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天然資源の開発利用に関する日米会議

耐風・耐震構造専門部会
第24回合同部会会議録

1992・メリーランド州
ゲイサースバーグ

THE 24ND JOINT MEETING
U.S.-JAPAN PANEL
ON
WIND AND SEISMIC EFFECTS

建 設 省 土 木 研 究 所

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土木研究所資料
第3162号1993年3月

UJNR
耐風・耐震構造専門部会
第24回合同部会会議録

要　旨

1992年5月にアメリカ・メリーランド州のNISTにおいて開催された耐風・耐震構造専門部会第24回合同部会の会議録をまとめたものである。主な課題は耐風工学，耐震工学，高潮・津波，日米共同研究，IDNDRについてである。

キーワード：耐風工学，耐震工学，高潮・津波

序　　文

天然資源の開発利用に関する日米会議（U.S. - Japan Conference on Development and Utilization of Natural Resources : 略称 ＵＪＮＲ）の耐風・耐震構造専門部会第２４回合同部会が、平成４年５月１９日から平成４年５月２２日の４日間にわたり、米国メリーランド州、ゲイサースバーグの国立標準技術研究所（ＮＩＳＴ）で開催されました。今回の会議には、日本側から委員９人、臨時委員５人の計１４名、米側からは委員５４名の参加を得て、論文の発表および１１作業部会での討論を通じ、両国の耐風、耐震構造分野における最近の研究情報と意見の交換、さらに、当部会が世界の自然災害の防除に果たすべき貢献策などについて活発な意見交換がなされました。

本会議におきましては、耐風工学、耐震工学をはじめとして日米合わせて４５編の論文が寄せられ、３６編の口頭発表が行われました。特に本会議では、吊橋の対風応答に関する風洞試験・解析による研究、群杭の特性についての実測・解析による研究、長期のデータを基にした高潮の FEMモデルの研究、免震構造に関する研究、耐震性に優れたコンクリートの研究、耐震補強に関する研究、および１９９２年３月のトルコでの地震に関する調査などについて報告がなされ、今後の耐風、耐震対策を推進する上で貴重な意見交換、情報交換を行うことができました。

合同部会に先立ち「第８回橋梁ワークショップ」が開催されより深い意見の交換が行われました。本ワークショップ成果ならびに先に実施された「第１回地震危険度及び設計地震力に関するワークショップ」、「第４回ライフライン施設の震災防止法に関するワークショップ」、および「第１回橋梁の免震ワークショップ」の成果は合同部会に報告され、さらに各作業部会から提案された７つのワークショップの開催計画が合同部会で承認されました。

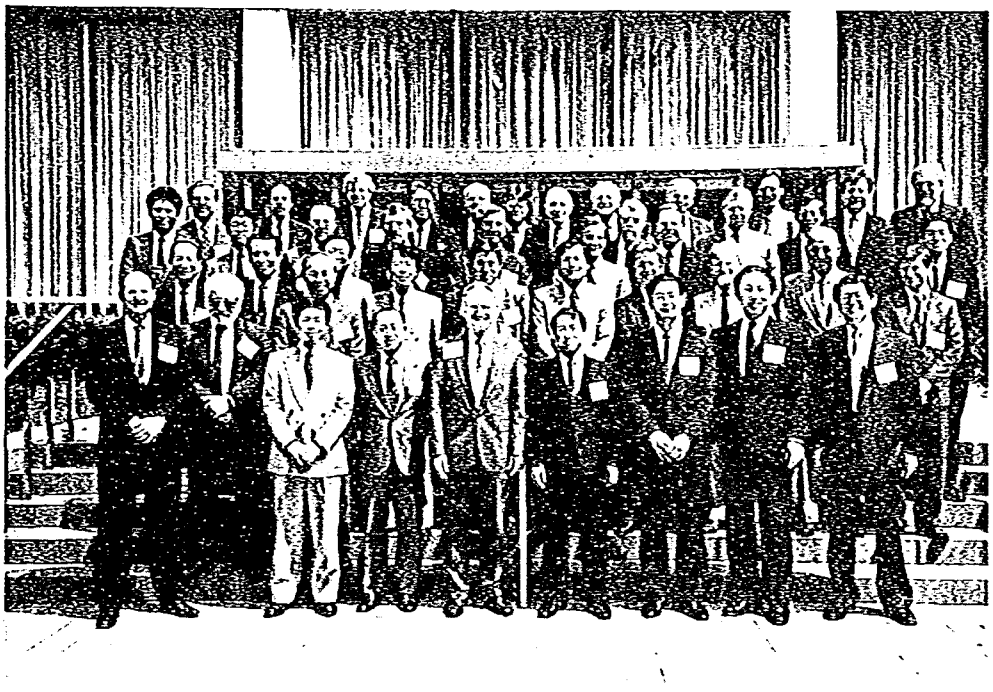
また、次回は耐風・耐震構造専門部会の２５周年にあたることから、これを記念して、第２５回合同部会中にシンポジウムの開催など特別な行事を予定することが決議されました。

本会議録は、第２４回合同部会に提出された論文と討議、並びに各作業部会の討議の概要をとりまとめたものです。本会議録を発刊し、当専門部会の活動状況を広くお知らせすることが、日米両国の耐風・耐震技術の向上に寄与するとともに、さらには、世界のこの分野における技術の発展貢献につながるものであると確信しております。

最後に、常日頃から我々の活動に絶ゆまぬ声援をお送りくださる関係各位に厚くお礼申し上げるとともに、当部会の活動を益々発展させることにより、今後とも皆様方のご期待に応えてまいりたいと願っております。

日本側部会長

住吉幸彦



耐風・耐震構造専門部会　第２４回合同部会　１９９２年５月１９日

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Ⅰ．合同部会日程

第２４回合同部会日程

5月１７日(日)	成田→Gaithersburg	(宿泊地)
１８日(月)	Capital Building視察	(Gaithersburg)
１９日(火)	開会式、合同部会、作業部会	(Gaithersburg)
２０日(水)	合同部会、作業部会	(Gaithersburg)
２１日(木)	合同部会	(Gaithersburg)
２２日(金)	合同部会、作業部会報告、最終結論採択、閉会式	(Gaithersburg)
２３日(土)	リーハイ大学、ATLSS実験施設等、New Orleansへ移動	(New Orleans)
２４日(日)	New Orleans市内視察	(New Orleans)
２５日(月)	Stennis航空宇宙ｾﾝﾀｰ視察	(New Orlrans)
２６日(火)	ＣＯＲＰＳ地区本部、港湾施設	(New Orleans)
２７日(水)	ＭＭＳ地方局、海洋構造物、San Franciscoへ移動	(San Francisco)
２８日(木)	San Francisco湾橋梁補修現場、UC Berkeley校	(San Francisco)
２９日(金)	補修及び免震プロジェクト現場、CORPS San Francisco湾模型	(San Francisco)
３０日(土)	San Francisco発	(機 中 泊)
３１日(日)	→ 成田着	

II. 会議プログラム

第２４回合同部会会議プログラム

開会式

１．日時　：　平成４年５月１９日（火）　１０：００～１１：３０

２．場所　：　商務省国立標準技術研究所（ＮＩＳＴ）

３．開会式　：

司会	米側事務局長	Noel J. Raufaste
開会		
挨拶	国立標準技術研究所長	John W. Lyons
	日本大使館科学技術参事官	林　幸秀
	米国側部会長	Richard N. Wright
	日本側部会長	住吉幸彦

米国側委員の紹介（米国側部会長）

日本側委員の紹介（日本側部会長）

第２４回合同部会常設議長の選出

議事採択

開会式終了

４．記念撮影

会議

５月１９日（火）

セッション　１　課題　Ｖ　ＩＤＮＤＲ　　議長　R.N. Wright

１２：３０　ＩＤＮＤＲに対する米国州政府の役割

(IDNDR, U.S. Federal Agency Role in IDNDR)

Kilho PARK*

１３：００　討議

セッション　２　課題　Ｉ　耐風工学　　議長　住吉　幸彦

１３：１０　超長大吊橋のフラッター特性

横山功一*　金崎智樹　保田雅彦

１３：３０　橋梁周辺の風の流れに対する電算シミュレーションを用いた風力の予測
(Predicting Wind Forces Using Computer Simulation of Air Flow around Bridges)
Okey U. ONYEWELUKWE* and Harold R. BOSCH

１３：５０　討議

１４：１０　休憩

１４：３０　作業部会

（A）「強震計配置計画と強震記録」　（野田、Brady）

（C）「既存構造物の補修及び補強」
及び（D）「構造物の性能評価」　（山内、Chong, Fuller）

（E）「地震動及び設計地震力」　（川島、Aigermissen）

（H）「地震時における地盤の挙動と安定」　（横山、Franklin）

（J）「交通施設の耐風耐風技術」　（藤原、Cooper）

１７：００　休止

5月２０日（水）

セッション　3　課題　Ⅲ　高潮・津波　議長　R.N.Wright

８：３０　1972年眉山の地滑りにより発生した津波の数値計算
後藤智明　高山知司　（発表：野田節男）

８：５０　海岸設計に用いる高潮の歴史に対する経験的解析
(Empirical Simulation of Strong Histories for coastal Design)
Martin C. MILLER* and Robin D. REINHARD

９：１０　討議

９：３０　休憩

セッション　4　課題　Ⅳ　日米共同研究　議長　R.N.Wright

９：５０　Development of Precast Seismic Structural System -Interium Report-
室田達郎　中田慎介　塩原等　（発表：山内泰之）

１０：１０　ＰＲＥＳＳＳ日米共同研究－現状報告
(U.S.-Japan Cooperative Research in Precast Seismic Structural Systems Program - Status Report)
Nigel Priestley and H.S. Lew*

１０：３０　バリアブルダンパーを用いた道路橋の地震応答制御
川島一彦*　運上茂樹　清水英之

１０：５０　討議

セッション　5　課題　Ⅱ　耐震工学（その１）　議長　R.N. Wright

11：10　Jordanelle　ダムに対する耐震設計上の考察
(Seismic Design Considerations for Jordanelle Dam)
John WILSON*, Perry HENSLEY, William ENGEMOEN and Francis MCLEAN

11：30　フィルダムに作用する地震力及びその簡易推定法に関する考察
藤沢侃彦　安田成夫　松本徳久　山辺健二　(発表：山口嘉一)

11：50　地震危険度解析における不確定性の解析
(Resolving Uncertainties in Seismic Hazard Models)
Jeffrey K. Kimball*

12：10　討議

12：30　昼食

セッション　6　課題　II　耐震工学(その1　続き)　　議長　R.N.　Wright

13：30　地震の発生の緊急連絡システムのフィージビリティスタディー
(Earthquake Alert System Feasibility Study)
Philip HARBEN*

13：50　大規模地下構造物の耐震設計法ガイドライン(案)
朝倉肇*　川島一彦　杉田秀樹

14：10　ライフラインの脆弱性
(ATC-25 Vulnerability of Lifelines)
William BIVINS*

14：30　討議

14：50　休憩

15：00　作業部会
(B)「大型実験計画」　(大谷、Lew)
(F)「ライフライン施設の災害防止」　(川島、Dikkers)
(G)「パッシブ、アクティブ、ハイブリッドコントロール」　(北川、Liu)
(I)「高潮・津波」　(横山、Meyers)
(K)「海洋施設の耐風耐震技術」　(野田、Smith)

17：00　休止

5月21日(木)

セッション　7　課題　II　耐震工学(その2)　　議長　住吉　幸彦

8：15　ビルの免震に関する設計クライテリア
(Design Criteria for Base Isolation of Buidings)
H. Tripp SHENTON*

8：35　道路橋の免震設計マニュアル(案)

川島一彦* 長谷川金二 長島博之

8：55 滑動免震構造システムに対する摩擦力可変型支承
(Friction Controllable Bearings for Sliding Base Isolation Systems)
Maria Q. FENG and Masanobu SHINOZUKA*

9：15 討議

9：35 休憩

セッション 8 課題 II 耐震工学（その2 続き） 議長 住吉 幸彦

9：50 日本における制振機構を有する建築構造物の現状とその動向
北川良和* 緑川光正

10：10 NSFにおける構造物の応答制御に関する研究プログラム
(Status of NSF Structural Control Research Program)
S.C. LIU*, Ken CHONG and Henry LAGORIO

10：30 免震用積層ゴム支承の標準化のための諸検討
大橋雄二 山内泰之*

10：50 免震橋梁の動的挙動に関する温度の影響
中野修* 谷口秀之 西弘明

11：10 討議

11：30 昼食

セッション 9 課題 II 耐震工学（その2 続き） 議長 住吉 幸彦

12：30 液状化性砂地盤上盛土の動的遠心模型実験
古賀泰之 古関潤一* 高橋晃浩

12：50 サンタクルツの地滑り
(Santa Cruz Landslide)
Mary Ellen HYNES*

13：10 液状化による地盤流動の予測及び杭に及ぼす影響に関する研究
常田賢一 松本秀應 東畑郁生 佐々木康 （発表：古関潤一）

13：30 討議

13：50 休憩

セッション 10 課題 II 耐震工学（その2 続き） 議長 住吉 幸彦

14：10 水平載荷実験による杭基礎の保有水平耐力
岡原美知夫 中谷昌一 木村嘉富 高木繁 （発表：藤原稔）

14：30 沖合い構造物の耐震性評価
(Seismic Requalification of Offshore Structures)

Charles SMITH*

14：50　港湾構造物に対する地震の影響

土田肇、野田節男*

15：10　討議

セッション　11　課題　II　耐震工学（その2　続き）　　議長　住吉　幸彦

15：30　行政命令12699号「連邦政府もしくは連邦政府によって補助を受けたり規準を定められた構造物の耐震安全性」

(Implementation of Executive Order 12699 "Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction")

Richard WRIGHT*

15：50　工兵隊における耐震工学研究プログラム

(CORPS Earthquake Engineering Research Program)

William ROPER*

16：10　世界地震危険度管理プログラム

(Worldwide Seismic Risk Management Program - Progress Report)

Ted ALGERMISSEN*

16：30　討議

16：50　休止

5月22日（金）

セッション　12　耐震工学（その3）　　議長　R.N. Wright

8：30　高強度材料を用いた鉄筋コンクリート造建築物の耐震性能（1991年度構造性能に関する研究成果

室田達郎　平石久廣　上之薗隆志　塩原等　藤谷秀雄　（発表：北川良和）

8：50　じん性のないコンクリートフレームシステムの動的試験

(Dynamic Testing of Non-ductile Concrete Frame Systems)

P.A. BRADY and S. SWEENY*

9：10　RC柱の振動台破壊実験

大八木規夫　小川信行*　箕輪親宏　御子柴正

9：30　討議

セッション　13　耐震工学（その3　続き）　　議長　R.N. Wright

9：50　中空のタイル壁の耐震性能プログラムの見直し

(A Hollow Clay Tile Wall Seismic Performance Program Overview)

J.E. BEAVERS*, W.D. JONES and W.C.T. STODDART

１０：１０　構造計算用プログラムの評定・指定制度

大谷圭一*

１０：３０　トルコ地震速報

(Preliminary Report on Reconnaissance of Turkish Earthquake)

Mehmet CELEBI*

１０：５０　討議

１１：１０　休憩

１１：２５　作業部会主催のワークショップ報告

作業部会（Ｅ）「地震動及び設計地震力ＷＳ」、平成３年５月２７日～３０日、つくば

（報告者：T. ALGERMISSEN）

作業部会（Ｆ）「第４回ライフライン施設の災害防止ＷＳ」、平成３年８月１９日～２１日、ロサンジェルス

（報告者：R. Dikkers）

作業部会（Ｇ）「橋梁免震設計ＷＳ」、平成３年９月４日～５日、バッファロー市

（報告者：川島一彦）

作業部会（Ｊ）「第８回橋梁ＷＳ」、平成４年５月１１日～１５日、シカゴ市

（報告者：J. Cooper）

１２：２０　昼食

１３：３０　作業部会報告

（Ａ）「強震計配置計画と強震記録」

（Ｂ）「大型実験計画」

（Ｃ）「既存構造物の補修及び補強」

（Ｄ）「構造物の性能評価」

（Ｅ）「地震動及び設計地震力」

（Ｆ）「ライフライン施設の災害防止」

（Ｇ）「パッシブ、アクティブ、ハイブリッドコントロール」

（Ｈ）「地震時における地盤の挙動と安定」

（Ｉ）「高潮・津波」

（Ｊ）「交通施設の耐風耐風技術」

（Ｋ）「海洋施設の耐風耐震技術」

１５：１０　最終結論の採択

１６：１０　休憩

閉会式

1．日時	：	平成4年5月22日（金）	16：20～16：45	
2．場所	：	商務省国立標準技術研究所		
3．閉会式	：	司会	米側事務局長	Noel J. Raufaste
		挨拶	日本側部会長	住吉　幸彦
			米国側部会長	Richard Wright

III. 開会式挨拶

III 開会式挨拶

国立標準技術研究所　所長　John W. Lyons

住吉所長、林参事官、そして日本、アメリカ両国からのすばらしい出席者の皆様、おはようございます、そしてＮＩＳＴへようこそ。

ここに、ＵＪＮＲ、第２４回耐風・耐震構造専門部会を開催し、皆様に来ていただけたことをたいへん喜ばしく思っております。これらの部会はＵＪＮＲの中でも活発で、もっとも成果を挙げている部会であると申し上げてもいいと考えています。

この部会は２４回目を数えるわけですが、Fire Research Panelというのがございまして、これは１２年から１４年続いております。私はそのパネルの第１回の副議長をさせていただいたもので、専門部会の作業がいかに難しいか存じ上げております。これらの分野での日米の協力関係はこの上もなく良く、両国の科学技術系の人々に対して模範となってきたといって良いと思っています。

そして、両国間の科学技術の協力がより進み、模範的な状態になることを望み、またそうなると信じております。

会議の前に住吉所長にお話ししたのですが、私はなんどか筑波を訪れ、土木研究所、建築研究所やそのほかの筑波にあります施設を訪問させて頂いたことがあります。ですから、（土木研究所のされております）研究については良く存じ上げているのですが、これまでMITIの実験室を含め、構造物に関するものやFire Programに関してなど、多くの共同研究や研究者の交流が行われてきました。

この専門部会の作業は全ての人々の生活の向上に役立ってきましたし、今回の会議の議論、そしてその成果はさらに役立つものとなると確信しております。また、十分な議論がなされることを望んでおります。

ここで、少し話しを脱線させたいのですが、来秋、土木研究所は創立７０周年を迎えられます。そこで、数カ月早いのですが、ここ、NISTで住吉所長に記念品を贈らせて頂き、お祝いを申し上げたいと思います。

日本大使館科学技術参事官　　林　幸秀

ライオンズ所長（Dr. Lyons）、ライト博士（Dr. Wright）、住吉所長、両国の優れた部会の皆様、おはようございます。

このような機会を与えてくださいましてありがとうございます。第２４回ＵＪＮＲ耐風・耐震構造合同部会に招待され、大変感謝しております。その開会に挨拶を出来ますこと

は大変に光栄に思っております。

ご存知のように、ＵＪＮＲは１９６４年１月に、天然資源に関する科学者、工学技術者の交流、研究情報の交換により両国の利益を図ることを目的として設立されました。ＵＪＮＲには１７の専門部会が設けられました。

この耐風・耐震構造合同部会は１９６９年に設けられ、部会の下で行われた両国の協力は強風、地震、高潮、津波による被害の軽減を目的とする技術の開発交流に大きな貢献しました。

この部会は、１９６１年科学技術協定のもと先駆的な役割をしてきたものと思っております。本部会が、毎年定期的に正式の会合を持った最初の部会であり、会議の後に視察を含めた部会でした。会議録は毎年日米両国でそれぞれ出版されています。部会の下に１１の作業部会があり、定期的に会合を持ち、会議録を出版しています。

ＮＩＳＴは米側部会長と事務局を持ち、日本側は土木研究所が日本側部会長と事務局を持っています。ここに、日本政府にかわりまして両機関に感謝するとともに敬意を表します。

最近、日本政府は２１世紀に向けての科学・技術基本方針を含む内閣決定を致しました。この中には、以下の３つの点が目標として掲げられています。

・地球と協調した人類の共存

・知的財産の拡大

・安心して住める明るい社会の構築

これら３点は本部会の目的と非常に深い関連を持っています。日本では何度も自然災害を被っています。ですから、自然災害の起きる機構、予測、および防止に関する技術を進展させる研究が重要であることが認識されています。日本は健康増進・保持、環境改善、社会経済的基盤の整備、災害予知・防止に科学技術を用いなければなりません。私は、この内閣の決定により本部会による両国の協力の重要性がさらに強調されたものと考えています。

両国は通行、貿易だけでなく科学技術の面でも完全な合意はなされていません。しかし、両国が協力し、様々の自然災害を軽減するよう技術を進展させるべきであることは明かです。本部会による力強く効果的な技術的活動は両国にとって非常に重要です。日本の古い諺に”災害は忘れた頃にやってくる”というものがあります。この諺は、人々は災害を忘れること無く被害を最小にするよう準備する必要があることを物語っています。本部会の結果はこの諺に語られた教訓を実行するのに非常に重要であり、結果として両国の生活水準の向上をもたらすと確信しております。

熱心な、深い議論を経て、貴重な部会の結論が導かれることを希望いたします。

どうもありがとうございました。

日本側部会長　住吉　幸彦

ライオンズ博士（Dr.　Lyons）、ライト部会長（Dr.　Wright）、ローファスト事務局長（Ｍｒ．Raufaste）ならびに米国部会の皆様、私は本日ここに日本部会を代表して御挨拶を述べる機会を頂きましたことを、大変光栄に存じる次第でございます。

まず、第２４回合同部会の開会にあたり、お世話頂いた米国商務省国立標準技術研究所、在米日本大使館の関係各位に厚くお礼申し上げるとともに、会議準備の労をとられた米国側事務局の各位に心から感謝いたしたいと存じます。

さて、この耐風耐震構造専門部会は、１９６８年に東京で第１回の合同部会が開催されてから、今回で第２４回目を迎えたわけでございます。この間、その活動分野は年々拡大してきており、合計１７あるＵＪＮＲの部会の中でも最も活発な活動を続けていると高い評価をいただいております。昨年「海洋施設の耐風耐震技術作業部会」が誕生しましたが、これを含めて、作業部会は合計１１になりました。作業部会が主催するワークショップも多数回開催され、日米両国の技術情報の交換に大きく寄与しております。

１９９１年５月までに２３回の合同部会と２７回のワークショップが開催され、日米あわせて１，２８１編の論文が発表されております。このように大きな技術情報を強風、地震、高潮、津波などの自然災害に苦しむ多くの国々にも広く活用していただけるように、この４月に日米両部会が協力して「文献目録」を公刊しました。今後、広く発展途上国に配布する予定でありますが、このような活動が、１９９０年から始まった「国際防災の１０年」に大きく貢献することを期待致しております。現在までの２４年間の活動を支えてこられた日米双方の関係各位のご努力に、改めて深く敬意を表する次第であります。

さて、この１年間を振り返ってみますと、日本では幸いなことに大きな地震は有りませんでしたが、それだけひずみエネルギーが蓄積されたと考えるべきでしょう。昨年の秋に日本列島を縦断した台風１９号では、農作物を中心に大きな被害を被りました。また、九州の雲仙岳の噴火により数千人の住民が１年近くも仮設住宅に住むという避難生活を余儀なくされております。海外に目を転じますと、トルコでは大きな地震があり、またバングラディシュではサイクローンによる高潮で多くの人々が亡くなっています。

このように自然災害の発生は世界各地でとどまることなく、耐風耐震分野において日米両国が協力して調査研究を進め、対策に反映して行かなければならないことが非常に多数あります。この部会の活動が益々重要になって来ると申せましょう。

私どもは、この４日間の合同部会で米国の新しい技術的知見をお伺いできることを期待しております。また、合同部会後の視察旅行におきましては、米国の代表的な研究機関における活動状況や実務における優れた技術を見聞できることを楽しみにしております。これらの機会を通じて、災害を軽減するための創造的な研究が推進されることを期待致して

おります。

最後に、日本側部会を代表して、第２４回合同部会の準備にあたられた米側部会のライト部会長、ローファスート事務局長、事務局ならびに米側委員の皆様に厚くお礼申し上げる次第であります。

今日から始まる合同部会が両国にとって有意義なものとなるように祈念いたしまして、私のご挨拶と致します。

ありがとうございました。

米国側部会長　Richard N. Wright

ライオンズ所長、林参事官、この専門部会のためにいろいろ御尽力頂きありがとうございます。

私たちは、これまで２３年間のこの専門部会での成果を誇りに思っております。

また、日本、アメリカそして広くは世界中において強風、地震により平静な生活を脅かされております。

そして、風、地震による被害をくい止めるために、共に学び、情報を共有し、防災対策案（recommendation）をまとめたいと強く思っています。

住吉幸彦土木研究所所長、日本側の専門部会議長を特に歓迎いたします。また、住吉所長に土木研究所の７０周年、および耐風耐震構造専門委員会での２３年間の御活躍にお祝い申し上げます。住吉所長のリーダーシップによりまして、ますますの発展されることと確信しております。

今回の２４回合同部会も密度の高い予定が組まれております。各々の研究所での研究成果、２３回合同部会以降にもたれた４回の作業部会の成果、プレキャスト構造系に関する共同研究の成果、日米両国での地震・強風に対する防災活動の進歩についての報告、そして、自然災害防止に関する国際年（１０年）の活動報告が予定されています。例えば、1969年から1991年の専門部会での出版物のリストを共同で作成し、全世界の人に利用可能なようにします。

合同部会の後、視察旅行が計画され、リーハイ大学の新しい構造実験室、ステニス宇宙飛行センター、ニューオリンズ港やメキシコ湾の海中構造物のハリケーン防災施設、サンフランシスコの建物、ライフラインの地震被害の修復の研究、および実情などを視察する予定となっています。

日本側の多くの準備をしてくださったこと、そしてはるばる会議出席のため御足労頂いたことに感謝いたします。日米両国の人々の生活の質、安全性の向上に役立つ十分で実りある交流がなされることを楽しみにしております。

IV. 会 議 参 加 者

第２４回合同部会参加者

日本側

部会長	住吉　幸彦	建設省土木研究所長
委員	藤原　稔	建設省土木研究所構造橋梁部長
〃	横山　功一	建設省土木研究所構造橋梁部構造研究室長
〃	川島　一彦	建設省土木研究所地震防災部耐震研究室長
〃	北川　良和	建設省建築研究所国際地震工学部長
〃	山内　泰之	建設省建築研究所企画部国際基準研究官
〃	野田　節男	運輸省港湾技術研究所構造部長
〃	大谷　圭一	科学技術庁防災科学技術研究所防災総合研究部長
〃	小川　信行	科学技術庁防災科学技術研究所防災総合研究部耐震工学研究室長
臨時委員	塩路　勝久	建設省土木研究所下水道部下水道研究室主任研究員
〃	古関　潤一	建設省土木研究所機械施工部動土質研究室研究員
〃	山口　嘉一	建設省土木研究所ダム部フィルダム研究室研究員
〃	中野　修	北海道開発局開発土木研究所構造部構造研究室長
〃	朝倉　肇	（財）先端建設技術センター研究第一部長

米国側

部会長	Dr. Richard N. Wright	商務省国立標準技術研究所建築防火センター所長
事務局長	Mr. Noel J. Raufaste	〃　〃　〃　共同研究計画課長
部会委員	Kharaiti I.Abrol	在郷軍人局
	Charles E.Anderson	内務省　開拓局
	James Beavers	エネルギー省、Martin Marietta Energy System
	A.Gerald Brady	内務省　地質調査所
	Mehmet Celebi	内務省　地質調査所
	Ted Algermissen	内務省　地質調査所
	Celso Barrientos	商務省　海洋気象庁
	William S.Bivens	緊急事態管理庁
	Harold Bosch	ＦＨＷＡ
	C.Y.Chen	国務省
	Vincent P.Chiarito	陸軍省　工兵隊水路試験所
	Robert Dikkers	商務省国立標準技術研究所
	G.Robert Fuller	住宅都市開発省
	Philip Harben	エネルギー省 、Lawrence Livermore国立研究所
	Walter Hays	内務省　地質調査所
	James D.Cooper	ＦＨＷＡ
	A.G.Franklin	陸軍省　工兵隊水路試験所
	R.Gausseres	国務省
	Mary Ellen Haynes	陸軍省　工兵隊水路試験所
	James R.Hill	エネルギー省

Larry C. Hultengren	国務省
Michael G. Katona	Air Force Civil Eng. Lab.
Jeff Kimball	エネルギー省
Shih-Chi Liu	国立科学財団
Herbert Meyers	商務省　海洋気象庁
Dov Jaron	国立科学財団
Roger Kenneally	原子力規則委員会
H. S. Lew	商務省国立標準技術研究所
Michael Mahoney	緊急事態管理庁
Martin Miller	陸軍省　工兵隊水路試験所
Ugo Morelli	緊急事態管理庁
D. Oh	国務省
K. O'Reilly	国務省
G. Ross	国務省
Howard D. Nickerson	Naval Facilities Eng. Command
O. Onyemelukwe	ＦＨＷＡ
Kilho Park	商務省　海洋気象庁
William E. Roper	工兵隊
Richard Sause	リーハイ大学
John B. Scalzi	国立科学財団
Charles E. Smith	内務省　鉱物管理部
S. Sweeny	U. S. Army Construction Eng. Research Lab.
Samuel Kramer	商務省国立標準技術研究所　次長
Masanobu Shinozuka	国立地震工学研究センター
Chris Stoddart	エネルギー省、Martin Marietta Energy System
John Wilson	内務省　開拓局
John W. Lyons	商務省国立標準技術研究所　所長
Tripp Shenton	商務省国立標準技術研究所
Riley Chung	商務省国立標準技術研究所
Erdal Safak	内務省　地質調査所

Ⅴ．合同部会最終結論

天然資源の開発利用に関する日米会議

耐風・耐震構造専門部会

第２４回合同部会最終結論

平成４年５月１９～２２日

国立標準技術研究所（米国メリーランド州ゲイザーズバーグ市）

ここに以下の決議を採択する。

１．第２４回合同部会は、両国にとって有益な技術情報の交換の機会を与えた。耐風・耐震問題に関する研究協力の重要性に鑑み、合同部会の継続は極めて重要である。

２．第２３回の合同部会以降、下記のような活動がなされた。

ａ）耐震・耐風技術の進歩を図るための研究を推進、実施しするために、両国の研究者の交流が行われた。

ｂ）技術資料、調査報告書、ワークショップ会議録が交換された。

ｃ）日米両国の機関及び国際機関、特に発展途上国の機関に配布するために、日米両部会は1969-1991年間の文献目録を公刊した。

ｄ）５つの重要な会議が開催された。

（ａ）第１回地震危険度及び設計地震力に関するワークショップ

…… 作業部会（Ｅ）

平成３年５月２８，２９日、つくば市

（ｂ）第４回ライフラインワークショップ …… 作業部会（Ｆ）

平成３年８月１９～２１日、ロサンジェルス市

（ｃ）第１回橋梁免震ワークショップ …… 作業部会（Ｇ）

平成３年９月４日、５日、バッファロー市

（ｄ）第２回プレキャスト耐震構造システム日米合同技術調整委員会

…… 作業部会（Ｂ）

平成３年１０月、つくば市

（ｅ）第８回橋梁ワークショップ …… 作業部会（Ｊ）

平成４年５月１１日～１５日、シカゴ市

3．強風と地震による災害に対する関心を高めるために、国際防災の１０年に関する国際防災シンポジウムが平成３年５月２０日に岐阜市で開催された。このシンポジウムで行われた５編の発表のうち、３編は本部会から行われた。シンポジウムには600人以上が参加した。

4．本部会は、両国の「国際防災の１０年」（IDNDR）国内委員会との合同部会会議録や作業部会ワークショップ報告書の交換等を通して、ＩＤＮＤＲに対する貢献を続ける。

5．本部会は、プレキャスト耐震構造システム(PRESSS)に関する日米共同研究の重要性を確認するものである。

6．本部会は、第２４回合同部会に際して作成された各作業部会の作業部会報告を承認する。これらの報告には、各作業部会の目的、活動範囲、成果、将来計画、情報交換、活動の効果および障壁が記述されている。

7．本部会は、各作業部会から提案された来年までに計画されている以下のワークショップの開催計画を承認する。

a．作業部会（B）　ハイブリット構造システムに関するワークショップ
平成４年９月、サンフランシスコ市

b．作業部会（E）　第２回地震動と設計地震力に関するワークショップ
平成５年５月、つくば市

c．作業部会（F）　第５回ライフライン施設の災害対策に関するワークショップ
平成４年１０月２６，２７日、つくば市

d．作業部会（G）　第２回橋梁の免震設計に関するワークショップ
平成４年１２月、つくば市

e．作業部会（G）　高性能・高性能材料とシステムに関するワークショップ
平成５年５月、日本

f．作業部会（J）　第９回橋梁ワークショップ
第２５回合同部会前に、つくば市

g．作業部会（K）　海洋施設の耐風耐震技術に関するワークショップ
平成５年５月、横須賀

さらに、第25回合同部会以後に、以下のワークショップの日程が定められている。

a．作業部会（A）　ディジタル記録技術に関するワークショップ
平成５年秋、ハワイ

b．作業部会（H）　液状化対策技術に関するワークショップ
平成５年１０月、日本

c．作業部会（I）　第３回高潮・津波ワークショップ
平成５年８月１９，２０日、日本

ワークショップの日程、プログラムは各作業部会長が両国部会長の同意を得て決定する。
第２５回の合同部会前に開催される各作業部会活動の成果は、第２５回合同部会において報告するものとする。

8．耐風耐震構造専門部会の２５周年を記念して、第２５回合同部会中に特別な行事を予定する。両国の事務局は、部会活動を記念すると同時に部会で得られた技術の実務への移転を促進するために、シンポジウムの開催や出版物の刊行等、適切な方法を検討する。日程調整や手配を平成４年中秋から開始できるように、平成４年初秋に両国の部会で事務局の提案を検討する。

9．本部会は、強風、強震による損害の軽減のための人材、技術情報、研究資料並びに観測記録の継続的交流の重要性を確認した。また、本部会は、両国が保有している大型実験施設の利用の重要性を確認した。したがって、今後ともこれらの活動を強化、拡大するとともに、必要に応じて各作業部会は類似の目的を有する他の作業部会とも活動を分担し合うものとする。このような交流を促進するため、本部会は公式な支援を用意する。

10．ＵＪＮＲ耐風・耐震構造専門部会の第２５回合同部会を平成５年５月に、日本のつくば市の建設省土木研究所において開催する。合同部会の実施時期、プログラム及び日程は、米国部会と協議の上、日本部会が提唱する。

〈以上〉

RESOLUTIONS

RESOLUTIONS OF THE TWENTY-FOURTH JOINT MEETING U.S.-JAPAN PANEL ON WIND AND SEISMIC EFFECTS (UJNR)

National Institute of Standards and Technology
Gaithersburg, MD 20899
May 19-22, 1992

The following resolutions are hereby adopted:

1. The Twenty-Fourth Joint Panel Meeting provided an opportunity to exchange valuable technical information which was beneficial to both countries. In view of the importance of cooperative programs on the subject of wind and seismic effects, the continuation of Joint Panel Meetings is considered essential.

2. The following activities have been conducted since the Twenty-Third Joint Meeting:

   a. Researchers were exchanged from both countries to promote and conduct research that advanced the state of earthquake and wind engineering.

   b. Technical documents, research reports, and proceedings of workshops were exchanged.

   c. Both sides published a List of Panel Publications 1969-1991 for distribution to U.S. and to Japan domestic organizations and to international organizations, especially those in the less developed countries.

   d. Five significant conferences were held:

      a) The First Workshop on Hazard/Risk Assessment and Design Earthquake Loading, Task Committee (E), Tsukuba, May 28-29, 1991.

      b) The Fourth Workshop on Earthquake Disaster Prevention for Lifeline Systems, Task Committee (F), at Los Angeles, CA, August 19-21, 1991.

      c) Workshop on Earthquake Protective Systems of Bridges, Task Committee (G), at Buffalo, NY, September 4-5, 1991.

      d) The Second U.S.-Japan Joint Technical Coordinating Committee on PRESSS, Task Committee (B), at Tsukuba, Japan, October 1991.

      e) The Eighth Bridge Workshop, Task Committee (J), at Chicago, IL, May 11-15, 1992.

A one day International Symposium on the International Decade for National Disaster Reduction was held in Gifu City on May 20, 1991, to increase the awareness of the people of Japan about the disastrous effects from strong winds and earthquakes. This Panel provided three of the five presentations. More than 600 persons attended.

4. The Panel will continue to seek methods to contribute to the International Decade of Natural Disaster Reduction (IDNDR) such as exchanging Proceedings of joint Panel Meetings and Task Committee Workshops with their respective Country's National Committees of IDNDR.

5. The Panel recognizes the importance of the work by both sides in the U.S.-Japan Joint Research Program on Precast Seismic Structural Systems (PRESSS).

6. The Panel accepts each Task Committee's report developed during the Twenty-Fourth Joint Meeting. Each report presents objectives, scope of work, accomplishments, future plans, information exchange, impacts, and barriers.

7. The Panel endorses the following proposed Task Committee Workshops during the coming year.

   a. Task Committee (B) plans a workshop on Hybrid Structural Systems, September 1992, San Francisco, CA.

   b. Task Committee (E) plans a 2nd workshop on Ground Motion and Seismic Design Forces, May 1993, Tsukuba.

   c. Task Committee (F) plans a 5th workshop on Earthquake Disaster Prevention for Lifeline Systems, October 26-27, 1992, Tsukuba.

   d. Task Committee (G) plans a 2nd workshop on Earthquake Protective Systems of Bridges, December 1992, Tsukuba.

   e. Task Committee (G) plans a workshop on Smart and High Performance Material and Systems, May 1993, Japan.

   f. Task Committee (J) plans a 9th Bridge Workshop before the 25th Joint Meeting, Japan.

   g. Task Committee (K) plans a Workshop on Wind and Earthquake Engineering for Offshore and Coastal Facilities, May 1993, Yokosuka.

   In addition, three Task Committees have scheduled workshops after the 25th Joint Panel Meeting:

   a) Task Committee (A) has scheduled a workshop on Digital Recording Technologies, Fall 1993, Hawaii.

   b) Task Committee (H) has scheduled a workshop on Remedial Treatment of Potentially Liquefiable Soils, October 1993, Japan.

   c) Task Committee (I) has scheduled the 3rd Workshop on Storm Surge and Tsunamis, August 19-20, 1993, Japan.

   Scheduling for the workshops shall be performed by the U.S. and Japan Chairmen of the respective Task Committee with concurrence of the Joint Panel Chairmen. Results of each activity conducted before the 25th Joint Meeting shall be presented at the 25th Joint Meeting.

8. In the spirit of celebrating the 25th Anniversary of the Panel on Wind and Seismic Effects, a special event will be conducted during the 25th Joint Panel Meeting to mark this occasion. Both

sides' Secretaries will identify an appropriate method that celebrates the Panel's activities and facilitates the transfer of the Panel's technologies into practice, e.g., special symposium, publication, joint activity. The Secretarys' suggestions will be discussed at the respective sides' early fall 1992 domestic meetings to begin scheduling and arranging this event during mid-fall 1992.

9. The Panel recognizes the importance of continued exchange of personnel, technical information, research results, and recorded data that lead to mitigating losses from strong winds and earthquakes. The Panel also recognizes the importance of using available large-scale testing facilities in both countries. Thus, these activities should continue to be strengthened and expanded and, as appropriate, share Task Committee activities at other meetings that have technical interests in the Task Committee activities. To facilitate these exchanges, the Panel will provide official endorsement.

10. The Twenty-Fifth Joint Meeting of the UJNR Panel on Wind and Seismic Effects will be held at PWRI, Tsukuba, Japan, May 1993. Specific dates, program, and itinerary will be proposed by the Japan-Side Panel with concurrence of the U.S.-Side Panel.

VI. 作業部会報告

（A）強震計配置計画と強震記録に関する作業部会報告

開　催　日：１９９２年５月１９日
場　　所：国立標準技術研究所　ゲイザーズバーグ市
出　席　者：日本側　野田節男（臨時作業部会長）　（港湾技術研究所）
　　大谷圭一　（防災科学技術研究所）
　　塩路勝久　（土木研究所）

　　米国側　A．G．Brady（作業部会長）（USGS）
　　H．Meyers　（NOAA）
　　P．E．Harben　（DOE）

［活動要約］

1．目的

本作業部会は、研究者および実務技術者への強震記録の提供の調整を目的とする。作業部会の委員は、強震計の設置・観測・記録の処理解析、利用者に提供する強震記録をより高品質なものとするための研究、および強震地動と構造物の地震応答の解析に従事している。

2．活動の範囲

作業部会は、強震地動に関連する研究、強震記録の処理、および構造物の応答記録に関する情報提供の調整を行う。定常的な記録と刊行物に加え、作業部会の技術的対応は以下のものを含む：

・一般的にはＵＪＮＲ合同部会の際に、作業部会ワークショップおよび会合を企画開催する。
・学協会の会合において本作業部会に関連する課題のセッションを開催する。
・強震記録の所有者、記録の利用者、および地震工学者社会それぞれの権利と期待とを考慮しつつ。重要な強震記録の数字化データを提供する方式を考慮実現する。
・数字化された強震記録のすべての利用者が記録の利用上の限界について適切な知識を有するようにするため、コンピューターによる強震記録の処理方法に関する最新の情報を交換する方法を考慮実現する。

免震機構または他の応答軽減機構を有する構造物に対する適切な地震計の配置や応答解析について研究計画を策定する。
個々の地点の応答特性を把握するために、恒久的におよび臨時に高密度地震計網を展開し、そのデータを活用することについて研究計画を策定する。

3．成果

・日米で取得した強震記録のカタログが交換されている。米国のデータは“Seismic Engineering Program Report”として、日本のデータは“Storong-Motion Earthquake Records in Japan”として刊行されている。数値化データがテープ、フロッピー、およびＣＤＲＯＭにより交換された。その他、多くの文献が交換された。
・北部および南部カリフォルニアに発生した地震の報告書（地質調査所およびカリフォルニア州鉱山地質局刊行）が日本側に提供された。
・１９９２年２月２日東京湾の地震速報（強震観測連絡会議：防災科学技術研究所編）は米国側に提供された。
・地質調査所よりＬ．Ｓｅｅｋｉｎｓによる北部および中部アメリカにの地震の加速度数値化記録：１９３３－１９８６（１９９２年５月）が、Ｍ．Ｃｏｖｅｒｓｅによる加速度強震記録の処理プログラムとユーザーマニュアル（１９９２年５月）と共に日本側に提供された。
・地理データセンター（国立海洋大気庁）より強震記録カタログのフロッピーディスクおよびソフト（１９９２年までの公開記録およびＬ．Ｗ．Ｒｏｗ Ⅲによるパソコン用強震記録カタログ：ＳＭＣＡＴ（１９９０年７月）のユーザーマニュアル）が日本側に提供された。

4．将来計画

・高密度強震観測の記録の解析の調整：３次元地中高密度観測網（地球物理学および地震工学）および構造物高密度観測（構造工学）

＊日米の研究者を含む２０名程度の数値記録技術に関するワークショップを１９９３年ハワイで開催するために、科学技術庁および国立科学財団に支援を申請する。ワークショップは２日間および現地視察を予定する。

5．情報交換

・国内関係者との間で、技術的集会、研究発表会、およびワークショップを通じて実施する。
・強震計による記録が得られた地点を中心とした地震後の現地調査への参加を通じて実

施する。

・相手側作業部会との間で、作業部会ワークショップ；相互訪問；セミナー；講演、および毎年の合同部会への参加を通じて実施する。

・最も重要な情報交換は、注目すべき地震についてに記録の得得状況の速報（およびそれに引き続くテープまたはディスクによる数字化記録）の交換である。

6．活動の効果

・世界中で強震記録の処理を統括している責任者の多くは、本作業部会およびその参加員の成果またはそれを応用した手法を用いている。これに該当する外国の観測網は以下のものを含む。

AUSTRALIA - Austrarian Seismological Center, Bureau of Mineral Resources PO Box 378, Canberra ACT 2601.

CANADA - Pacific Geoscience Center, PO Box 6000, Sidney, BC. V82.

ITALY - The Italian National Commission for Research and Development on Alternative Energy Sources(ENEA), Rome.

MEXICO - Japan-Mexico Earthquake and Disaster Prevention Center, Mexico City

PERU - Japan-Peru Enrthquake and Sisaster Prevention Center, Lima.

TAIWAN - AMART Array

TURKEY - University of Bogazici, 80815 Bebek Istanbul.

YUGOSLAVIA - Institute of Earthquake Engineering and Engineering Seismology, Skopje.

・強震記録が収得されたところ以外の国における利用上都合のよい状態に処理された地震データおよび記録の利用

・注目すべきじしんの数字化記録が活動の範囲の第３項に基づいて配布されることにより、研究用データベースが充実される。設計事務におけるこれらのデータの利用は地震災害の軽減に貢献する。

7．問題点

・作業部会の共同活動のための資金不足。

・訪問研究なくしては、参加者の濃密な相互啓発を伴う共同研究作業は困難である。（資金とスタッフの不足がしばしば障害となる。）

(A) STRONG-MOTION INSTRUMENT ARRAYS AND DATA

Date: May 19, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees:

U.S. Side -	A. G. Brady (Chairman)	(USGS)
	H. Meyers	(NOAA)
	P. E. Harben	(DOE)
Japan Side -	S. Noda (Chairman)	(PHRI)
	K. Ohtani	(NIED)
	K. Shioji	(PWRI)

MISSION STATEMENT

1. Objective:

The objective of the Task Committee is to coordinate the provision of strong-motion earthquake data to researchers and to the practicing engineering community. The Task Committee focuses on instrumentation; recording, processing, and analyzing strong-motion data; research to ensure high-quality data dissemination; and analysis of ground motion and structural motion in dynamic response to earthquakes.

2. Scope of Work

The T/C coordinates strong-motion research, processing strong-motion data, and disseminating information on the recorded behavior of structures. In addition to regular exchange of data and publications, the T/C's technical approach includes:

* Planning and conducting T/C workshops and meetings, held generally in conjunction with UJNR Joint Meetings.

* Conducting related technical sessions at professional society meetings.

* Creating procedures for the dissemination of significant strong-motion digital data with regard for the rights and expectations of (a) the owner, (b) the users of data, and (c) the earthquake engineering community.

* Creating an up-to-date information exchange on computer processing procedures and related research so all users of processed data are aware of valid ranges and practical limitations.

* Coordinating research programs for appropriate instrumentation and dynamic analysis of structures with base isolation or other response-reducing systems, under earthquake excitation.

* Coordinating research programs using both permanent and temporary arrays and data derived to study site response.

3. Accomplishments

* Catalogs of strong-motion earthquake records observed both in the United States and Japan are being exchanged. U.S. data are published in the "Seismic Engineering Program Report", and the Japanese data are published in "Strong-Motion Records in Japan". Digital data is exchanged on tape, floppies, and CDROMs. Many other publications also were exchanged.

* Preliminary Reports of the Southern and Northern California earthquakes, from both the U.S. Geological Survey and California Division of Mines and Geology, were exchanged.

* Prompt report of the Tokyo Bay Earthquake, February 2, 1992, from the Strong-Motion Earthquake Observation Council, NIED, was exchanged.

* From the USGS, a CDROM containing "Digitized Strong-Motion Accelerograms of North and Central American Earthquakes 1933-1986", May, 1992, by Linda Seekins was exchanged, together with a software package for PC processing and a User Manual: "Basic Strong-Motion Accelerogram Processing Software";Version 1.0, May 1992, by April M. Converse.

* From the National Geophysical Data Center, NOAA, disks containing the "Strong-Motion Data Catalog", and software (all records in the global public domain through 1992) were exchanged. The User Manual is "An Earthquake Strong-Motion Data Catalog for PC's, SMCAT" Version 2.0, July 1990, by Lee Wesley Row III).

4. Future Plans

* Coordinate the analysis of records from arrays: downhole 3-D arrays (geophysics and earthquake engineering) and structural arrays (structural engineering).

* The Science and Technology Agency, and the National Science Foundation will be asked to sponsor a joint workshop in Hawaii, in the fall of 1993, on Digital Recording Technologies, for approximately 20

invited participants, including Japanese and US researchers. The workshop will be for 2 days, followed by a technical site visit.

5. Information Exchange

* Performed within the domestic community through technical meetings, conferences, seminars, and workshops.

* Performed through participation in post-earthquake field investigations with concentration in areas where strong-motion stations have recorded data.

* Performed with the counterpart side through T/C workshops; visitor exchanges, seminars, lectures; and participation in annual Joint Panel meetings.

* The most crucial information exchange is rapid description of recovered strong motion records (and subsequently, digital strong-motion data on tape or disk) from significant earthquakes.

6. Impact

* Most managers who process strong-motion recordings use techniques based, or closely associated with, the work of this T/C and its members. Foreign networks fitting this category include:

AUSTRALIA - Australian Seismological Center, Bureau of Mineral Resources, PO Box 378, Canberra ACT 2601.

CANADA - Pacific Geoscience Center, PO Box 6000, Sidney, BC. V8L 4B2.

ITALY - The Italian National Commission for Research and Development on Alternative Energy Sources (ENEA), Rome.

MEXICO - Japan-Mexico Earthquake and Disaster Prevention Center, Mexico City.

PERU - Japan-Peru Earthquake and Disaster Prevention Center, Lima.

TAIWAN - SMART Array

TURKEY - University of Bogazici, 80815 Bebek, Istanbul.

YUGOSLAVIA - Institute of Earthquake Engineering and Engineering Seismology, Skopje.

* Use of seismological and strong-motion data, in convenient form, is facilitated in seismically active countries having no such data of their own.

* Dissemination of digital data from significant earthquakes, taking into account the conditions of the 3rd paragraph in the Scope of Work, increases the research data base; application of this data in design practice aids in reduction of earthquake hazards.

7. Barriers

* Lack of adequate funding for T/C joint endeavors.

* Establishing joint research efforts and maintaining close interaction between participants are difficult unless visiting research appointments are established. (Funding and staff availability are often barriers).

（B）大型実験計画に関する作業部会報告

開　催　日　：1992年5月20日
場　　所　：国立標準技術研究所　ゲイザースバーグ市
出　席　者　：日本側　大谷　圭一　（作業部会長）　（防災科学技術研究所）
　　山内　泰之　（建築研究所）
　　山口　嘉一　（土木研究所）
　　米国側　H. S. Lew　（作業部会長）　（国立標準技術研究所）
　　R. E. Ropper　（陸軍工兵隊）

［活動要約］

1．目的

本作業部会の目的は原型構造物の実験室での実験と建設現場での実験を通して、より耐風・耐震性能の高い構造物を建設するために、構造物の動的特性の性能資料を開発することである。大型模型の実験データは小型模型の実験結果を確認し、構造物の挙動解析の結果を実証するのに必要とされる。

2．活動の範囲

作業部会では建物及び橋梁を除くその他の構造物の実大規模の構造性能の評価のために作業部会（C）、（D）と協力して、研究計画の項目を開発する。

＊　共同実験計画を計画し、ワークショップ、個々の研究課題ごとの合同会議を開催し、共同研究計画を推進する。
＊　日米双方の各研究機関で実施される研究計画を調整する。研究成果の出版及び基準類への成果の適用を強化する。
＊　研究者、技術情報、利用可能な実験施設の交換を強化する。

3．成果

＊　第2回プレキャスト耐震構造システム（PRESSS）米国技術調整委員会がカリフォルニア州サンディエゴにて1991年6月に開催された。
＊　日本のPRESSS研究計画が建築研究所，東京大学，京都大学，横浜国立大学を含む多くの研究機関によって継続されている。過去1年間に数回の日本技術調

整委員会が開催された。

＊ 第２回ＰＲＥＳＳＳ日米合同調整委員会が茨城県つくば市で１９９１年１１月１８、１９、２０日に開催された。

＊ ハイブリッド構造に関する日本計画グループが１９９１年５月から５回の会合を持った。

＊ ハイブリッド構造に関する第１回米国計画会議が１９９２年２月にサンフランシスコで開催され，日本の研究者もその会議に参加した。

４．将来計画

＊ 組積造日米共同研究の第三段階（５層構面実験）の予算が米国研究者に対して国立科学財団によって決定され，実大５層組積造の実験がカリフォルニァ大学サンディエゴ校に於て１９９２年に実施される。

＊ 第３回ＰＲＥＳＳＳ研究日米合同技術調整委員会が米国に於いて１９９２年１０月に開催される予定である。米国技術調整委員会は第３回日米合同技術調整委員会の前に会合する予定である。

＊ 「国際防災の１０年」計画を支援する為に，この作業部会の賛同の基で実施された共同研究の成果を，高地震危険度を有する国に広く普及するための方策を探求する。

＊ ハイブリッド構造に関する日米合同ワークショップを１９９２年９月サンフランシスコ湾岸地域にて開催することが計画されている。

＊ 作業部会は適切な日米の機関において、次の課題について継続的な審議を促進する。

ａ）高地震地域に建設される構造物へのコンクリート、鉄筋などの高強度構造材料の応用

ｂ）建築物，橋梁以外の構造物の実大実験計画

＊ 大型水関連構造物の動的挙動が特殊であることに着目し，本作業部会はこれらの構造物，例えば，ダム，こう門，ポンピングステーション等，の地震時挙動について研究する新しい作業部会の創設についてのフィージビリティスタディを行うことを勧告する。具体的な提案は第２５回合同部会に於て作成するものとする。

５．情報交換

＊ 技術報告書及び技術書類が両国の共同研究に参加している機関の間で交換された。

６．活動の効果

＊ 鉄筋コンクリート，鉄骨構造及び組積造に関する共同研究の成果が日米両国の技

術ジャーナルに掲載された。多くの研究成果が日米の建物設計基準などの改良に利用されている。

* 共同研究計画の成果として、建設実施及び耐震工学の理解の状況が双方において改善された。共同計画の成果は両国の研究を刺激している。
* 両国の実験施設を使用した研究計画を促進した。

7．問題点

* 作業部会の活動を妨げる主要な技術的障壁は存在しない。しかしながら、建設技術には相違があり、効果的な共同研究の実施にあたっては十分な注意が必要である。
* 日米両国とも研究者の交換を支援する予算の確保が困難な状態である。両国の財政組織にこの合同部会活動から得られる技術的利益を宣伝し，研究者の交換・共同研究の推進を強化すべきである。

補足

大型水関連構造物の地震時安全性は，下流地域の人口増加が年々進行し，これらの構造物の破壊が深刻な災害をもたらす危険性をはらんでいる故に，特に重要なこととなってきている。

実験や観測の結果と解析の結果を比較することが必要である。例えば，線形及び非線形解析結果を実験，観測結果により実証することが必要である。解析，設計，建設方法の詳細にわたる情報交換を行うことにより将来の研究の成功と要求を明確にすることが出来る。

Report of Task Committee on

(B) LARGE-SCALE TESTING PROGRAM

Date: May 20, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees:

U.S. Side -	H. S. Lew (Chairman)	(NIST)
	W. E. Roper	(COE)
Japan Side -	K. Ohtani (Chairman)	(NIED)
	H. Yamanouchi	(BRI)
	Y. Yamaguchi	(PWRI)

MISSION STATEMENT

1. Objective:

The objective of the Task Committee is to develop performance data of full-scale dynamic properties of structures to better resist wind and seismic loads through both laboratory testing of prototype structures and field testing of structures in situ. Improved full-scale test data will validate the results of small-scale model tests and substantiate the results of computer analyses of structural behavior.

2. Scope of Work

The Task Committee develops its research agenda in coordination with other appropriate T/Cs, such as T/C's "C" and "D", for full-scale evaluation of buildings, and other structures except bridges.

* Plans and conducts workshops and joint meetings to identify research topics and develops joint research programs.

* Coordinates research projects carried out by various laboratories in the U.S. and Japan. Facilitates publication of research results and implementation of findings in codes and standards.

* Facilitates exchange of research personnel, technical information and available testing facilities.

3. Accomplishments

* The second U.S. Technical Coordinating Committee on Precast Seismic Structural Systems Program (PRESSS) was held in June 1991 at San Diego, California.

* The Japan PRESSS Research program is continuing at several institutions including BRI, University of Tokyo, Kyoto University, and Yokohama National University. The Japan TCC Meetings were held during the past fiscal year.

* The second U.S.-Japan Joint Technical Coordinating Committee on PRESSS was held in October 1991 at Tsukuba, Japan.

* The Hybrid structural system is being considered as the fifth topic for the joint research program following PRESSS. In preparation for the joint research, the Japanese Planning Group on hybrid structures met five times since September 1991.

* The first U.S. planning meeting on hybrid structures was held in San Francisco, February 1992. Several Japanese researchers participated in the meeting.

4. Future Plans

* The National Science Foundation (NSF) has funded the third phase(5-story structural test) of the U.S.-side research of the U.S.-Japan Coordinated Program for Masonry Building Research. The testing of a five-story full-scale masonry structure will be carried out at the University of California, San Diego during 1992.

* The third Joint Technical Coordinating Committee (JTCC) on PRESSS will be held in the U.S. in October 1992. The U.S. TCC on PRESSS will meet prior to the third JTCC.

* In support of the International Decade for Natural Disaster Reduction (IDNDR) program, techniques will be explored to disseminate findings of Joint Research Projects carried out under the auspices of T/C "B" to countries with high seismic risks.

* A joint U.S.-Japan workshop on hybrid structural systems is planned to be held in September 1992 in the San Francisco bay area.

* Information exchange of following topics between appropriate U.S. and Japan Organizations will be continued;

a. Application of high strength construction materials, such as concrete and steel, to structures in high seismic zones,

b. Testings of large-scale structures other than buildings and bridges.

* Because the dynamic response of large hydraulic structures is unique, it is recommended that a study be conducted on the feasibility of creating a new task committee on seismic behavior of these structures such as dams, navigation locks and pumping stations (see Appendix). A proposal for creating a new Task Committee will be made to the full panel at the 25th Joint Meeting.

5. Information Exchange

* Technical reports and research documentation were exchanged with participating organizations in both countries.

6. Impact

* The results of the past joint research projects on reinforced concrete, steel and masonry structures have been published in both U.S. and Japanese technical journals. A number of key findings have been incorporated into U.S. and Japanese building codes and design guidelines.

* As a result of joint research projects, the state of practice and understanding in earthquake engineering has been improved in both countries. The results from the joint program above further stimulated research in both countries.

* Facilitate joint research projects utilizing available testing facilities in both countries.

Barriers

* There are no major technical barriers for the attainment of the Task Committee mission. However, different building practices and needs require careful planning for effective joint research projects.

* Securing funding to support exchange of research personnel is difficult in both countries. Funding agencies of both countries should be informed about the technical benefits resulting from the joint panel activities and encouraged to participate in researcher exchange and joint research programs.

APPENDIX

The seismic safety issue of large hydraulic structures is important since the risk of a major disaster due to a failure of such structures increases each year with the growth of population downstream.

Experimental and observed measures need to be compared to analytical procedures. For example, the analyses for linear and nonlinear response need to be verified with experimental and observed data. An exchange of details concerning analysis, design, and construction practices can highlight the success and needs for further research.

（C）既存構造物の補修および補強に関する作業部会報告

開　催　日　：１９９２年5月１９日
場　　所　：ＮＩＳＴ，ゲイザスバーグ
出　席　者　：日本側　山内　泰介（臨時作業部会長）（建築研究所）
　　北川　良和　（　〃　）
　　朝倉　肇　（先端建設技術センター）
　　米国側　Ken Chong (T/C "C", Chair) (NSF)
　　Bob Fuller (T/C "D", Chair) (HUD)
　　Jack Scalzi (T/C "D", Incoming Chair) (NSF)
　　Howard Nickerson (T/C "C")(Naval Facilities Engineering Command)
　　Kharaiti Lal Abrol(T/C "C") (Department of Veterans Affairs)
　　Larry Hultengren (DOS)
　　John R. Hayes, Jr.(USACERL)
　　Chris Stoddart (Martin Marietta-(observer)
　　Charles E.Anderson(Bureau of Reclamation)
　　Nicholas J.Carino (Visitor,NIST)
　　Ugo Morelli (FEMA)

[活動要約]

1．目的

C作業部会の目的は既存の建築物や他の構造物の補修・補強に関連して材料、部材、全建築物および構造に関連する研究を推進し、技術移転の場を提供することにある。対象範囲としては原構法と同様に構造材料や構造部材に関する研究を含むものである。

2．活動の範囲

部会の仕事は情報交換を推進し、ワークショップやセミナーを企画し、運営することにある。ワークショップは通常、毎年の専門部会会議と同時に行う。また、ワークショップはＴ／Ｃメンバーに関連研究のレビューの機会を与えると共に、将来の研究計画の立案を

指導しその計画案を分担し、実行する機会を与えるものとする。

また、T／Cは専門機関による会議に於て関連する技術報告を行う。またT／Cでは建物の補修　補強工事を行うための新しい材料や方法－例えばグラスファイバー補強樹脂、高施工性コンクリート、あと施工アンカー、炭素繊維線、構造用接着剤など及び補修・補強の機械的自動化やロボットの開発的応用等を研究する。

作業は日米の共同研究の実施に重点を置き、重複を最小とし得られる成果を最も効果的なものとする。

3．成果

＊国立科学財団が主導している最近の関連研究として次のものが紹介された。

○大型構造システムのための定量的非破壊評価方法

この課題は１９９０年に発表された。１９９２年には第２次計画に入るが学際的な研究チーム作りが進められている。４つの研究課題が第１次の関連課題として選択された。

これらの４つのプロジェクトの詳細や当初計画についての記述は第２３回合同部会のプロシーディングスの中に記されている。

○構造物の耐震性に対する補修・補強関連研究

１９９０年の初期に、ＮＳＦは既存構造物の耐震補修や補強の分野での総合研究をその内容とした新しい５ヶ年計画を発表した。

この研究計画の第１年度である１９９０年度の間に、国立科学技術財団は９つのプロジェクトに対して予算を配分した。

上記に加えて、国立地震工学研究センター（ＮＣＥＥＲ）、地震工学研究センター（ＥＥＲＩ）、連邦危機管理庁（ＦＥＭＡ）の既存建物委員会、建築・防火研究所（ＮＩＳＴ）、カリフォルニア地震安全協会（ＣＳＳＣ）ならびに他の組織などにより行われているその他の地震補修・補強計画との間の協力と推進のための努力が続けられている。１９９１年度の第１会組織委員会が１９９１年２月１３日にSalt Lake Cityで開催された。

＊コンクリート造と組積造の補強、計画および構造補修に関する３回のＡＳＣＥ１日セミナーが行われた。これらは１９９０年３月にカリフォルニアオークランドで、次は１９９０年４月にボストンで、また１９９０年５月にワシントンで行われた。

＊土木構造物と材料の非破壊評価（ＮＤＥ）に関するワークショップが１９９０年１０月にコロラド、ボールダー市で行われた。４０６頁のハードカバーのプロシーディングスがコロラド大学の土木工学部から出せれており、利用可能となっている。会議には大学や産業界の工学関係者が多数参加した。

＊非破壊評価に関するフォロー・アップ・ワークショップが１９９２年５月にコロラド、ボールダー市で開催された。

＊１９９０年５月にＮＩＳＴで開催された構造物の評価および補修・補強に関するワークショップのプロシーディングス（NISTIR 4515）が発刊され、つくばで開催された第２３回ＵＪＮＲ合同部会の時に配布された。プロシーディングスのコピーは著者、ワークショップの参加者および米国内のその他の関係者に配布された。この会議はＮＳＦの支援により開催されたもので２２の論文が報告されたが、その内の１７編は構造物の補修・補強に関するものであった。２４人の参加者があり、１２人は米国から、１２人は日本からであった。

＊米国ではＦＥＭＡがＡＴＣ２１既存建物の耐震診断ハンドブックとＡＴＣ２２、既存建物の耐震改修技術を発刊した。米国エネルギー庁と米国郵便サービスは関連する補修・補強工事を展開している。

＊“耐震安全性のための補強”に関するレポートが国立アカデミー出版から１９９１年２月に刊行された。

＊日本では既存建築物の耐震診断・耐震改修設計指針が改訂され、被災建築物の被災度判定を復旧に関する指針が発刊された。両指針は１９９１年８月に出版され、現在実施されつつある。

＊いくつかの補修・補強プロジェクトが米国工兵隊の建設工学研究所（ＣＥＲＬ）で進行している。すなわち、

・靭性の少ないＲＣ骨組の補修・補強、柱はり部分架構及びかりに取付けた粘障性ダンパーを用いてグレードアップしたものと元のままの縮尺骨組の振動台実験。骨組筋
・ＲＣ骨組の二方向応答、組積造壁をはめ込んだＲＣ骨組の面内対面外の振動台実験、震後の補修を施した被災耐震壁の再実験のための将来的な実験。
・１９９３年に計画されている、プレキヤスト・コンクリートパネルの接合詳細部の補強研究

4．将来計画

＊ＮＳＦによる１９９２年度における第２次計画は次のようなものである。

・大型構造システムのための定量的非破壊検査方法
・構造物の耐震性に関する補修・補強計画

・ＮＳＦ／ＳＨＲＰによるストラクチュア・コントロール研究

・新高分子材料（ＮＳＦ／ＥＰＲＩ）

＊公共インフラストラクチュア・システム研究計画策定のためのフイージビリティ・スタディがＮＳＦのもとで検討されている。

＊２１世紀へ向けてのインフラストラクチュア研究のためのＮＡＥ／ＮＲＣの援助

＊ＮＳＦ，ＡＩＳＩ及びＡＩＳＣの新たな協力による建設分野での革新的鋼構造研究

＊既存構造物の補修・補強は全体的な変更（ＭＡＲＳ計画）により連邦の主導のもとに継続的に研究されている

＊ＮＳＦは米国の中央および東部地方の既存構造物や建物に焦点をあて、新たな展開を進めている。

＊ＩＤＮＤＲの活動を支援するために、Ｃ作業部会は種々の災害による災害の補修・補強に関連した委員会活動に参加することを検討する。

＊Ｃ作業部会は、Ｇ作業部会と共に“高機能、高性能材料およびシステム”に関する日米ワークショップをサポートすることを検討する。このワークショップは１９９３年の第２５回ＵＪＮＲ専門部会を連ねて開催される可能性がある。

＊既存構造物の評価を助けるための実験、補強法の定量化及び構造物の現況の特性化の役割を慎重に検討すべきであり、多分これは将来のワークショップの課題となろう。

＊日本では、建築研究所が、要求性能に基づく新たな構造設計手法に関する新しい総合的研究計画を提案することを検討している。

＊日本では、ＩＩＳＥＥの３０周年を記念した地震災害軽減技術に関するＩＤＮＤＲ国際シンポジウムで被災建物の診断、補修、補強に関する技術の現況を紹介・討議する予定である。

５．情報交換

＊ワークショップのレポートや個人的な研究成果のレポートが米国側および日本側の委員を通して交換された。

＊第２４回ＵＪＮＲ合同部会で報告された多数の論文はＣ作業部会の関心をひくものであった。

６．活動の効果

＊鉄筋コンクリート造、鉄骨造その他の構造による建物の補強に関する日米両国の設計クライテリアを推進するのに役立てられた。

＊作業部会の勧告は、例えば、地震力に対する既存組積造建物の性能をグレードアップするためにロスアンジェルス市によって作成された再建計画に示されているように建物規準や専門手引書の中に組み込まれた。

＊既存の構造物の性能をグレードアップするにあたり、日米両国の実施設計に影響を及ぼした。

7．問題点

＊利用可能な研究成果を得るためにより大きな進展が必要とされる。作業部会の役割を支援するために、また、合同部会のワークショップに参加することを支援するために、付加的な予算が必要である。

Report of Task Committee on

(C) REPAIR AND RETROFIT OF EXISTING STRUCTURES

Date: May 19, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees: U.S. Side: Ken Chong (T/C "C", Chair) (NSF)
Bob Fuller (T/C "D", Chair) (HUD)
Jack Scalzi (T/C "D", Incoming Chair) (NSF)
Howard Nickerson (T/C "C") (Naval Facilities Engineering Command)
Kharaiti Lal Abrol (T/C "C") (Department of Veterans Affairs)
Larry Hultengren (DOS)
John R. Hayes, Jr. (USACERL)
Chris Stoddart (Martin Marietta/DOE) (Observer)
Charles E. Anderson (Bureau of Reclamation)
Nicholas J. Carino (Visitor, NIST)
Ugo Morelli (FEMA)

Japan Side: Hajime Asakura (Temporary member) (ACTEC)
Hiroyuki Yamanouchi (Temporary Chairman "C&D") (BRI)
Yoshikazu Kitagawa (BRI)

MISSION STATEMENT

1. Objective:

The objectives of the Task Committee are to encourage research and provide technology transfer activities in materials, components, and whole buildings and structural systems related to the repair and retrofit of existing buildings and other structures. The scope will encompass studies related to materials of construction, structural components as well as prototype structural systems.

2. Scope-of-Work

The T/C work includes information exchanges, planning, and hosting workshops and seminars. Workshops are generally held in conjunction with the annual Panel's meeting. They provide T/C members with an opportunity to review related research and to direct plans for the future, share and implement results. The T/C conducts related technical sessions at professional society's meetings. The T/C studies new materials and methods to accomplish repair and retrofit operations on buildings; e.g., fiberglass reinforced plastics, high performance concrete, post-installed anchor bolts, carbon-fibre wire, structural adhesives, and exploits use of automation and robotics

for repair and retrofit. Work focuses on coordinating research projects in U.S. and Japan to minimize duplication and to maximize benefits.

3. Accomplishments

*Status reports were delivered on recent related NSF initiatives:

Quantitative Nondestructive Evaluation for Large Structural Systems. This initiative was issued in June 1990. Plans are underway for the second round in 1992. Interdisciplinary research teams are encouraged. Four projects were selected for support in the first round. Details of these projects and a description of the initiative are included in the proceedings of the 23rd UJNR Joint Meeting in Tsukuba, Japan.

Repair and Rehabilitation Research for The Seismic Resistance of Structures. In early 1990, NSF announced a new 5-year research program that focuses on a coordinated research effort in the seismic repair and rehabilitation of existing structures. During the first year of the research program, FY 1990, NSF funded a total of nine projects.

In addition, cooperation and coordination efforts were initiated with other on-going seismic repair and rehabilitation programs, conducted by the National Center for Earthquake Engineering Research (NCEER), the Earthquake Engineering Research Institute (EERI) through its Existing Building Committee, the Federal Emergency Management Agency (FEMA), the Building and Fire Research Laboratory (NIST), The California Seismic Safety Commission (CSSC), and other organizations. The first coordination meeting of FY 1991 was held in Salt Lake City on February 13, 1991.

* 3 day ASCE seminars on Rehabilitation, Planning and Structural Repair of Concrete and Masonry were held in, Oakland, CA (3/90); Boston (4/90); and Washington D.C. (5/90).

* A workshop on Nondestructive Evaluation, (NDE) of Civil Structures and Materials was held in October 1990, in Boulder, Colorado. Participants came from many disciplines of engineering in academics and industries. A hardcover proceedings (406 pages) is available from the University of Colorado, Department of Civil Engineering.

* A follow-up workshop on NDE was held in May 1992, in Boulder, Colorado.

* Proceeding of the Workshop on Evaluation, Repair, and Retrofit of Structures, (NISTIR 4515), held at NIST in May 1990, was published by NIST and distributed at the 23rd UJNR Joint Meeting in Tsukuba, Japan. Copies of the proceedings were also distributed to authors, workshop participants, and other researchers and practitioners in the U.S. Twenty-two papers were presented

at the NSF sponsored workshop, of which 17 were on repair and retrofit of structures. Delegates totaled 24, 12 were from the U.S. and 12 from Japan.

* In the U.S., FEMA published ATC 21, A Handbook for Seismic Evaluation of Existing Buildings, and ATC 22 Techniques for Seismically Rehabilitating Existing Buildings. The U.S. Department of Energy and U.S. Postal Service are involved in extensive repair and retrofit projects.

* A report on "Retrofitting Buildings for Seismic Safety" was published by the National Academy of Press in 2/91.

* In Japan, the Guidelines for Evaluating and Strengthening Existing Buildings have been revised, and the Guidelines for Evaluation of Earthquake Damage and Repair Methods of Buildings was published. Both guidelines were available in August 1990, and are being implemented.

* Several on-going repair/retrofit projects at the US Army Construction Engineering Research Laboratory (CERL) include:

  o Repair/Retrofit of Non-Ductile Reinforced Concrete Frames. Shake table tests of beam-column subassemblages and scaled frames, both as-built and as upgraded using viscoelastic dampers in frame bracing.

  o Biaxial Response of Reinforced Concrete Frames, shake table tests of in-plane vs out-of-plane response of reinforced concrete frames with masonry in-fills. Future research to retest damaged walls after post-earthquake repair.

  o Strengthening of Precast Concrete Panel Connection Details Planned FY93 new start project.

4. Future Plans

* A second round of related NSF initiatives are planned for FY 92:

  - Quantitative Nondestructive Evaluation for Constructed Facilities.

  - Repair and Rehabilitation Research for the Seismic Resistance of Structures.

  - NSF/SHRP Initiative on Structural Control

  - Advanced Polymeric Materials (NSF/EPRI)

* The feasibility of establishing a Civil Infrastructure Systems Research Program is being studied at NSF.

* Supports NAE/NRC on a study of infrastructure research for 21st century.

* New initiative on innovative steel research for construction, NSF with AISI and AISC.

* Repair and retrofit of existing structures are being studied in association with a possible federal initiative on adaptation due to the global changes (MARS) program.

* New initiatives are being developed by NSF to focus on central and eastern U.S. existing structures/buildings.

* In support of IDNDR activities, T/C "C" plans to participate in task committee activities related to repair and retrofit of damages due to various disasters.

* The T/C "C" will consider joint support of a proposed U.S.-Japan workshop on "Smart and High-performance Material/Systems", by T/C "G". The workshop will possibly be conducted in 1993 in conjunction with the 25th Joint UJNR panel meeting in Tsukuba, Japan.

* A possible session on wind damages will be planned in the future in the U.S.

* The role of testing in support of evaluation of existing structures, characterization of the current conditions and qualification of retrofit approaches should be carefully examined and might be a possible topic for a future workshop.

* In Japan, BRI is considering to propose a new comprehensive research program on new structural design methodology based on the desired structural performance.

* In Japan, state-of-the-art technology for diagnosis, repair and retrofitting of damaged structures will be introduced and discussed at the IDNDR International Symposium on Earthquake Disaster Reduction Technology in commemoration of the 30th Anniversary of IISEE.

5. Information Exchange

* Published workshop reports and individual research papers; they were exchanged with the U.S. and JAPAN-side members.

* Numerous papers presented at the 24th UJNR Joint Panel Meeting are of interest to T/C "C".

6. Impact

* Contributed to development of U.S. and Japan design criteria for repairing and strengthening reinforced concrete, steel, and other buildings.

* T/C recommendations were incorporated into building codes and professional practices, as illustrated by rehabilitation programs used by the City of Los Angeles to upgrade existing masonry buildings to resist earthquake forces.

* Influenced changes to U.S. and Japan design practices to upgrade existing structures.

7. Barriers

* Greater progress needs to be made to implement available research results. Additional funding sources need to be identified to support technical activities to support committee missions and to support participant attendance at Joint Panel Meeting workshops.

（D）構造物の性能評価に関する作業部会報告

開　催　日：1992年５月19日（作業部会Ｃとの合同打ち合せ）

場　　所：国立標準技術研究所　ゲイザーズバーグ市

出　席　者：日本側　朝倉　　肇（臨時委員）　（先端建設技術センター）

山内　泰之（作業部会Ｃ及びＤ臨時作業部会長）（建築研究所）

北川　良和　（　〃　）

米国側　K.Chong　（作業部会Ｃ部会長）　（ＮＳＦ）

B.Fuller　（作業部会Ｄ部会長）　（ＨＵＤ）

J.Scalzi　（作業部会Ｄ次期部会長）　（ＮＳＦ）

H.Nickerson（作業部会Ｃ）（Naval Facilities Engineering）

K.Abrol　（作業部会Ｃ）　（Department of Veterans Affairs）

L.Hultengren　（ＤＯＳ）

J.R.Hayes,Jr.　（ＵＳＡＣＥＲＡＬ）

C.Stoddart　（Bureau of Reclamation）

N.J.Carino　（Visitor）

U.Morelli　（ＦＥＭＡ）

［活動要約］

1．目的

本作業部会の目的は、風及び地震力に抵抗する既存構造物の性能を改善するための被害度軽減政策及び、プログラムを開発することである。適切な性能評価をするために、両国は共同して、現状診断、スクリーニング及び構造解析手法を開発すべきである。そして、構造物が解析され、計測器の設置がされ被災後の評価がされるべきである。

2．活動の範囲

※風及び地震時挙動を計測するために、解析されかつ観測機器が設置されたベンチマーク構造の詳細目録を作成する。

※スクリーニングの統一的手法ならびに両国構造物の風及び地震に対する耐力の解析法の共同開発。

※既存構造物の現状診断するためのセンサー技術、機器による計測及びエキスパートシステムの開発。

※組積造建築物、プレキャストコンクリート造建築物及び制震システム建築物の評価をして、被災後の性能評価を行うために両国のベンチマーク構造物のカタログにこれらの建築物を含めること。

※作業部会Ｃと協力してＵＪＮＲの合同部会開催時にワークショップを開催する。

3．成果

※EnglekirkおよびHartによるTCCMARレポートが1992年２月に出版された。日本側にコピーが提供された。

※最近の米国の法令について第24会合同部会においてステイタスレポートが発表された。1990年米国余暇住宅条例（Affordable Housing）の耐震安全性に関する第947項、米国地震危険度低減プログラムの再制定条例：法律101－614、1990年11月16日付における耐震基準に関する第８項

※国際防災の10年（ＩＤＮＤＲ）の耐風、耐震関連の両国の活動について情報を交換した。「危険度の検証、自然災害（低減のための国家戦略）」と題するレポートが科学技術に対する米国連邦調整会議の下の自然災害低減に関する小委員会でまとめられ、1992年６月に出版された。その研究及び実行計画には、危険度評価及び低減プログラムが含まれている。コピーが日本側メンバーに送付されよう。

※既存構造物の地震危険度評価を含め、いくつかのFEMA-ATCプロジェクトの現状が討議された。FEMAは今後数か年にわたる損失評価研究プロジェクトを計画している。

4．将来計画

※建物目録のための、報告システム及びデータ収録様式を開発し、両国間の情報交換の為にコンピューターデーターファイルの収録を推進させる。各合同部会時に報告する。

※既存構造物の計測、評価、現状診断に関係する民間企業、設計業界、大学、資金協力機関及び他の政府機関の参加の奨励の継続。

※構造物の現状診断をするため、また、建築部材に及ぼす経年劣化の影響を知るために、両国においてモニター用計測技術の開発を行うこと。

※構造性能評価に関連して、国際防災の10年(IDNDR) に関する合同部会の活動に協力して参加すること。

※1993年に日本で構造性能評価手法に関するワークショップを開催することが検討された。このワークショップでは非破壊評価(NDE) に力点が置かれよう。

5．情報交換

※技術資料が交換された。また、関係の日本側ペーパーが第２４回合同部会に持たらされた。

※日本側はTCCMAR-Japanによってまとめられた「中層補強組積造の設計ガイドライン」のコピーを米側議長に送付する。米側はこれを英訳するのにつき可能性を検討する。

6．活動の効果

※強風及び大地震による破壊を防止するための建物に関する標準及び法規の見直しを進展させる。

※経済性の高い補修・補強、あるいは解体の技術を発展させるために既存構造物の現状診断実施能力を備えた技術者を育成する。

※主要な構造物が強風及び地震による力に適切に耐えることが確かめられていることは、公衆、行政官、政策立案者、技術者の建物の性能に対する信頼性のレベルを増進する。

7．問題点

※発生する可能性の高い強風及び大地震に対する既存建築物の耐力性能を登録するに当たっての予算不足。

※竜巻、台風、ハリケーンに対する危険度マップの開発の困難性。

※建築物への計測装備及び被災構造物の残存性能評価の実施及び諸活動の調整に必要な外国出張旅費に係わる予算不足。

Report of Task Committee on

(D) EVALUATION OF STRUCTURAL PERFORMANCE

Date: May 19, 1992 (Joint Meeting with T/C "C")

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees: U.S. Side: Ken Chong (T/C "C", Chair) (NSF)
Bob Fuller (T/C "D", Chair) (HUD)
Jack Scalzi (T/C "D", Incoming Chair) (NSF)
Howard Nickerson (T/C "C") (Naval Facilities Engineering Command)
Kharaiti Lal Abrol (T/C "C") (Department of Veterans Affairs)
Larry Hultengren (DOS)
John R. Hayes, Jr. (USACERL)
Chris Stoddart (Martin Marietta/DOE) (Observer)
Charles E. Anderson (Bureau of Reclamation)
Nicholas J. Carino (Visitor, NIST)
Ugo Morelli (FEMA)

Japan Side: Hajime Asakura (Temporary member) (ACTEC)
Hiroyuki Yamanouchi (Temporary Chairman "C&D")(BRI)
Yoshikazu Kitagawa (BRI)

MISSION STATEMENT

1. Objective:

The objectives of this Task Committee are to develop disaster mitigation policies and programs to improve the capacity of existing structures to resist wind and seismic forces. To provide adequate performance evaluation, each country will coordinate development of condition assessment, screening, and structural analysis methodologies. Structures will be analyzed and instrumented, and then evaluated after disasters.

2. Scope of Work

* Develop an inventory of "benchmark" structures that have been analyzed and instrumented to measure wind and seismic responses.

* Develop a uniform system for screening and analyzing wind and seismic resistance capacity of structures in each country.

* Develop sensor technology, instrumentation, and "expert" systems to provide condition assessment of existing structures.

* Evaluate masonry and precast concrete buildings, and seismic base isolation systems; include results in each country's catalog of "benchmark" structures for post-disaster performance evaluation.

* Conduct workshops at future UJNR Joint Panel Meetings, cooperatively with T/C "C".

3. Accomplishments

* TCCMAR Reports by Englekirk and Hart were published in Feb. 1992. Copies were furnished to the Japanese side.

* Further status reports on recent U.S. legislation were presented at the 24th Joint Panel Meeting: 1990 National Affordable Housing Act, Sec. 947 on Seismic Safety Standards. National Earthquake Hazards Reduction Program Reauthorization Act; Public Law 101-614, Nov. 16, 1990. Sec. 8 on Seismic Standards.

* Information was exchanged on the activities of each country in the International Decade on Natural Disaster Reduction (IDNDR), pertaining to winds and earthquakes. A report entitled: Hazard-Proofing the Nation, Strategy to Reduce Natural Disasters was developed by the Subcommittee on Natural Disaster Reduction, a part of the U.S. Federal Coordinating Council for Science and Technology, and printed in June 1992. Included in the research and implementation strategies are risk assessment and mitigation programs. Copies will be mailed to the Japanese members.

* Status reports of several FEMA - ATC projects were discussed, including those on seismic risk assessment of existing structures. FEMA is planning a Loss Assessment Research Project to be conducted over the next several years.

4. Future Plans

* Develop a reporting system and data collection form for building inventory and performance standard, and compile a computer data file for exchange between countries. Report at each Joint Panel Meeting.

* Continue to encourage participation by private industry, consulting engineering firms, universities, funding agencies and State and local government agencies involved in instrumentation, evaluation, and condition assessment of existing buildings.

* Monitor development of sensor technology and instrumentation of structures for condition assessment, and to measure long-term aging effects on building components.

* Participate in coordination of Joint Panel activities in the International Decade for Natural Disaster Reduction (IDNDR), pertaining to structural performance evaluation.

* A workshop on Structural Performance Evaluation was discussed to be held in Japan 1993. The workshop will place emphasis on Non-Destructive Evaluation (NDE).

5. Information Exchange

* Technical documents exchanged, and pertinent Japanese papers delivered at the 24th Joint Panel Meeting.

* Japanese side will send to the U.S. T/C Chairman a copy of the "Design Guidelines for Medium Rise Reinforced Masonry Buildings" developed by TCCMAR-Japan. U.S. Side will explore feasibility of translating this manuscript into English.

6. Impact

* Enables the development of revised standards and building code provisions to prevent destruction from high winds and earthquakes.

* Provides engineers with the capability to conduct condition assessment of existing structures for the purpose of developing cost effective repair, retrofit, strengthening or demolition programs.

* Will increase the confidence level of the public, regulators, policy makers and engineers, to be assured that the majority of major structures will perform adequately to resist high wind and earthquake forces.

7. Barriers

* Insufficient resources to document load-carrying capacity of existing buildings subjected to probable extreme winds and earthquakes.

* Difficulty in developing probability risk maps for tornadoes, typhoons, and hurricanes.

* Limited resources for instrumenting buildings, for conducting post-disaster evaluations, and for participating in foreign travel for coordination of activities.

（E）地震動及び設計地震力に関する作業部会報告

開　催　日：１９９２年５月１９日
場　　所：国立標準技術研究所、ゲイザーズバーグ市
出　席　者：日本側　川島一彦（作業部会長）　(PWRI)
　　大谷圭一　(NIED)

　　米国側　Walter Hays（臨時作業部会長）　(USGS)
　　Jim Hill　(DOE)
　　Jeff Kimball(オブザーバー)　(USGS)
　　Mike Mahoney(オブザーバー)　(USGS)
　　Mehmet Celebi　(USGS)

［活動要約］

1．目的

本作業部会の目的は、以下の５項目について技術開発を図るとともに、情報及び経験を交換するものである。

１）地震動の工学的な特性の性格付け
２）地震動データを表示するためのマッピング技術
３）マイクロゾーネーション
４）設計への応用
５）地震危険度評価

2．活動の範囲

共同研究、作業部会ワークショップ及び集会を計画・実施する。これらのワークショップ及び集会は一般に耐風・耐震構造専門部会の合同部会にあわせて開催する。作業部会の活動には次の項目を含む。

－　共同研究を実施する。
－　研究者の交流を図る。
－　時機を得た情報を交換する。

－　ＩＤＮＤＲの目標に向けた協力

3．成果

「第１回　地震危険度・ハザード及び設計地震力」に関するワークショップを１９９１年５月２８～２９日に土木研究所で開催した。ワークショップでは、米側から４編、日本側から１０編の論文発表が行われた。また、スタディーツアーとして、大規模橋梁の建設現場を訪問し、設計地震力の取扱いに関して現場打ち合せを行った。

4．将来計画

サブダクション地震に重点をおいて、第２回地震危険度・ハザード及び設計地震力に関する日米ワークショップの開催を計画する。本ワークショップは、１９９３年５月に日本で開催する。テーマとしては、１）データ解析とモデル化、２）地震動と地震時地盤ひずみの工学的特性、３）設計への応用、の３つを取り上げる。トピックスとしては、マグニチュードと距離による地震動の距離減衰、岩盤と地盤に対する線形、非線形及びエネルギー応答スペクトル、地震動の予測、確率的な表示、不確定性、構造物のタイプと重要性、地盤と構造物の動的相互作用、シナリオ地震、損害の評価、ゾーニング・マップ、設計と施工、を考慮する。

5．情報交換

両国の関連機関に出席することにより技術報告及び研究資料が交換された。

6．活動の効果

作業部会の努力は、入力地震動に関する耐震設計基準に反映される。

7．問題点

ワークショップ及び関連する集会を開催するための海外出張に要する費用の不足

Report of Task Committee on

(E) GROUND MOTION AND SEISMIC DESIGN FORCE

Date: May 19, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees:	U.S. Side -	Walter Hays, (Acting Chairman)	(USGS)
		Jim Hill	(DOE)
		Jeff Kimball	(DOE)
		Mike Mahoney	(FEMA)
		Mehmet Celebi	(USGS)
	Japan Side -	Kazuhiko Kawashima (Chairman)	(PWRI)
		Keiichi Ohtani	(NIED)

MISSION STATEMENT

1. Objective:

The objectives of this Task Committee are to develop techniques and exchange information and experiences on: 1) engineering characterization of ground motion, 2) mapping techniques for displaying ground motion data, 3) microzonation, 4) siting and design applications, and 5) earthquake hazard risk assessment.

2. Scope of Work

Plan and conduct cooperative research, T/C workshops and meetings, These meetings and workshops are generally held in conjunction with the annual Panel Meeting on Wind and Seismic Effects. Activities include:

- Conduct cooperative research programs

- Exchange research personnel

- Exchange information on a timely basis

- Cooperate in contributing to the goals of the IDNDR

3. Accomplishments

The First U.S.-Japan Workshop on Hazard/Risk Assessment and Design Earthquake Loading was sponsored by the Task Committee E, and was held on May 28-29, 1991

at the Public Works Research Institute, Tsukuba, Japan. Four presentations from U.S. -side and ten presentations from Japan-side were made. During the technical site visits, tours to the sites of large bridge constructions were made and technical discussions on seismic design loading were conducted.

4. Future Plan

Plan the 2nd Workshop on "Ground Motion and Seismic Design Force" with emphasis on subduction zone earthquakes. This Workshop will be held in Japan in May 1993. Three themes will be developed: 1)Data analysis and modeling, 2) Engineering characterization of ground motion and ground strain, and 3) Siting and design applications. Topics will include: attenuation of ground motion with magnitude and distance; linear, nonlinear, and energy response spectra for rock and soil; prediction of strong motion; probabilistic representation; uncertainty; types and importance of structures; soil structure interaction; scenario earthquakes; loss estimation; and zoning maps for siting, design and construction.

5. Information Exchange

Technical reports and research documentation were exchanged with participating organizations in both countries.

6. Impact

Task committee efforts influence the national seismic codes concerning input seismic forces.

7. Barriers

Lack of funding for travel to conduct workshops and related meetings has been the principal barrier in the past. To increase benefits to funding agencies of each country, a study tour is being planned.

（F）ライフライン施設の災害防止法に関する作業部会報告

開　催　日：１９９２年５月２０日

場　　所：国立標準技術研究所　ゲイザースバーグ市

出　席　者：日本側　川島　一彦　（ＰＷＲＩ）

中野　修　（ＨＤＢ）

浅倉　肇　（ＡＣＴＣ）

米国側　R．Ｄｉｋｋｅｒｓ　（ＮＩＳＴ）

H．Ｂｏｓｈ　（ＦＨＷＡ）

J．Ｃｏｏｐｅｒ　（ＦＨＷＡ）

R．Ｃｈｕｎｇ　（ＮＩＳＴ）

J．Ｈｉｌｌ　（D.OE）

P．Ｈａｒｂｅｎ　（ＬＬＮＬ－ＤＯＥ）

［活動要約］

１．目的

作業部会（Ｆ）は、１）地震時におけるライフライン施設の挙動、２）被害予測技術や被害調査法のような技術ならびにその他の地震対策、の２点に焦点をあてている。

これらの技術は、ライフライン施設の安全とサービス性の確保のために既存の基準等の改善をもたらすと考えられる。ガス、石油、水道、下水道の各パイプライン、電力、通信、交通施設等のライフラインは、都市や地域社会の生存と健康に極めて重要なものである。これらの施設に影響を与えるような風および地震の影響は、地域社会に大きな社会的、経済的混乱と人的被害をもたらすものである。

２．活動の範囲

・作業部会主催のワークショップと会合の計画および実施：技術的な知見と実施の現状について情報交換するとともに、研究成果を有効に実施に活かすための情報交換ならびに研究に重要な機会を共同して設定する。

・専門部会の会合において関連の技術のセッションを運営する。

・特定のライフライン施設に関する研究計画を作る（大型実験計画に類似した研究計画）

。例えば、合同デモンストレーション等により、ライフライン管理のための緊急評価とオンライン監視システムのための基準を作る。

・建設省土木研究所が作成した「土木構造物の震災復旧技術マニュアル（案）」を翻訳、印刷、配布したと同様な特別な活動を行う。

・ライフライン施設の復旧および補強に対する性能基準およびマニュアルを開発する。

3．成果

・ライフライン施設の地震災害の防止に関する第4回ワークショップを１９９１年8月１９日～２１日に米国ロスアンジェルスにおいて開催した。

・第２４回合同部会においてライフライン施設に関する論文の報告に寄与した。

・ＩＤＮＤＲの目標達成のための積極的参加を奨励する。

・地下構造物の耐震設計法マニュアルを開発した（日本側）

4．将来計画

・ライフライン施設の地震災害の防止に関する第5回ワークショップを１９９２年１０月２６日、２７日本国つくば市で開催する。ワークショップの後にスタディーツアーを行う。

・日米両国においてライフライン施設の耐震設計基準やガイドラインを作成するため努力を奨励する。基準作成に関連した情報の交換を図るためにＵＪＮＲチャンネルを十分に活用する。ライフライン施設の耐震基準やガイドラインの作成を共同で行うための可能な協力を図る。

・米国のカリフォルニア州バークフィールドおよび日本の千葉市における埋設管の計器観測を継続し、データを交換する。

5．情報交換

・国内においては、技術打ち合せ、会議のセッション、セミナーおよびワークショップを通じて。

・日米間においては、作業部会主催のワークショップおよび合同部会を通じて。

・ＮＣＥＥＲ、ＥＥＲＩ及びＡＳＣＥなどが主催する各種の集会への研究者の参加と視察。

・進行中のライフライン関係の研究に参加する研究者の短期（３～１２ヵ月）交換。

6．活動の効果

・建設省の震災構造物の復旧技術マニュアル（案）の講習会が東京、大阪を含む日本の9都市で開催され、作業部会のメンバーが講義した。約2千人の実務技術者が参加し

たが、このことにより技術の普及に貢献した。

・作業部会のメンバーが委員会（ＴＣＬＥＥ／ＡＳＣＥ）での作業に参加することにより、ライフライン施設の設計・施工および運用基準の評価に貢献した。

7．問題点

・作業部会の年次ワークショップを開催し、報告書を発行するための予算が不十分であること。

Report of Task Committee on

(F) DISASTER PREVENTION METHODS FOR LIFELINE SYSTEMS

Date: May 20, 1992

Place: National Institute of Standards and Technology
Gaithersburg, Maryland

Attendees:	U.S. Side -	R. Dikkers (Chairman)	(NIST)
		H. Bosh	(FHWA)
		J. Cooper	(FHWA)
		R. Chung	(NIST)
		J. Hill	(DOE)
		P. Harben	(LLNL-DOE)
	Japan Side -	K. Kawashima (Acting Chairman)	(PWRI)
		O. Nakano	(HDB)
		H. Asakura	(ACTC)

MISSION STATEMENT

1. Objective

The objectives of Task Committee "F" are to improve: 1) the behavior of lifeline systems during earthquakes and 2) engineering and other seismic countermeasures such as damage estimation techniques and inspection procedures. These technologies provide users with improvements to existing standards of practice for safety and serviceability of lifeline systems. Lifeline systems such as gas, oil, water, and sewage pipelines; and power, communication, and transportation systems are crucial to the survival and health of a city or community. Earthquakes affecting such systems cause severe social and economic disruptions to communities and cause human suffering to the residents.

2. Scope of Work

* Plan and conduct T/C workshops and meetings. The purpose is to exchange the state-of-the-art knowledge and practice, and to identify key cooperative opportunities for exchange and studies which effectively implement research results.

* Conduct related technical sessions at professional society meetings.

* Develop research programs on selected lifeline systems (similar in concept to the Large-Scale Testing Program). For example, through joint demonstration, produce criteria for emergency design and loss evaluation and on-line

monitoring systems for lifeline operation.

* Conduct special activities such as translating, printing, and distributing MOC PWRI's publication Manual for Repair Methods for Civil Engineering Structures Damaged by Earthquakes.

* Develop performance standards and a manual on repair, restoration, and retrofit of lifelines.

3. Accomplishments

* Conducted the 4th Joint Workshop on Earthquake Disaster Prevention for Lifeline Systems on August 19-21, 1991, in Los Angeles, California.

* Contributed technical papers on lifeline systems at the 24th joint panel meeting.

* Participated in promoting the goals of the IDNDR.

* Developed a seismic design manual for underground structures (Japan Side).

4. Future Plan

* Conduct the 5th Joint Workshop on Earthquake Disaster Prevention for Lifeline Systems on October 26 - 27, 1992, in Tsukuba, Japan. A technical study tour will follow the technical sessions.

* Encourage current efforts in both countries to establish seismic design guidelines and standards for lifeline systems. Existing UJNR channels should be fully utilized to facilitate the exchange of relevant information concerning standards development. Possible collaboration of joint development lifeline systems standards and guidelines should be pursued.

* Continue observation of instrumented buried pipes in Parkfield, California, and Chiba, Japan, and exchange data.

5. Information Exchange

* Domestic community: through technical meetings, conference sessions, seminars, and workshops.

* Between U.S. and Japan: through T/C Workshops and annual Joint Panel Meetings.

* Visits by researchers and participation in technical meetings sponsored by organizations such as NCEER, EERI and ASCE.

* Short-term (3 to 12 months) researcher exchanges to participate in on-going lifeline research.

6. Impact

* Contributed to the increase of field engineers' technical knowledge on repair methods, through seminars with lectures by T/C members using the MOC Manual n Japan: more than two thousand participants attended seminars.

* Contributed to the development and assessment of design, construction, and operating standards for lifeline earthquake engineering (TCLEE/ASCE) via T/C members' participation in committee work.

7. Barriers

* Lack of sufficient funds to conduct annual Task Committee.Workshops and to publish proceedings and reports.

（G）パッシブ，アクティブおよび
ハイブリッドコントロールに関する作業部会報告

開　催　日：１９９２年５月２０日

場　　所：国立標準技術研究所　ゲイザーズバーグ市

出　席　者：日本側　北川良和（作業部会長）　（建築研究所）

藤原　稔　（土木研究所）

山内泰之　（建築研究所）

米国側　S. C. Liu（作業部会長）　（NSF）

J. Hayes　（CERL）

C. Stoddart　（Martin Marietta/DOE）

R. Sause　（Lehigh Univ.）

［活動要約］

1．目的

本作業部会の目的は以下の５つである．

(1) パッシブ，アクティブおよびハイブリッドコントロールに関する研究の推進

(2) 風・地震時における構造物応答の低減・抑制に必要な制御技術，特に実設計・施工を考慮した装置・システムの開発

(3) 構造物制御に関する日米共同研究の推進

(4) 構造物制御に関する官・学・民共同研究の推進

(5) 国際共同による各種研究活動の推進及びＩＤＮＤＲへの貢献

2．活動の範囲

ＵＪＮＲ作業部会（Ｇ）はＵＪＮＲ組織を通じて構造物制御研究に関する情報・データ交換を行うと共に関連機関との共同作業を推進する．活動内容は以下の６項目である．

1) 構造物制御研究の開始・展開・実行に関するＵＪＮＲ関連研究機関への技術支援，協議，調整の準備

2) 技術移転と実用化を推進する為の官・学・民協力の振興

3) 各種共同研究，ワークショップ，打合せの支援・企画及び実用面での知識と経験

に関する情報交換

4) 社会的コンセンサスを得るための推進的，論証的活動

5) 既存構造物の補強，修復及び構造物に対する制御に関する目標性能，設計仕様，安全評価等のガイドライン（指針）の作成

6) 新しいインテリジェント部材，高精度センサー，最適制御システム設計，模型及び実大構造物実験に関する研究の推進

3．成果

1) 構造物制御研究に関する日米両国関係者は積極的に関連情報の交換及び共同研究の展開について意見を交換を行った．

2) １９９１年９月４，５日Ｔ／Ｃ“Ｇ”主催による第１回橋梁の免震設計に関する日米ワークショップが米国ニューヨーク大学バッファロー校の国立地震工学研究センター（NCEER）に近いホテル・マリオットで実施された．ワークショップの一環として米国の免震橋１１橋の実地見学を行った．このワークショップにＮＣＥＥＲのI. G. Buckle，ＵＪＮＲ委員のS. C. Liu，M. Shinozuka，川島と米国，日本，台湾から６１名の研究者・技術者が参加した．その内容はプロシーデングとしてＮＣＥＥＲより公表される予定である．

3) １９９１年３月２３～２５日アクティブ制震（振）シンポジウム／ワークショップが東京で実施された．このシンポジウムは日本学術会議地震工学研究連絡委員会制振小委員会（SCJ）によって組織されたもので，ＵＪＮＲ委員であるS. C. Liu，M. Shinozuka，北川，川島はこのシンポジウムに積極的に参加した．ワークショップは３つの分野，すなわち制御理論，装置とシステム，構造設計から構成された．論文集はSCJにより公表された．

4) １９９２年４月２７～３０日免震構造技術に関する国際ワークショップが東京で実施された．このワークショップは日本建築学会免震小委員会（AIJ）によって組織されたもので，ＵＪＮＲ委員である北川，川島が積極的に参加した．ワークショップの主テーマは設計概念と基準，地震入力及び応答解析，アイソレータとダンパーの挙動，実施設計及び地震観測である．プロシーデングはＡＩＪより公表された．

5) １９９１年６月２７～２９日インテリジェント・システムに関する日・米・伊ワークショップ／セミナーがイタリーペルギアで実施された．このワークショップにＵＪＮＲ委員のS. C. Liu，H. J. Lagorio，K. Chong，W. Anderson と米国，日本，イタリア他多数の西ヨーロッパ諸国から研究者が参加した．

6) 性能・安全性及び危険緩和のための構造物制御研究に関するＮＳＦによる大規模研究プロジェクトに大学等の研究機関より６７テーマの応募があった．このうち

１９９２年より５年プロジェクトとして１０テーマ程度が選定される予定である.

４．将来計画

1) Ｔ／Ｃ“Ｇ”主催による「第２回地震時の橋梁パッシブ及びアクティブ制御に関する日米ワークショップ」が１９９２年１０月又は１２月，つくば市建設省土木研究所において計画されている．日本国内の免震橋梁の視察旅行がワークショップの前後に予定されている（ワークショップ議長はI. G. Buckle/ ＮＣＥＥＲと川島一彦／土木研究所）
2) 性能・安全性及び危険緩和のための構造物制御研究に関するＮＳＦによる大規模研究プロジェクトは１９９２年より２年度目を迎える予定である.
3) 構造物アクティブ，ハイブリッド制御システムに関する日米共同研究プロジェクトが建設省（建築研究所，土木研究所） ＮＳＦによって検討されている.
4) 橋梁のハイブリッドコントロールに関する共同研究プロジェクトがＰＷＲＩ，ＮＩＳＴ，ＦＨＷＡ，ＮＣＥＥＲ及びカリフォルニア大学（Irvine）によって検討されている.
5) ７階建鉄骨造免震構造モデルによる振動台実験がＣＥＲＬで進行中である.
6) 更に，１９９２年６月より形状記憶合金を用いたアクティブ制振構造物の実験がＣＥＲＬで予定されている.
7) 高性能・高品質材料システムに関する日米合同ワークショップを近い将来実施する．つくば（日本）での第２５回ＵＪＮＲ合同部会の前後に，Ｔ／Ｃ“Ｃ”との協調・支援のもと実施されることが望まれる.
8) 建築物のアクティブ，ハイブリッド制御に関する共同研究プロジェクトがＢＲＩ，ＮＣＥＥＲ及びカリフォルニア大学（Irvine）によって検討されている.
9) 来年度（1992-1993）及び今後５ヶ年の明確な作業計画を作成する.
10) Ｔ／Ｃ“Ｊ”と協調して，引き続き橋梁構造物に対する免震技術の開発及び実施を支援する.
11) 官民連帯共同研究（建築構造物の制振構造の実用化技術の開発）がＢＲＩでＵＪＮＲ委員の北川により実施されている．１９９３年１月縮小モデルによる振動台実験が予定されている.

５．情報交換

構造物応答及び制御システムに関する技術資料が交換された.

６．活動の効果

Ｔ／Ｃ“Ｇ”での構造物応答制御に関する学歴的研究及び共同研究を開始するために，

両国からの活発な研究グループが形成される.

7. 問題点

現状での技術的障害は認められない. しかしながら, 両国からの十分な財政的裏付けが無い場合には, T／C "G" の目的を達成する上で困難を来たすものと思われる.

Report of Task Committee on

(G) PASSIVE, ACTIVE, AND HYBRID CONTROL SYSTEMS

Date: May 20, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees:

U.S. Side -	S. C. Liu (Chairman)	(NSF)
	Jack Hayes	(CERL)
	Chris Stoddart	(Martin Marietta/DOE)
	Richard Sause	(Lehigh Univ.)
Japan Side -	Y. Kitagawa (Chairman)	(BRI)
	M. Fujiwara	(PWRI)
	H. Yamanouchi	(BRI)

MISSION STATEMENT

1. Objective:

The objectives of the Task Committee are to: (1) develop research plans in passive, active, and hybrid control of structures and equipment, (2) implement such control techniques as computerized motion reduction or modification devices/systems in actual design and construction of structural engineering facilities under seismic or wind environments, (3) promote U.S. -Japan cooperation in structural control research (4) bring together governmental, academic, and industrial participants in joint pursuit of such efforts, and (5) contribute to IDNDR by organizing joint research and other technical activities in structural control on the basis of international cooperation.

2. Scope of Work

The UJNR T/C "G" works closely with other organizations to provide the leadership in control research efforts by facilitating the exchange of technical data and information through the established UJNR mechanisms. Work includes:

i) Providing technical assistance, consultation and coordination to UJNR affiliated research agencies in the initiation, development, and execution of their programs in structural control research.

ii) Promoting joint government-university-industry collaborative efforts to facilitate technology transfer and practical implementation.

iii) Sponsoring and conducting multi-disciplinary workshops and meetings to identify key areas of research and opportunities for cooperation, and to exchange new knowledge and experience in practice.

iv) Developing promotional and demonstrative activities to stimulate public awareness and interest in this emerging field of research.

v) Establishing performance standards, design specifications, guidelines and code recommendations for application in new construction as well as retrofit/rehabilitation of existing structures with control systems.

vi) Initiating research in new intelligent materials, high-tech sensors, optimal control systems design, and laboratory and field experiments of prototype and full-scale structures.

3. Accomplishments

i) The U.S. Panel on Structural Control Research worked closely with its counterpart Japanese Panel in exchanging technical information and developing joint research plans.

ii) In cooperation with T/C "J", a "U.S.-Japan Workshop on Earthquake Protective System (Passive Control System)" was held at NCEER, Buffalo, New York, on September 4 and 5, 1991. A study tour to inspect base-isolated bridges across the U.S. was conducted by participants prior to and following the Workshop. I. G. Buckle of NCEER and UJNR Panel members S. C. Liu, M. Shinozuka, K. Kawashima, and 61 researchers from the U.S., Japan, and Taiwan participated in the technical activities. Proceedings of the Workshop is being published by NCEER.

iii) The "Japan National Symposium/Workshop on Active Structural Response Control" was held on March 23-25, 1991, in Tokyo, Japan. The Symposium was organized by Japan National Working Group on Active Structural Response Control, Earthquake Engineering Research Liaison Committee, Science Council of Japan (SCJ). UJNR Panel members S. C. Liu, M. Shinozuka, Y. Kitagawa, and K. Kawashima participated in the Symposium's technical activities. The workshop identified three major areas of research in structural control. They are control theory, devices and systems, and structural design. Preliminary Proceedings of the Symposium was published by SCJ.

iv) The "International Workshop on Recent Development in Base Isolation Techniques for Building" organized by the Architectural Institute of Japan (AIJ) was held in April 1992, in Tokyo, Japan. UJNR panel members Y. Kitagawa and K. Kawashima participated in the Workshop's technical

activities. The Workshop identified four specific topics for future efforts: design concept and code requirements, estimation of seismic input and structural response, behavior of isolators and dampers, and practical applications and field observations. Proceedings of the International Workshop was published by AIJ.

v) A trilateral U.S.-Italy-Japan Workshop/Seminar on Intelligent Systems was held in Perugia, Italy, June 27-29, 1991. Investigators from the U.S., Italy, Japan, and other Western European countries gave presentations of papers and participated in group discussions. UJNR members S. C. Liu, H. J. Lagorio, Ken Chong, William Anderson, etc., contributed papers and H. J. Lagorio participated in the meeting.

vi) Under the NSF initiative on "Structural Control Research for Safety, Performance, and Hazards Mitigation," sixty seven (67) proposals were submitted by the U.S. research community. Approximately ten (10) proposals are being selected for funding during the first year (FY 92) of the 5-year program.

4. Future Plans

i) In cooperation with T/C "J", a "Second U.S.-Japan Workshop on Passive and Active Control of Bridges Against Earthquakes" is scheduled for December 1992 at PWRI, Tsukuba. A study tour to inspect base-isolated bridges across Japan is being planned for the participants prior to and following the Workshop (Workshop Co-Chairmen: Ian Buckle/NCEER and K. Kawashima/PWRI).

ii) In cooperation with T/C "J", support of development and implementation of innovative base-isolation technology for bridge structure will continue.

iii) Specific Task Committee work plans are being developed for coming year (1992-1993) and for the next five years.

iv) Research cooperation between the U.S. and Japan will be pursued using such channels as the UJNR and other inter-governmental mechanisms as the U.S. and Japan Panel on Structural Control. Specific plans to promote "The U.S.-Japan Cooperation Research Project" on active and hybrid control systems are being studied by NSF, BRI, and PWRI.

v) Joint research projects on hybrid control of bridges are on-going by PWRI, NIST, FHWA, NCEER, and University of California, Irvine.

vi) The Construction Engineering Research Laboratory (CERL) will initiative work in July 1992 on active control of building structures using shape memory alloy. UJNR member Jack Hayes is project coordinator of this

research.

vii) A joint U.S.-Japan Workshop on "Smart and High-Performance Materials Systems" will be planned in the near future. If possible this workshop shall be in conjunction with the 25th Joint UJNR panel meeting in Tsukuba, Japan. T/C "C" will be invited to join and help sponsor this Workshop.

viii) Joint research projects on active and hybrid control of buildings are being developed by BRI, NCEER and University of California, Irvine.

ix) At BRI, research programs on structural control of buildings have been developed and implemented under the direction of UJNR member Y. Kitagawa. Shaking-table tests of scaled control models will be carried out in January 1993. Collaborative research program involving possible organizations and private companies are being developed.

5. Information Exchanges

Technical materials on structural response and control systems were exchanged.

6. Impact

The T/C "G" has stimulated research communities from both sides to initiate multi-disciplinary studies and cooperative investigation in structural control.

7. Barriers

No technical barriers are identified at the present time. However, lack of sufficient financial backing from both sides may present difficulties in achieving the objectives of T/C "G".

（H）地震時の地盤の挙動と安定作業部会報告

開　催　日：１９９２年５月１９日
場　　所：米国商務省国立標準技術研究所　ゲイサスバーグ市
出　席　者：日本側　横山功一　（臨時作業部会長）　（土木研究所）
　　山口嘉一　（　〃　）
　　古関潤一　（　〃　）

　　米国側　A.G. Franklin　（作業部会長）　（陸軍局工兵隊水路試験所）
　　M.G. Katona　（空軍土木研究所）

「活動要約」

1．目的

土木構造物を所管する政府機関は、地震災害に対して、これらの地震時安定性を確保し、また経済的な対策をとることが求められている。本部会の目的は、地盤や構造物の地震時の安定性を確保することを目的として、地盤の動的挙動の予測、地盤と構造物の動的相互作用の解析さらに基礎および土構造物の地震被害の軽減のために有効な技術の向上を図ることにより、上記の求めに応じられるよう支援することである。

2．活動の範囲

・毎年の合同部会における、地震時の地盤の挙動と安定に関係する技術開発、技術の動向ならびに技術の現況を記述した技術論文の提出。
・現地調査、研究ならびに設計・施工の現況に関連する技術データや関連する情報の交換。
・すでに提案された、または実施中の共同研究プログラムについての作業部会ワークショップの計画と実行。
・作業部会は、研究課題の調整、日米両国の政府関係研究機関の間での研究者交流、経験と研究成果に関する技術情報の交換による共同研究プログラムを策定・実行するとともに、研究成果や推奨できる設計施工についての共同執筆・出版を通じて技術移転を図る。そして両国が関心を持つ問題に関して衆知を集め、両国のそれぞれの研究能力を最大限に活用することによって、研究の成果を高めることができる。
・公式あるいは非公式な共同プログラムに基づく研究者の交流。交流により共同研究プロ

グラム目的の推進、技術情報交換の促進、両国の設計施工法の習熟、さらに両国の担当者間のより深い相互理解と協調関係を促進する。

3．成果

・第23回および第24回合同部会において、地震時の地盤の挙動と安定に関する論文が米側から5編、日本側から10編提出された。……1991年5月，1992年 5月

・土木研究所の主催により液状化の対策に関する国際ワークショップが、1991年1月22日から25日まで日本のつくば市に於て開催された。合計19編の論文が提出された。……1991年 1月

・日本側作業部会長の常田賢一氏はセントルイスを訪れ、水路試験所の R.L.Ledbetter氏と会い、1991年1月に開催された液状化の対策に関する国際ワークショップの報告書の取りまとめ方法、今後のワークショップの開催計画について打ち合せを行った。……1991年3月

・液状化の対策に関する国際ワークショップの報告書が印刷され、作業部会（H）の委員及びワークショップの参加者に配布された。……1991年3月、5月

・1990年7月16日に発生したフィリピン地震に対する、 日本政府の災害緊急援助隊・専門家チームの英文レポートが米国側に提供された。……1991年 5月

・作業部会（H）は、第２３回専門部会で承認された”海洋構造物の耐風・耐震技術に関する作業部会（K）”に対して、地震に対する海洋構造物及び海底地震観測の検討の役割を引き継いだ。……1991年 5月

・土木研究所の古関潤一氏がマサチューセッツ工科大学において、１年間の予定で研究員として、液状化の危険度解析に関する研究を行っている。……1991年 8月

・米国側作業部会長代理の水路試験所のR.H.LEDBETTER氏が来日し、液状化の対策に関する日米共同研究を開始するための情報交換と討議を行い、基本事項について合意が得られた。……1991年 9月

・土木研究所の山口嘉一氏が水路試験所において、１年間の予定で研究員として、”ダム基礎の浸透に関する研究”を行っている。……1991年12月

4．将来計画

・水路試験所は、新しい研究プロジェクト”補修、診断、維持と復旧に関する研究（ＲＥＭＲ） ”（研究期間：１９９２年～１９９６年）を開始した。…… 1991年10月

・土木研究所及び建築研究所は、新しい研究開発”大都市地域における地震防災技術の開発”（研究期間：１９９２年～１９９６年）を開始した。……1992年 4月

・防災科学技術研究所、建築研究所及び米国Wayne州立大学は、共同研究”液状化地盤における杭基礎の地震抵抗に関する研究”（研究期間：１９９２年～１９９４年）に着手した。……1992年 4月

・1989年5月及び1991年9月の計画調整会議の結果に従って、両国が共通の関心を持っている液状化対策に関する研究項目を設定する。……1992年 9月
・液状化の対策に関する次回のワークショップは、1993年10月に日本で開催することを提案する。……1993年10月
・第22回合同部会の作業部会（H）では、地震災害における人命救助を支援するための技術開発や研究の必要性について議論した。必要とされる主な分野は、生存者を探査するための高度な技術及び破壊した構造物から生存者を救出するガレキの安全な堀削のための新しい装置やシステムの開発である。　作業部会（H）は、この課題が部会活動の範囲外であることは理解しながらも、この分野は重要であり、合同部会の場で議論されるべきものと考えている。

本作業部会では、この課題の実行のために、次の事項を提言する。

（1）この分野に対応し、この分野の経験や研究を担うのにふさわしい新しい作業部会の設置。

（2）この分野を他の作業部会への組み込み。

作業部会（H）は、この分野が適切に評価されるような方法を模索してゆく。

5．情報交換

・作業部会のワークショップ、研究者交流、毎年の合同部会への参加と論文の提出。
・日米両国政府機関研究所の研究者の交流。
・震後の現地調査などの技術的活動への参加と訪問。
・両国での専門的な活動、刊行物への論文の掲載、両機関による研究論文の刊行、専門的な学会、会議、セミナー、ワークショップへの参加。

6．活動の効果

・研究者交流と地震に関連した技術データ、経験、設計・施工法などについての情報交換を通じて、土質地震工学分野の両国の技術水準が向上した。
・1983年日本海中部地震のあとに実施された地震の原位置試験法の共同研究により、地盤の液状化判定のための原位置試験法がより正確になり改善された。この改善された方法は1985年以降米国で一般に用いられている。
・ロマ・プリータ地震に関する調査により、地盤液状化及び軟弱地震に関する一層の研究の必要性が認識された。
・１９８８年及び１９９１年の液状化の対策に関するワークショップにより、最近の技術に関する貴重な情報が交換され、日米双方の将来の研究推進に貢献した。

7．問題点

・地震工学分野の研究に投資される研究予算の不足。

・海外への出張の制約。

Report of Task Committee on

(H) SOIL BEHAVIOR AND STABILITY DURING EARTHQUAKES

Date: May 19, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees: U.S. Side - A. G. Franklin (Chairman) (WES)
M. G. Katona (AFCEL)

Japan Side - K. Yokoyama (Temp. Chairman) (PWRI)
Y. Yamaguchi (PWRI)
J. Koseki (PWRI)

MISSION STATEMENT

1. Objective:

Government agencies with responsibility for public works have the need to assure their seismic safety and to provide economical protection against earthquake hazards. The objective of this Task Committee is to assist in meeting those needs by enhancing the available technology for predicting the dynamic behavior of soils, analyzing dynamic soil-structure interaction, and modifying the earthquake behavior of foundations and earth structures in order to assure their safe performance during earthquakes.

2. Scope of Work

* Present technical papers at annual joint panel meetings on technological developments and on state-of-the-art and practice related to soil behavior and stability during earthquakes.

* Exchange information and technical data relating to field performance, research, and methods of practice.

* Plan and conduct T/C workshops in coordination with proposed or on-going cooperative research programs.

* The Task Committee plans and conducts programs of cooperative research, exchange of researchers between U.S. and Japanese research institutions, publication of research results and recommended practice. The benefits realized from available research funds can be amplified by

bringing greater resources to bear on problems of mutual interest, and by coordinating research tasks to make the best use of the respective strengths of the two sides' research capabilities.

* Exchange visiting researchers using mechanisms provided by formal and informal cooperative research programs. Such exchanges promote technical information exchanges, familiarization with methods of practice on the respective sides, and mutual understanding and cooperative relationships between researchers on the two sides.

3. Accomplishments

* U.S. researchers presented 5 papers and Japanese researchers presented 10 papers, on soil behavior and stability during earthquakes, at the 23rd and 24th Joint Panel Meetings. 5/91, 5/92

* The International Workshop on Remedial Treatment of Potentially Liquefiable Soils was conducted at PWRI in Tsukuba from 22 to 25 January 1991. Nineteen papers were presented. 1/91

* Mr. Ken-ichi Tokida, the T/C (H) Japan Chairman, visited St. Louis and met Mr. R. H. Ledbetter of the U.S. Army Engineer Waterways Experiment Station and discussed the Proceedings of the International Workshop held in January and plans for the next Workshop. 3/91

* Proceedings of the International Workshop on Remedial Treatment of Potentially Liquefiable Soils was published and distributed to the members of Task Committee (H) and participants of the Workshop. 3/91, 5/91

* The English language report on the expert team of Japan Disaster Relief Team (JDR) on the earthquake in the Philippines of July 16, 1990, was provided to the U.S. side. 5/91

* T/C (H) has relinquished its responsibility for consideration of marine structures subject to earthquakes and the measurement of sea floor earthquake motions to the new Task Committee (K) on Wind and Earthquake Engineering for Offshore and Coastal Structures, recognized at the 23rd Joint Panel Meeting. 5/91

* Mr. Jun-ichi Koseki of the PWRI is a one-year Guest Researcher at Massachusetts Institute of Technology, working on Liquefaction Risk Analysis. 8/91

* Dr. Richard H. Ledbetter, of the U.S. Army Engineer Waterways Experiment Station, visited Japan in September 1991. Information

exchange and fundamental discussion to start the U.S.-Japan joint research through T/C "H", on remedial treatment of liquefiable soils were conducted, and fundamental subjects were agreed upon. 9/91

* Mr. Yoshikazu Yamaguchi of the PWRI is a one-year Guest Researcher at the U.S. Army Engineer Waterways Experiment Station where he is studying water seepage in dam foundations. 12/91

4. Future Plans:

* The Waterways Experiment Station has initiated a new research project "Repair, Evaluation, Maintenance, and Rehabilitation Research (REMR)" (for the U.S. fiscal-years 1992 - 1996). 10/91

* The Public Works Research Institute and the Building Research Institute have initiated a new research and development project "Research and Development on Earthquake Disaster Prevention in Large Urban Area" (for the Japan-fiscal years 1992 - 1996). 4/92

* The National Research Institute for Earth Science and Disaster Prevention, the Building Research Institute, and Wayne State University have initiated the cooperative research "Research on Earthquake Pile Resistance in Liquefied Soils". 4/92

* Identification of research tasks on preventive measures against soil liquefaction that are of common interest, in accordance with the results of the planning meeting held at Tsukuba in May 1989 and at Yokohama in September 1991. 9/92

* T/C (H) proposes to hold the next U.S.-Japan Workshop on Remedial Treatment of Potentially Liquefiable Soils in Japan, in October 1993. 10/93

* T/C (H) at the 22nd Joint Panel Meeting, discussed the need for technological innovation and research to assist in saving lives following natural disasters caused by earthquakes. The principal areas of need were identified as improved technology for locating survivors and the development of new equipment and systems for safe excavation of debris to remove survivors from collapsed structures. T/C (H) recognizes this subject is outside their area of responsibility but feels it is important and should be discussed by the full Joint Panel.

The Task Committee suggests the following options for implementation:

(1) Explore a new Task Committee which will address this topic area and provide the focal point to share experience and research in this area.

(2) Incorporate this topic into another existing T/C.

T/C (H) will continue to explore methods whereby this topic will receive appropriate attention.

5. Information Exchange:

* T/C workshops, exchange of visitors, participation in annual joint panel meetings, and presenting technical papers.

* Exchange of researchers between U.S. and Japanese government laboratories.

* Visits by researchers and participation in technical activities such as post-earthquake field investigation.

* Fostering professional community contacts in the U.S. and Japan, publication of papers in journals, publication of research reports by the respective agencies, participation in professional society meetings and conferences, and workshops.

6. Impact

* Through the exchange of researchers and the diffusion of earthquake related technical data, experience, and information on methods of practice, the state of technology in geotechnical earthquake engineering has been raised on both sides.

* Through the cooperative research program on in-situ testing of soils which was carried out after the 1983 Nihonkai-Chubu Earthquake, improved and more accurate methods of using in-situ tests for evaluating the liquefaction potential of soils were achieved: these have been in general use in the United States since 1985. The Loma Prieta Earthquake stimulated renewed awareness of the need for further research in soil liquefaction and amplification effects of soft soils.

* Through the workshops on remedial treatment of potentially liquefiable soils held in 1988 and 1991, beneficial information on recent technology were exchanged and contributed to progress future research of both U.S.-and Japan-sides.

7. Barriers

* Insufficient U.S. and Japan Government funds allocated to adequately address earthquake engineering research.

* Department of Army administrative limitation on foreign travel.

（１）高潮及び津波に関する作業部会報告

開　催　日：１９９２年５月２０日
場　　所：国立標準技術研究所、ゲイザーズバーグ市
出　席　者：日本側　横山功一　(PWRI)
　　塩路勝久　(PWRI)

　　米国側　H. Meyers（作業部会長）　(NOAA)
　　W. E. Roper　(COE)

［活動要約］

1．目的

本作業部会の目的は技術、研究、情報の交換と共同研究によって、高潮・津波による災害を軽減することにある。高潮および津波は災害を起こすにたる危険を有する。高潮はハリケーンや台風によって生じ、強風と台風中心の気圧低下は大洪水と強い波の作用をもたらし得る。高潮は、上陸前数日間の追跡は可能であるが、上陸地点の中心を予測することは難しい。津波は主に海底地震によって引き起こされるが、海底火山の活動によることもある。津波は発生源からの距離に応じて、発生後数分から１日までの間に来襲する。

2．活動の範囲

・高潮・津波・の発生、発達、伝播および影響に関する研究成果の交換を行う。これは、歴史津波と最近の津波の観測および理論的検討を含む。特に、日米による深海津波計測を実施するよう努力する。
・高潮・津波対策の検討、計画、警報、技術課題に関する成果、現状の情報交換を行う。
・高潮・津波に関する計画または実行中の事業の情報交換を行う。
・波の到達時間、地すべりが発生した地点、遡上高、波の特性等を予測するコンピュータプログラム、機器の改良、検知および警報のための衛星技術の利用等、技術開発に関する情報交換を行う。
・合同部会での文献、技術報告書の交換、ワークショップ、共同事業および部会員の直接的な交流によって、研究の普及を助ける。

3．成果

・1992年4月の北カリフォルニア地震・津波の予備観測記録が日本側に提供された。
・米国側の部会員についてＣＯＥ、コロラド大学、ＮＯＡＡ、ＮＳＦから増員があった。
・米国側は、本土と領土に対し影響のあった津波について詳細な情報収集を行った。これをもとに "United States Tsunamis 1690-1989" を出版し、日本側に配布した。
・第２回の津波ワークショップを１９９０年１１月５～７日にハワイで実施した。日本側からは９名、米国側からは２０名が出席した。ワークショップでは討議終了後、ヒロへの現地視察も実施した。また、論文集が印刷され、関係者に配布された。

4．将来計画

・深海津波計測を含む津波観測に関する活動のデータおよび情報の交換を進める。
・機器の試験、共同出版、研究者の交換、衛星通信、日米間の交流の援助等共同作業について検討するとともにそれらを推進する。
・高潮・津波災害の様々な側面に関する研究および技術の交流を図るため、２～３年おきにワークショップを開く必要があることに米国側と日本側は合意する。第３回ワークショップを１９９３年８月に日本で開催することを計画する。ワークショップでは高潮と津波を話題として取り上げる。ワークショップの開催時期はＩＵＧＧの津波シンポジウムの前後の時期とする。ワークショップは、アメリカ側と日本側の両方から同数ずつ出される約１０人の学術研究員と、幾人かの政府機関の作業部会のメンバーにより構成され、高潮部門と津波部門の人数は同程度とする。
・1996年は三陸沖地震・津波の100周年と、1946年のアリューシャン地震・津波50周年である。これらの地震・津波は警報システムの開発とこの分野での活動を活発にさせるのに役立ったものであるが、作業部会ではこの合同記念シンポジウムもしくは公式のプログラムの準備にとりかかる。
・アメリカ側は全ての有用な潮位計の記録を含む、アメリカ西海岸へ作用を及ぼす津波に関するの詳細な研究を仕上げ、報告書を１９９２年秋に印刷する。また、日本側へも配布する。ユネスコ-ITCUの要求について　もアメリカ側は、メキシコ西海岸における津波による作用についてのカタログをSanchezとFarrerasにより翻訳・出版する。

5．情報交換

・刊行物、文献目録データ、技術論文の交換および研究者の交流を継続する。

6．活動の効果

・この分野における研究者間の協力関係が増す。
・米国と日本による新技術の完成が早められる。
・日米両国の科学者、技術者に対する情報と技術の広範囲な普及を援助できる。

7．問題点

・政府または私的な組織による作業部会員および合同部会パネルへの資金援助が不十分である。

・津波問題は米国において日本ほど深刻ではなく、その結果研究者の集団が小さい。津波に関心を持つ研究者の数は少ない。このことは定期的な作業部会とワークショップの重要性を示している。

・問題を扱うにおいて、日本と米国とではその方法、重要性に違いがある。米国では多くの災害は遠隔地で発生した津波によるものである。このため、米国政府は警報や避難方法の開発を強調する。それに対し、日本では、津波はしばしば近海で発生し、到達時間が短いため適切な警報を発令したり、避難をするに十分な時間がない場合がある。このため工学的手法が必要となる。この障壁に関しては、米国側から臨時会員または本会員として年会議に出席した技術者や民間の専門家により部分的に指摘されている。

・高潮調査委員会は十分に組織化されておらず、専門の学会、学会誌または国際的な組織、さらにはその分野における刺激的な研究テーマが不足している。作業部会（Ｉ）はこのギャップを埋めることを助ける。

Report of Task Committee on

(I) STORM SURGE AND TSUNAMI

Date: May 20, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees: U.S. Side - H. Meyers (Chairman) (NOAA)
W. E. Roper (COE)

Japan Side - K. Yokoyama (Acting Chairman) (PWRI)
K. Shioji (PWRI)

MISSION STATEMENT

1. Objective:

The objective of this Task Committee is to mitigate damages from storm surges and tsunamis through shared technologies, research, information sharing, and cooperative work. Storm surge and tsunami are hazards capable of inflicting damage of disastrous proportions. Storm surges are associated with hurricanes and typhoons where high winds and the mounding of water under the low barometric pressure of the storm's eye can cause extensive flooding and severe wave action. These often can be tracked for days in advance of a landfall but the location of the landfall center is difficult to predict. Tsunamis are predominately caused by underwater earthquakes and, to a lesser extent volcanic and land slide activity. Depending on the distance from the source, tsunamis arrive within minutes to up to a day after generation.

2. Scope of Work:

* Exchange results of research on storm surge and tsunami occurrence, generation, propagation, and coastal effects. This includes observations on historical, current, and theoretical tsunamis. Of particular interest is the effort by U.S. and Japan to acquire deep ocean tsunami measurements.

* Exchange results and status of anti-storm surge and tsunami activities including analysis of the problem, planning, warning, and engineering approaches.

* Exchange information on planned and on-going projects relating to storm surge and tsunamis.

* Exchange information on development of technologies such as computer programs to predict travel times, land-fall locations, run-up heights, and wave characteristics and analysis; improved instrumentation, and use of satellite communications for detection and warning.

* Facilitate dissemination through exchange of literature, technical reports at joint meetings, special workshops, joint projects, and direct interaction among participants.

3. Accomplishments

* Copies of preliminary marigrams from the April 25, 1992 Northern California earthquake and tsunami were distributed to the Japanese side.

* The U.S. side has compiled detailed information on all tsunamis affecting the U.S. and its possessions. The publication, "United States Tsunamis 1690-1989" has been printed and given to the Japanese side.

* A Second Tsunami Workshop was held on November 5-7, 1990 in Honolulu, Hawaii. Nine members participated from the Japanese side and about 20 from the U.S. side. Site visits were made to Hilo as part of the Workshop. Proceedings were published and were sent to all members.

4. Future Plans

* Develop exchanges of data and information on activities related to tsunami measurements including deep ocean measurements. Deep ocean bottom gages will be recovered in June 1992 from the ocean bottom stations which are expected to have recorded the small tsunamis for the April 25, 1992, Northern California earthquake and tsunami. This event was recorded by Hawaiian and U.S. West Coast shore based marigraphs. The seafloor topography of the source area will be re-surveyed. These will be shared with the Japan side.

* Develop activities and information exchanges to support the International Decade of Natural Disaster Reduction.

* Explore and promote joint undertakings such as tsunami gauge testing, scientist exchange, satellite communication for tsunami warning, and assistance in contacts within the U.S. and Japan.

Increase the participation of storm surge experts in the Panel and T/C meetings.

The U.S. and Japanese sides agree that there is a need to hold workshops at about 2 or 3 year intervals to exchange the latest research and technology relating to various aspects of storm surge and tsunami hazards. A third workshop including both storm surge and tsunamis is planned for August, 19-20, 1993 in Tskuba, Japan, near the time of the International Union of Geodesy and Geophysics (IUGG) Tsunami Commission Symposium. The Workshop will include about 10 academic researchers and some additional Government Task Committee members from the US side and similar numbers from Japan. There will be about an even split between the storm surge and tsunami problems.

1996 will be the 100th anniversary of the great Sanriku Earthquake and tsunami in Japan and the 50th anniversary of the April 1, 1946 Aleutian earthquake and tsunami. These events led to the creation of warning systems and much of the current activity in this field. The Task Committee is beginning to plan for a Joint Commemorative Symposium or other observance.

The U.S. side is completing a detailed study of tsunamis affecting the US West Coast including copies of all available marigrams. This will be published in the fall of 1992 and shared with the Japan side. Also, at the request of UNESCO-ITSU, the U.S. side will publish a catalog of tsunamis affecting the West Coast of Mexico compiled by Sanchez and Farreras, the Mexican authors.

5. Information Exchange

Continue exchange of publications, bibliographic data, technical reports, and personal contacts.

6. Impact

Increased cooperation among scientists and engineers working in these fields.

Hasten implementation of new modeling and design techniques developed by the U.S. and Japan.

7. Barriers

Insufficient funding from government and private sector organizations to participate in T/C workshops and the Panel's annual joint meetings.

The tsunami problem is less severe in the U.S. than in Japan which leads to a smaller community of researchers. There are few U.S. researchers focusing on tsunamis at any specific location; thus highlighting the importance of T/C periodic meetings and workshops.

There are different approaches and degree of significance in dealing with problems in the U.S. and Japan. For example, in the U.S. much of the hazard is from remote sources which leads the government to emphasize developments of warning and evacuation schemes. In Japan the engineering approach is needed because there are more locally generated tsunamis which often do not provide sufficient times for adequate warning and evacuation. This barrier is partially being addressed by inviting U.S. engineers and non-government specialists to participate during the annual meetings.

The storm surge research community is less well organized and lacking the specialty societies, journals, and international organizations to define and stimulate work in the field. The Task Committee (I) can help fill this gap.

（Ｊ）交通システムの耐風耐震技術に関する作業部会報告

開　催　日：１９９２年５月１９日
場　　所：国立標準技術研究所　ゲイザーズバーグ市
出　席　者：日本側　藤原　稔　（土木研究所）
　　中野　修　（北海道開発局）

　　米国側　Ｊ.Ｄ.Ｃｏｏｐｅｒ　（作業部会長）　（ＦＨＷＡ）
　　Ｃ.Ｂａｒｒｉｅｎｔｏｓ　（ＮＯＡＡ）
　　Ｒ.Ｄｉｋｋｅｒｓ　（ＮＩＳＴ）
　　Ｊ.Ｓｃａｌｚｉ　（ＮＳＦ）
　　Ｈ.Ｓ.Ｌｅｗ　（ＮＩＳＴ）

［活動要約］

１．目的

交通システムは、人々の交流及び物資の輸送等において重要な役割を有する。この交通システムの中で、特に道路橋はその建設条件から地震や風の影響を受け易い。こうした点を考慮し、本作業部会では、地震や風に対する道路橋の挙動に関する調査研究を計画、推進し、発展させるとともに、それらの調査研究の成果に基づいて、道路橋の耐震、耐風に関するガイドラインや指針を作成することを目的とする。

２．活動の範囲

本作業部会では、道路橋の諸元と機能の多様性にかかわらず、あらゆる道路橋を主要な対象とし、既設橋の維持管理および新設橋の設計施工に関する事項の調査研究を行うものとする。本作業部会では、これらの調査研究を進めるために、橋梁の設計における数値計算や試験および橋梁の点検・補修補強・維持管理等に関してワークショップ・人事交流・共同研究などを実施するものとする。

３．成果（過去２年間）

日本側はＪ作業部会のもとで第１回日米橋梁補修補強ワークショップを１９９０年１２月１７日から２２日に開催し、耐震補強についての議論を行った。米側５名、日本側３

０名の研究者によって１９の発表が行われた。議題は橋梁の補修補強技術の開発の歴史、ロマプリエータ地震の被害評価、橋梁の耐震補強についての条件評価および優先度評価についての議論、耐震補強法、現在進行中の研究等である。首都高速道路の橋梁に適用された補強技術に関する視察旅行が行われた。ワークショップの論文集が両国に配布された。

・　Ｇ部会の協力の下で米国側は橋梁構造の地震応答制御に関するアドホックワークショップを１９９１年９月に国立地震工学研究センター（ＮＣＥＥＲ）において開催し、討議を行い、両国におけるアクティブ、パッシブおよびハイブリッドコントロールの実用性について検討を行った。参加者は米国の免震橋の視察を行った。

・　橋脚および柱の耐震性能に関して、動的繰返し荷重に対する鉄筋コンクリート橋脚や柱の耐荷性能、鉄筋コンクリート橋脚の破壊に関する模型実験、実物大および大型鉄筋コンクリート柱供試体の実験およびコンクリート充填鋼管柱の耐荷性能についての協同研究を実施した。

・　１９９２年５月にイリノイ州シカゴで第８回橋梁ワークショップを開催し、またミシシッピ河沿いに、アイオワ州バーリントン、イリノイ州クインシー，アルトン、およびミズーリ州セントルイスにおいて重要橋梁の現地視察を行った。ワークショップの主なテーマは、(1)長大橋、(2)風の影響および基準、(3)地震関係、(4)設計、(5)支承、(6)基礎、(7)評価、ランク付け、マネジメントである。１９９１年５月に日本つくば市で第７回橋梁ワークショップを開催し、また東神戸大橋、関西国際空港、明石海峡大橋の現場視察を行った。

・　日米両サイドは、第２３回及び第２４回ＵＪＮＲ合同部会において数多くの報告を行うとともに、建築物、橋梁、製作工場、建設現場および計測現場等へのスタディツアーを行った。

４．　将来計画

・　第９回橋梁ワークショップを第２５回合同部会に先だってつくば市で開催する。

・　橋梁の耐震補強に関する第２回日米ワークショップを米国において開催する計画である。このワークショップは耐震補強法の実用性に焦点をあてる予定である。

・　実験・数値解析および現場研究に基づく、改善された道路橋耐震補強についての調査や情報交換を継続する。情報交換には既設道路橋の維持管理についての情報を含む。

・　Ｇ作業部会と共同で橋脚および柱の耐震性能に関する実験的研究を継続するとともに免震技術およびハイブリッドコントロールに関する研究を行う。地震制御に関する第２回ワークショップを１９９２年１２月つくば市で計画している。

・　吊橋や斜張橋の耐震性能、空力弾性学特性および空気力学特性、ケーブルの検査技術および振動や腐食の防止対策についての共同研究を推進する。

・　日米双方における橋梁の耐震設計基準の比較のための共同研究を推進し、柱の設計法

および解析について議論を行い、限界状態設計法の適用についての情報交換を行う。

・　橋梁の地震応答制御、システム同定技術および非破壊診断技術に関しての共同研究を推進する。

・　新素材を含めた構造材料の利用と性能に関する共同研究を推進する。

・　ジョイントレス橋梁の挙動に関する共同研究を推進する。

5．情報交換

・　１９９０年フィリピン地震調査データを含む、技術報告、研究プログラム、建設計画、設計計画およびこれらに関する写真やビデオを交換した。

・　ＦＨＷＡはＰＷＲＩの河野氏、神崎氏、藤本氏、井上氏の訪問を受けた。井上氏は５ヶ月間Turner-Fairbank道路研究センターで研究に従事し、バージニア、メリーランド、ペンシルバニア、カリフォルニア州の道路関係機関を訪問して橋梁のメンテナンスとマネジメントについての研究を行った。井上氏は“道路橋のメンテナンスに関する日米比較”のレポートを作成した。

・　ＣＡＬＴＲＡＮＳのＭｏｈｓｅｎ　Ｓｕｌｔａｎ氏はつくばのＰＷＲＩとの１年間の人事交流において橋梁の耐震設計の日米比較の研究を行った。

6．活動の効果

・　文献、測定、技術および地震応答測定データの交換により、設計基準や補修指針の改善が図られた。

・　耐風に関する調査研究における試験方法やモデルの統一を図り、これにより耐風技術に関する諸問題に対する有効な方策を見出した。

・　日米双方における橋梁ワークショップやスタディツアーの実施により、斜張橋のケーブルの防食対策および振動防止対策をはじめとする多くの項目の技術交換ができた。

・　国際的な観点からその実施が必要とされる研究の優先性を明らかにすることにより、日米双方で同種の研究を行うことを省くことができた。

7．問題点

本作業部会における調査研究を進めるにあたっては、現在のところ特設の問題点は見あたらない。ただし、合同部会やワークショップへの参加、実橋を対象とした試験や大型モデル供試体を用いた実験の遂行にあたって、これらの予算の不足が続いており、本作業部会の活動が制限される状況にある。

Report of Task Committee on

(J) WIND AND EARTHQUAKE ENGINEERING FOR TRANSPORTATION SYSTEMS

Date: May 19, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees: U.S. Side - J. D. Cooper (Chairman) (FHWA)
C. Barrientos (NOAA)
R. Dikkers (NIST)
J. Scalzi (NSF)
H. S. Lew (NIST)

Japan side - M. Fujiwara (Chairman) (PWRI)
O. Nakano (Hokaido Development Bureau)

MISSION STATEMENT

1. Objective:

The objectives of this Task Committee are to plan, promote, and foster research on the behavior of highway bridges when subjected to wind and seismic forces and to disseminate research results and provide specifications and guidelines based on the Task Committee's findings. Surface transportation systems play a vital role in the movement of goods and people. Highway bridges are especially influenced by the forces of wind and earthquakes because of their open exposure to those forces.

2. Scope of Work:

The scope of work is applicable mostly to highway bridges without limitation on their size and function including: existing bridge and new bridge designs; whole system of bridges; and/or single components of a bridge.

The mission is performed through:

Conducting workshops, exchanging researchers, developing methods for design evaluations and test procedures, developing inspection techniques, rehabilitation and maintenance specifications and policies, and performing cooperative research programs and other relevant cooperative administrative activities.

3. Accomplishments (past 2 years)

The Japanese side hosted the first U.S.-Japan Workshop on Seismic Retrofit for Bridges under the auspices of T/C "J" December 17 to 22, 1990 to discuss and review practical seismic retrofit applications in both countries. Nineteen technical presentations were made by 5 U.S. researchers and 30 Japanese researchers. Technical topics include the history of developing retrofit techniques for bridges; damage assessment from the Loma Prieta earthquake; discussion on condition assessment and prioritization of bridges for seismic retrofit; seismic strengthening methods; and research in progress. Site visits were made to view retrofit technology applied to bridges on Tokyo metropolitan expressways. Workshop Proceedings were disseminated in both countries.

In cooperation with T/C "G", the U.S. side hosted an Ad Hoc Workshop on Seismic Response Control of Bridge Structures in September 1991 at the National Center for Earthquake Engineering Research to continue discussion and review practical seismic active, passive and hybrid control applications in both countries. Participants toured seismically isolated bridges in the U.S.

Conducted coordinated research studies on seismic performance of bridge piers and columns. Details of this work involved determining the performance of reinforced concrete piers and columns subjected to dynamic cyclic loading, performing model tests on the failure of reinforced concrete piers, testing full and large-scale concrete columns and for the behavior of concrete filled steel tubes.

Held the eighth bridge workshop in Chicago, Illinois in May 1992 which was followed by site visits to major bridges along the Mississippi River at Burlington, Iowa; Quincy, Illinois; Alton, Illinois; and St. Louis, Missouri. Main themes of the workshop included: (1) Long-Span Bridges; (2) Wind Effects and Codes; (3) Seismic Issues; (4) Design; (5) Bearings; (6) Foundations; and (7) Evaluation, Rating and Management. Held the seventh bridge workshop in Tsukuba, Japan in May 1991, which was followed by site visits of Higashi Kobe Bridge, Kansai International Airport, and Akashi Kaikyo Bridge.

Both sides presented technical papers at the 23rd and 24th UJNR Joint Meetings and participated in technical study tours to buildings, bridges, factories, construction sites, and instrumented structures.

4. Future Plans

* Conduct the 9th bridge workshop just prior to the 25th Joint Panel

meeting in Tsukuba, Japan.

* Plan to host a 2nd U.S.-Japan Workshop on Seismic Retrofit of Bridges in the U.S. The Workshop will focus on practical seismic retrofit measures.

* Continue to investigate and exchange technical information on improved seismic retrofit and strengthening procedures for highway bridges based on experimental, analytical, and field studies. This exchange should include information on maintenance of existing bridges.

* Continue experimental research study on the seismic performance of bridge piers and columns, and support emphasis on base isolation and hybrid control of bridges in cooperation with T/C "G". The 2nd Workshop on seismic control is planned for December 1992 in Tsukuba.

* Develop a coordinated research study on seismic, aeroelastic, and aerodynamic response of cable-supported bridges with emphasis on cable inspection, vibration, control and corrosion protection.

* Continue a coordinated research study to compare the seismic design criteria for bridges in Japan and the U.S. and discuss the method and analysis procedures for bridge column design. Information on application of limit state design method continues to be exchanged.

* Encourage a coordinated research study on seismic response and control, system identification techniques, and nondestructive evaluation of bridge structures.

* Encourage a coordinated research study on use and performance of structural materials including new materials.

* Encourage a coordinated study on performance of jointless bridges.

5. Information Exchange

* Exchanged technical reports, research program documentation, construction logs, design plans, and assorted photographs and videos, including technical data appropriate for investigating the Philippine earthquake of 1990.

* FHWA hosted PWRI visitors Messrs. K. Kohno, H. Kanzaki, A. Fujimoto, and J. Inoue. Mr. Inoue studied for five months at the Turner-Fairbank Highway Research Center and visited State highway agencies in Virginia, Maryland, Pennsylvania and California to study bridge maintenance and management. Mr. Inoue prepared a report entitled "Comparison of

Highway Bridge Maintenance Between the U.S. and Japan".

* Mr. Mohsen Sultan, California Department of Transportation conducted a study on the comparison of Seismic Design for U.S. and Japanese bridges during a one year exchange with PWRI in Tsukuba.

6. Impact

* The free exchange of literature, instrumentation technology, and earthquake response data has led to the advancement in design technology, and development of preliminary retrofit guidelines, and to the calibration and verification of specifications.

* Greater uniformity was achieved in wind engineering test procedures and modeling resulting in more efficient solutions to the aerodynamic bridge problem.

* The workshop and study tours in the two countries have facilitated technology transfer. Examples include retrofit technology, cable protection and vibration suppression of cable-stayed bridges.

* A set of priorities for needed research from a more international viewpoint has reduced the conduct of duplicative research efforts.

7. Barriers

There are no major technical barriers to achieve the Task Committee mission. However, financial barriers, especially funding needed to participate in workshops and panel meetings and for conducting actual bridge tests and large-scale model tests, continue to limit the effectiveness of the work in this Task Committee's work.

（K）海洋施設の耐風耐震技術に関する作業部会報告

開　催　日　：１９９２年５月２０日
場　　所　：国立標準技術研究所　ゲイザーズバーグ市
出　席　者　：日本側　野田節男（臨時作業部会長）（港湾技術研究所）
　　横山功一　（土木研究所）
　　古関潤一　（土木研究所）

　　米国側　C. Smith（作業部会長）　（NMS）
　　G. Brady　（USGS）
　　W. Roper　（COE）
　　M. Miller　（WES）

［活動要約］

1．目的

海洋施設の設計基準は陸上施設のそれとはかなり異なっている。この差は、水－構造物あるいは風－構造物の相互作用、地震時に構造物に作用する動水圧、軟弱な基礎海底地盤、海洋施設の環境データの不足、陸上施設と比較して不十分な工学的な経験等によるものと考えられる。

本作業部会では、風、地震による海洋施設の災害防止のための技術的な調査・研究を行うことを目的とする。本作業部会はこの目的にかなった調査・研究を計画、助成、実行し、これらの研究成果を将来の設計基準に生かす活動を行う。

2．活動の範囲

本作業部会の活動の範囲は理学から工学までの多岐にわたっており、地球科学、構造工学、機械工学、海洋工学等における検討を行う。対象とする構造物は石油掘削施設、航行援助施設、防波堤、岸壁、海岸保全施設等である。本作業部会は上述の様々の分野の情報交換を実施する場を提供する。具体的な検討課題は下記の通りである。

・主要課題の摘出のための会議やワークショップ、研究協力、情報交換を支援、実行する。

・海底地震観測結果により海底地形、ローカルな地盤条件が海底地震動に与える影響を

検討し、海洋施設の設計のための強震動を推定する。

・地震、風による動的応答、構造物－基礎－地盤系の動的相互作用を検討する。

・海洋施設内の各種操作機器の動的応答を推定する。

・震源（断層）の特性、地震活動度を海洋施設の設計にどのように反映させるかを検討し、その推定手法を開発する。

・新しい研究成果の現行設計法、施工法への活用を促進する。

・強風、強震動に対する海洋施設の動的応答に関するデータ取得のための室内実験、現地観測を推し進める。

・新設、あるいは既設構造物の補修に適用するための作業基準、設計基準、ガイドライン、コードを作成する。

3．成果

・本作業部会の活動の範囲、活動計画を作成した。

4．将来計画

・第２５回ＵＪＮＲ合同部会の前に、横須賀の港湾技術研究所で海洋施設の耐風・耐震技術に関する日米ワークショップ開催する。

・ワークショップの課題と日米両国の発表者を決定する。

・作業部会のメンバーが関係している海洋施設の耐風・耐震工学に関する研究を調整する。

5．情報交換

・合同部会、作業部会での日米両メンバーによる情報交換

・国際会議、国際会議における各セッション、セミナー、ワークショップを通じての工学研究者、科学者間の情報交換

6．活動の効果

・海洋施設の耐震設計における科学者、エンジニアの間の協力を促進する。

・海洋施設に影響を与える地震の危険度、問題点の認識を深めさせる。

7．問題点

・研究を強力に支援し、研究者の交換のための旅費を援助する予算が不十分である。

・海洋施設の地震による危険度に関する調査・研究の重要性の認識が、特に米国側において不足している。

Report of Task Committee on

(K) WIND AND EARTHQUAKE ENGINEERING FOR OFFSHORE AND COASTAL FACILITIES

Date: May 20, 1992

Place: National Institute of Standards and Technology
Gaithersburg, MD

Attendees:			
	U.S. Side -	C. Smith (Chairman)	(MMS)
		G. Brady	(USGS)
		W. Roper	(COE)
		M. Miller	(WES)
	Japan Side -	S. Noda (Temporary Chairman)	(PHRI)
		K. Yokoyama	(PWRI)
		J. Koseki	(PWRI)

MISSION STATEMENT

1. Objective:

The objective of this Task Committee is to develop technical insights necessary to mitigate damage to offshore and coastal facilities due to extreme wind and seismic effects. The Task Committee will plan, promote, and develop research initiatives to meet this objective, and will disseminate the results of their research for incorporation into future specification or design guidelines.

Criteria for the design of offshore and coastal facilities may differ greatly from their onshore counterparts. These differences can arise due to their unique design or mass distribution, to the fluid/structure or wind/structure interaction, to the placement of foundation elements in or on soft, fully saturated soils that can be subject to large hydrodynamic pressures, and to the lack of specific environmental data or the engineering experience that has been developed for most onshore sites.

2. Scope of Work

The Task Committee's work spans a broad spectrum of scientific and engineering disciplines to include a variety of geoscience, structural, mechanical, and marine topics. Structures of interest include offshore oil and gas facilities, aids to navigation, breakwaters, bulkheads, and other coastal facilities. The Task Committee will provide

the forum for the communication between the above disciplines as they impact the stated objective. Items of specific interest include the following:

* Sponsoring and conducting workshops and meetings to identify key areas of research, opportunities for cooperation, and the exchange of knowledge.

* Predicting strong ground motions for offshore and coastal sites including assessing the effects of basin geometry, linear, and nonlinear local geological effects using actual seafloor response measurements.

* Determining the dynamic response and the interaction of structure/foundation/soil systems to seabed motions and/or extreme wind forces.

* Assessing the dynamic response and behavior of various operational facilities mounted on offshore and coastal structures.

* Developing assessment methodologies for seismicity and other characteristics of potential seismic sources (e.g. faults) for offshore and coastal sites in regards to how these conditions relate to structural design criteria.

* Promoting the implementation of new research results into current design and construction processes.

* Developing research efforts to include laboratory and field programs to obtain data on the response of offshore and coastal facilities to extreme wind and seismic forces.

* Creating performance standards, design specifications, guidelines, and code recommendations for application to new construction as well as remedial action for existing facilities.

3. Accomplishments

* Established T/C scope of work and T/C operational plan.

4. Future Plans

* Hold a U.S./Japan Workshop on Wind and Earthquake Engineering for Offshore and Coastal Facilities the week before the 25th Joint UJNR Meeting at the Port and Harbour Research Institute in Yokosuka, Japan.

* Develop workshop agenda and finalize list of speakers from both the U.S. and Japan Side.

Coordinate, where possible, on-going research on wind and earthquake engineering for offshore and coastal facilities of interest to the members of the Task Committee.

5. Information Exchange

With Japan and U.S. members through T/C meetings and annual joint meetings.

With the Engineering and Scientific community through meetings, conference sessions, seminars, and workshops.

6. Impact

Increased cooperation among scientists and engineers concerned with the seismic design of offshore and coastal facilities.

Increased awareness of problems and risks associated with seismic events affecting offshore and coastal facilities.

7. Barriers

Insufficient funding to sponsor a strong research initiative and to support travel and the exchange of personnel.

Not enough awareness, especially on the U.S.-side of the importance of investigating the risks associated with seismic concerns for offshore and coastal facilities.

VII. 課題、論文及び討議

課題Ⅰ．耐風工学

超長大吊橋のフラッター特性

横山　功一[1]、金崎　智樹[2]、保田　雅彦[3]

【要　約】

タコマ・ナロウズ橋の落橋以来、長大吊橋などの変形し易い構造物の設計では、風洞試験により耐風安定性を確認することが行われてきた。我国では、長大橋の耐風安定性の確認は、部分模型を用いたバネ支持模型試験によるのが一般的であり、バネ支持条件としては、実橋の最低時のねじれ振動と、それに近いモード形状のたわみ振動とが対象とされる。吊橋のような3次元構造物の振動が部分模型という2次元剛体の振動に置き換えられるということは、橋軸方向に空気力が一定であり、かつねじれ振動とたわみ振動のモード形状もほぼ等しいというブライヒ（F.Bleich）の仮定のもとに導かれたものである。

ところが、明石海峡大橋の全橋模型試験で観測された連成フラッターは、これまでの概念とまったく異なり、たわみモード形状が非常に複雑な連成フラッターであった。これは、バネ支持模型試験による耐風安定性の照査方法の限界を示したものであるといえる。

本文では、一様流中でみられたフラッター特性を分析し、バネ支持試験による耐風安定性評価手法の問題点を整理するとともに、今後の検討方針について報告する。

【キーワード】

超長大吊橋、耐風安定性、連成フラッター、
全橋模型、風洞試験

1．まえがき

明石海峡大橋は神戸市と淡路島の間に建設されている全長3，９１０mの超長大吊橋であり、中央支間長１，９９０mは、現在世界最長のハンバー橋を一挙に１．５倍近く凌ぐこととなる。図－１に明石海峡大橋の一般図を示す。

図－２に吊橋のスパンと振動数および暴風時水平たわみ量の関係を示すが、スパンが大きくなるに従い振動数が低下し、暴風時の水平たわみ量が急激に増大することが分かる。このため、本橋の耐風安定性を確保する事は従来の長大吊橋に比べてはるかに難しく、風荷重による変形量を精度良く評価するということはこれまで以上に重要であると認識されている。

吊橋の耐風安定性を検証するための風洞試験法には、①バネ支持試験、②タウトストリップ模型試験および③全橋模型試験があり、精度の高い模型が製作されれば全橋模型試験の信頼度が最も高い。

そこで、従来の長大吊橋のように部分模型による耐風性の評価だけでなく、できるだけ実橋を忠実に再現した全橋模型を用いて風洞試験を行うことにより本橋の耐風性を検証することとして、現在大型風洞施設において実験が進められているところである。実験は、一様流中での試験、乱流中での試験が終了したところであり、今後さらに模型を旋回させた斜風試験、架設時の試験を予定している。

ここでは、一様流中試験でみられた連成フラッターに着目し、バネ支持試験結果のみから超長大橋の耐風安定性を評価することが困難であることを示した。

なお、一連の実験は、建設省土木研究所と本州四国連絡橋公団とによる「超長大橋梁の耐風設計法に関する共同研究」の一環として行われているものである。

2．目的

これまで我国では橋梁の耐風性を調査する場合、部分模型を用いたバネ支持風洞試験が実施されてきた。吊橋のような３次元構造物の振動が部分模型という２次元剛体の振動に置き換えられるということは、橋軸方向に空気力が一定であり、かつ、ねじれ振動とたわみ振動のモード形状もほぼ等しいという仮定（ブライヒの仮定）のもとに導かれたものであり[1]、この仮定は、吊橋の架設中や部分的に遮音壁を取り付けた吊橋を除き、完成後の吊橋に関してはほぼ妥当な仮定であるとされてきた。これにより、最低次のねじれおよびたわみ振動（通常は対称１次モード）を対象としたバネ支持模型試験により、完成後の吊橋の耐風安定性が確認できるとされてきたわけである。

ところが、今回実施した明石海峡大橋の全橋模型を用いた一様流中での風洞試験結果では、このような仮定に反する結果が顕著に認められた。

そこで、試験結果を分析し、現行の部分模型試験による耐風性の評価方法の問題点を考察することとする。

3．試験条件

試験を実施した大型風洞施設（測定部断面：幅 41m × 高さ 4m）における気流は、主流方向の乱れ強さが０．５％以下であり、断面内の平均風速分布の偏差は

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3)本州四国連絡橋公団設計部設計第一課長

±３％以内である。従って、気流の一様性は十分保たれている。大型風洞施設の一般図を図－３に示す。

全橋模型は縮尺１／１００で製作されており、全長約４０ｍである。全橋模型の概要を図－４に示す。模型設計については昨年度報告した[1]ので、ここでは模型の振動試験において計測された各モードの振動数を所要値と比較して表－１に示す。なお、吊橋の全橋模型風洞試験ではフルード数を相似させる必要があり、風洞風速の１０倍が実橋換算風速となり、振動数の所要値は実橋の解析結果の１０倍となる。振動数の計測結果から十分高次モードまで相似されており、精度の高い模型であることがわかる。

４．試験結果

以下に基本断面（図－５参照）の全橋模型風洞試験結果をバネ支持試験結果と比較して報告する。

４．１　静的ねじれ変形

風速と中央径間中央のねじれ変形量の関係を図－６に示す。本橋の設計風速である６０ｍ／ｓ（風洞風速６ｍ／ｓ）において約－２度（頭下げ）のねじれ変形が生じている。このねじれ変形量は、従来の吊橋で考えられていた値をはるかに超えるものである。既に行われているバネ支持試験の結果より、本橋補剛トラス断面のフラッター特性は、迎角の影響を比較的強く受けることが分かっており、このことは、ブライヒの橋軸方向に空気力が一定であるという仮定が既に成り立たなくなっていることを示している。

なお、今回の実験と解析の過程において、吊橋補剛桁の風によるねじれ変形量は、定常空気力のモーメント成分により発生する変形よりも、抗力により補剛桁が水平方向にたわんだ際に、ハンガーが補剛桁を引き戻そうとする力がトルクとなって発生する変形量が支配的であることが分かった（図－７参照）。

また、模型のねじれ変形量と実橋の解析結果とでは図－６に示す様に若干の差が見られる。これは主に模型製作上の制約によるものであり、具体的には、①塔の伸び剛性が相似できないこと、②ハンガーの補剛桁への取付部の剛性により見かけ上のハンガー取付位置が高くなっていること、さらに③桁の剛性を相似させるためのＶ字型バネが水平たわみにより力を受け、ねじれ剛性が低下すること等によるものである。

なお、今回の全橋模型試験結果については、上述したねじれ変形量の誤差を解析により補正することとしている。

４．２　フラッター特性

４．２．１　減衰率の変化

風速と減衰率の関係を図－８に示す。ねじれ振動の減衰率は、風速７ｍ／ｓ程度まではほぼ一定であり、７ｍ／ｓを超えたあたりから急激に低下している点が特徴的である。減衰率が正から負に変化する風速８．５ｍ／ｓ（実橋換算で８５ｍ／ｓ）がこの断面のフラッター発現風速である。ただし、ここでのねじれ振動は無風時と同じ純粋なねじれ振動ではなく、比較的低風速からたわみ振動をともなった連成振動である。

たわみ振動の減衰率は風速とともに急激に大きくなっている。

一方、迎角０度のバネ支持試験結果では大型風洞の風速に換算して２ｍ／ｓ程度から徐々にねじれの減衰率が低下し、６ｍ／ｓでフラッターが発現した。

風洞試験では、まず空気力により構造系が負減衰となるフラッター発現風速が問題となる。静的ねじれ変形の影響があり、バネ支持試験結果と全橋模型試験結果とを単純に比較することはできないが、両試験のフラッター発現風速の一致はよくない。

４．２．２　振動数の変化

風速と振動数の関係を図－９に示す。ねじれ振動数は風速の増加とともに減少し、フラッター発現風速においては、無風時の振動数の約９０％まで低下する。一方、たわみの振動数は、ほとんど変化していない。

ねじれ振動数の低下に関しては、バネ支持試験結果とほぼ同じ傾向であった。

４．２．３　回転中心の移動量

フラッター発生時の回転中心の移動を模式的に図－１０に示す。中央径間中央付近では回転中心が上流側に移動しているのに対し、中央径間１／４点付近では回転中心が下流側へ移動しており、さらに側径間では再び上流側へ移動している。ブライヒの仮定によれば、橋軸方向に一定でなければならない回転中心の移動量が明らかに変化していることになる。

なお、バネ支持試験でみられたフラッターは、回転中心が上流側に移動した連成フラッターであり、その挙動は全橋模型の中央径間中央のそれに類似していた。

４．２．４　フラッター時のモード

フラッター発生時におけるねじれおよびたわみ振動モードを固有周期の１／８時間ステップ毎に図－１１に示す。また、同図には無風時のねじれおよびたわみの対称１次モードも併せて示す。ねじれ振動モードに関しては、フラッター時もほぼ無風時のねじれ対称１次モードと同じモードであった。しかしながら、たわみ振動モードは、無風時の単一のモードではなく、複雑なモードが現れている。

なお、気流に対して頭上げをねじれの正、鉛直下向きをたわみの正と定義すると、中央径間中央付近および側径間ではたわみ振動が約２０度ねじれ振動に先行していた。

この位相差は、中央径間中央付近では回転中心が上

流側に移動していることと対応する。ただし、中央径間ではねじれ振動が卓越しているのに対し、側径間ではたわみ振動が卓越しているために、側径間では移動量が大きく、回転中心が桁幅よりも外に出てしまっている。したがって、側径間の振動は見かけ上たわみ振動のように見えるものであった。

ところが、中央径間の１／４点付近では、位相差が１８０度に近づいている。これは、回転中心が風下側に移動した事実と一致する結果ではあるが、このように橋軸方向に位相差が変化することもブライヒの仮定に反するものである。

５．考察および今後の課題

明石海峡大橋の全橋模型試験結果では、「部分模型を用いた２自由度のバネ支持試験により、３次元構造物である吊橋の耐風安定性が評価できる」とするブライヒの仮定に著しく反するフラッター時の挙動が観測された。すなわち、ねじれ振動モードはフラッター時も無風時と変化しないが、たわみ振動モードはねじれ振動モードと類似したモードではなく、まったく別のしかも無風時の単一モードではない複雑なモードが現れるということである。このため、バネ支持試験で得られたフラッター発現風速が、実橋のフラッター発現風速の精度のよい推定値となっていない可能性がある。

この原因をブライヒの仮定にさかのぼって考えると以下の原因が考えられる。

（１）空気力が橋軸方向に一定であるという仮定は、静的ねじれ変形により大きく乱されている可能性がある。すなわち、静的ねじれ変形により全橋模型では気流に対する迎角が橋軸方向に大きく変化している。

暴風時のねじれ変形量は、従来の１，０００ｍ級吊橋ではわずかであり、耐風性を評価する上でそれほど重要視されていたとは言えない。しかしながら、今回の試験及び解析により、２，０００ｍ級の吊橋では従来考えられていた以上に大きなねじれ変形が生じる可能性が示された。したがって、このねじれ変形量が耐風安定性に及ぼす影響に対しては、十分考慮する必要があると考えられる。

（２）無風時のねじれとたわみの振動モード形状が等しいという仮定は、図－１１に示したように中央径間に関してはほぼ成立するものの、側径間では成立せず、たわみ振動の比率に比べねじれ振動は非常に小さい。

これら（１）、（２）の原因により、全橋模型試験では、バネ支持試験結果から予想されたねじれとたわみの一次モードからなる連成フラッターとは異なり、たわみ振動モードが複雑な連成フラッターが観測されたものと考えられる。

また、今回対象としたフラッターが従来のトラス補剛吊橋でみられたねじれフラッターではなく、連成フラッターであることもバネ支持試験結果と全橋模型試験結果とでフラッター発現風速が異なる要因であったと考えられる。したがって、バネ支持試験結果のみから超長大橋のフラッター特性を推定することには限界があるものと言えよう。

今後は、別途計測した非定常空気力を用い、最近提案されている多くのモードを考慮したフラッター解析手法等による解析も併せて行う予定である。バネ支持試験結果と全橋模型試験結果の差の要因を分析し、それぞれの要因の影響を把握することにより、部分模型試験の適用範囲を明確にし、実橋の耐風性をこれまでよりも精度よく推定することが可能になるものと考えられる。

６．まとめ

大型風洞施設を用いた明石海峡大橋の全橋模型風洞試験では、バネ支持試験結果からは予測されなかった複雑な振動性状の連成フラッターが観測され、フラッター発現風速も両試験で一致していない。すなわち、超長大橋の耐風安定性を評価するためには、バネ支持試験のみでは不十分であり、全橋模型試験を実施する意義が大きいということである。

部分模型試験結果と全橋模型試験結果の差の要因としては、ブライヒの仮定より以下が考えられる。

１）静的ねじれ変形量が大きく、相対迎角が橋軸方向に変化しているため、空気力が橋軸方向に一定でない。

２）ねじれ対称１次モードとたわみ対称１次モードの形状の差が大きい。

これらの要因は、いずれも明石海峡大橋が従来の吊橋に比べはるかにスパンが大きいことに起因している。

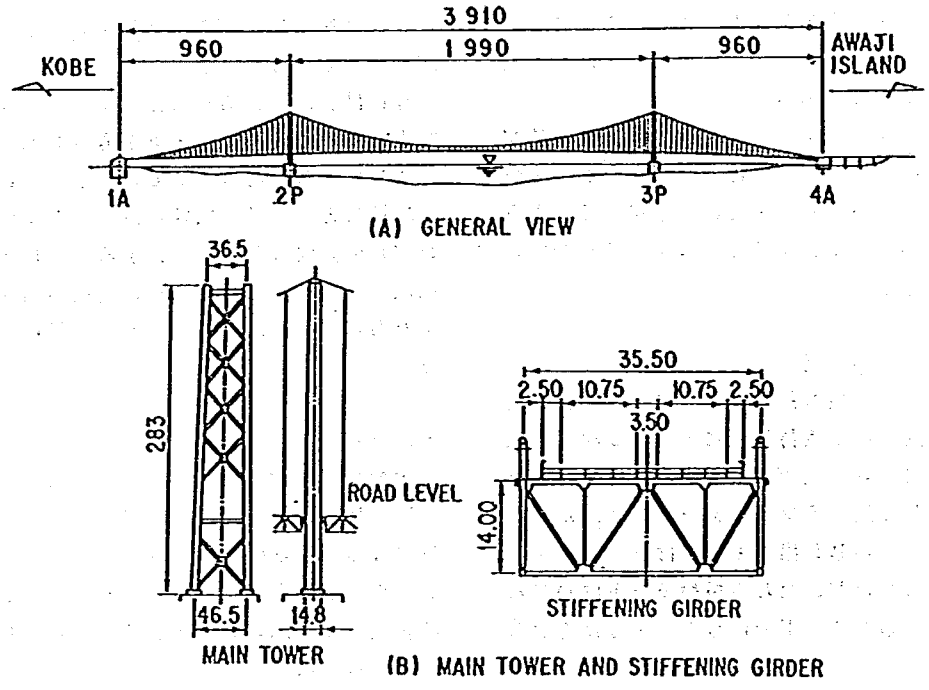
今後は、全橋模型試験でみられたフラッター特性を正確に推定できる理論を構築することにより、これまで以上に精度のよい超長大橋の耐風性評価手法を提案していきたい。

＜参考文献＞

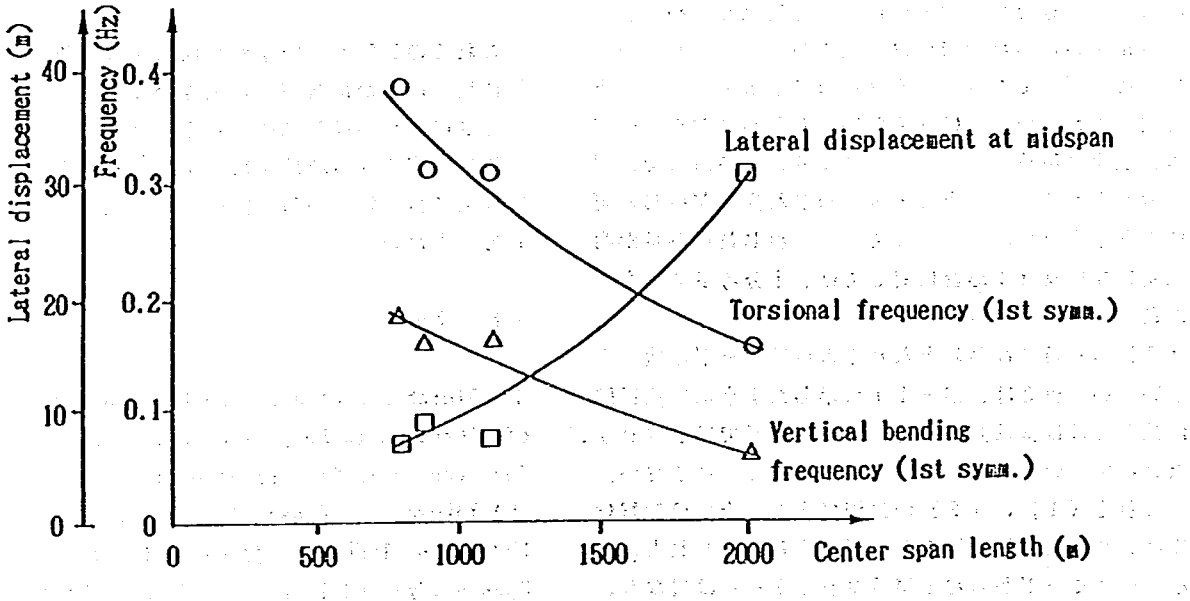
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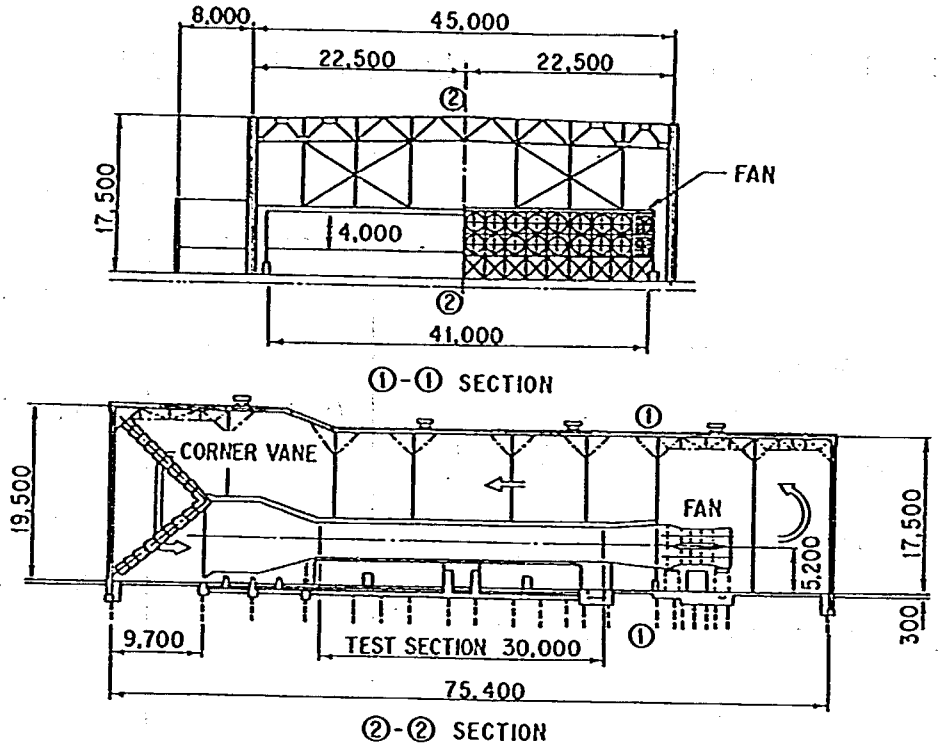
３）Miyata T., Yokoyama K., Yasuda M., Hikami Y., Akashi Kaikyo Bridge: Wind effects and full model wind tunnel tests, Proc. 1st ISALB, Copenhagen, 1992.



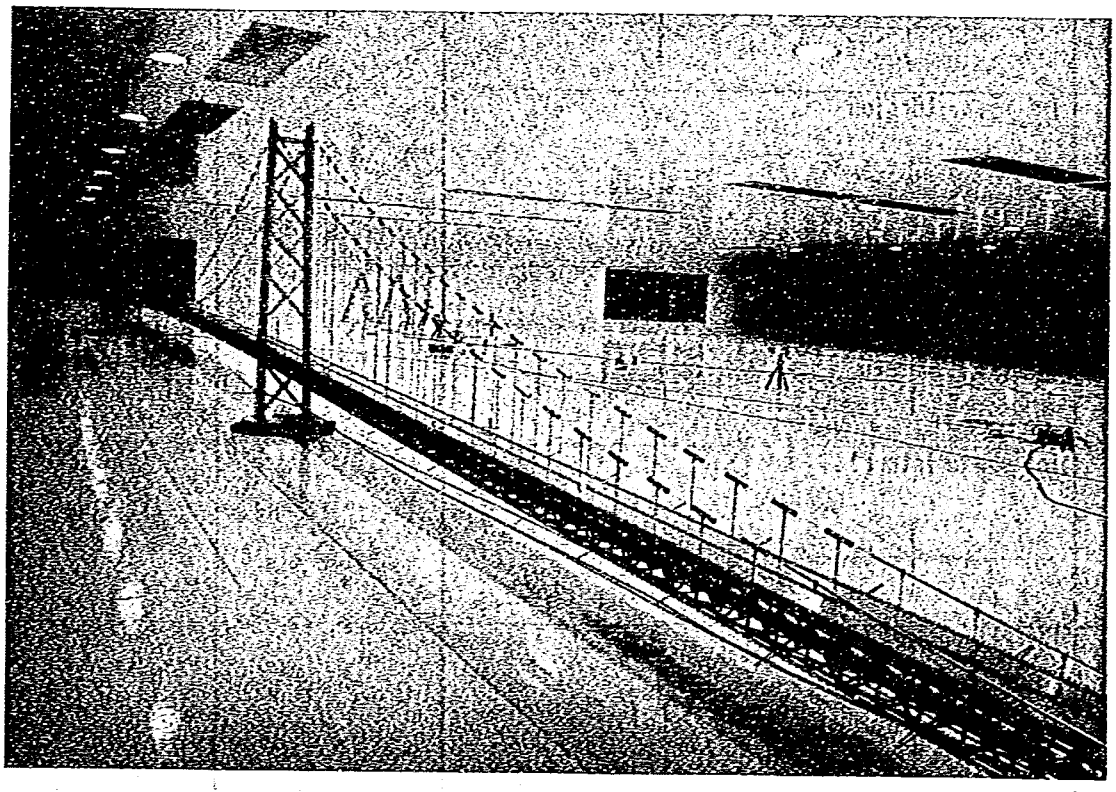
Figure—1 General veiw of the Akashi Kaikyo Bridge (unit:m)



Figure—2 Structural properties of long-span bridges

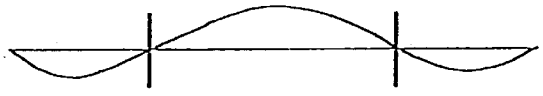
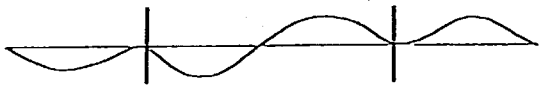
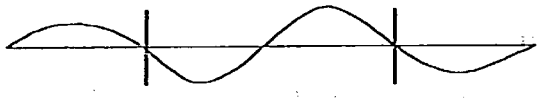
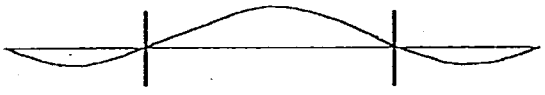
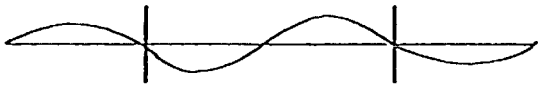
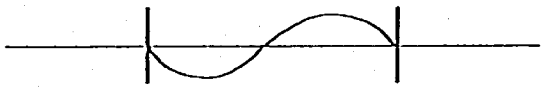


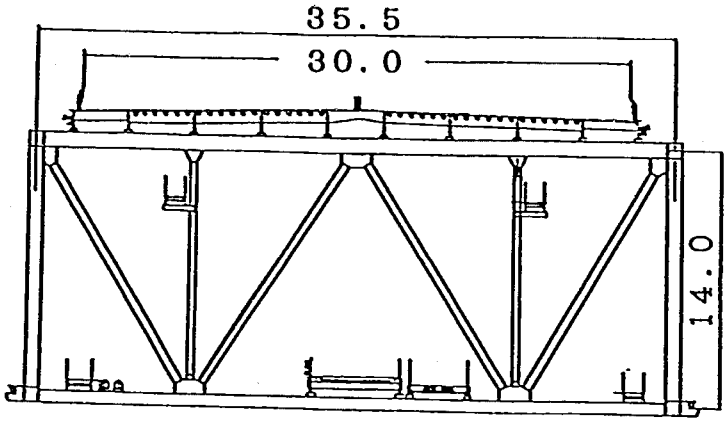
Figure—3 General layout of large boundary layer wind tunnel (unit:mm)



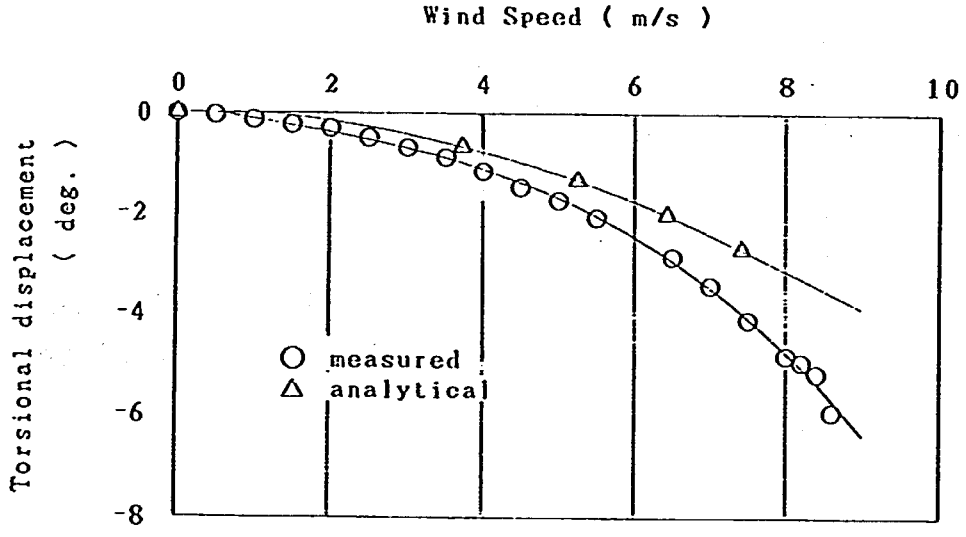
Figure—4 Full bridge model of the Akashi Kaikyo Bridge

Table—1 Natural frequency of the full bridge model

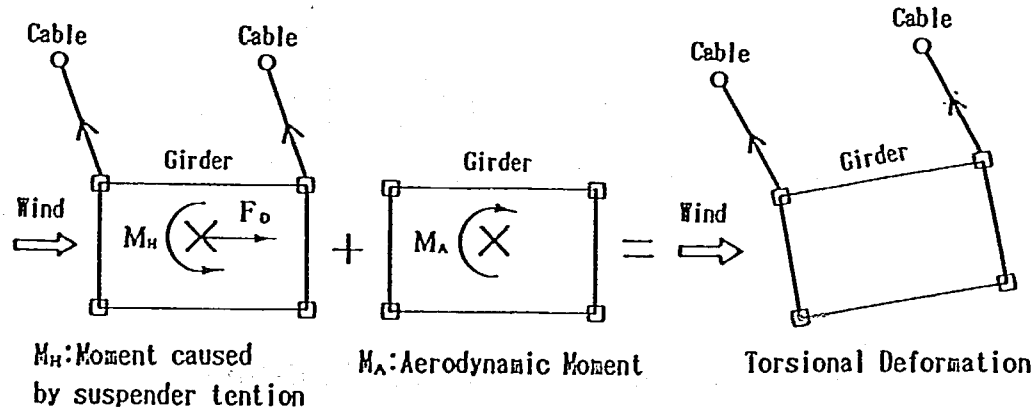
No.	Mode	frequency	
		required	full model
1	1st sym. bending 	0.638 Hz	0.647 Hz
2	1st anti-sym. bending 	0.745 Hz	0.757 Hz
3	2nd anti-sym. bending 	0.835 Hz	0.855 Hz
4	2nd sym. bending 	1.213 Hz	1.245 Hz
5	1st sym. torsion 	1.497 Hz	1.557 Hz
6	1st anti-sym. torsion 	2.077 Hz	2.075 Hz
7	1st sym. lateral 	0.387 Hz	0.391 Hz
8	1st anti-sym. lateral 	0.775 Hz	0.781 Hz



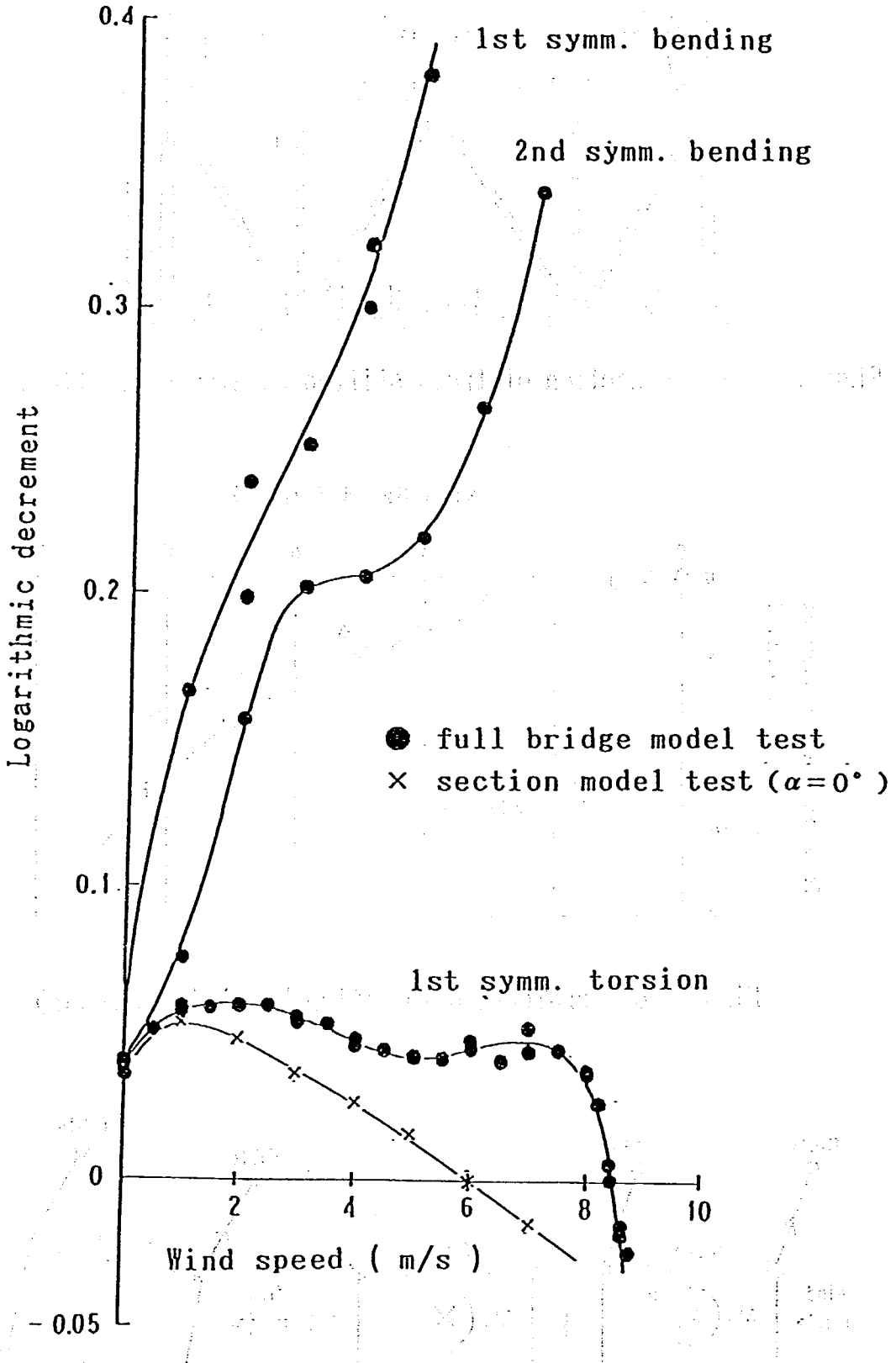
Figure—5 Cross section of truss stiffening girder (unit:cm)



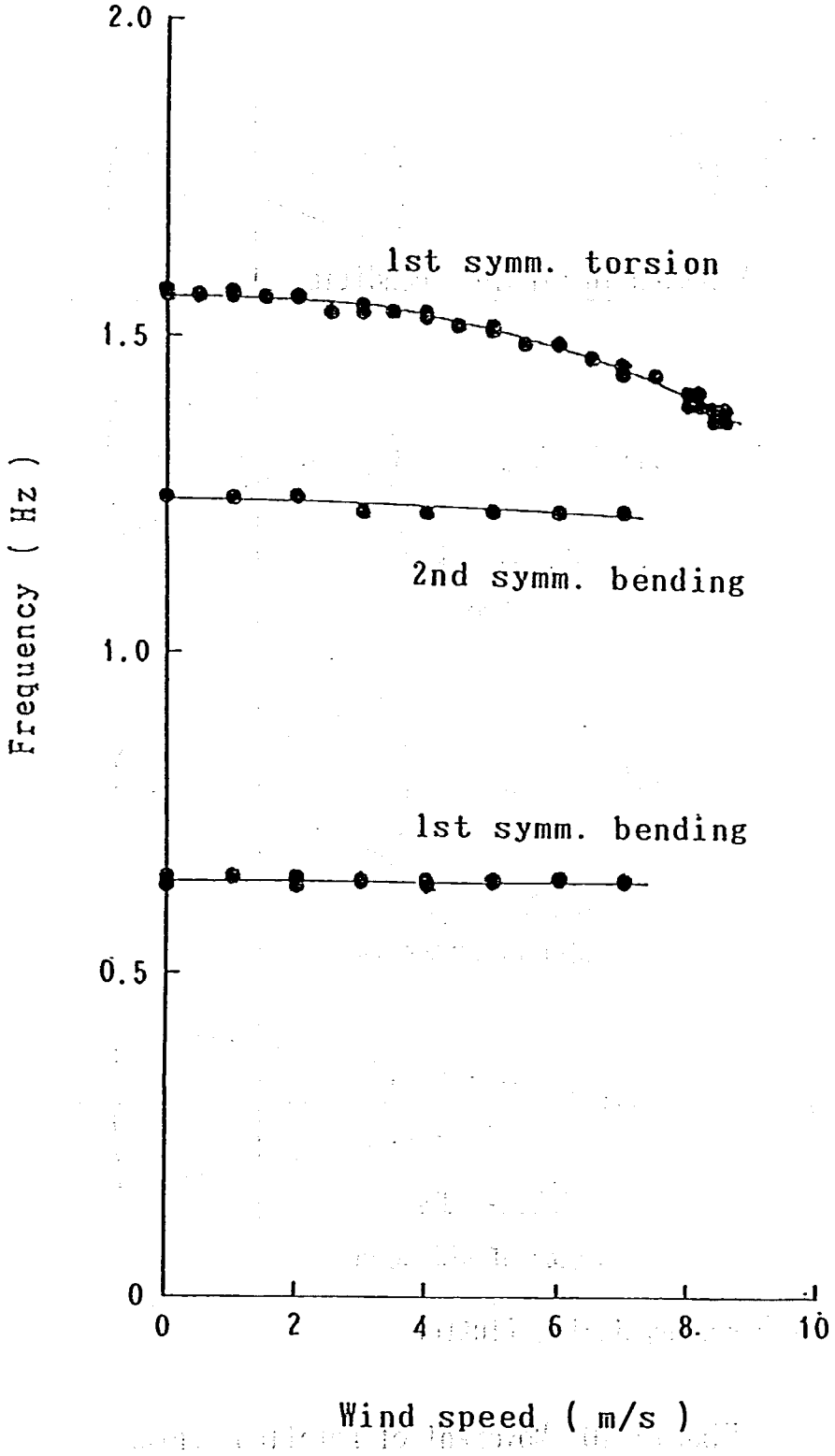
Figure—6 Torsional deformation by wind (midspan)



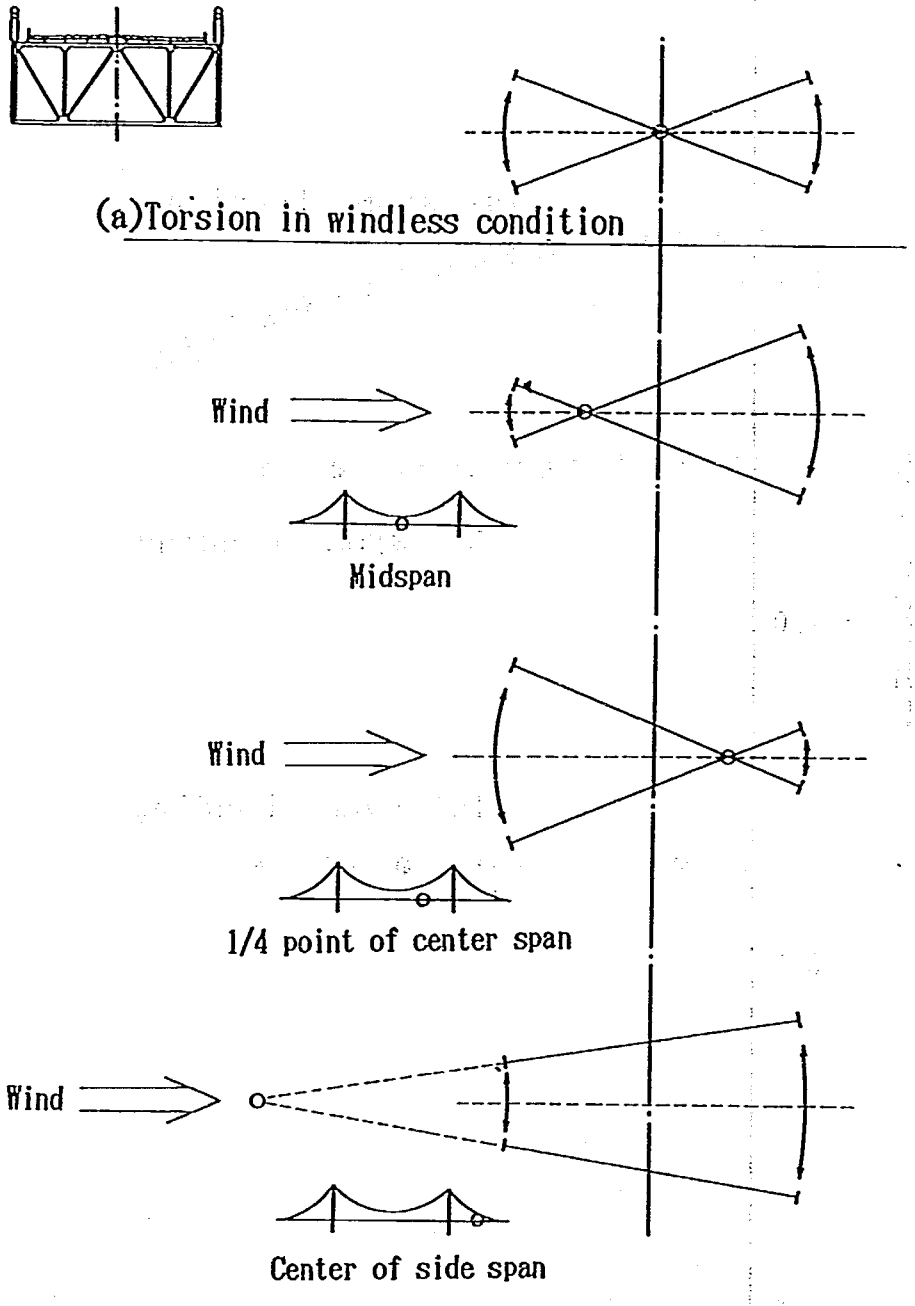
Figure—7 Moment caused by lateral deflection



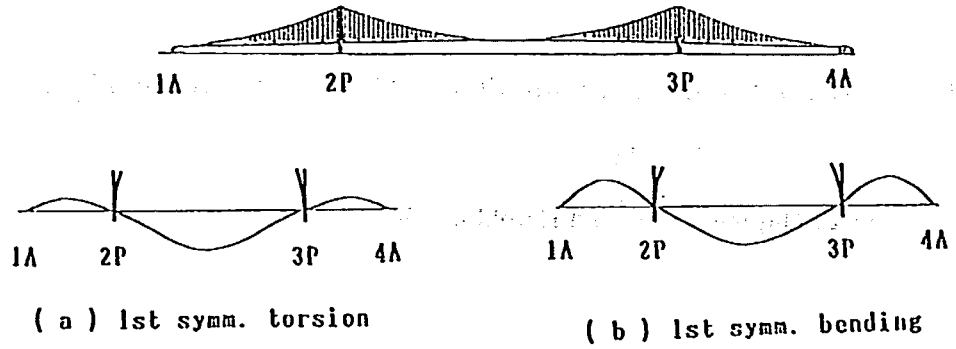
Figure—8 Change of aerodynamic damping



Figure—9 Change of response frequency



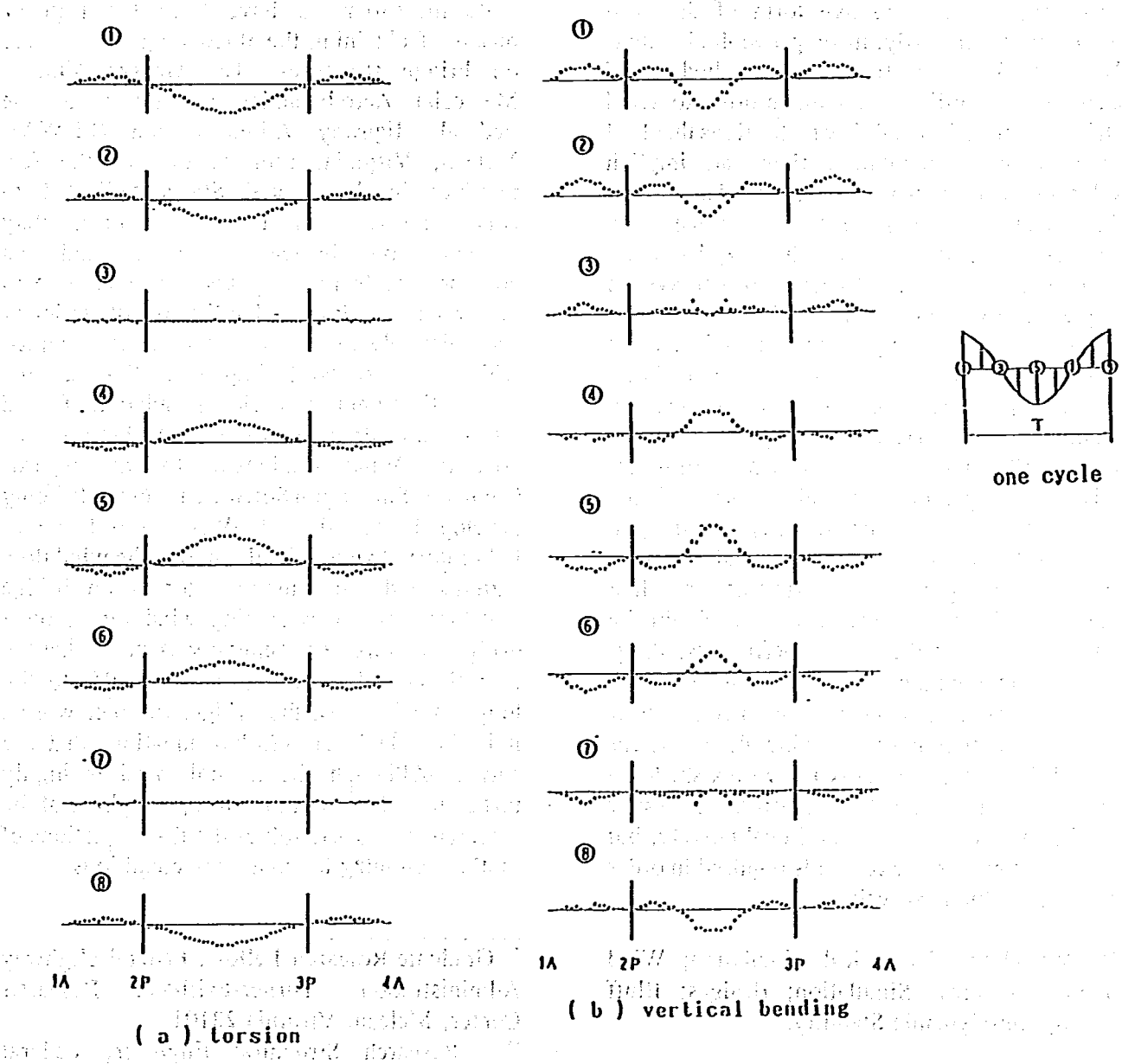
Figure—10 Movement of rotation center



(a) 1st symm. torsion

(b) 1st symm. bending

(1) Mode shapes in windless condition



(a) torsion

(b) vertical bending

(2) Response during flutter

Figure—11 Response of girder in one cycle during flutter

Predicting Wind Forces Using Computer Simulation of Air Flow Around Bridges

by

Okey Onyemelukwe* and Harold Bosch**

ABSTRACT

The finite-difference procedure has been used to solve the non-conservative form of the two-dimensional, unsteady, incompressible, laminar, Navier-Stokes equations in a body-fitted curvilinear coordinate system. In order to avoid stringent numerical stability restrictions that lead to excessive computation time, an implicit solution scheme was employed. The finite-difference mesh was generated by using an automatic grid generation technique, based on the boundary-fitted curvilinear coordinate system suggested by Thompson et al. (1977). The advantage of using the boundary-fitted coordinates being that the irregular body geometries inherent in bridge shapes can be handled more generically, than by the use of the more familiar rectangular cartesian coordinates. Solving the primitive variable form of the Navier-Stokes equations directly yields the values of the velocity components and pressure. The resulting pressure coefficients are then integrated along the body surface to obtain the desired values of the force coefficients, drag, lift, and pitching moment. Application to various bluff shapes and bridge components, show excellent prediction of the wind flow patterns around the body, especially the vortex shedding phenomenon. The fluctuating pressure coefficients obtained are good qualitatively, but sufficient time history record is required in order to compute the mean values.

KEYWORDS: Numerical Simulation; Wind Simulation; Flow Simulation; Bridges; Bluff Bodies; Aerodynamic Stability.

1. INTRODUCTION

For many years, wind tunnel testing and full scale measurements have been the primary means of obtaining the fluctuating wind forces on bridge structures. The George Vincent Memorial Aerodynamics Laboratory of the Federal Highway Administration (FHWA), Mclean, Virginia, remains one of the few facilities in the United States dedicated to conducting wind related research studies, solely focusing on bridge structures and its components. In recent years, however, due to the ever increasing availability of fast, efficient and relatively inexpensive computers, coupled with the successful implementation in the aeronautical field, numerical simulation of wind effects on bridges has become a viable research area for Wind Engineers. In this regard, Computer Simulation Software is currently being developed at the FHWA Aerodynamics Laboratory, to numerically predict the wind flow patterns and wind induced forces on bridge structures. When evaluating wind forces on a bridge structure, it is generally assumed that the critical wind direction is perpendicular to the longitudinal axis of the bridge. In other words, it is the 2-D Navier-Stokes equations that are solved. Although the natural wind is highly turbulent and random in nature, this phase of the research, as a start, solves the flow equations of motion assuming laminar flow conditions.

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The numerical scheme used, employs the finite-difference procedure in solving the governing 2-D, unsteady, incompressible, laminar, Navier-Stokes equations, in a body-fitted curvilinear coordinate system. This paper discusses the grid generation technique used, the governing equations both in the physical and the transformed planes, the finite-difference scheme, and the results from application of this numerical scheme to various bridge elements.

2. GRID GENERATION TECHNIQUE

The body-fitted curvilinear coordinate system is one in which, at least one coordinate line coincides with the boundary surface of the body in the flow field. This automatically eliminates the need for interpolation in the finite-difference implementation of boundary conditions for an irregularly shaped body. This grid generation technique involves the solution of an elliptic partial differential system in the physical plane. To solve the system of equations, a coordinate transformation from the physical plane (x,y), to a uniform rectangular computational plane (ξ,η) is necessary. Mathematically, this transformation is stated as,

$$\begin{aligned} \xi &= \xi(x,y) \\ \eta &= \eta(x,y) \end{aligned} \qquad (1)$$

where the transformed coordinates (ξ,η) are the independent variables, and the physical coordinates (x,y) are the dependent variables. The elliptic system of equations solved are either of the Laplace type or the Poisson type, which are both stated below:

Laplace Equation-

$$\begin{aligned} \nabla^2 \xi &= 0 \\ \nabla^2 \eta &= 0 \end{aligned} \qquad (2)$$

Poisson Equation-

$$\begin{aligned} \nabla^2 \xi &= P(\xi,\eta) \\ \nabla^2 \eta &= Q(\xi,\eta) \end{aligned} \qquad (3)$$

After transformation of these equations to the computational plane by using the chain rule of differentiation, the resulting form of the Laplace equation that is actually solved for the grid point values are,

$$\begin{aligned} a x_{\xi\xi} - 2b x_{\xi\eta} + c x_{\eta\eta} &= 0 \\ a y_{\xi\xi} - 2b y_{\xi\eta} + c y_{\eta\eta} &= 0 \end{aligned} \qquad (4)$$

Dirichlet boundary conditions consisting of the physical coordinates of the body geometry, and the outer flow boundary must be specified, before the solution can proceed. The coefficients a, b, c, are transformation metrics given in terms of the following difference expressions,

$$\begin{aligned} a &= x_\eta^2 + y_\eta^2 \\ b &= x_\xi x_\eta + y_\xi y_\eta \\ c &= x_\xi^2 + y_\xi^2 \end{aligned} \qquad (5)$$

The functions $P(\xi,\eta)$ and $Q(\xi,\eta)$ in the Poisson equation are chosen to enable the control of the grid lines within the flow field. Thus the grid lines can be clustered towards the body surface where high flow gradients exist. Figures 1a. and 7a. show the finite-difference grid network generated by this procedure, around a square cylinder and a typical bridge deck cross-section respectively. Note that the grid lines are packed close to the body surface.

3. FLOW EQUATIONS

3.1 Physical Plane

In the physical plane, the governing flow equations are the 2-dimensional, incompressible, laminar momentum equations, and the continuity equation. The dimensional form of the flow equations are usually non-dimensionalized with respect to the flow parameters of the undisturbed flow region. The primitive variable form of the

non-dimensional flow equations are :

$$u_x + v_y = 0$$
$$u_t + uu_x + vu_y = -p_x + \frac{\nabla^2 u}{Re} \quad (6)$$
$$v_t + uv_x + vv_y = -p_y + \frac{\nabla^2 v}{Re}$$

Instead of solving the continuity equation, a new poisson equation in pressure form is solved. The need for the poisson pressure equation is to eliminate the pressure-velocity coupling that exist between the momentum and continuity equations. Also, the elliptic nature of this poisson type pressure equation allows for the elliptic characteristic of the physical flow to be introduced into the model. Again in the physical plane, the poisson pressure equation solved is:

$$\nabla^2 p = S_p - D_t \quad (7)$$

where,

$$S_p = -(u_x)^2 - 2v_x u_y - (v_y)^2$$
$$D = u_x + v_y$$

The derivation of this pressure equation ensures the satisfaction of the continuity requirement. However, the temporal D term is purposely not set to zero according to Harlow and Welch (1965), to avoid numerical instabilities that would otherwise occur.

3.2 Transformed Equations

Before the flow equations can be solved in the computational plane, a similar transformation procedure stated earlier must be applied. After introducing the applicable transformation metrics for the equations, the form of the transformed momentum equations solved are :

u-momentum-

$$u_t + \frac{u(y_\eta u_\xi - y_\xi u_\eta)}{J} + \frac{v(x_\xi u_\eta - x_\eta u_\xi)}{J} = -\frac{y_\eta p_\xi}{J} + \frac{y_\xi p_\eta}{J} + \frac{(au_{\xi\xi} - 2bu_{\xi\eta} + cu_{\eta\eta} + du_\eta + eu_\xi)}{ReJ^2} \quad (8)$$

v-momentum-

$$v_t + \frac{u(y_\eta v_\xi - y_\xi v_\eta)}{J} + \frac{v(x_\xi v_\eta - x_\eta v_\xi)}{J} = -\frac{x_\xi p_\eta}{J} + \frac{x_\eta p_\xi}{J} + \frac{(av_{\xi\xi} - 2bv_{\xi\eta} + cv_{\eta\eta} + dv_\eta + ev_\xi)}{ReJ^2} \quad (9)$$

Pressure Equation-

$$\nabla^2 p = S_p - D_t \quad (10)$$

where,

$$\nabla^2 p = \frac{(ap_{\xi\xi} - 2bp_{\xi\eta} + cp_{\eta\eta} + dp_\eta + ep_\xi)}{J^2}$$

$$S_p = -\frac{(y_\eta u_\xi - y_\xi u_\eta)^2}{J^2} - \frac{(x_\xi v_\eta - x_\eta v_\xi)^2}{J^2} - \frac{2(y_\eta v_\xi - y_\xi v_\eta)(x_\xi u_\eta - x_\eta u_\xi)}{J^2}$$

$$D = \frac{(y_\eta u_\xi - y_\xi u_\eta - x_\eta v_\xi + x_\xi v_\eta)}{J}$$

a, b, c, d, e, x_ξ, x_η, y_ξ, y_η are all transformation scale factors or metrics, while J is the Jacobian of transformation and Re is the flow Reynolds number.

3.3 Boundary Conditions

The boundary conditions used for the momentum equation are the no-slip boundary conditions i.e. $u=0$, $v=0$. For the pressure equation, a Neumann type boundary condition is used at the wall surface $\partial p/\partial n$, and $p=0$ is specified at the outer boundary. Note that the pressure being referred to is the non-dimensional pressure, hence, setting the outer boundary value which occurs far away from the body in the undisturbed flow region to zero is satisfactory. In the computational plane, periodicity was employed in implementing the boundary conditions along the ξ coordinate in the numerical solution routines.

4. FINITE-DIFFERENCE SCHEME

The finite-difference formulation is based on an Upwind-Implicit scheme. A backward difference is used in time, and central difference expressions used for the space derivatives. Due to numerical instability resulting from using a central difference for the first derivative convection terms, the transformed form of a third order upwind difference expression suggested by Kawamura and Kuwahara (1984) is used instead. For the u-velocity convection term in the transformed momentum equations, the finite-difference expression used is :

$$\left(\hat{u}\frac{\partial u}{\partial \xi}\right)_i = \hat{u}_i \frac{-u_{i+2} + 8(u_{i+1} - u_{i-1}) + u_{i-2}}{12\Delta\xi} + |u_i| \frac{u_{i+2} - 4u_{i+1} + 6u_i - 4u_{i-1} + u_{i-2}}{4\Delta\xi} \qquad (11)$$

The $\hat{u}_i$ refers to the transformed form of the non-linear velocity term in the flow equations of motion.

5. SOFTWARE APPLICATION

The simulation software developed in this study has been applied to a square cylinder, a rectangular cylinder with depth to width ratio of 0.5, and to a typical bridge deck cross-section. In each case, the flow was impulsively started, using the results from a potential flow simulation as the initial iteration guess. A Reynolds number of 1×10^5 was used for all the flow problems. This value is considered typical for the physical flow regime in which most bridge structures exist. The outer boundary for the flow around a square cylinder was placed at 10 times the body length. Figure 1 show a portion of the mesh in the physical plane, and also plots of the flow streamlines, velocity vectors and pressure contours at the Instantaneous non-dimensional time (NDT= $u_\infty t/L$) of 50. The computed pressure coefficient C_p is given by the non-dimensional expression:

$$C_p = \frac{p - p_\infty}{\frac{1}{2}\rho u_\infty^2} \qquad (12)$$

The variables with ∞ represent the flow parameters of the undisturbed flow region far from the body. In Figures 2 and 3, the fluctuating nature of the wall surface pressure distribution is depicted for non-dimensional times of 50 and 75. In both cases, negative suction pressures exist on the top, bottom, and leeward sides as expected. On the windward side, the pressures are characteristically positive i.e. pressures directed into the wall. An illustration of the vortex shedding phenomenon is shown in Figure 4, for flow around a rectangular cylinder of depth to width ratio of 0.5. The distribution of pressure on the body surface of this rectangular cylinder at non-dimensional time of 75 is shown in Figure 5. The instantaneous flow streamlines and wall surface pressure distribution on both a square cylinder, and a rectangular cylinder computed by Inamuro and Saito (1985), using the discrete vortex method are shown in Figure 6a. In Figure 6b, they show a comparison of their calculated mean pressures to experimental values from the work of Otsuki et al. (1974, 1978 & 1980). Note that their pressure plots are for values of pressure coefficients times -1, i.e ($-C_p$). The values of instantaneous pressure in Figures 2, 3, and 5 are in qualitative agreement with those in Figure 6. Finally, the results from

application to a bridge deck section are shown in Figures 7 and 8. Figure 7 shows the clustered grid used, the instantaneous flow streamlines, velocity vectors, and pressure contours at a non-dimensional time of 20. The wall surface pressure distribution is plotted in Figure 8. Again these values are qualitatively acceptable.

6. CONCLUSION

The computer flow visualization plots of the flow streamlines, show excellent prediction of the wind flow patterns around the various bodies. On the other hand, the resulting pressure coefficients are qualitatively acceptable at this time. Sufficient time history record is required in order to compute representative mean values, that can be directly compared with mean values from wind tunnel experiments.

7. ACKNOWLEDGMENTS

Much of this work is from the first author's PhD dissertation. The study was supported by a Federal Highway Administration grant.

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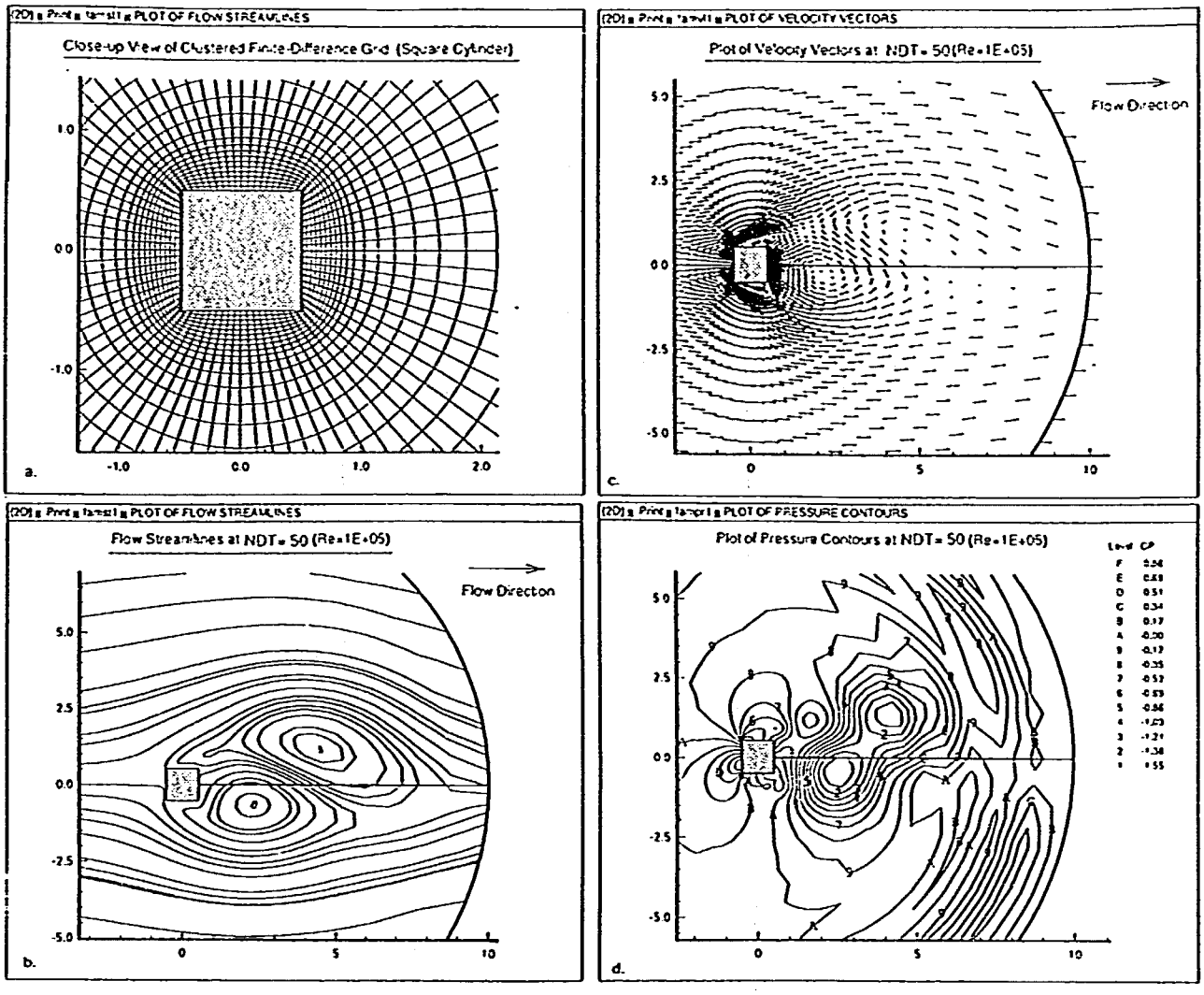


Figure 1. Instantaneous Flow around a Square Cylinder at NDT= $u_\infty t/L$ = 50

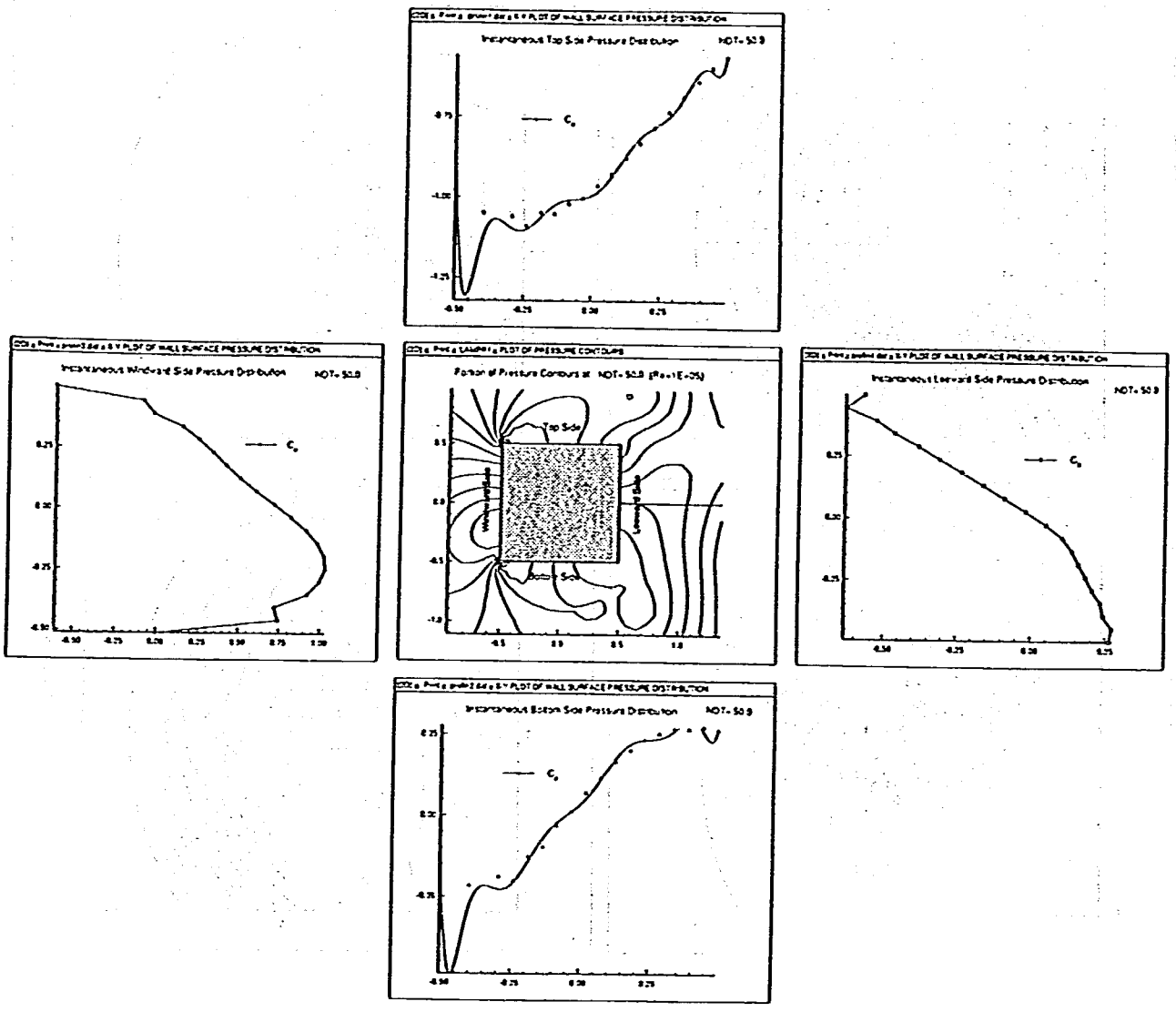


Figure 2. Instantaneous Body Surface Pressure Distribution around a Sqaure Cylinder at NDT= $u_\infty t/L = 50$

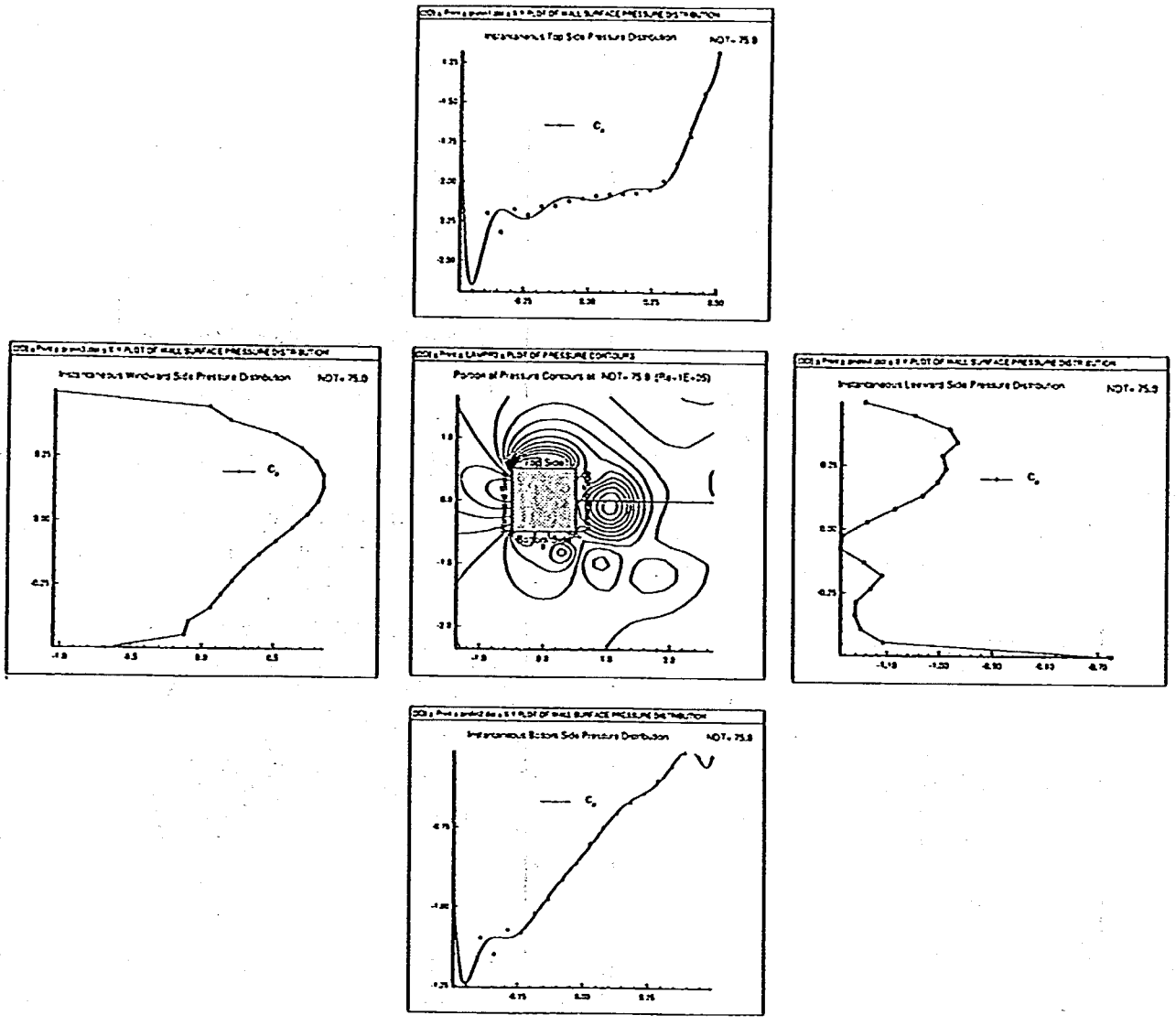


Figure 3. Instantaneous Body Surface Pressure Distribution around a Sqaure Cylinder at NDT= $u_\infty t/L$ = 75

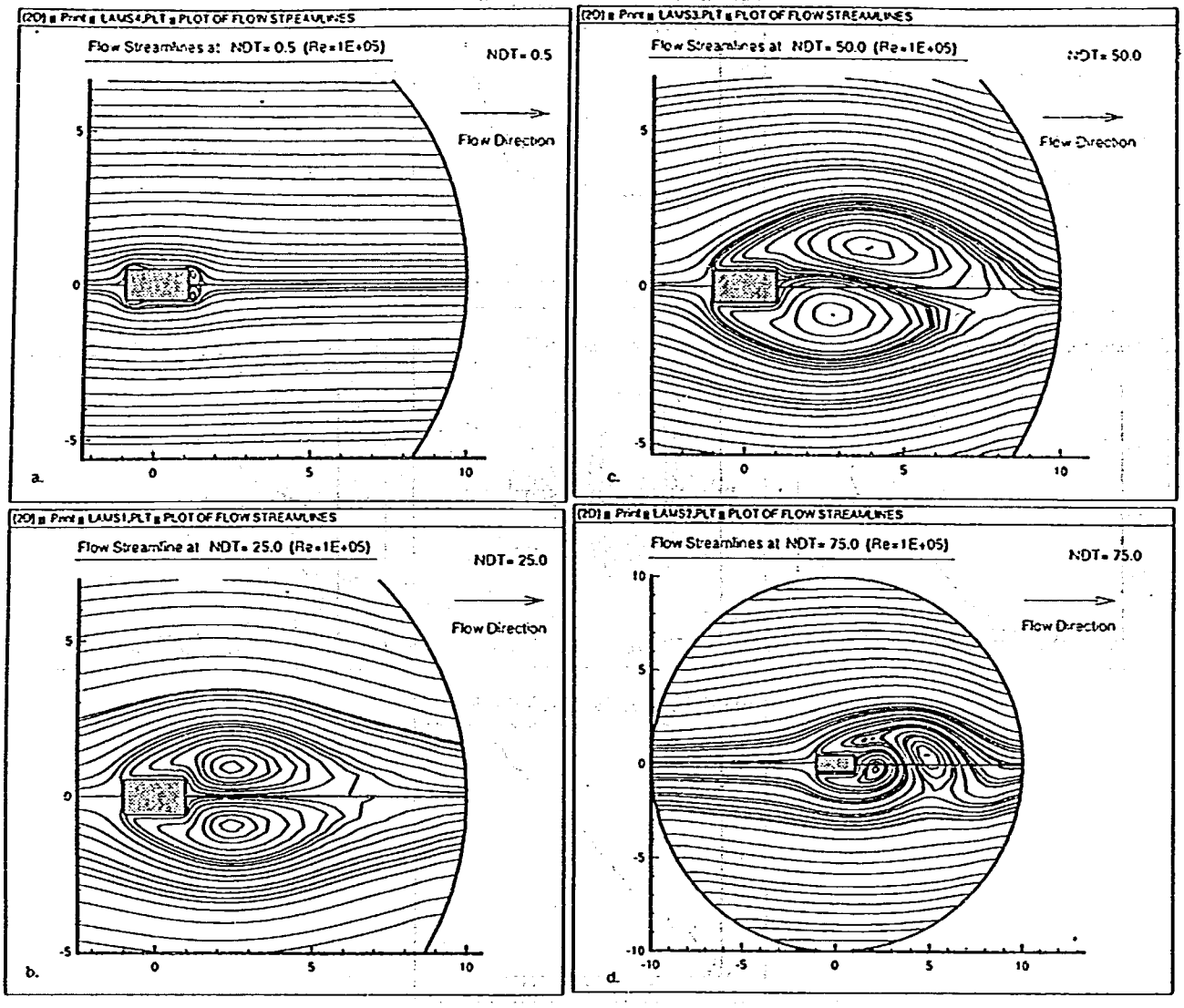


Figure 4. Time History of Flow around a Rectangular Cylinder (d/w=0.5)

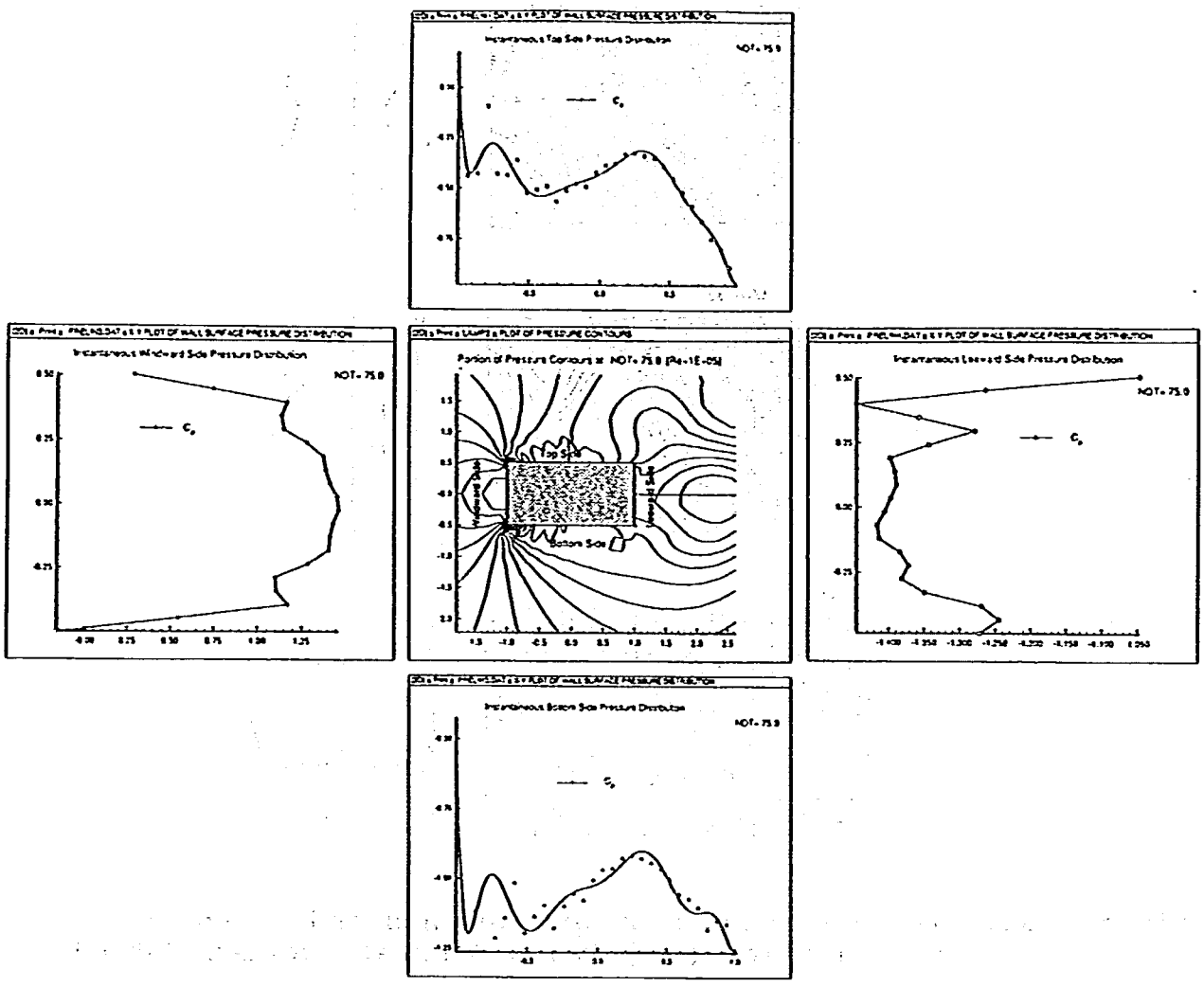


Figure 5. Instantaneous Body Surface Pressure Distribution around a Rectangular Cylinder (d/W=0.5) at NDT= $u_\infty t/L$ = 20

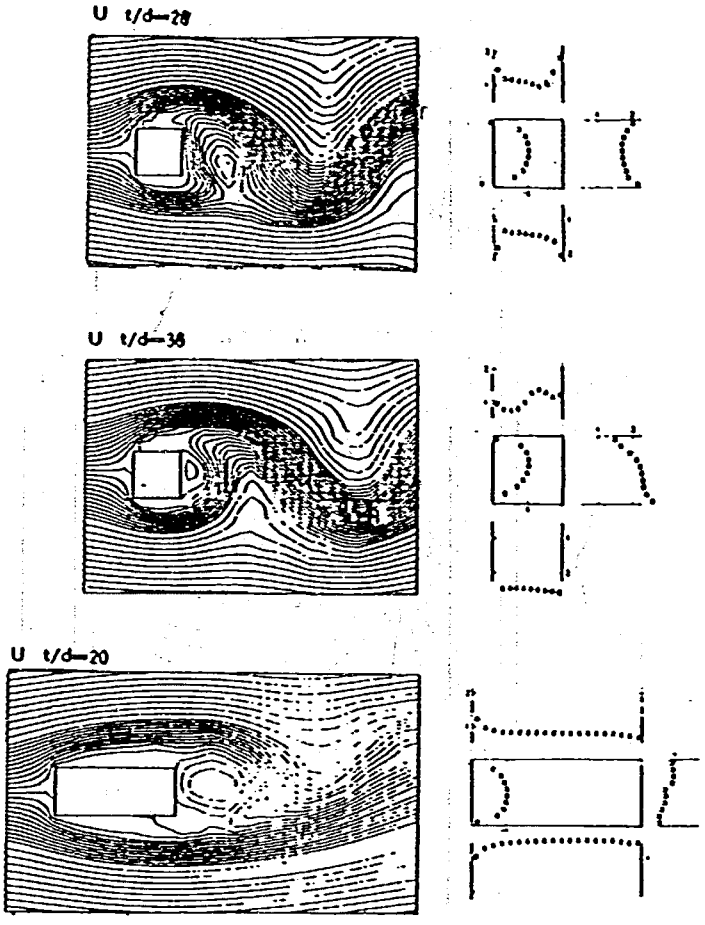


Figure 6a. Instantaneous Flow Streamlines and Pressure Distribution ($-C_p$) Using Discrete Vortex Method

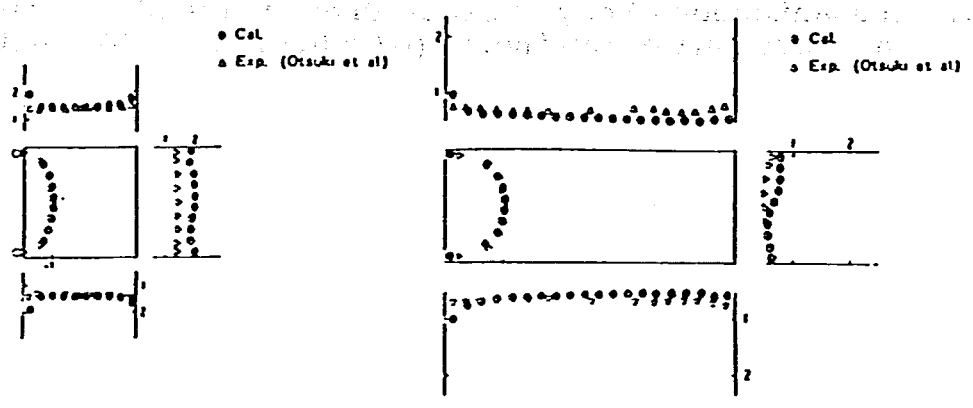


Figure 6b. Mean Pressure Distribution from Discrete Vortex Method and Wind Tunnel Experiment

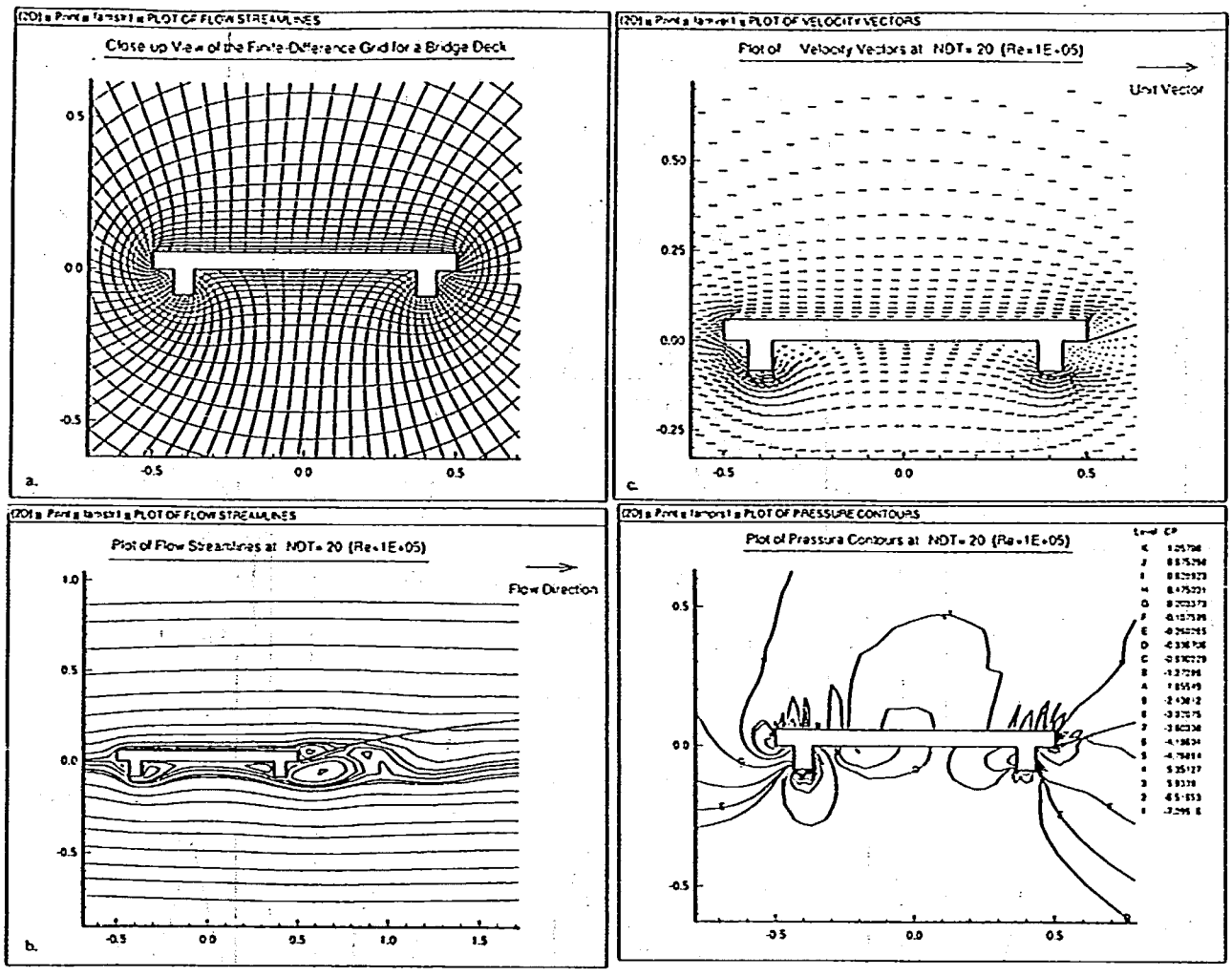


Figure 7. Instantaneous Flow around a typical Bridge Deck Section at NDT= $u_\infty t/L = 20$

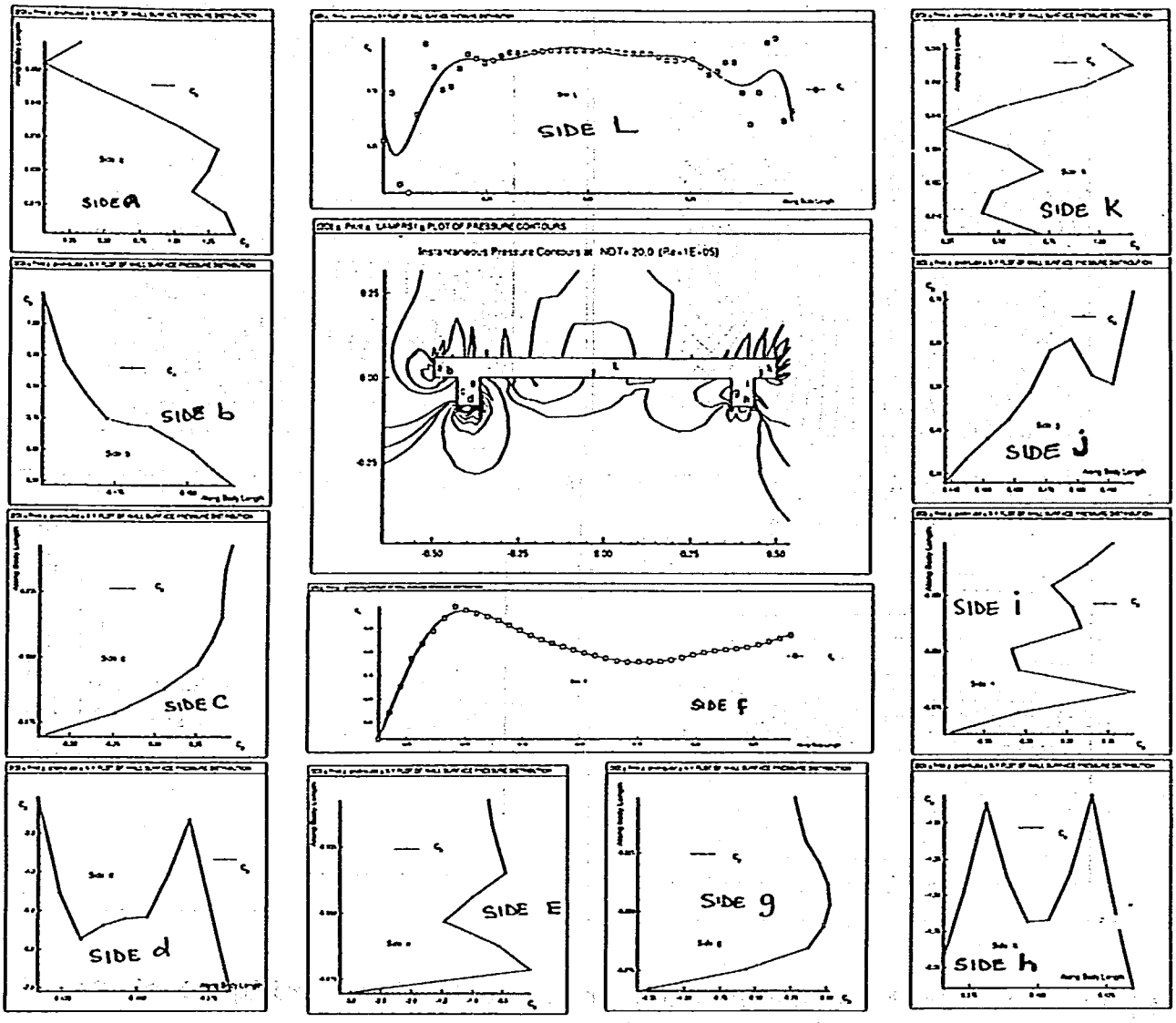


Figure 8. Instantaneous Body Surface Pressure Distribution around a typical Bridge Deck Section at NDT= $u_\infty t/L$ = 20

Determination of Windborne Missile Design Parameters

by

James R. McDonald*

ABSTRACT

Tornadoes and other high winds pick up and transport objects and debris, which can cause extensive damage to structures. This paper addresses the problem of windborne missile impact on facilities which, if damaged, could result in injury to people or damage to the environment. The problem has two parts: (1) what missiles should be included in the criteria, and (2) what is the impact resistance of different wall and roof panels to the postulated missiles. This paper briefly addresses both issues. The document, *Guidelines for Design and Evaluation of Department of Energy Facilities Subjected to Natural Hazard Phenomena*, (UCRL 15910) specified a missile criteria for moderate and high hazard facilities. The selection of the missiles to be considered and their parameters by the Natural Hazards Phenomena Committee are described in the paper. A series of impact tests were then conducted at Texas Tech University to identify a set of wall and roof panels that could satisfy the UCRL 15910 missile impact criteria. The deterministic criteria is selected to satisfy stated probabilistic performance goals of the DOE facilities.

KEYWORDS: debris; impact; missiles; tornadoes; wind; wind resistant design

1. INTRODUCTION

Model building codes (e.g. SBCCI, 1991) and national wind load standards (e.g., ASCE, 1988) do not require building designs to consider loads produced by tornadoes. The probability of a building at a specific geographic location experiencing a tornado with particular wind speed is considered too small for ordinary buildings. However, there are circumstances where design for tornadoes is necessary or desirable. Facilities that have high dollar value materials or perform critical functions, such as a fur storage warehouse or a corporate computer data center, may need to be designed to be tornado resistant. Facilities that store or process toxic or radioactive materials may also require consideration of tornado effects. Nuclear reactors have traditionally been designed to be tornado resistant. Critical equipment and facilities needed to conduct a safe shut down of the reactor are housed in tornado-hardened buildings or enclosures.

A tornado damages buildings or facilities by three principal mechanisms: (1) wind, (2) atmospheric pressure change (APC), and (3) missile impact. This paper addresses the third damage mechanism: tornado missile impact.

High winds and tornadoes pick up and transport various objects and debris. Under the right set of circumstances, these objects may be accelerated to high speeds, carried to great heights, and travel long distances. Common terms for debris transported by the winds are *wind-generated* or *tornado-generated* missiles. Tornado winds are more efficient in transporting missiles than "straight" winds in hurricanes and thunderstorms.

When a tornado missile strikes the wall or roof of a building, two independent responses are possible: *penetration* or *structural response*. A penetration impact is caused by a long, prismatically-shaped object such as a timber plank, pipe, or pole. When the missile strikes the wall, one of four possible events takes place:

(1) the object strikes the wall at some oblique angle and bounces harmlessly off the wall (oblique strike)

(2) the missile strikes essentially on end, and penetrates some distance into the wall (penetration)

(3) the missile strikes the wall with sufficient force to scab material from back face of wall; it may penetrate but does not perforate the wall (back face scabbing)

(4) the missile passes through the wall and exits at some slower speed (perforation with residual velocity).

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Structural response occurs when a large, heavy object strikes a structural component and produces excessive plastic deformations in the material. An example is an automobile striking a reinforced concrete wall panel with sufficient force to produce yielding of the reinforcing steel and crushing of the concrete. If a heavy object strikes a column with sufficient force, the column could collapse and initiate a progressive failure as other columns in the structural system collapse one after the other. Structural response impact is not addressed in this paper.

Many buildings and facilities operated by the U.S. Department of Energy (DOE) house toxic or nuclear materials that present potential danger to plant personnel or the environment should a high wind or tornado damage the facility. Recognizing the potential problem, DOE through Lawrence Livermore National Laboratories, supported the development of *Design and Evaluation Guidelines for DOE Facilities Subjected to Natural Phenomena Hazards*. A committee consisting of wind, earthquake and flood experts produced the guidelines, which are referred to as UCRL 15910. The approach adopted by the Natural Hazards Phenomena (NPH) Committee is to use a deterministic design and evaluation criteria to meet probabilistic performance goals. Hill et al. (1991) describes the treatment of wind and APC loads on DOE facilities. The objective of the study described herein is to establish the basis for the missile criteria and impact-resistant requirements for design or evaluation of DOE facilities.

Four facility-use categories are defined in the current version of UCRL 15910 (DOE, 1990): (1) general use, (2) important or low hazard, (3) moderate hazard, and (4) high hazard facilities. Only the latter two categories require consideration of windborne missiles. A facility will be subject to either high wind or tornado design criteria, depending on its geographic location. Both criteria require consideration of windborne missiles. Tornado missile criteria is more conservative than straight wind.

The remainder of this paper is divided into three sections that present (1) basis for missile trajectory parameters, (2) impact resistance of wall and roof panels, and (3) conclusions and recommendations.

2. BASIS FOR MISSILE PARAMETERS

The principal questions to be answered about tornado missiles are (1) what types are transported by the winds, (2) how fast do they fly, (3) how high, and (4) how far.

2.1 Windborne Missiles

Field damage experience is the best means for identifying the types of missiles that can be picked up and transported by high winds or tornadoes. For purposes of discussion, objects can be classified as lightweight, medium, and heavy missiles (McDonald and Kiesling, 1988). Lightweight missiles include roof gravel, pieces of gypsum board, fiberglass insulation, sheet metal panels, and small tree branches. These missiles primarily break glass or damage building finishes.

Medium weight missiles are, by far, the most common. The category includes pieces of timber plank from damaged or destroyed wood buildings, electrical conduit, plastic pipe, small communication antennas, fence posts, small steel joists, purlins, or girts. These missiles will be picked up by thunderstorm or hurricane winds, but are more likely to achieve the highest trajectory speeds when picked up by tornado winds. They can achieve speeds sufficient to perforate ordinary stud wall construction, walls with brick veneer, unreinforced and ungrouted concrete masonry, and concrete wall panels less than four inches thick.

Heavy missiles include a variety of objects such as large diameter pipes, utility poles, automobiles, trash dumpsters, storage tanks, and railroad cars. Except for the pipes and poles, these objects tend to roll and tumble on the ground rather than become airborne. The poles and pipes can penetrate or perforate all walls with equivalent resistance less than or equal to a 12 in. thick reinforced concrete wall. Large, heavy objects cause structural response rather than penetration failure.

Field investigations of many windstorms have led to the identification and classification of windborne missiles. Lightweight and medium weight missiles are common in relatively weak tornadoes, hurricanes and straight winds. Transportation of heavy missiles is an indication of high winds in an intense tornado.

Because of the adverse consequences of core damage to nuclear power reactors, the USNRC design criteria requires plants to be designed for a very conservative list of heavy weight missiles (USNRC, 1981). As a result of the USNRC

missile criteria, facilities that house equipment or systems required for safe shutdown of power reactors are required to have 24-in. thick concrete walls and 18-in. roof slabs. This very conservative criteria controls design and contributes to high construction costs. Probabilistic risk assessments (PRAs) of heavy tornado missile events suggest that the probability of reactor core damage from missile impact is of the order of 1×10^{-8}.

The probabilistic performance goals established for DOE facilities in UCRL 15910 can be achieved with much less conservative windborne missile criteria. Because PRAs are time consuming and expensive, a deterministic approach was adopted, which leads to designs that meet the performance goals. The approach is based on post-storm damage data and judgments of experts in wind damage assessments.

2.2. Missile Flight Characteristics

Flight characteristics are the parameters that determine how fast, how high, and how far missiles travel in a windstorm. The important parameters are weight, surface area, drag and lift coefficients, release mechanism and height of missile at release. The weight, area and aerodynamic coefficients determine if a missile, when released, will be transported by the wind or merely fall to the ground under the influence of gravity. The height above ground at release determines how long a missile will remain airborne before striking the ground. Another factor is whether the missile is restrained or unrestrained before being picked up by the wind. If it is unrestrained, it may be simply pushed along the ground by the wind. If it is restrained and the winds suddenly overcome the restraint, the missile is more likely to fly higher and faster than if it were not restrained.

In the case of a tornado, the location within the tornado wind field where the missile is released determines the trajectory it will follow. Missiles that release on the right hand side of the tornado path centerline (looking in direction of tornado travel) will generally travel in the direction of tornado translation with the trajectory converging toward the centerline. Objects picked up on the left side of the path centerline are carried by the tangential winds and may initially move in the opposite direction to tornado tranlation and travel around the back side of the tornado vortex.

In field investigations, the initial location ofthe missile and its final impact location usually can be determined. What happens in between can only be determined if movies or videos of the storm were available. Such films are very rare. Thus, the only feasible approach to understanding missile trajectories is by computer simulation.

2.3 Trajectory Simulation

Two approaches to trajectory simulation have been used in the past: deterministic and probabilistic.

2.3.1 Deterministic

Missile aerodynamics are modeled using flight parameters, injection mechanism, wind field model and dynamic equations of motion (McDonald, 1975). Flight parameters include drag and lift coefficients, area, and weight of missile. Drag and lift coefficients are obtained from wind tunnel tests. The flow of wind in the tornado vortex is modeled as realistically as possible. Photogrammetric analyses of the flow observed in movies or videos of tornado funnels are used to identify the wind flow patterns as well as theoretical considerations. Several tornado wind field models have been published in technical literature. The DBT-77 model by Fujita (1978) is based on photogrammetric analysis and extensive field observations of damage patterns by Dr. Fujita. Other models include Hoecker (1960) and Simiu and Cordes (1976).

Injection of the missile into the wind field can be as simple as releasing the missile at some preselected point within the three-dimensional tornado wind field where it will achieve maximum speed, height and/or distance traveled. Another approach is to systematically place the missiles at regular intervals across the tornado path and release them at some predetermined wind speed. Trajectories are then calculated for each missile and the worst conditions are tabulated.

Missiles can be modeled as a three-degree-of-freedom point model or a six-degree-of-freedom rigid model. Calculations are simpler for the point model. The approach was used to successfully predict the impact locations of the steel beams that were picked up by the Bossier City Tornado (McDonald, 1981).

The six-degree-of-freedom model requires a set of drag and lift coefficients for each orientation

of the missile in space relative to the wind direction vector. Such measurements have been made in the wind tunnel for a few objects. The computational effort for the six-degree-of-freedom model is many times larger than for the three-degree-of-freedom model, with no solid evidence that it is more accurate or realistic (Redman et al., 1976). Neither model considers the effects of small scale turbulence in the tornado wind field.

2.3.2 Probabilistic Simulation

The probabilistic approach to trajectory simulation is essentially a Monte Carlo simulation. Tornado wind field characteristics (maximum wind speed, path width and direction) are randomly selected as are certain missile parameters at each time step of the trajectory calculations. The missiles are randomly selected according to some weighted frequency scheme. By simulating many years of tornado missile experience, statistics on the types of missiles and their trajectory characteristics can be obtained. See for example Twisdale and Dunn (1978). The site specific approach, which attempts to model the real situation as accurately as possible, must be applied at each facility being considered for tornado effects. To carefully model a site by this method requires extensive effort and considerable cost.

2.4 UCRL 15910 Missile Criteria

Since UCRL 15910 is based on a deterministic approach to satisfy probabilistic performance goals, the deterministic approach to tornado missile criteria was adopted by the NHP Committee.

Since general use and low hazard facilities are designed by criteria in model codes or national standards, which have no provisions for winborne missiles, UCRL 15910 also does not specify a missile criteria for these facilities. Moderate and high hazard facilities located in areas where tornadoes are not a serious threat have a minimum windborne missile criteria. The minimum missile criteria accounts for objects or debris that could be picked up by straight winds, hurricanes or weak tornadoes.

Moderate and high hazard facilities located in tornado prone areas must satisfy tornado missile criteria. Because the tornado missile trajectory calculations require specialized software and involve considerable time and effort, the NPH Committee established missile criteria that was independent of tornado design wind speed. This was possible because the range of tornado design wind speeds for the 26 DOE sites was relatively narrow. Thus a single criteria for all moderate and high hazard facilities could be established.

Two medium and one heavy weight missiles are considered in the design and evaluation of DOE facilities. A timber missile is typical of debris found in the destruction of office trailers, storage sheds, residences and other light framed timber structures. Hundreds of these missiles can be generated in the destruction of a residential neighborhood. A steel pipe missile represents a class of debris that includes electrical conduit, liquid or gas piping, fence posts and small columns. The pipe missile is less frequently available for transport and does not fly as high or as fast as the timber missile, but is more destructive upon impact. Tornadoes can roll and tumble automobiles, pickup trucks, small vans, forklifts and storage tanks of comparable size and weights. These missiles do not become airborne, but can damage wall panels, frames or columns by excessive structural response.

The three missiles produce varying degrees of damage. A specific type of construction is required to stop each missile. The timber missile is capable of breaking glass and perforating curtain walls or unreinforced masonry walls. Reinforced concrete or masonry walls are required to stop the pipe missile. Timber and pipe missiles can perforate weak exterior walls and emerge with sufficient speed to perforate interior walls and damage HVAC ducts, filter enclosures or control equipment. The impact of a rolling or tumbling automobile produces failure by excess structural response. Bearing walls, rigid frames and exterior columns are particularly susceptible to heavy missile impacts.

Specific missile criteria in UCRL 15910 are summarized in Table 1. Critical missile parameters include (1) material, (2) weight, (3) impact speed and (4) maximum height achieved. Impact speeds are expressed in terms of horizontal and vertical components. They should not be combined vectorially, because the maximum horizontal speed does not occur at the same point in the trajectory as the maximum vertical speed. The maximum horizontal speed generally occurs at maximum height of trajectory above ground, while the maximum vertical speed occurs at impact with the ground.

In general, the maximum horizontal speed is used in the design of walls or other vertical barriers. The maximum vertical speed is used to design roof decks and other horizontal barriers. Both maximum horizontal and vertical speeds should be considered (separately) on sloping surfaces.

The designer or evaluator using UCRL 15910 criteria is not required to determine missile impact speeds. He designs new barriers or assesses existing barriers based on results of impact tests. The impact test results for the timber plank and steel pipe missiles are described in the next section. From the test results, specific wall or roof barriers are known to meet the UCRL 15910 missile criteria.

3. IMPACT RESISTANCE OF WALL AND ROOF PANELS

3.1 Background

Penetration, perforation and scabbing equations used for tornado missile impact design in the 1960s were derived from military applications of ballistic missile impacts. The masses and shapes of the military projectiles were quite different from those generated by tornado winds. Gwaltney (1968) presented a comprehensive review of missile generation and protection in nuclear power plants. A number of penetration formulas are referenced in the Gwaltney paper:

(1) Modified Petry formula

(2) Ballistic Research Laboratory (BRL) formula

(3) National Defense Research Committee (NDRC) equation

(4) Army Corps of Engineers formula

Because none of the ballistic formulas were directly applicable to tornado missile impact design, several agencies and companies initiated impact test programs. Most of the missiles and barriers tested in the 1970s related to the U.S. Nuclear Regulatory Commission (USNRC) criteria for design of commercial nuclear plants. Test results were reported for steel rods (rebar), steel pipes, timber planks, utility poles, and automobiles. Based on test results, a modified version of the NDRC equation was proposed by Kennedy (1975). Rotz (1975) developed an empirical equation based on a series of impact tests sponsored by Bechtel Power Corporation (Velasso, 1975). The NDRC and Rotz equations are applicable to steel pipe missiles for impacting on reinforced concrete panels. The test results were for impact speeds higher and panels much thicker than those required by UCRL 15910.

3.2 Missile Impact Tests

The Tornado Missile Impact Facility (TMIF) at Texas Tech University is capable of testing the resistance of various wall and roof panels to the impact of medium weight missiles. The tests include both the 2x4 timber plank (15 lbs) and the 3-in. diameter steel pipe (75 lbs). A compressed air cannon is capable of propelling the 2x4 plank to 150 mph and the pipe to 75 mph. Figure 1 shows the general arrangement of the facility. Figure 2 is a photo of the cannon. A rigid reaction frame supports the test panels. The speed of the missile just prior to impact is measured by an optical timing gate.

3.2.1 2x4 Timber Plank Missiles

Three series of tests were conducted with the 2x4 plank missile. The first series of tests determined the impact resistance of conventional residential wall construction as defined in *Architectural Graphics Standards* (Packard, ed., 1981). Wall panels 4 ft wide by 8 ft high were constructed with 2x4 Douglas fir studs at 16 in. on center. The inside wall material was 1/2 in. gypsum board. The exterior cladding is described in Table 2. As shown in Table 2, the 2x4 plank perforates the stud walls at speeds of about 55 mph. Missile trajectory simulations of the 2x4 plank suggest that if the missile is transported by the winds, it will most likely achieve speeds greater than 55 mph. Thus, ordinary stud wall construction will be perforated by the 2x4 timber plank missile. Stud walls clad with insulation board and brick veneer are capable of resisting the 2x4 plank missile at speeds less than 120 mph (see Table 2).

The second series of tests involved impact of the timber plank missile on concrete masonry walls (McDonald and Bailey, 1985). Eight-in. and 12-in. concrete masonry unit (CMU) walls with various combinations of reinforcement and grout in the vertical cells were tested. Unreinforced and ungrouted walls offer little resistance to perforation. If the missile strikes the face shell of an 8-in. wall, it perforates at impact speeds as low as 60 mph. The 12-in. CMU wall offers slightly more resistance

because the face shell is thicker. Reinforcing each vertical cell with a 1/2-in. diameter (#4) rebar and filling the cell with a high slump concrete grout produces a wall that is capable of stopping the 2x4 plank at any impact speed. At impact speeds greater than 100 mph, the missile will splinter. Horizontal trussed joist reinforcement at 16 in. on center helps prevent cracking of the masonry panel. Table 3 summarizes the results of the second series of tests on CMU walls.

The third series of tests involved impact of the 2x4 timber plank on unreinforced and reinforced concrete wall and roof panels. The 2x4 timber plank will not perforate a 4-in. unreinforced concrete panel (4000 psi nominal compressive strength), but it will badly crack it. Adding 1/2-in. diameter rebar at 12-in. on center each way on the inside face of the panel will prevent the cracking. Impacts of the plank at speeds greater than 100 mph on slabs thicker than 4 in. will cause the missile to splinter.

3.2.2 Three-In. Diameter Steel Pipe Missiles

A series of impact tests were conducted on reinforced concrete and masonry panels that were designed to satisfy the UCRL 15910 missile impact criteria. Six concrete walls with 6-in., 8-in., 9-in., or 10-in. thicknesses were tested at impact speeds of 50 and 75 mph. All concrete panels performed better than the Modified NDRC equation predicted.

Eight-in. and 12-in. CMU walls that were grouted and reinforced with 1/2 in. diameter rebar were tested. The 3-in. diameter pipe perforated the 8-in. CMU wall at an impact speed of 75 mph. Back-face scabbing occurred at an impact speed of 50 mph. Pieces of material weighing approximately 7.5 lbs came off the back face of the panel. The threshold perforation speed of the 12-in. CMU wall was found to be approximately 60 mph.

Two other masonry panels were tested with the pipe missile. One was a 9 1/2-in. cavity wall consisting of two wyths of clay brick masonry and 2500 psi concrete in the cavity. Number 3 rebar was placed vertically at 8-in. on center. Horizontal joint reinforcement was placed at 16-in. on center. The panel was impacted by the steel pipe at 50 mph. Cracks appeared on both the front and back face, but no perforation on scabbing occurred. This panel was not tested at 75 mph. The second masonry panel was constructed with 8-in. reinforced and grouted CMUs and a 4-in. clay brick veneer. This panel was impacted by the pipe at 50 mph. The impact speed appeared to be near the threshold of back-face scabbing. Table 4 summarizes the results of these tests.

4. CONCLUSIONS AND RECOMMENDATIONS

Several series of impact tests using the 2x4 timber plank and the 3-in. diameter steel pipe have been conducted at the Texas Tech University TMIF. Results of these tests form the basis for the following recommended wall and roof panels for missile protection of moderate and high hazard facilities as defined in UCRL 15910. (See Table 2 for missile criteria.)

4.1 Moderate Hazard - Straight Wind

The 2x4 timber plank impact at 50 mph requires a minimum 8-in. CMU wall with one #3 rebar grouted in each vertical cell and trussed horizontal joint reinforcement at 16 in. on center.

4.2 Moderate Hazard - Tornado

The timber plank impact at 100 mph requires a minimum 8-in. CMU wall with one #3 rebar grouted in each vertical cell and trussed horizontal joint reinforcement at 16 in. on center; 6-in. concrete wall with #3 rebar at 12 in. on center each way 1 1/2-in. from the inside face. The 3-in. pipe impact at 50 mph requires a minimum 12-in. CMU wall with #4 rebar grouted in each vertical cell and trussed horizontal joint reinforcement at 16 in. on center; 8-in. concrete wall with #4 rebar at 12-in. on center, each way 1 1/2-in. from each face.

4.3 High Hazard - Straight Winds

High hazard facilities for straight winds have the same criteria as moderate hazard, except the wall panels are susceptible to impact up to 50 ft above ground level.

4.4 High Hazard - Tornado

The 2x4 timber plank impacting at 150 mph requires a minimum of 8-in. CMU wall with one #4 rebar grouted in each vertical cell and trussed horizontal joint reinforcement at 16-in. on center; 6-in. concrete wall with #4 rebar at 12-in. on center, each way, 1 1/2-in. from each face. The 3-in. diameter pipe impacting at 75 mph requires a minimum 10-in. concrete wall

reinforced with #4 rebar at 12-in. on center, each way, 1 1/2-in. from each face.

4.5 Recommendations

The wall panels described above will satisfy the criteria of UCRL 15910. The criteria will meet the performance goals for the category of facility considered. Additional tests are needed to determine the resistance of doors, grills and other wall cladding systems. The impact resistance of steel plate panels is untested. Additional tests are needed to validate the BRL equation for steel plates.

Considerable additional study is also needed to establish and verify procedures for calculating structural responses to heavy weight missiles.

5. ACKNOWLEDGEMENTS

Financial support for the Texas Tech University missile impact tests from the National Science Foundation and Lawrence Livermore National Laboratory are gratefully acknowledged.

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Table 1. Windborne Missile Criteria in UCRL 15910

	Moderate Hazard	High Hazard
Straight winds	2x4 timber plank, 15 lb @ 50 mph (horiz.); max. height 30 ft.	2x4 timber plank, 15 lb @ 50 mph (horiz.); max. height 50 ft.
Tornadoes	2x4 timber plank, 15 lb @ 100 mph (horiz.); max. height 150 ft; 70 mph (vert.)	2x4 timber plank, 15 lb @ 150 mph (horiz.); max. height 200 ft; 100 mph (vert.)
	3 in. dia. steel pipe, 75 lb @ 50 mph (horiz.); max. height 75 ft; 35 mph (vert.)	3 in. dia. steel pipe, 75 lb @ 75 mph (horiz.); max. height 100 ft; 50 mph (vert.)
		3000 lb automobile @ 25 mph; rolls and tumbles

Table 2. Results of 2x4 Timber Plank Impact Tests on Typical Stud Wall Panels (McDonald, 1990)

Panel Description	Perforation Speed, mph
Masonite Siding	54
Insulation Board and Masonite Siding	54
Plywood (1/2 in.) and Masonite Siding	52
Plywood (1/2 in.)	52
Plywood (3/4 in.)	53
Stucco	53
Lapboard Siding	53
Insulation Board and Lapboard Siding	52
Insulation Board and Brick Veneer	120

Table 3. Estimated Speed for Perforation of CMU Walls by 2x4 Timber Plank Missile (McDonald, 1990)

Wall Construction	Perforation Speed, mph
8-in CMU Wall	
Grout and rebar in every cell	>130[1]
Grout only in every cell	>130[2]
Intermittent vertical reinforcement	65[3]
Horizontal Joint Reinforcement Only	65[3]
Unreinforced	60
12-in. CMU Wall	
Grout and rebar in every cell	>130
Grout only in every cell	>130
Intermittent vertical reinforcement	70[3]
Horizontal Joint Reinforcement Only	70[3]
Unreinforced	65

[1]Missile will splinter.

[2]Missile will not perforate, but wall may crack.

[3]Face shell of ungrouted cell will be perforated.

Table 4. Results of 3-in. Diameter Steel Pipe Impact Tests

Panel Description	Impact speed, mph	Damage
6-in. reinforced concrete panels	35	Small cracks, no perforation or scabbing
8-in. reinforced concrete panels	50	Small cracks, no perforation or scabbing
9-in. reinforced concrete panel	50	No cracks, perforation or scabbing
9-in. reinforced concrete panel	75	Radial cracks on back face, 1 1/2-in. penetration, impending scabbing
10-in. reinforced concrete panel	75	No significant cracks, perforation or scabbing
8-in. CMU panel with vertical cells reinforced and grouted	50	Missile perforated panel; 7.5 lb pieces came off back face
12-in. CMU panel with vertical cells reinforced and grouted	60	Cracks on front and back face; impact speed near threshold for scabbing

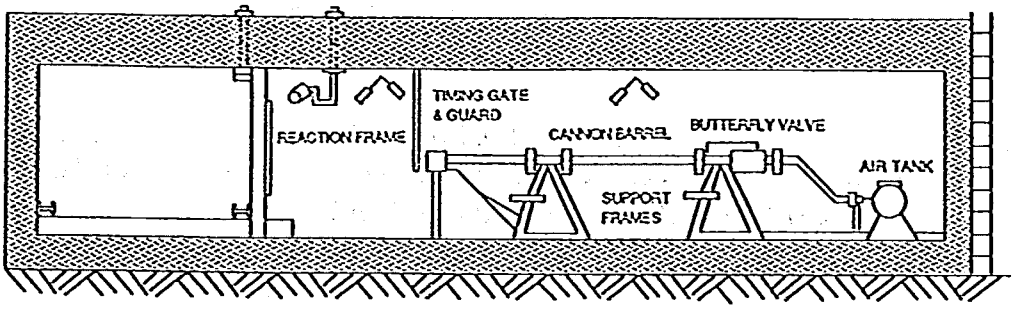


Figure 1. Elevation View of Tornado Missile Impact Facility

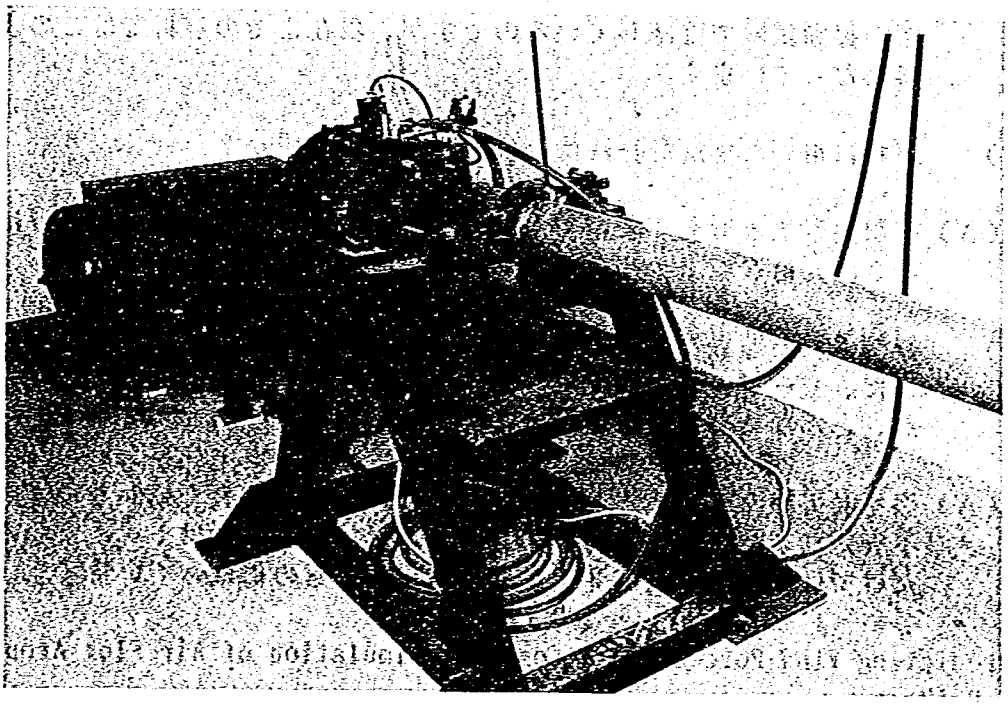


Figure 2. Tornado Missile Cannon

課題Ⅰ　耐風工学　討議

（1）　Flutter Characteristics of a Super Long-span Suspension Bridge
(Yokoyama, Koichi*, Kanazaki, Tokida. and Yasuda, M.)

Cooper（Q）　吊橋の耐風安全性に関して２次元バネ支持模型試験と３次元弾性模型試験の結果に違いがあるということが分かりましたが、従来の規模の吊橋でもこのような違いが現われるのでしょうか？

横　　山（A）パラメトリックな試験をしていないので、どの程度の長大橋になったら両者に差が出てくるのか良く分かりません。しかし、明石海峡大橋のように中央支間2,000m級という長大橋のためにこのような特異な現象が見られ、従来規模の吊橋では差は余りないのではないかと考えられます。

Chong（Q）　解析モデルの剛性がやや剛なのではないのでしょうか？　そのために、ねじれ変形の差が出てきているのだと思いますが。

横　　山（A）ねじれ変形量には多くの要因が関係します。模型の構造特性は静的載荷実験、振動試験で確認しているのですが、現在までの解析ではこのような結果になっています。

Chong（Q）　解析に用いた構造計算法はどのようなものですか？

横　　山（A）有限要素法と考えてよいでしょう。吊橋は“魚の骨”モデルと呼ばれている３次元構造をモデル化したものになっています。

Bosch（Q）　観測された連成フラッターはモード形状は安定していたのでしょうか？
また、モードはねじれ１次、たわみ１次の組合せではないということですが、どんなモード形状となっていたのでしょうか？

横　　山（A）図を見ていただけると１サイクルのモード形状が良く分かります。多少、不安定な挙動が認められます。モード形状はねじれは１次、たわみは１次だけではなく、２次、３次などが混在したものになっています。

（2）　Predicting Wind Forces Using Computer Simulation of Air Flow Around Bridges
(Okey U. Onyelukwe* and Harold R. Bosch)

Barientoss（Q）①計算で考慮する領域の大きさは構造物と比べてどの程度の大きさになっているのか？　また、②計算機の規模はどの程度のものですか？

Onyemelukwe（A）①構造物の１０倍程度までの領域を数値計算に取り入れています。②現在はＰＣを使っており、言語はFortranです。この計算はスーパーコンピューターがふさわしいと思っていますが、近々ＥＭＳが使えるようにな

るので活用する予定です。

藤　　原（Q）　発表された両人にお尋ねしたいと思いますが、風洞試験と比較して数値計算はどの程度設計において実用になるのでしょうか？

Onyemelukwe（A）数値計算は実験には置き換えられないと考えています。補足的に使うことになるでしょう？

横　　山（A）　その通りだと思いますが、設計の初期の段階でおおよその検討をつけるようなときには使えるのではないでしょうか。

横　　山（Q）　構造物周辺の流れの数値計算は土木研究所でも試みたことがありますが、機械的にプログラムを作るのではなく、良い計算結果を得るためには何か数値計算上の工夫が必要になるようです。発表された計算の場合、特別な工夫をされたのでしょうか？

Onyemelukwe（A）計算のアルゴリズムに従って数値計算をしているので、特別なノウハウはありません。

課題II. 耐震工学

Seismic Design Considerations for Jordanelle Dam

by

John A. Wilson[1], Perry J. Hensley[1], William O. Engemoen[1], and Francis G. McLean[1]

ABSTRACT

When completed in 1993, Jordanelle Dam will be a compacted, zoned earthfill dam having a sloping core. It is being constructed in northern Utah, USA, an area of significant seismicity. Maximum Credible Earthquakes (MCE) for the damsite were determined to be a magnitude 7.5 at a distance of 19 miles, and a local magnitude 6.0 to 6.5. The dam has been designed to take maximum advantage of the excellent construction materials available at the site. Use of these materials in combination with a sloping core has permitted the use of economical 1-1/2:1 downstream and 2:1 upstream slopes. Seismic analyses confirmed that the structure could withstand seismic loadings with minimal damage. This paper will summarize the design and analysis of the structure, with particular emphasis on seismic design considerations.

KEYWORDS: Embankment; Dam; Stability; Earthquakes; Slopes; Seismicity; Analysis; Design

1. INTRODUCTION

Jordanelle Dam and Reservoir, located on the Provo River near Heber City, Utah, are an important feature of the United States Bureau of Reclamation's Central Utah Project. Its primary function is to store and deliver municipal and industrial water to the growing Salt Lake City/Provo-Orem metropolitan areas. The Reservoir will have a total capacity of over 395,000,000 cubic meters (320,000 acre-feet), impounded by constructing Jordanelle Dam to a height of approximately 91 meters (300 feet) above original ground. The dam is a compacted embankment dam containing over 10.7 million cubic meters (14 million cubic yards) of soil and rock material. It has a crest length of 1,125 meters (3,700 feet) and a crest width of 12 meters (40 feet) as shown in Figure 1. Waterways include a fuse plug controlled emergency spillway and a tunnel outlet works, consisting of both low level and multi-level intakes. Jordanelle Dam was designed in two stages between 1986 and 1989, and construction began in 1987. It is anticipated that it will be essentially complete in early 1993, with reservoir filling to commence later that year.

This paper describes the design concepts for Jordanelle Dam, as related to the consideration of seismic loadings anticipated in this location. Additional analyses undertaken during the construction currently in progress are also presented.

2. DESCRIPTION OF EMBANKMENT DESIGN

The embankment cross section is shown on Figure 2. As shown, it consists primarily of compacted zone 4 material, described in Table 1. The upstream slope is 2 horizontal to 1 vertical, and the downstream slope is 1-1/2

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horizontal to 1 vertical. A relatively thin upstream sloping clay core, zone 1, is the primary impervious element. It is protected by a two-stage filter-drain system (zones 2 and 3) to prevent piping and control seepage. Slope protection upstream consists of a 1 meter (3-foot) thickness of riprap obtained from nearby quarries. Downstream slope protection for the relatively steep 1-1/2:1 slope consists of cobbles and boulders (zone 5) processed from the borrow area.

The upstream sloping core is a key feature in the configuration of the embankment. It was made relatively thin to minimize the amount of foundation preparation and treatment required beneath the zone 1. It was sloped upstream to reduce the potential for differential settlement between the more compressible zone 1 core and the relatively incompressible zone 4 shells. In addition, the inclusion of the core and filter-drain system in the upstream portion of the dam leaves the vast downstream bulk of the embankment in an unsaturated condition. This ensures both static and seismic stability. The filter-drain system immediately downstream of the core serves as a defense mechanism to ensure that any cracking created by earthquake loading is properly protected by a filter and that any resulting seepage is safely conveyed downstream by the drainage system. The ability to keep the downstream shell unsaturated, plus the availability of good quality, strong materials, enabled the construction of a relatively steep 1-1/2:1 downstream slope.

A significant factor in the final configuration of the embankment was the quality and quantity of available materials. Fortunately, the reservoir area contained an abundant supply of both impervious materials (gravelly clays) and pervious materials (sandy gravels with cobbles and boulders) of excellent quality. The pervious materials, zone 4, are particularly desirable due to their low compressibility, high strength, durability and relatively high permeability. Because of these nearly ideal properties, the design utilized this material to the largest extent possible. The high strength made relatively steep slopes possible. This reduced the overall embankment volume and lowered the construction cost significantly.

Although the pervious materials are plentiful, there was little contingency in the design for a possible underrun of available zone 4 materials. For that reason, provision was made for a 1,700,000 cubic meter (2,200,000 cubic yards) zone 4A fill within the downstream zone 4 shell. Pervious materials were the preferred material for the zone; however, the zone was designed to permit the use of other excavated materials, such as gravelly clays. This provides for the use of suitable, but unspecified, materials from the borrow area and excavations. The impact of this zone on embankment stability was evaluated, as discussed later.

3. SEISMOTECTONIC SETTING

Jordanelle damsite is located in the Wasatch Mountains within the Intermountain Seismic Belt (ISB) in north-central Utah (Figure 3). The ISB is a zone of contemporary seismicity extending from Arizona to Montana along the eastern margin of the Basin and Range province. It is considered to have the highest level of earthquake risk in the contiguous United States outside of California and Nevada. Within the ISB, three historical earthquakes with magnitudes ranging from 6.6 to 7.5 have been accompanied by surface rupture on late Quaternary faults.

Potential earthquake sources for damsites in the ISB are considered to be of two types: 1) large-magnitude earthquakes on faults with evidence of late Quaternary surface rupture and 2) a local random earthquake that occurs on "blind" subsurface faults. The most likely sources of significant ground motions in the region that could affect Jordanelle damsite are the Wasatch fault, faults in Round Valley, and the random earthquake. A maximum credible earthquake (MCE) of magnitude 7.5 is assigned to the Wasatch fault at a distance of 30 km (18 mi)

from the damsite. An MCE of magnitude 6.5 to 6.75 is assigned to the faults in Round Valley at a distance of 20 km (12 mi) from the damsite. An MCE of magnitude 6 to 6.5 is assigned to a local random earthquake source.

Geologic investigations have shown that late Quaternary surface displacements have not occurred on faults in the vicinity of the damsite. We have, therefore, concluded that there is no potential for significant coseismic surface rupture in the dam foundation. However, ground deformation has been associated with recent moderate-magnitude earthquakes that have occurred on blind faults in the ISB. Tectonic subsidence accompanying the 1975 Pocatello Valley earthquake of magnitude 6.0 suggests that a similar phenomenon could accompany a magnitude 6 to 6.5 earthquake on a blind fault in the vicinity of the dam. Historic data and model simulations indicate that a tectonic displacement of about 15 cm could occur on favorably oriented faults in association with a moderate-magnitude earthquake.

4. MATERIALS CHARACTERIZATION

Shear strength parameters used for the principal embankment materials in the preconstruction stability analyses are shown in Table 2 for each loading condition.

The strengths shown in Table 2 were determined as follows:

Zone 1 - Core: Unconsolidated Undrained (UU) triaxial tests were performed on specimens compacted to 98 percent Proctor Density. These were performed in order to obtain undrained shear strengths representative of the gravelly clay core material during construction. Consolidated Undrained (CU) triaxial tests (with measured pore pressures subtracted) were used to develop the drained strength used in the steady seepage analysis. The same tests (without measured pore pressure subtracted) were used to develop consolidated undrained strengths representative of undrained strength available in the consolidated clay core during a rapid drawdown loading.

Zone 2 - Filter: Laboratory testing of the well graded sand and gravel filter material was not performed. Drained strength parameters for the well compacted filter materials were conservatively selected from available literature and past Reclamation experience with similar materials.

Zone 3 - Drain: Laboratory testing was not considered practical due to the large particle size (3.8 to 20.3 cm) of this material. Conservative strength parameters, considering the rounded and highly uniform gradation of the material, were also selected from available literature and past Reclamation experience.

Zone 4 - Shell: Initially, the zone 4 material was assumed to behave dilatively during shear. Drained strengths were, therefore, assumed to represent all loading conditions for this material. The zone 4 was assumed to have drained strengths with an effective ϕ of 44° (Marachi, 1972) below a normal stress of 690,000 N/m^2 (100 lb/in^2) and a ϕ of 39° with a cohesion intercept of 107,600 N/m^2 (15.6 lb/in^2) above a normal stress of 690,000 N/m^2 (100 lb/in^2). These strengths are supported by the work of Leps (1970) with large scale triaxial tests on Oroville Dam material.

Zone 4 shell material is placed in two-foot lift thicknesses and compacted using vibrating rollers. Test fills prior to construction were evaluated to establish the optimum procedures for placing and compacting this material. At the beginning of the stage 2 construction, samples of this material were shipped to the University of California, Berkeley for testing in their 30 centimeter (12-inch) diameter triaxial apparatus (Seed, 1990). Tests were performed on samples scalped to a 5-cm (2-inch) maximum particle size and compacted at a density corresponding to material placed in stage 1. The results indicated that the material might not behave as originally anticipated. The tests

indicated that materials compacted during stage 1 could be slightly contractive rather than dilative during shear. As a result of these tests, a heavier vibratory roller (11-ton-per-drum dual-drum) was specified for stage 2, resulting in higher densities than those achieved during stage 1 construction. In addition, a re-evaluation of the design was made to check the impact of these in-place stage 1 materials on embankment performance.

Based on the test results, the zone 4 material placed during stage 1 would be represented by a consolidated stress phi angle of 19° for a post-earthquake analysis. This conclusion is based on the assumption that the total material will contract and build up pore pressure. In reality, though, the oversized material would likely inhibit the increase in pore pressure. Due to the greater densities of the zone 4 material being placed in the upstream shell during stage 2, the strength is conservatively represented by a phi of 30° for a post-earthquake analysis. The downstream zone 4 material is also being placed at the higher densities and will remain unsaturated. An effective friction angle of 39°, as indicated by the Berkeley results (Seed, 1990) was used in the reanalysis.

Zone 4A - Fill: The strength used for the gravelly clays allowed in the Zone 4A mass fill portion of the dam were assumed the same as the drained and undrained strengths obtained from laboratory tests on the zone 1 core materials. This provides for the flexibility of placing materials of lesser strength in this zone.

5. STATIC STABILITY EVALUATION

The static stability was evaluated for three loading conditions. These were: steady-state seepage, rapid drawdown, and end of construction. The analyses were performed using the computer program SSTAB2 (Chugh, 1981), based on the Spencer method, to compute factors of safety for various failure surfaces. The downstream slope was evaluated for a range of zone 4A configurations. For these analyses, it was conservatively assumed that the zone 4A consisted of impervious gravelly clay materials.

The end of construction case was examined using the conservative assumption that pore pressure developed during placement in either the core or the zone 4A portion of the downstream shell would not dissipate prior to completion of the dam. The gravel and cobble zone 4 material placed in the embankment was assumed to be free draining in the analyses. The factors of safety obtained for the end of construction condition are shown on Table 3.

The steady seepage condition was examined with the reservoir water surface at the maximum elevation of 1884.3 m (6182 ft) and the phreatic surface established by the top of joint use water surface elevation of 1879.5 m (6166 ft). The factor of safety was obtained for different circular searches and wedge surfaces for the steady seepage condition assumptions. The minimum factor of safety obtained for a significant failure surface is shown on Table 3 and meets the Reclamation design requirements (USBR, 1987).

For the steady seepage condition, the internal drainage systems were assumed fully functional. The clayey zone 4A placed in the downstream shell was assumed to remain unsaturated at a moisture content near optimum. Pore pressures in the relatively thin sloping core were estimated by assuming full hydrostatic pressures on the upstream face of the core and atmospheric pressures on the downstream underside of the core.

For the rapid drawdown condition, no pore pressure dissipation due to drainage in the zone 1 core material was assumed. The zone 4 gravel and cobble upstream shell was assumed pervious enough to drain completely during the reservoir drawdown. Construction pore pressures in the clayey zone 4A materials were assumed to have dissipated completely.

The rapid drawdown condition was examined for a complete draining of the reservoir. This was assumed to follow the establishment of steady seepage conditions with the reservoir at elevation 1879.5 m (6166 ft). The values of the factor of safety obtained for the different surfaces for the rapid drawdown conditions are shown on Table 3.

6. SEISMIC GROUND MOTIONS

The maximum credible earthquakes (MCE's) derived for the Jordanelle damsite are shown in Table 4 (USBR, 1988). The Wasatch fault event (No. 3, Table 4) is considered to be the more probable event. It has a recurrence interval for a magnitude 7.0 to 7.5, earthquake of 330 (± 90) years for the entire fault zone (Youngs, 1987). The two segments of the Wasatch fault nearest to Jordanelle Dam were assigned recurrence intervals of about 2,000 years.

Three separate accelerograms were used to represent potential earthquakes for analysis. The first was a record developed at the University of California, Berkeley, by Professor H. Bolton Seed. It represents a near-field magnitude 7.25 earthquake on the Hayward Fault in the San Francisco Bay area (Seed, 1983, Pike, 1991). It was scaled to a peak acceleration of 0.67g. This was to represent a magnitude 6.5 event in the immediate vicinity, not associated with a specific fault. This resulted in a maximum velocity of 76 cm/s (30 in/s) and a velocity spectrum intensity of 286 cm (113 inches). The latter value is very large in comparison with available data from historical earthquakes (Von Thun, 1988). The record has a total duration of 23.7 seconds and contains a long-period section referred to as a "fling". This fling causes the motion to have a great deal of energy in the 1.0 to 1.5 second period range. Figures 4 and 5 show the acceleration time history and the acceleration response spectrum for this earthquake record.

The second design earthquake record is a synthetically generated accelerogram considered to be representative of a mean magnitude 6.5 local event. It was developed for use in this type of seismic setting, (Munoz, 1986). It is characterized by a peak acceleration of 0.55g, a peak velocity of 35 cm/s (14 in/s), a total duration of 20.5 seconds, and a velocity spectrum intensity of 163 cm (64 in).

The third earthquake record represented a magnitude 7.5 event on the Wasatch fault at a distance of 30 kilometers (19 miles). An accelerogram developed by H. B. Seed and I. M. Idriss to represent the far field of a magnitude 8.25 earthquake was used. It was scaled to a peak acceleration of 0.29g. This resulted in a record with a peak velocity of 25 cm/s (10 in/s), a velocity spectrum intensity of 171 cm (67 inches) and a total duration of 48 seconds.

7. DYNAMIC DEFORMATION ANALYSIS

Seismically induced embankment deformations were evaluated using the computer program DYNDSP (Von Thun, 1981). This program is based on the sliding block procedure by Newmark (1965) for estimating permanent displacements. It is assumed that slide planes develop within the mass and that the material above the slide plane behaves as a rigid body.

The DYNDSP program uses the limit equilibrium method. It computes an incremental permanent displacement along the sliding plane each time the earthquake loading causes the driving forces along the plane to exceed the resisting forces. The effective angle of motion is calculated as the weighted average of the angle of sliding of each individual slice. The computed displacement is, therefore, calculated as the movement of the sliding mass along the failure surface in the downslope direction. Selective incorporation of the effects of horizontal and vertical inertial forces, excess pore pressure development, shear strength reduction, and dilation is included.

The program estimates the relative displacement of a specific failure mass given the applicable time history of both horizontal and vertical accelerations. The critical surfaces for a structure must be found by the examination of many failure surfaces. The total displacement is determined by integration of the results from these individual surfaces.

Several failure surfaces and driving acceleration records were analyzed for determining response to the MCE's. The results are shown in Table 4.

The shear strength parameters used in the original, preconstruction dynamic analysis are shown in Table 2. These strengths reflected no loss in shearing resistance due to dynamic loading conditions. It was assumed that the embankment would be compacted to a density sufficient to preclude significant loss of shear resistance associated with excess pore pressures in the saturated zone 4 material (upstream). The materials within the downstream shell (zone 4 and 4A) were assumed unsaturated. The downstream failure surfaces were, therefore, located primarily in unsaturated material, a planned result of the sloping core design.

The results of the analysis indicate that up to 1.4 m (4.6 ft) of vertical deformation is possible. This occurred on a circle which passes through half of the crest to one-fourth of the distance down the downstream slope (Fig. 6). The remaining material from the crest would be expected to slough into a more stable position. Total loss of crest elevation would not be expected to exceed the 1.4 m (4.6 ft). All of the deeper failure surfaces evaluated resulted in less deformation. Hence, it was concluded that with 5.7 m (18.6 ft) of designed freeboard above normal reservoir elevation, the embankment would not lose its reservoir-impounding capacity.

The first design earthquake record previously discussed caused significantly larger deformations than the synthetic record or the Wasatch fault earthquake (Table 3). Even so, the displacements of up to 1.4 m (4.6 ft) on shallow upper surfaces, if they were to occur, would require repair, but would cause no loss of the reservoir water.

The largest computed vertical displacement from the synthetic earthquake record was 0.55 m (1.8 ft). This is believed to be the largest seismic loading with a reasonable probability of occurrence. Under this earthquake loading the embankment is expected to retain approximately 90 percent of the design freeboard.

The magnitude 7.5 earthquake on the Wasatch fault has a predominance of long period vibrations in the design record. This caused amplification of the motions within the embankment up to a peak acceleration of 1.2 g near the crest of the dam. This, coupled with a total duration of 48 seconds, yielded calculated vertical displacements up to 0.85 m (2.8 ft). The largest displacement was located on the shallow upstream shear surface (Table 3). Over 4.7 m (15.5) ft of freeboard would remain after this seismically induced deformation.

As a check on the reasonableness of the results presented above, a simplified analysis was also performed. The procedure described by Makdisi and Seed (1977) was used to estimate the earthquake-induced deformations of the embankment with and without the clayey zone 4A in the downstream shell. Deformations were computed for potential failure surfaces through the full height of the embankment, the upper half of the embankment, and the upper quarter of the embankment. The analysis was performed using the computer program SEIDAKA (Adhya, 1982), which uses the Makdisi-Seed (1977) procedure. Maximum estimated deformations were:

0.06 m (0.2 ft) through the full height,
0.24 m (0.8 ft) through the upper half, and
2.0 m (6.5 ft) through the upper quarter.

These displacements compare reasonably with the values obtained using the more rigorous DYNDSP procedure, and confirm the acceptable performance of this design.

As a further check on the probable performance under earthquake shaking, literature reports were reviewed. Several gravelfill or rockfill dams, of varying size and design, which have been subjected to severe seismic shaking suffered minimal, or no damage (Bureau, 1986, and Seed, 1977). Two modern, well-instrumented rockfill dams with thin impervious zones survived strong shaking from the 1985 magnitude 8.1 Mexico earthquake with minor cracking and settlement (Bureau, 1986). LaVillita Dam is located about 24 km (15 miles) from the epicenter of the main shock. It recorded about 50 seconds of ground motion with a peak acceleration of 0.13 g. El Infiernillo Dam, about 74 km (46 miles) from the epicenter recorded an estimated peak acceleration of 0.5 g on the dam crest. These two dams have been subjected to 5 other earthquakes of magnitude 6.5 or larger since 1964, as well as a magnitude 7.5 aftershock to the 1985 event, and are continuing to perform well. Jordanelle Dam has been designed to provide seismic behavior comparable to these two dams.

Following the completion of the laboratory testing on the zone 4 soils at the University of California, Berkeley, the deformation analyses were repeated using the reduced shear strengths. Only materials placed during stage 1 were considered capable of experiencing significant loss of effective shear resistance. It was conservatively assumed that widespread pore pressure increase occurred in zone 4 materials placed during stage 1 in the saturated upstream shell. The analysis results indicated that the embankment would retain sufficient strength to remain stable. Seismically induced deformations would still be considerably less than the available freeboard.

8. CONSTRUCTION MONITORING

Construction began in 1987 as stage 1 of a two-stage construction process. It included stripping the abutments and west side of the valley to bedrock, producing zone 2 and 3 filter and drain materials, and placing embankment material on the west side of the valley up to the original ground elevation. Stage 2 construction, which includes all remaining construction activities, began in 1989. It is scheduled to be completed in early 1993. At the time of this writing, construction of the outlet works tunnels is completed, the embankment is complete to approximately elevation 1838 m (6030 ft), or 40 m (130 ft) above original ground, and construction of the emergency spillway is complete.

As anticipated, material placed in the downstream zone 4A portion of the embankment have varied in origin and composition. Much of the material placed to date has been material from required excavations, with a wide range of gradations. In general, the material placed has been superior to the "poorest" material considered during design analyses.

Due to an overabundance of zone 3 drainage material from the stage 1 material processing, several upstream horizontal drainage layers have been incorporated into the design of the embankment. These layers will serve to allow rapid drainage and dissipation of pressures in the event of rapid drawdown or seismic shaking, providing an extra margin of stability.

9. CONCLUSIONS

Testing of the materials being placed in the embankment confirm that the dam is being constructed in accordance with specifications requirements. Close monitoring and inspection of construction activities is being carried out by construction and design personnel. This assures that the design is being properly adapted to site

conditions, and that design concepts and assumptions are valid for the as-built structure.

The availability of excellent materials for embankment dam construction made construction of Jordanelle Dam with a relatively steep 1-1/2:1 downstream slope in a seismically active area feasible. Investigations and analyses indicate that the structure will be stable and perform well for all anticipated static and dynamic loading conditions. The analyses were carried out using commonly available methods to calculate the static factors of safety and anticipated seismic performance of the dam. Careful construction monitoring and reevaluation of the design assumptions during construction have indicated satisfactory conformance to design concepts with regard to both materials and densities in the field.

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EMBANKMENT ZONE	TYPE OF MATERIAL
Zone 1	Lean clays, sandy-gravelly clays
Zone 2	Processed sands and gravels, max size 5 cm
Zone 3	Clean gravels and cobbles, max size 20 cm
Zone 4	Well graded sand, gravel, cobbles and boulders, max size 61 cm
Zone 4A	Variable, ranging from lean clays to rockfill
Zone 5	Cobbles and boulders, max size 61 cm

Table 1. Types of Materials in Embankment Zones

Principal Embankment Zone	Loading Conditions			
	End of Const.	Steady Seepage	Rapid Drawdown	Dynamic
Zone 1 Conf. stress <275,000 N/m² (40 psi)	c=11 psi ϕ = 21°	c=2.5 psi ϕ = 28°	c=2.5 psi ϕ = 28°	c=2.5 psi ϕ= 28°
Conf. stress >275,000 N/m² (40 psi)	c=2.5 psi ϕ = 28°	c=2.5 psi ϕ = 28°	c=16 psi ϕ = 13°	c= 16 psi ϕ = 13°
Zone 2	c=0 ϕ = 35°	c=0 ϕ = 35°	c=0 ϕ = 35°	c=0 ϕ = 35°
Zone 3	c=0 ϕ = 30°	c=0 ϕ = 30°	c=0 ϕ = 30°	c=0 ϕ = 30°
Zone 4	c=0 ϕ = 44°	c=0 ϕ = 44°	c=0 ϕ = 44°	c=0 ϕ = 44°
Zone 4A Full pore press. dissip.	----	c=2.5 psi ϕ = 26°	c=2.5 psi ϕ = 26°	c=2.5 psi ϕ = 26°
No pore press. dissipation				
a) conf. stress < 550,000 N/m² (80 psi)	c=11 psi ϕ = 17°	c=11 psi ϕ = 17°	c=11 psi ϕ = 17°	----
b) conf. stress > 550,000 N/m² (80 psi)	c=40 psi ϕ = 0	c=40 psi ϕ = 0	c=40 psi ϕ = 0	----

Table 2. Material Strengths Used for Preconstruction Stability Analysis.

		End of Const	Steady Seepage	Rapid Drawdown
Downstream	Infinite slope	1.5	1.5	---
	Circle	1.5	1.5 1.4*	---
	Wedge	---	---	---
Upstream	Infinite slope	1.9	1.9	---
	Circle	2.1	2.0	2.0
	Wedge	2.7	4.5	2.4
*No dissipation of construction pore pressure in zone 4A.				

Table 3. Factors of Safety for Static Stability

EQ.	Magnitude	Distance	Earthquake Record Used	Peak Velocity	Maximum Vertical displacement (m)	
					Clay zone 4A	Gravel zone 4A
No. 1	M_L 6.5	local	Scaled Pacoima-Taft	76 cm/s	0.4 u/s 0.5 d/s	1.3 u/s 1.4 d/s
No. 2	M_L 6.5	local	Synthetic, USBR	35 cm/s	0.3 u/s 0.3 d/s	0.5 u/s 0.5 d/s
No. 3	M_s 7.5	30 km.	Scaled, Seed-Idriss	25 cm/s	0.4 u/s 0.1 d/s	0.9 u/s 0.3 d/s

Table 4. Maximum Credible Earthquakes and Seismically-Induced Deformations.

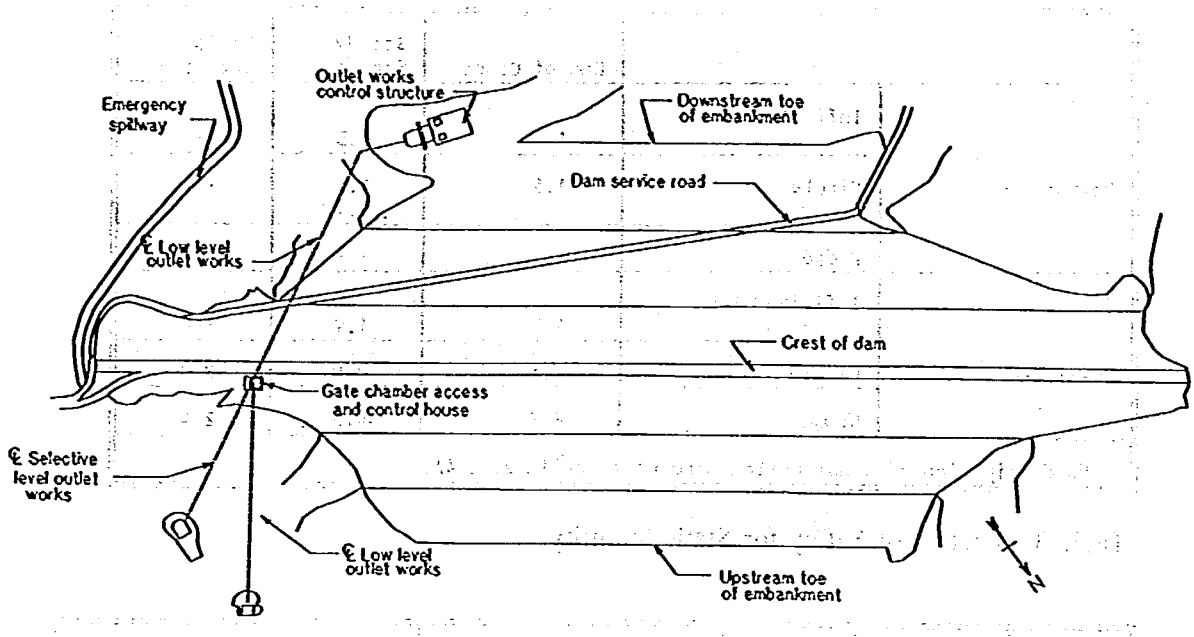


Figure 1 - Jordanelle Dam General Plan

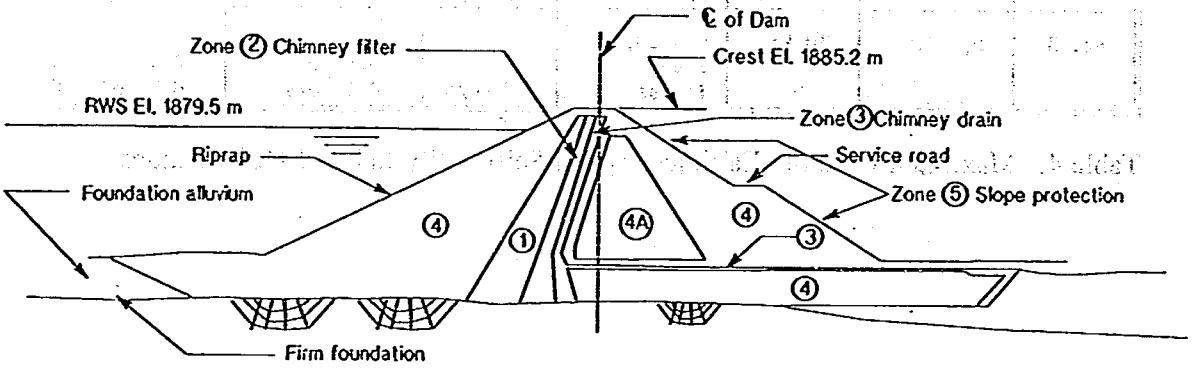


Figure 2 - Jordanelle Dam Embankment Section

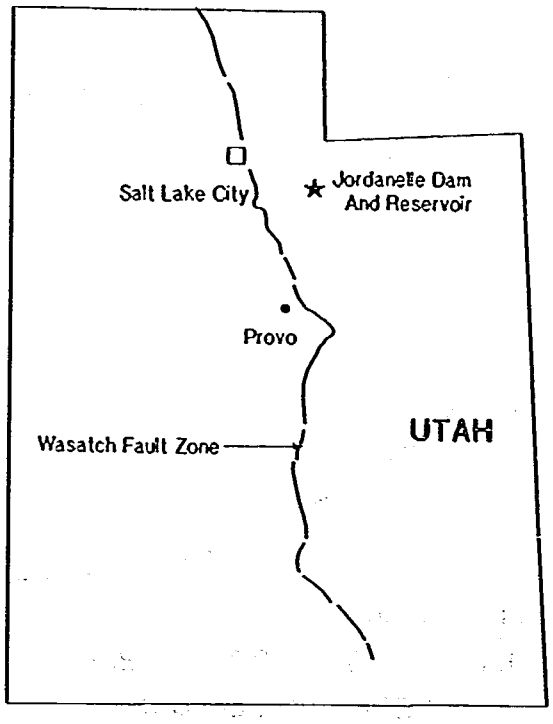


Figure 3 - Location of Jordanelle Dam and Wasatch Fault Zone

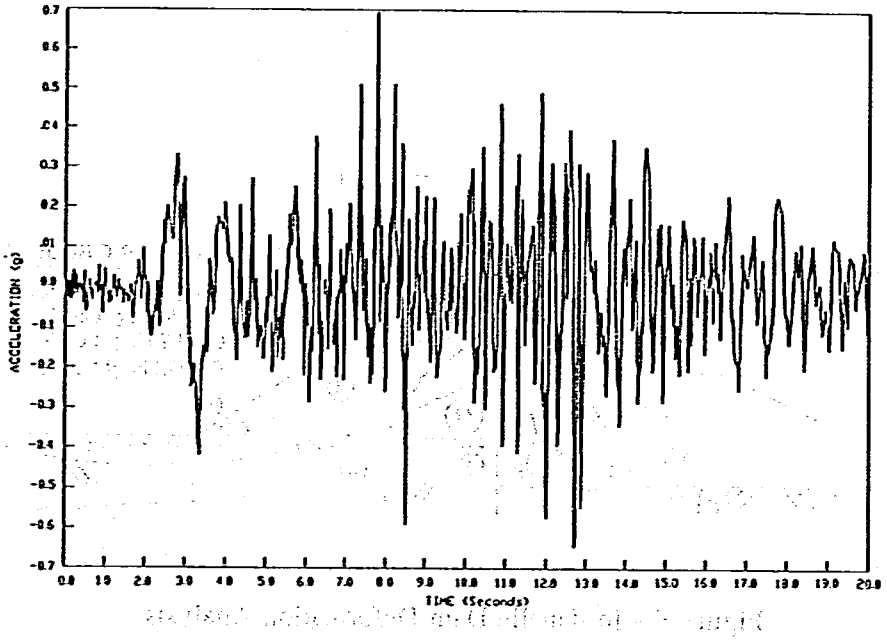


Figure 4 - Jordanelle Dam Design Accelerogram

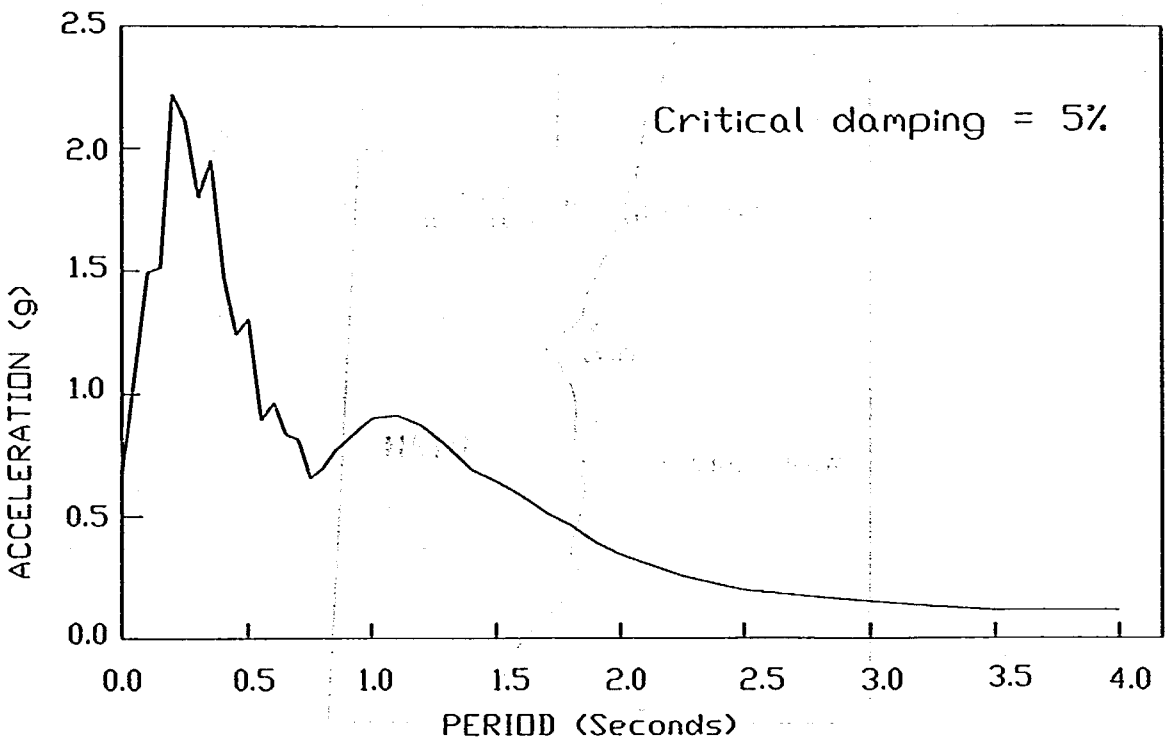


Figure 5 - Jordanelle Dam Acceleration Response Spectrum

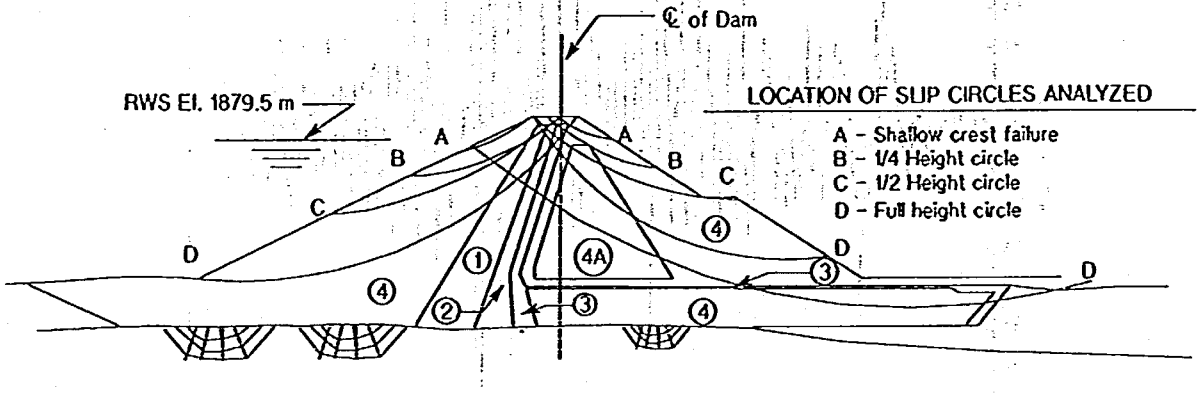


Figure 6 - Jordanelle Dam Deformation Analysis

フィルダムに作用する地震力およびその簡易推定法

原沢保彦[*1]、安田成夫[*2]、松本徳久[*3]、山邊建二[*4]

要　約

フィルダム堤体に作用する地震力が、1次元、2次元の動的解析により検討された。本報告では、実地震動記録を用い、地震時におけるダム堤体の地震力分布を計算した。種々の周波数成分を有する入力地震動を用いた応答計算により、地盤動の周波数特性と最大応答加速度の関係が明かとなった。また、地盤動の加速度応答スペクトルを用い、任意の深さのすべり土塊に作用する最大平均加速度を推定する方法も示した。

キーワード：フィルダム、最大加速度、動的解析、地震動、永久変形

1．まえがき

本論文は、基盤における地震動の周波数特性とフィルダム堤体に作用する地震力の関係を論じたものである。すなわち種々の周波数特性の入力地震動に対して堤体の応答を求め、地震力の周波数依存性を明らかにし、さらに、地盤の地震動応答加速度スペクトルから簡易に地震力を求める手法を開発した。

Newmark[1]やSeed[2]以来、繰返し荷重を受けて強度低下することのない材料で築造されているフィルダムでは、地震時の堤体の安全性を地震によって生じる堤体の永久変形から評価する方法が発展してきた。この永久変形量の計算では、堤体内に仮定したすべり土塊に作用する地震時慣性力をその土塊の質量で除すことによって得られる平均化された加速度（Average acceleration:以下$\overline{a}$）とその地震動継続時間中の最大値（Maximum average acceleration:以下$\overline{a}_{max}$）が主要なパラメーターとなる。すなわち、すべり土塊の安全率を1.0とする加速度を限界加速度（Yield acceleration:以下a_y）とすると、フィルダムの地震による変形量は$\overline{a}$がa_yを越えた時刻よりすべりが始まるとの考えから求められるが、近似的には単にa_yと$\overline{a}_{max}$の比から推定できる。例えばWatanabe et al.[3]は周期0.125秒から0.625秒までの5種の正弦波入力に対して、Newmarkの方法によるすべり計算を行い、天端の永久変形量は、すべり土塊の位置によらず$\overline{a}_{max}$と$\overline{a}$がa_yを越える地震動継続時間の2乗に比例するとしている。また Makdisi et al.[4]は実地震波に対するNewmarkの方法によるすべり計算から、地震のマグニチュード別の永久変形量と$a_y/\overline{a}_{max}$の関係を示している。さらに、$\overline{a}_{max}$は、実務で使用されている震度法による安定計算において、震度の概念と密接な関係にあり、フィルダム堤体に作用する地震力を表現している。土の強度特性を考えると1回の短時間の荷重では大きなひずみは進行せず、ある一定回数以上の繰返し荷重を受けたときにひずみが進行する。そこで、ある大きさ以上の$\overline{a}$が一定回数以上繰返すときその$\overline{a}$を等価加速度（Equivalent accelera-tion:以下$\overline{a}_{eq}$）とし、これをもって震度法の震度とする考え方もある[5]。当初Seed et al.[5]やAmbraseys et al.[6]によってダム堤体を均質な三角形でモデル化しすべり面を直線で近似したウェッジ形のすべり土塊の$\overline{a}_{max}$は、すべり土塊の標高および堤体の固有周期によって異なることが明らかにされた。これらの数値解析においてすべり土塊の$\overline{a}_{max}$の算定はMononobe et al.[7]によるせん断梁理論の1次元動的解析手法が用いられた。のちにChopra[8]によって、1次元のせん断梁理論で求められた$\overline{a}_{max}$が有限要素法による2次元動的解析のそれと比べて平均的に10～20%程度、大きめとなることが示された。

$\overline{a}_{max}$は、堤高をH、天端標高とすべり円弧の最低標高の差をyとしたとき、y/Hの関数となる。$\overline{a}_{max}$を天端の最大応答加速度$\ddot{u}_{max}$で除し無次元化し、これとy/Hの関係でプロットすると無次元化した$\overline{a}_{max}$の曲線（以下単に加速度曲線）が得られる。Makdisi et al.[9]によれば、加速度曲線は解析方法の1次元と2次元とでは大きな差異はなく、堤体の固有周期によってある程度の変動があるものの、平均値を採用したとき、変動の上下限は上位標高で±10%～±20%、下位標高で±20%～±30%の範囲にあり、平均値を実務上の設計曲線として使用してよいとされる。ただし安全側の設計を行うときには平均値よりも10～30%大きめの値を使うことが推奨されている。加速度曲線から地震力を求めるときには、$\ddot{u}_{max}$を定める必要があるが、Makdisi et al[9]は設計地震動の加速度応答スペクトルと1次元せん断梁理論によるモード重畳法から2乗平均によって簡易に求める手法を提案している。Luan et al[10]は堤体法勾配、減衰定数、材料のせん断弾

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度から定まるすべり面形状が加速度曲線に与える影響を研究し、加速度曲線はNakdisi et al.の結果が中位標高以下では過小の側にあること、堤体の法勾配および材料強度にはほぼ無関係であるとの結論を得た。

$\overline{a}_{max}$は以上のように、解析手法、堤体の形状、すべり土塊の標高、堤高、物性値との関連で研究されているが、入力地震動の周波数特性との関係についての研究がほとんどない。そこで本論文では、$\overline{a}_{max}$と入力地震動の周波数特性の関係を論じ、かつ加速度応答スペクトルから簡易に求める手法をとりまとめたものである。

2. 解析モデルと解析手法

2．1. モデルダム

解析に用いたモデルダムは上下流方向断面を対象とし、上流面勾配1:2.6、下流面勾配1:2.0、高さ63mと設定している。1次元動的解析では、Fig. 1(a)、2次元動的解析の場合はFig. 1(b)を用いる。

2．2. モデルダムの物性値

ロックフィルダムの築堤材料は大半がロック材料であり、フィルタ材料およびコア材料の物性値を解析に考慮してもその与える影響は小さいと考え、モデルダムの物性値は堤体全体についてロック材料の物性値を与えた。せん断弾性係数は、ロック材料の室内繰返し三軸試験より設定し、拘束圧依存性も考慮した。減衰定数は堤体の応答加速度に大きな影響を与え、その設定は解析上重要な問題である。減衰定数は、材料自体の内部減衰と、基礎岩盤・貯水池などの境界条件に関わる逸散減衰から成っている。Fig. 2はロックフィルダムのロック材料の基準化したせん断弾性係数および減衰定数のひずみ依存曲線を表したものであり[11]、ロック材料の内部減衰は10^{-3}近傍のひずみに対して約10%程度の大きさを示している。また、貯水位がある場合逸散減衰が増加することは実験的に示されており[12]、Ohmachi et al.[13]の実測からは満水時には約10%程度になることが示唆されている。したがって、満水時における大地震時の減衰定数は全体で約20%程度ということになる。また、既往の研究[5][7][9][10]においても20%の減衰定数が解析に用いられており、ここでは設定する地震の規模や既往の研究結果を踏まえ減衰定数をせん断ひずみ、平均主応力に関わらず一律20%と設定した。

2．3. 解析手法

動的解析手法としては、1次元動的解析のモード重畳法，SHAKEH[14]と2次元動的解析のQUAD-4[15]，FLUSH[16]の4手法を用い、各々の手法ですべり土塊の$\overline{a}$を算出した。

モード重畳法は、多質点系構造物の振動を規準振動形の直交性を利用することにより1自由度系の振動の重ね合わせで表現し、これにより地震時における構造物の応答解析を行う手法である[17]。モード重畳法をフィルダムに適用するにあたっては、Fig. 3に示すようにダムを質点とバネでモデル化した。質点は各層の全重量 m_iを各層の重心位置に設定し、バネは質点間に配置する。添字のiは質点番号を表している。バネ定数k_iは式(1)で与えられる。

$$k_i = \frac{\ell_i G_i}{d_i} \qquad \cdots(1)$$

ここに、ℓ_iは質点（重心）位置におけるダムの幅で、d_iは質点間の距離、G_iはせん断弾性係数を表している。

i層より上部のすべり土塊の$\overline{a}$は次式で算定される。

$$\overline{a} = \frac{F_i}{\sum_{n=i}^{N} m_n} \qquad \cdots(2)$$

ただしF_iはi層と(i－1)層間で働くせん断力であり、$F_i = k_i \times (y_i - y_{i-1})$で表される。ここに、$y_i$はi層の質点の変位である。

SHAKEHはSHAKE[18]を有限地盤に適用できるように改良したものである。この手法におけるすべり土塊の$\overline{a}$は、円弧内の各層の面積で重みをつけた平均値で算出した。

QUAD-4、FLUSH におけるすべり土塊の$\overline{a}$は、円弧内の節点を共有する要素面積の重み付きで算出した。

2．4. 入力地震動

入力地震動は本来設計対象とする地震の特性に応じて決められるべきものである。岩盤における実測加速度記録によると、同一のダムサイトで、同じ程度のマグニチュードの地震であっても地震そのものが異なると地震動の周波数特性が著しく異なる場合があり、安全側の設計の立場からは、入力地震動には種々の周波数特性のものを想定しておくべきと考える。したがって、堤体に作用する地震力の評価において、周波数特性の異なる種々の実地震動を用いた応答解析を実施した。ここでは、ダムサイト岩盤で観測された基盤の地震動記録から、地震動のフーリエスペクトルの卓越周波数に着目して周波数特性の異なる8地震動を選定した。地震動の諸元をTable 1に示す。入力地震動最大加速度$\bar{g}_{max}$が 0.2G（G：重力加速度）となるように、実地震動の加速度の補正を行った。Fig. 4はTable 1の8個の地震動の減衰定数が20%の時の加速度応答スペクトルを示したものである。

3. 予備的検討

3．1. 鉛直動の影響

まず予備的検討として、すべり土塊の$\overline{a}$の算定にあた

り、応答加速度における鉛直成分および入力地震動における鉛直地震動の影響について検討した。式(3)から求まるすべり土塊の加速度モーメントM_sから鉛直応答加速度と鉛直地震動の影響を評価することとした。

$$M_s=\sum_{i=1}^{n}\{m_i\times(a_{xi}\times r_{yi}+a_{yi}\times r_{xi})\}\ /\sum_{i=1}^{n}m_i \quad \cdots(3)$$

ここで、a_{xi}, a_{yi}はそれぞれ水平、鉛直応答加速度であり、r_{xi}, r_{yi}は円弧の中心から要素質量m_iの中心までの水平、鉛直距離である。また、鉛直地震動の影響を併せて評価するにあたって、以下の３ケースを設定した。

①地震動として水平地震動のみを入力し、動的解析で得られる水平応答加速度時刻歴を用いてM_sを算出する(Case@)。

②地震動として水平および鉛直地震動を入力し、動的解析で得られる水平応答加速度時刻歴を用いてM_sを算出する(Case⑥)。

③地震動として水平および鉛直地震動を入力し、動的解析で得られる水平および鉛直応答加速度時刻歴を用いてM_sを算出する(Case©)。

計算は二次元解析のQUAD-4を用い、Fig. 1(b)のモデルダムに対して実地震波を入力し、すべり土塊のM_sの地震動継続時間中の最大値を求めた。なお、地震動はTable 1に示した実地震動のCase5を用いるが、ダムサイト岩盤で記録された地震動のうち、水平最大加速度と鉛直最大加速度の比が1/4～1/2の間にあることから[19]鉛直地震動の最大加速度は水平の1/2とした。

Fig. 5は計算結果であるが、鉛直地震動の影響については、Case@とCase⑥はほぼ同様な曲線形状を示し、@が全体にわたって⑥よりも4.0%程度大きめになっている。Case@とCase©を比較するとy/H<0.6で最大4.0%程度©が小さい。その関係はy/H=0.7を境にして逆転し、y/H=1.0で最大約10%Case©の方が大きくなる。このことから、入力地震動に鉛直成分および鉛直応答加速度を考慮した場合のM_sへの影響は水平成分のみの場合に比べて高々10%と判断される。なお田村・岡本[20]はフィルダムの安定性に及ぼす鉛直動の影響について正弦波形を用いた実験および解析結果に基づく評価を行い、水平動の1/2の鉛直動が作用する場合の効果は水平動が10～20%程度大きくなったのと同じ程度の効果としている。

以上のことから鉛直動及び鉛直応答加速度の影響は少ないと判断し、計算を簡便にすることからも、入力としては水平地震動のみ、応答加速度も水平方向のみのCase@とすることとした。

３．２．　平均加速度$\bar{a}$の算定方法

すべり土塊の$\bar{a}$は、一般に土塊に作用する水平力から求められるが、土塊のモーメントから計算することも考えられる。そこで以下で両者を比較する。

①土塊の円弧中心に関するモーメントが等価となる$\bar{a}$を求める。すなわち、水平応答加速度時刻歴a_{xi}と要素質量m_i、円弧の中心から各要素重心までの鉛直距離r_{yi}を用いて、$\bar{a}$を次式で算定する(Case@)。

$$\bar{a}=\sum_{i=1}^{n}\{m_i\times a_{xi}\times r_{yi}\}\ /\sum_{i=1}^{n}\{m_i\times r_{yi}\} \quad \cdots(4)$$

②土塊の水平力が等価となる$\bar{a}$を求める。すなわち、各要素のa_{xi}とm_iを用いて、$\bar{a}$を式(5)で算定する(Case©)。

$$\bar{a}=\sum_{i=1}^{n}\{m_i\times a_{xi}\}\ /\ \sum_{i=1}^{n}m_i \quad \cdots(5)$$

計算の結果、Fig. 6に示すように式(4), (5)で得られる$\bar{a}_{max}$がほぼ一致することから、以下では式(5)により$\bar{a}_{max}$を算定している。

4．　最大平均加速度$\bar{a}_{max}$

各地震動に対する円弧すべり土塊の$\bar{a}_{max}$とy/Hの関係をFig. 7に示す[21]。y/Hが大きくなると、すなわち円弧が深くなると$\bar{a}_{max}$は小さくなり、ダム基礎に接するすべり土塊(y/H=1)の$\bar{a}_{max}$はCase 1を除いて基盤加速度の最大値$\bar{g}_{max}$にほぼ等しいかそれより小さい。また、地震動の卓越周波数が高いCase6～8は特にダム基礎における$\bar{a}_{max}$は小さい。卓越周波数が0.59HzであるCase 1は任意のy/Hに対して８波中最大となり、天端での$\bar{a}_{max}$は$\bar{g}_{max}$に対してほぼ3.5倍となっている。Case 4の地震動の卓越周波数は堤体の固有周波数に最も近いが、Case 4の$\bar{a}_{max}$はCase 1と比べて天端で約300gal程度小さく、卓越周波数だけでは天端における$\bar{a}_{max}$の大小の傾向はつかめないことがわかる。なお、Case 1のフーリエスペクトルは、0.3秒以上の長周期成分が卓越した正弦波に近い波である。また、Fig. 4の応答加速度スペクトルとFig. 7の$\bar{a}_{max}$を比較すると、堤体の固有周期である0.6秒以上の長周期成分が多いCase 1、2、3、5において$\bar{a}_{max}$が大きい。

Fig. 8は４種類の解析手法毎に８波の入力地震動に対する$\bar{a}_{max}$の平均値を示したものである。この図から、１次元解析は２次元解析に比べ上位標高で15%程度大きいが、下位標高では差は10%程度である。$\bar{a}_{max}$の算定にあたって最も計算時間が短く経済的なモード重畳法を用いても、他の解析手法に比べほぼ満足できる結果が得られることがわかる。Fig. 9(a)は標高別のy/Hにおける最大応答加速度比$(\bar{a}/\bar{g})_{max}$の$\bar{g}_{max}$に対する倍率をTable 1の８入力波に対して求め、８波の平均値を示したものである。１次元解析は２次元解析に対して10%程度大きい応答になるが、全体に解析手法による応答の差は小さい。

y/H＞0.5では$\ddot{g}_{max}$に対する応答倍率は約1.0、y/H＝0.5より高位標高では標高とともに増加しy/H＝0（天端）で2.2～2.5となっている。Fig. 9(b)はQUAD-4によって計算された8波に対する$\bar{u}(y/H)_{max}/\ddot{g}_{max}$の分布である。

Figs. 8,9に示される計算値を実物ダムの地震時挙動と比較してみよう。Fig. 10(a) は建設省所管のフィルダムにおいて実測された天端での応答倍率と$\ddot{g}_{max}$の関係である。この図によれば$\ddot{g}_{max}$が20gal以下では応答倍率は3～6倍程度あるが$\ddot{g}_{max}$が増加すると低下し、特に100gal以上では2～2.5倍となっている。このことはひとつには$\ddot{g}_{max}$の増加とともに減衰が増加する非線型性による。ふたつには$\ddot{g}_{max}$の小さな地震は震央距離の遠いものが多く地震動の周波数が比較的長く堤体の共振点付近にあるが、$\ddot{g}_{max}$の100galを越えるものはダムサイトに近いマグニチュードの小さな地震によるもので高周波であることによる。つまり、Fig. 10(a)の$\ddot{g}_{max}$が100galを越えた地震動の周波数特性は、Table 1の入力地震動で言えばCase6～8に相当するわけで、Fig. 9においてCase6～8のy/H=0 における$\bar{a}_{max}$が$\ddot{g}_{max}$の1.5～2.5倍にあることは実測に一致しており、動的解析における物性値の仮定がほぼ妥当であると言える。またFig. 10(b) は手持ちの建設省のダムの実測データに、文献[23),24),25),26)]に記されている結果を加えて整理したフィルダムの標高別の最大加速度分布である。この図には、$\ddot{g}_{max}$が5gal程度のものも含まれており、減衰が小さいために応答倍率はFig. 9(a)に比し全体に大きめであるが、Fig. 9(b)の標高別加速度分布は、Fig. 10(b)の下限値と平均値の範囲にある。すなわち、$\ddot{g}_{max}$が0.2Gのときには減衰が増加しFig. 10(b)の下限値と平均値の中間程度と考えるとFig. 9の解析結果は実測と一致していると言える。以上のことから本論文における解析は実測の結果に照らして妥当と考えられる。Fig. 11は、Fig. 9の8波の$\bar{a}_{max}$の平均値を$\bar{u}_{max}$で除して求めた加速度曲線である。なお、Fig. 11は、Makdisi et al.[1)]とLuan et al.[10)]の結果も記入してある。本論文の2次元動的解析の加速度曲線は、Luan et al.の分布とほぼ一致しており、Makdisi et al.の結果とは若干異なっている。Luan etal. は築堤材料のせん断弾性係数を拘束圧に依存させて解析しているのに対し、Makdisi et al.ではせん断弾性係数を一定と仮定しており、このせん断弾性係数の設定方法が$\bar{a}_{max}$の分布に影響を与えているものと考えられる。1次元動的解析による加速度曲線は2次元動的解析に比べて左側にずれ、円弧高さy/Hが0.3以上でほぼ平行であるが、これは1次元動的解析における$\bar{u}_{max}$が大きいために相対的にずれたものである。

Makdisi et al.やLuan et al.は、加速度曲線をFig. 11の曲線で表してすべり量の算定に用いているが、この分布は単に平均値にすぎず、地震動の周波数特性と堤体の固有周波数の関係は省略されている。$\bar{a}_{max}$はFig. 7に示すようにこれらの関係に大きく左右され、Case2，3の地震動に対してFig. 11の分布を適用すると中低標高部での$\bar{a}_{max}$を過小評価してしまうことになる。したがって、加速度曲線を用いる手法が妥当なものとしても、これらのことを考慮せずに単純に$\bar{u}_{max}$を設定して各標高の$\bar{a}_{max}$を推定すると大きな誤差を生じる恐れがある。設定された加速度曲線で各標高における$\bar{a}_{max}$の分布を推定する手法は簡便であるが、以上のような問題点があり、むしろ$\ddot{g}_{max}$を基準にしたFig. 7を基本として入力地震動の周波数特性に応じて$\bar{a}_{max}$の分布を求めた方が実務上誤りが少ないと考えられる。

5．　最大平均加速度簡易推定手法

円弧すべり土塊の$\bar{a}_{max}$は入力地震動の周波数特性に依存しており、周波数特性が異なれば応答は異なってくる。以下では、実地震動において$\bar{a}_{max}$に与える影響の大きい周波数成分の評価方法について検討した。

一般に構造物の任意の点の変位や応力などの物理量Rが、単位の振幅で周波数がf Hzの正弦波が入力したときf の関数R(f)で表されるとすれば、R(f)は周波数応答関数である。そこであるy/Hのすべり土塊の$\bar{a}_{max}$を多くの周波数f に対して計算により求めた。Fig. 12はFig. 1のダムの周波数応答関数をQUAD-4を用いy/H=0.0, 0.4, 1.0 の3標高で求めたものである。各すべり土塊の$\bar{a}_{max}$はf/f_0=1.0付近でピーク値をとっているが、すべり土塊の深さにより大きさが異なっている。いま$\bar{a}_{max}$に影響を及ぼす周波数成分は$\bar{a}_{max}/\ddot{g}_{max}$が1よりも大きくなる範囲としたが、すべり土塊の深さにより異なる。しかしながら計算の簡略化のため周波数範囲はFig. 12に基づき、すべり土塊の深さに依らず式(6)で示されると判断した。すなわち、フィルダムの地震時の応答には固有周波数が5～6次までの低次の周波数が支配的であり、f/f_0の上限はf/f_0=1.0の2倍までをとれば十分とし、下限については各すべり土塊の応答倍率が半分となるf/f_0=0.8とした。

$$0.8 \leq f/f_0 \leq 2.0 \qquad \cdots(6)$$

前述のようにMakdisi et al.は地盤の加速度応答スペクトルが荷重条件として与えられたとき、これを用いて$\bar{u}_{max}$を推定し、さらにFig. 11の加速度曲線と組み合わせて$\bar{a}_{max}$の設計値を求める手法を提案している。一方、Fig. 12に示す$\bar{a}_{max}$の周波数応答関数が存在することとその特性に着目すれば、荷重条件として与えられた加速

度応答スペクトルから直ちに精度よく$\overline{a}_{max}$を求めることが可能なはずである。すなわち入力される地震動の加速度応答スペクトルが与えられたとき、式(6)で示される周波数領域における加速度応答スペクトルの平均値S_aを算出し、これと動的解析から算出される$\overline{a}_{max}$の間に相関があれば、S_aから簡易に$\overline{a}_{max}$を算定することが可能となる。入力地震動としてTable 1の8個の地震動を用い、Fig. 4からS_aを算出し$\overline{a}_{max}$との関係を図示したものがFig. 13である。

また、前述のようにダムの堤高（固有周期）がすべり土塊の$\overline{a}_{max}$に及ぼす影響ついては、モード重畳法の計算結果をFig. 14に示す。これによれば、堤高が異なっても$\overline{a}_{max}/S_a$はy/Hに対して差がないことを示している。

以上は減衰定数hを20%とした結果である。h＝10%とした場合について新たに応答スペクトルを求め直し、$\overline{a}_{max}/S_a$とy/Hの関係を求めh＝20%の場合と比較するとFig. 15となる。なお、図中の$R_{0.1}$、$R_{0.2}$は、それぞれ10%と20%の減衰定数の場合、表－1の8波の入力地震動に対する$\overline{a}_{max}/\bar{g}_{max}$のばらつきの図中の回帰曲線との相関係数である。QUAD-4を用いた2次元動的解析の結果では、h＝10%と20%の差は小さく、近似式として次式が成り立つ。

$$\overline{a}_{max}=\{2.0-1.35\,(y/H)\}\times S_a \qquad \cdots(7)$$

以上の結果から、すべり土塊の$\overline{a}_{max}$は以下の手順を踏むことで簡易に推定できる。

① 地震動の加速度応答スペクトルを算出する。このとき減衰定数は、対象とするダムの減衰定数とする。

② ダム堤体の固有周波数を算定する。

③ 加速度応答スペクトルの平均値S_aを式(6)の周波数領域を対象として算出する。

④S_aを式(7)に代入して、あるいはFig. 15から各標高におけるすべり土塊の$\overline{a}_{max}$が算出される。

6. 永久変形量の推定と室内繰返し試験の載荷応力

堤体内のあるすべり土塊の限界加速度をa_yとし、入力地震動によるそのすべり土塊の最大平均加速度が$\overline{a}_{max}$であるとき、$a_y/\overline{a}_{max}\geq 1.0$であれば永久変形量$U_y$は発生しないが、$a_y/\overline{a}_{max}<1.0$のときには、$U_y$が生じ、$U_y$は$a_y/\overline{a}_{max}$の値と強い相関があり、$U_y$を$a_y/\overline{a}_{max}$から推定するチャートがMakdisi et al[9]、Luan et al[10]により提案されている。この考えは基本的にはNewmark[11]の方法を発展させたものであるが、本論では$\overline{a}_{max}$を用いて室内試験により永久変形量を推定する方法を考える。石原[27]は堤体内のすべり土塊について地震前の常時荷重による安全率と地震時の安全率をスライス法によって求めこれから静的および動的せん断応力を推定し、この応力条件での室内試験から永久ひずみを求める方法を提案している。本論文もこの考え方によるが、$\overline{a}_{max}$を用いて直接動的せん断応力を推定することとする。地震前に常時荷重によるすべり土塊のモーメントをM_sとすると、常時荷重による平均的なせん断応力τ_sはrを円弧半径、lを円弧長さとして

$$\tau_s=\frac{M_s}{r\cdot l} \qquad \cdots(8)$$

一方、地震荷重によるモーメントM_dは$\overline{a}_{max}$から求めることができ、地震による平均的なせん断応力τ_dは

$$\tau_d=\frac{M_d}{r\cdot l} \qquad \cdots(9)$$

堤体材料の供試体を平均的な拘束圧σ_xで圧密し、排水条件でτ_sを載荷し$\pm\tau_d$の繰返しせん断応力を非排水条件で載荷し、累積ひずみを求める。これにより、地震時の変形が推定可能となる。なお、載荷応力の波形は$\overline{a}$の時刻歴とすることが望ましいが、一定振幅のときには$\overline{a}_{max}$の代りに$\overline{a}_{eq}$を用いる。Fig. 16はFig. 2(b)のモデルダム上流面のすべり土塊の$\overline{a}_{max}$とすべり面のせん断応力τ（$\tau_s+\tau_d$）の関係についての計算結果である。Fig. 16の$\overline{a}_{max}=0$のときのτは式(8)より求められたτ_sであり、$\tau_d\leq\tau_s$となるτ_dの範囲が図中に破線で示されている。この図から$\overline{a}_{max}$が0.2G程度以下ではτ_dはτ_sより小さく、応力反転は生じてないことがわかる。Fig. 16を利用すれば室内繰返し試験により永久ひずみを求めるときの載荷条件を精度よく定めることができる。

Fig. 17は、ロック材料の繰返し載荷試験[28]より求めた繰り返し載荷回数N＝10回のU_yとτ_dの関係を示す。Fig. 16から、あるいは式(8)、(9)から対象とするすべり土塊に対するτ_sとτ_dを求め、このτ_sとτ_dをFig. 17に適用すれば、N＝10回の繰返し載荷したときの永久ひずみU_yを推定することができる。ただし、Fig. 17は平均主応力σ_x＝6.0kgf/cm²の場合であり、ロック材のせん断強度には拘束圧依存性があるから実験では平均主応力を何段階か変えてFig. 17の曲線を求めておく必要がある。

7. 結　論

種々の周波数特性を持つ地震動が、フィルダム堤体の地震力に及ぼす影響を4種類の動的解析手法を用いて評価し、$\overline{a}_{max}$の簡易推定手法について検討した。得られた結論を以下に要約して示す。

（1） 入力地震動は、水平成分のみとし、堤体応答加速度も水平成分のみを用いて$\overline{a}_{max}$を求めても、入力および応答において水平、鉛直2成分を考慮したときに比べ、高々10% 程度の誤差であり、水平成分のみから$\overline{a}_{max}$を求めることができる。

（2） $\overline{a}_{max}$は、対象とするすべり土塊について等価な

モーメントおよび水平合力から求める２方法があるが、両者の差は最大で５％程度である。すなわち計算がより簡単な水平合力から$\overline{a}_{max}$が求められる。

（３） 種々の周波数特性を持つ８波の地震動に対して４種類の解析方法で$\overline{a}_{max}$の分布を求めると、１次元解析である質点系モード重畳法およびSHAKEMは、８入力波に対する平均で、天端における応答加速度が15% 程度２次元解析よりも大きい。y/H＞0.3の標高では解析方法による$\overline{a}_{max}$のばらつきは10%以下であり、１次元解析による天端付近の応答がやや大きめの点を除けば全体に解析方法によるばらつきは小さい。

（４） $\overline{a}_{max}$は$\overline{g}_{max}$が同一であっても、周波数特性によって８波の平均を基準として±50%変動する。したがって、実務において、$\overline{u}_{max}$で無次元化されたHakdisi et al.による加速度曲線の平均値を用いるときには、当該サイトで想定される地震動の周波数特性との関連を検討すべきである。

（５） 入力地震動の加速度応答スペクトルが与えられたとき、y/Hの値に応じて式(6)の周波数範囲についてのスペクトルの平均値S_aを求めれば、式(7)あるいはFig. 15から$\overline{a}_{max}$を簡易に推定することができる。

（６） $\overline{a}_{max}$から式(8)および(9)を用いて静的および地震力によるせん断応力を推定し、これを用いて堤体材料の室内繰返し試験を実施すれば、地震力により堤体に生じる永久変形推定の精度を高めることができる。

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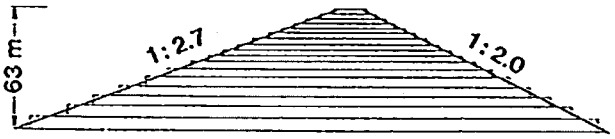
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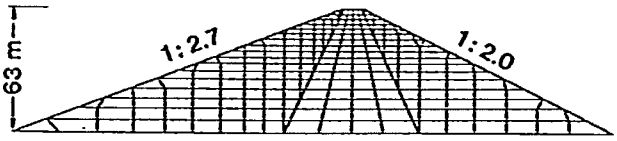
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(a) One-dimensional



(b) Two-dimensional

Fig. 1 Finite element idealization used in the analysis

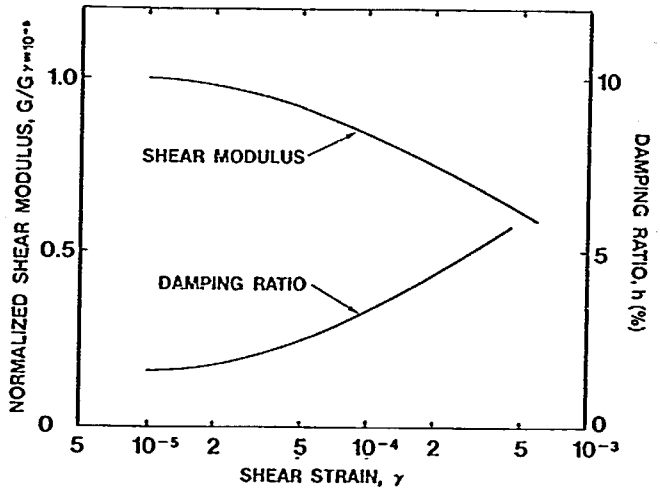


Fig. 2 Strain dependent curves of shear modulus and damping ratio used in response computations

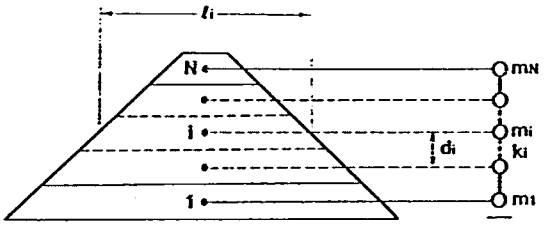


Fig. 3 Shear beam model with lumped mass (Modal Analysis)

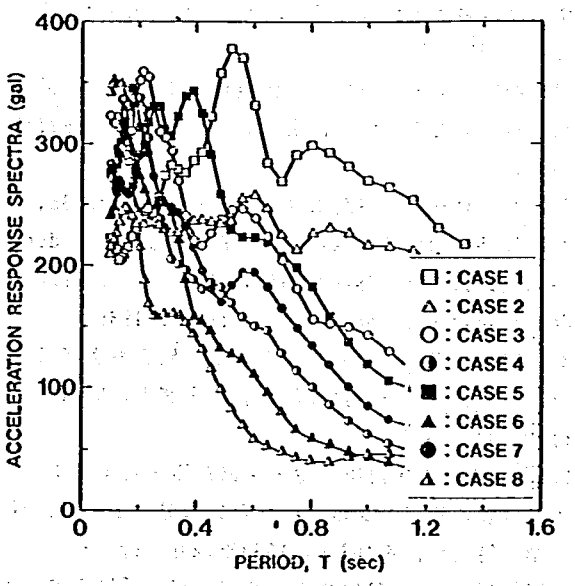


Fig. 4 Acceleration response spectra for input ground motions

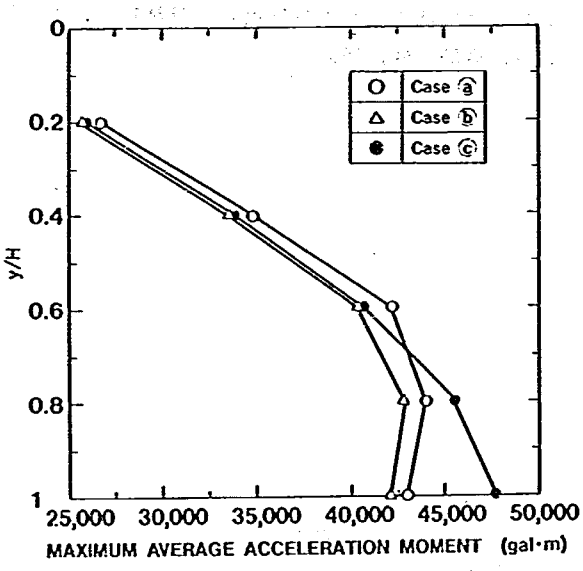


Fig. 5 Maximum average acceleration moment

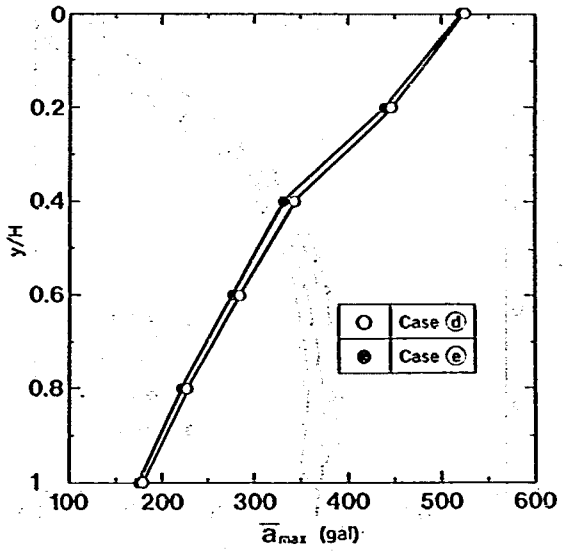
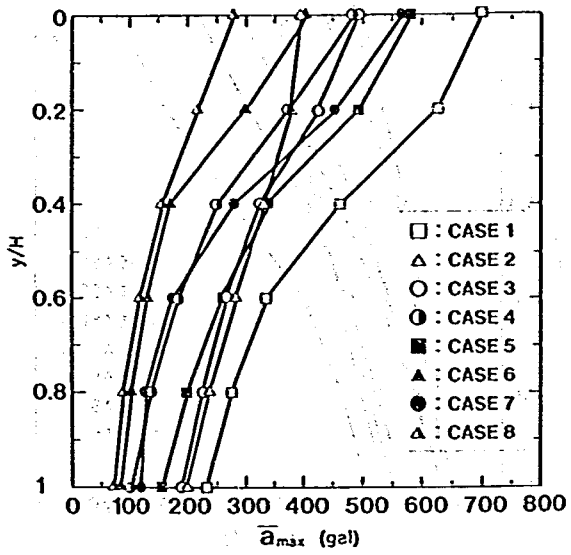
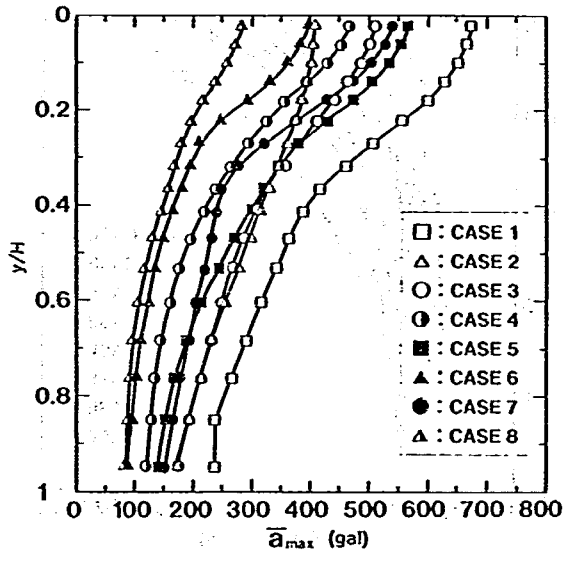


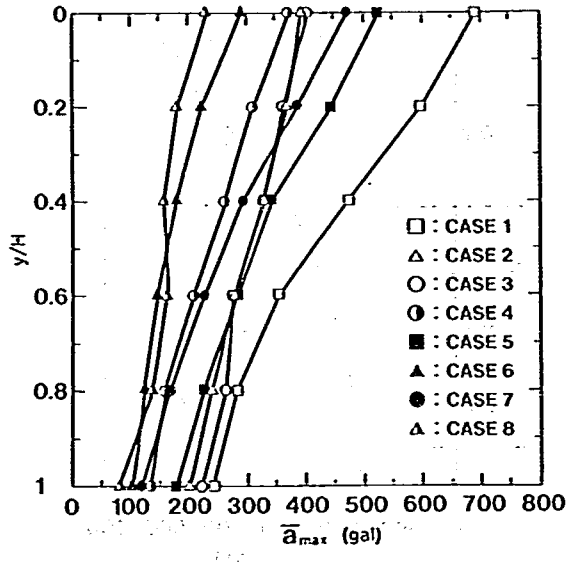
Fig. 6 Maximum average acceleration computed from moment and horizontal force



(b) SHAKEM Fig. 7(b)



(a) Modal Analysis Fig. 7(a)



(c) QUAD-4 Fig. 7(c)

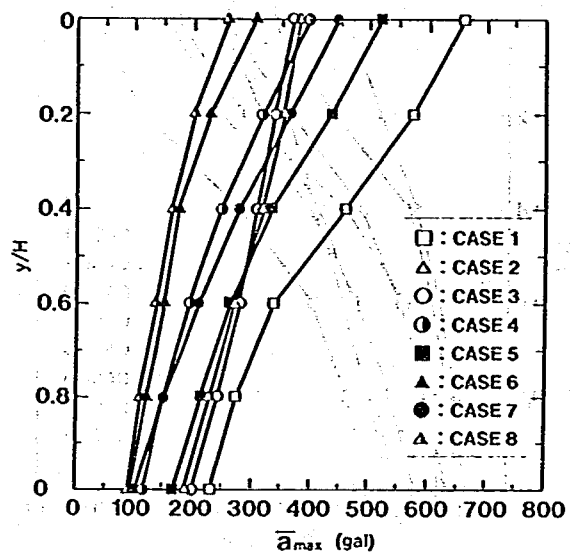


Fig. 7 Maximum average acceleration

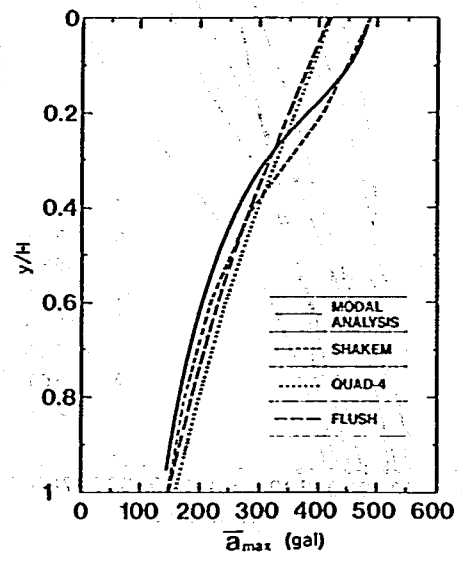
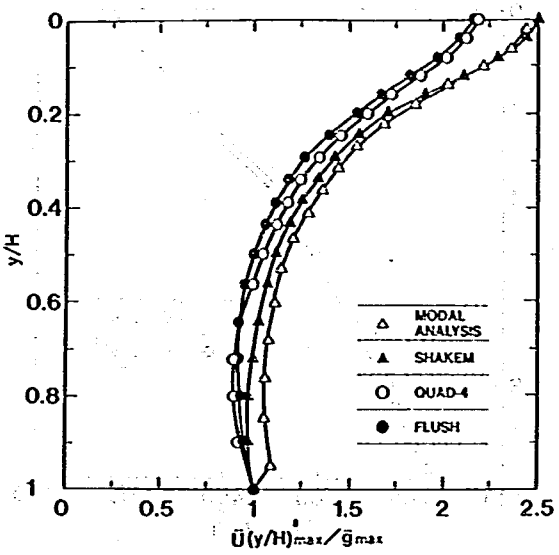
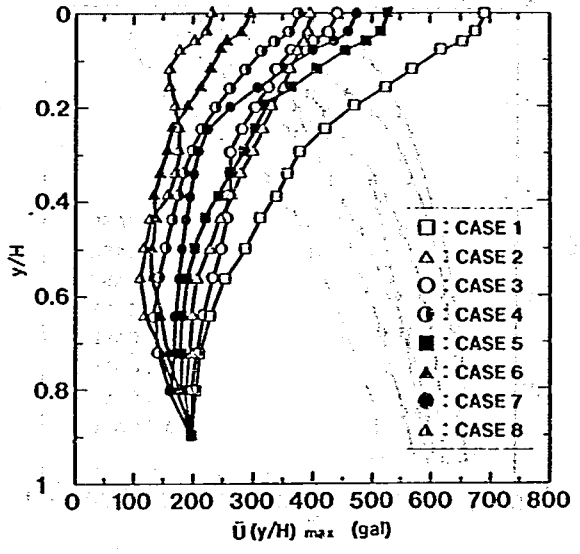


Fig. 8 Relationship between y/H and a_{max} for eight input motions

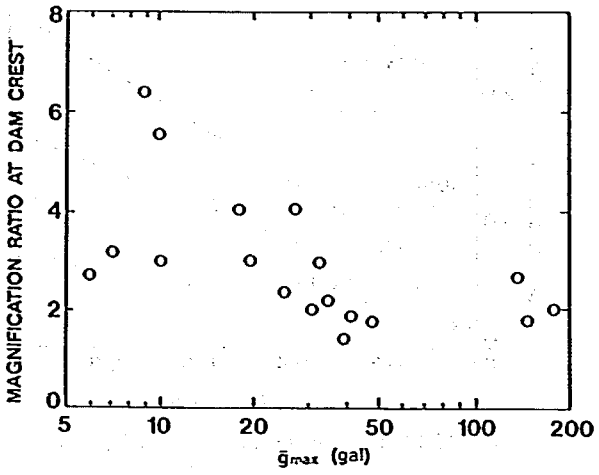


(a) Normalized maximum acceleration from different analytical methods



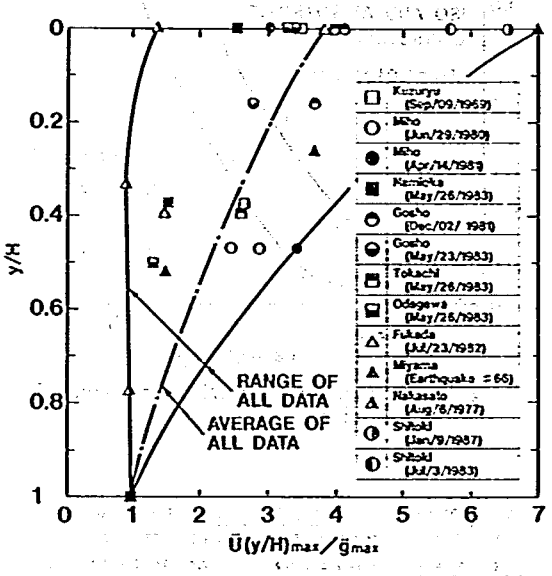
(b) Maximum acceleration from QUAD-4

Fig. 9 Maximum acceleration at various elevations



(a) Maximum acceleration magnification ratio at dam crest with maximum ground acceleration

Fig. 10 Acceleration response measured at rockfill dams



(b) Maximum acceleration distribution with elevations

Fig. 10 (b)

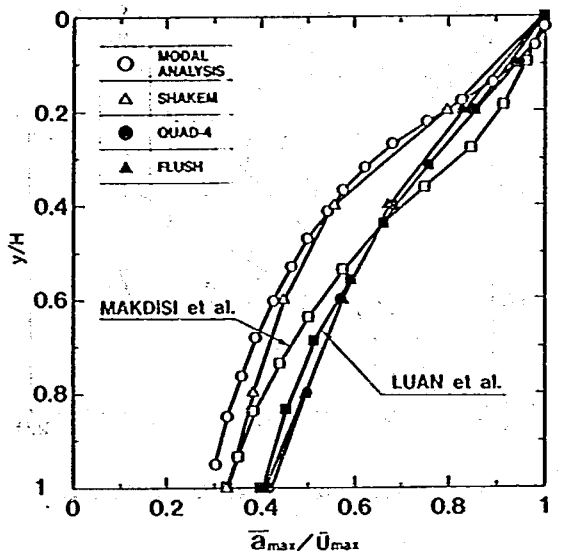


Fig. 11 Comparison of normalized maximum acceleration curve with depth of sliding mass

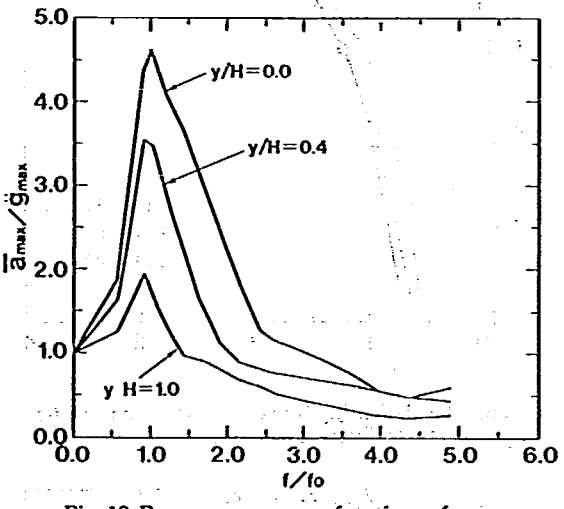


Fig. 12 Frequency response functions of a_{max} calculated by QUAD-4

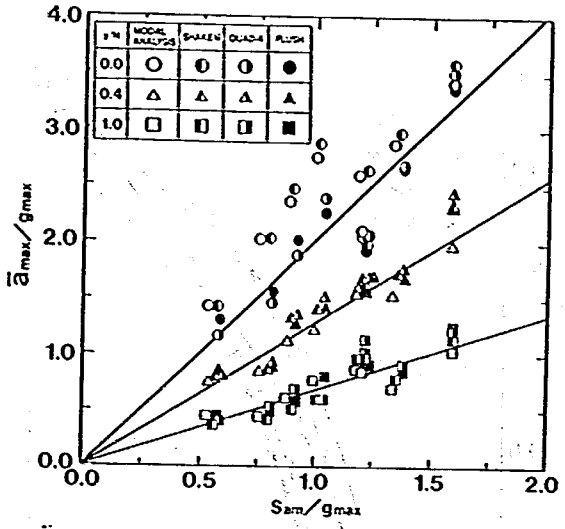


Fig. 13 Relationship between a_{max}/g_{max} and S_{am}/g_{max}

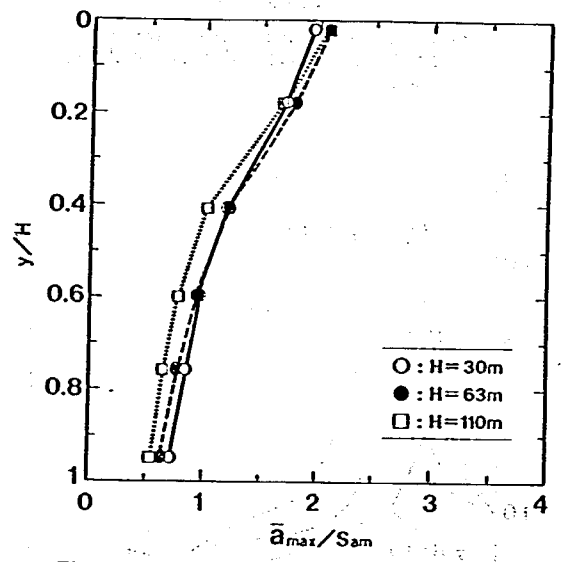


Fig. 14 Relationship between a_{max}/S_{am} and y/H for three different dam-heights

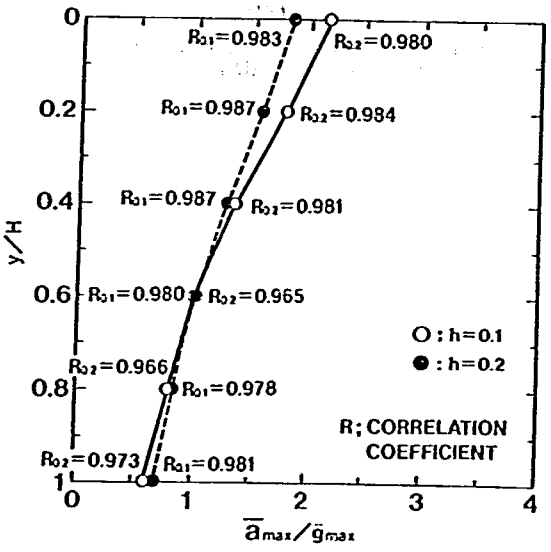


Fig. 15 Relationship between a_{max}/S_{am} and y/H for h=10% and 20% (QUAD-4)

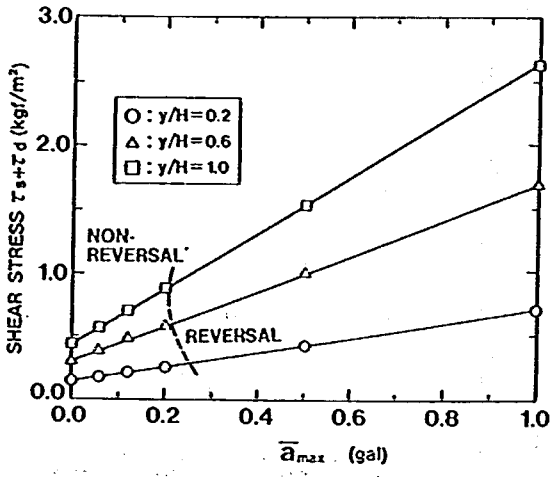


Fig. 16 Shear stresses estimated by using a_{max}

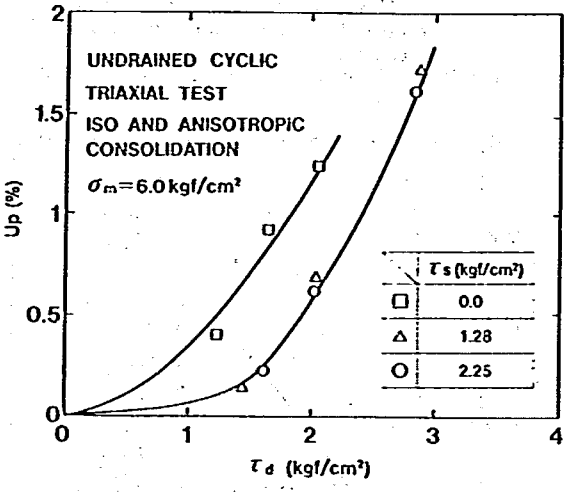


Fig. 17 Permanent strain development in undrained cyclic triaxial

Case	DAM site	Earthquake	M	L(KM)	f(Hz)	f/fo
1	Gosho	Mid Japan Sea	7.7	198.0	0.59	0.35
2	Minase	Miyagi offshore	7.4	167.0	0.78	0.46
3	Yokoyama	Central Gifu	6.6	60.0	1.07	0.63
4	Tokachi	Northern Hidaka Mts.	7.0	62.0	1.61	0.95
5	Tase	Mid Japan Sea	7.7	264.0	2.39	1.41
6	Gosho	Central Iwate	6.6	69.0	3.20	1.88
7	Miho	Izu peninsula offshore	6.7	56.0	4.15	2.44
8	Miho	Western Kanagawa	5.8	12.0	6.59	3.88

(Note) M: Magnitude L: Epicentral distance
fo: Natural frequency of model dam f: Predominant frequency

Table 1 Characteristics of earthquake motions observed at dam foundation

Resolving Uncertainties in Seismic Hazard Models

by

J. K. Kimball*

ABSTRACT

This paper describes the types of uncertainties inherent in seismic hazard analysis, and identifies those steps that can be taken to resolve these uncertainties. The focus is on the recent advances that have been made in performing probabilistic seismic hazard studies, with particular emphasis on issues related to identifying uncertainties in the basic seismic hazard models. Comprehensive probabilistic seismic hazard studies have been completed over the past five years by Lawrence Livermore National Laboratory for the United States Nuclear Regulatory Commission and by the Electric Power Research Institute for the commercial utility industry. The United States Department of Energy is actively using these studies to evaluate seismic hazard at Department of Energy sites. Results from these studies can be used to quantify existing uncertainties in seismic source understanding, earthquake occurrence rates, and ground motion attenuation. The resulting uncertainties can then be evaluated to determine those that are reducible with additional study. Additionally, a clear understanding of modeling uncertainties can provide insight into the attributes that future probabilistic seismic hazard analysis should have.

KEYWORDS: seismic hazard; seismicity; seismic sources; uncertainty evaluation; ground motion

1. INTRODUCTION

The purpose of this paper is to provide a summary description of the types of uncertainties inherent in seismic hazard analysis, and to identify those steps that can be taken to resolve these uncertainties. This purpose will be accomplished by describing how the Department of Energy (DOE) is utilizing probabilistic seismic hazard results, including the identification and evaluation of uncertainty. The general purpose of a probabilistic seismic hazard analysis is to evaluate the vibratory ground shaking at a particular site by considering the viable sources of earthquakes, the likelihood of earthquake occurrences for each source, an estimate of the particular ground motion parameter of interest, and an analytical model for estimating the probability of exceeding the ground motion parameter. These general steps are shown on Figure 1. Each of the general steps will be discussed below, in terms of modeling uncertainty.

The DOE currently specifies the loading associated with the Design Basis Earthquake (DBE) based on probabilistic acceptance criteria. Mr. J. Hill, DOE, presented the DOE approach at the twenty-third annual joint meeting of the United States/Japan Panel on Wind and Seismic Effects, May 1991. The DOE facilities range from office buildings to complex

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nuclear facilities. The design basis earthquake (DBE) acceptance criteria represents a range of probabilistic input levels from those associated with standard building codes (about 1/500 or .002 per year) to those associated with nuclear reactors (about 1/1,000 to 1/10,000 or .001 to .0001 per year). Because the DBE is specified probabilistically, the values selected are associated with mean estimates, to account for uncertainty in seismic hazard.

Because the DBE is probabilistically derived, particular attention has been focused on the types of approaches and inputs that have gone into existing seismic hazard analyses. Separate comprehensive studies were completed by Lawrence Livermore National Laboratory (LLNL) for the U.S. Nuclear Regulatory Commission (NRC) and by the Electric Power Research Institute (EPRI) for the commercial utility industry. These studies have been used at a number of DOE Eastern United States sites to establish the DBE seismic hazard curves. The LLNL and EPRI results can be used to highlight the strength of probabilistic results along with potential shortcomings that must be considered by decisions makers when using such results. Table 1 provides a brief outline of how the LLNL and EPRI studies were developed.

For the reader who is interested in background material regarding probabilistic seismic hazard analysis, two references are particular useful. The first is a National Research Council report from the Panel on Seismic Hazard Analysis (Committee on Seismology) on Probabilistic Seismic Hazard Analysis, 1988. The second is the Probabilistic Safety Analysis Procedures Guide, NUREG/CR-2815, Rev. 1, 1985, which describes how the various types of seismic hazard input should be characterized, including the treatment of uncertainties. The summary references for the LLNL and EPRI studies are D.L. Bernreuter et al, 1987, and Electric Power Research Institute, 1986. Two useful references regarding seismic source characterization are Coppersmith, 1991, and Thenhaus et al, 1987.

2. SOURCES ON UNCERTAINTY IN SEISMIC HAZARD ANALYSIS

It is important in any probabilistic seismic hazard analysis to make an estimate of the confidence in hazard results. In the LLNL and EPRI seismic hazard studies this was accomplished by providing constant percentile seismic hazard curves (such as 15th, 50th and 85th percentiles) and estimates of mean hazard. Figure 2 shows an example of such results for the DOE Savannah River Site in South Carolina. Several observations are noteworthy. First, the median (50th percentile) results are similar between the two studies. Second, the overall estimates of uncertainty are quite different between the two studies, which result in drastic differences between mean estimates of seismic hazard. Because DOE is attempting to use results such as these to establish the DBE, it has been important to understand why the two studies give such divergent estimates of uncertainty.

A convenient way of addressing seismic hazard uncertainty is to segregate uncertainty into two components. For the purpose of this paper these two components are labeled randomness and modeling uncertainty. Randomness represents variability that cannot be reduced with more data or better models. Modeling uncertainty represent variability that can be reduced (in concept) with better data and analysis techniques. The discussion provided below will identify modeling uncertainty issues using the LLNL and EPRI results. The

type of data that could be used to address such modeling uncertainty will also be identified. Such an observation has generic implications for any probabilistic seismic hazard analysis.

2.1 Seismic Source Uncertainty

In the LLNL and EPRI studies each of the experts and expert teams were requested to provide seismic source maps which generally represent areas of uniform earthquake potential. Figure 3 shows examples of such maps from two of the LLNL experts for the southeastern United States. The alternatives shown on the two maps represent very different opinions regarding the identification of seismic sources. Such divergent alternatives represents the ability of earth scientists to identify active seismic features (modeling uncertainty). Presently in intra-plate regions such as the Eastern United States, the identification of specific earthquake causal structure remains elusive. It could be expected that in more active tectonic regions, such as plate margin regions in California and Japan, earth scientists can more readily define the seismic sources. Such an assumption, however, may not be totally warranted given the occurrence of potentially damaging earthquakes such as the 1983 Coalinga, California earthquake. Thus, the current state of practice should recognize the potential large uncertainty in seismic source identification.

Since modeling uncertainty is potentially reducible, it is important to identify the type of data that should be assessed to reduce uncertainty in seismic sources. This data includes:

- Analysis of historic seismicity;
- Accurate instrumental seismic recordings;
- Geologic assessment of paleo-seismic features (such as fault scarps and liquefaction features); and
- Integration of geologic/seismologic/geophysical data to provide a three dimensional picture of the seismogenic crust.

As this type of data evolves, one can expect the uncertainty in seismic sources such as shown on Figure 3 to diminish.

2.2 Earthquake Occurrence Rate Uncertainty

In the LLNL and EPRI studies each of the experts and expert teams were requested to provide estimates of earthquake activity rates (commonly termed the "a" value), slopes of the earthquake occurrence curve (commonly termed the "b" value), and estimates of largest magnitude. Figure 4 displays the earthquake occurrence curves from the LLNL and EPRI studies for the Charleston, South Carolina seismic source, for the best estimate earthquake recurrence parameters ("a" and "b" values). Similar to seismic sources estimation, there is wide divergence in earthquake recurrence estimates. This divergence is even more dramatic if the uncertainty in "a" and "b" values is reviewed. Should such divergence be expected? The answer to this question is not simple considering that the earthquake occurrence information represents both randomness and modeling uncertainty.

In attempting to assess whether the estimates of uncertainty on earthquake occurrence rates are realistic, expert

opinion studies must include sufficient feedback and interaction to assure that input has a scientific basis. Clearly seismicity experts are not necessarily uncertainty experts. To illustrate this problem, the LLNL data for the DOE Savannah River Site has been evaluated in detail. One of the seismicity experts gave a range of recurrence intervals for magnitude 5 earthquakes for the Charleston, South Carolina, seismic source as 20 days to over 2000 years. Such dramatic examples may illustrate more of the problem of expert assessment of uncertainty than true modeling uncertainty.

The modeling uncertainty associated with earthquake occurrence rates, like seismic sources, is potentially reducible. The type of data that should aid this process includes:

- o Quantification, on a consistent basis, of the size of historic earthquakes compared to instrumental seismicity;
- o Integration of geologic slip rate from paleo-seismic studies with seismicity data; and
- o Assessment of short term strain rates in active tectonic regions.

While this type of data should aid in reducing modeling uncertainty in earthquake occurrence rates, it may be more important to ensure that earth scientists better understand the process of uncertainty assessment.

2.3 Ground Motion Attenuation Uncertainty

In the LLNL and EPRI studies separate efforts were completed to develop ground motion attenuation relationships. In the LLNL study five attenuation experts selected attenuation models and provided uncertainty estimates. In the EPRI study one expert selected three attenuation models with one estimate of attenuation uncertainty. Figure 5 displays a comparison of predicted peak ground acceleration from the LLNL and EPRI studies for magnitudes 5 and 7. Figure 5 also shows wide divergence of opinion, similar to other components of seismic hazard input. In a region such as the eastern United States where there are few available strong motion records one might expect relatively large peak acceleration modeling uncertainty. In reviewing Figure 5, however, one sees that the modeling uncertainty predicted by the experts spans over an order of magnitude at the distance of one hundred kilometers. When such divergence exists, the user should require that each model selected be clearly justified. The LLNL seismic hazard is particularly sensitive to one of the attenuation models at rock sites. Much effort has been expended by other experts in an attempt to show that this model is inappropriate. While DOE and the NRC have not agreed with this conclusion, decisions made using the LLNL and EPRI results have had to recognize the sensitivity of this issue.

The type of data that should be used to reduce such modeling uncertainty includes:

- o Collection and analysis of strong motion data near damaging earthquakes;
- o Assessment of crustal attenuation properties both on a local and regional basis;
- o Evaluation of earthquake source characteristics such as stress drop; and

- o Assessment of the effects of local site conditions including the impact of soil consolidation at deep soil sites.

2.4 Overall Summary of LLNL/EPRI Evaluation

Based on the evaluation of the LLNL and EPRI data DOE has reached the following general assessment of these two studies with respect to modeling uncertainty. This assessment provides insight into the types of attributes that any probabilistic seismic hazard should have. There is a high degree of similarity between the LLNL and EPRI seismic hazard studies ranging from overlap of experts used to general overlap in parameter input from the experts. The key difference between the two studies relates to the topic of uncertainty assessment, particularly modeling uncertainty assessment for all input variables. Identified issues relate to the process of expert opinion elicitation, particularly the issues of how and whether experts assess and understand uncertainty. This puts increased emphasis on the user or decision maker to fully understand the uncertainties inherent in the probabilistic results.

The DOE review has also found that there are several stable trends in the existing LLNL and EPRI data. These trends include:

- o General consistency in median probabilistic estimates of seismic hazard;
- o General consistency within each study of the ratio of 85th percentile hazard curves to the median hazard curves;
- o General consistency between each study in the ratio of 85th percentile hazard curves to the median hazard curves; and
- o Relative consistency in hazard trends; studies provide similar ranking of high seismic hazard to low seismic hazard sites.

The DOE has used the above trends to develop an interim position regarding how the two studies should be used at DOE sites. This position is built on the stable median hazard curves with a "correction factor" to account for uncertainty. The correction factor will be used to develop mean estimates of seismic hazard. The "correction factor" is based on the above identified trends, and was estimated to have a value of 1.65 for peak ground acceleration at probabilities of about 1/5000 or .0002 per year.

3. ATTRIBUTES OF AN ADEQUATE PROBABILISTIC SEISMIC HAZARD ANALYSIS

Given the types and the order of magnitude of modeling uncertainties described above, it is important to outline the attributes that any probabilistic seismic hazard analysis should contain. It is particularly important that any decision maker who is using a probabilistic seismic hazard analysis require that these attributes be properly addressed. These attributes include:

- o Clear documentation of input including modeling assumptions and how seismicity parameters were selected. It is important to understand the basis for the seismicity parameters (geologic versus seismologic) and attenuation models;
- o Comparison of model input to historic seismicity. While there should not necessarily be an over-reliance on historic seismicity, this type of comparison can identify issues which may otherwise be overlooked;

- o Incorporation of alternative models that represent the current state-of-knowledge. Hazard analyses without this attribute can lead to a false sense of security by not recognizing viable alternatives. It is important, however, that a basis be provided for all models used;

- o An analytical approach that allows the user to understand dominate seismic hazard contributors versus simply showing the bottom-line hazard curve. For many applications it is important to know which sources, magnitudes and distance most contribute to the seismic hazards; and

- o Clear documentation and presentation of uncertainties.

If the above attributes are addressed by a probabilistic seismic hazard analysis, then the hazard results are likely to provide consistent estimates from site to site. Even considering the large absolute differences in results such as LLNL and EPRI, it is at least reassuring to know that both studies provide consistent trends of seismic hazard.

4. CONCLUSIONS

The state-of-knowledge regarding the causes of earthquakes in much of the United States is limited, resulting in large uncertainties in seismic hazard estimates. Probabilistic seismic hazard studies must clearly describe and document this uncertainty to be useful for decision makers. Continued improvements regarding the assessment, quantification and documentation of expert opinion is needed before the actual probabilistic numbers can be used in an absolute sense. At the same time, continued study regarding the seismic and ground motion input should aid in reducing modeling uncertainties. Such efforts fall within the following major areas:

- o Continued geologic, seismologic, and tectonic study of the causes, mechanisms and rates of crustal deformation and the validity of alternative seismicity models;

- o Evaluation of instrumental seismic data including the variability of earthquake occurrence rates; and

- o Enhanced understanding of strong motion characteristics, regional attenuation properties and the effect of local surficial material.

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TABLE 1

Summary of LLNL and EPRI Study

LLNL STUDY:

Used individual expert opinion to evaluate all seismic hazard input.

1. 10 Seismicity experts provided zonation, earthquake occurrence rates and upper magnitude values including estimates of uncertainty.

2. 5 Attenuation experts provided attenuation models and model uncertainty.

3. The LLNL used Monte Carlo methods to evaluate overall seismic hazard uncertainties. Each site included 2750 seismic hazard curves, which were processed into constant percentile seismic hazard curves.

EPRI STUDY:

Used expert teams to evaluate seismic hazard input except for attenuation models.

1. 6 Expert teams provided zonation, earthquake occurrence rates and upper magnitude values including estimates of uncertainty.

2. One expert provided attenuation models and model uncertainty.

3. EPRI used logic trees to evaluate overall seismic hazard uncertainty, which were processed into constant hazard seismic hazard curves.

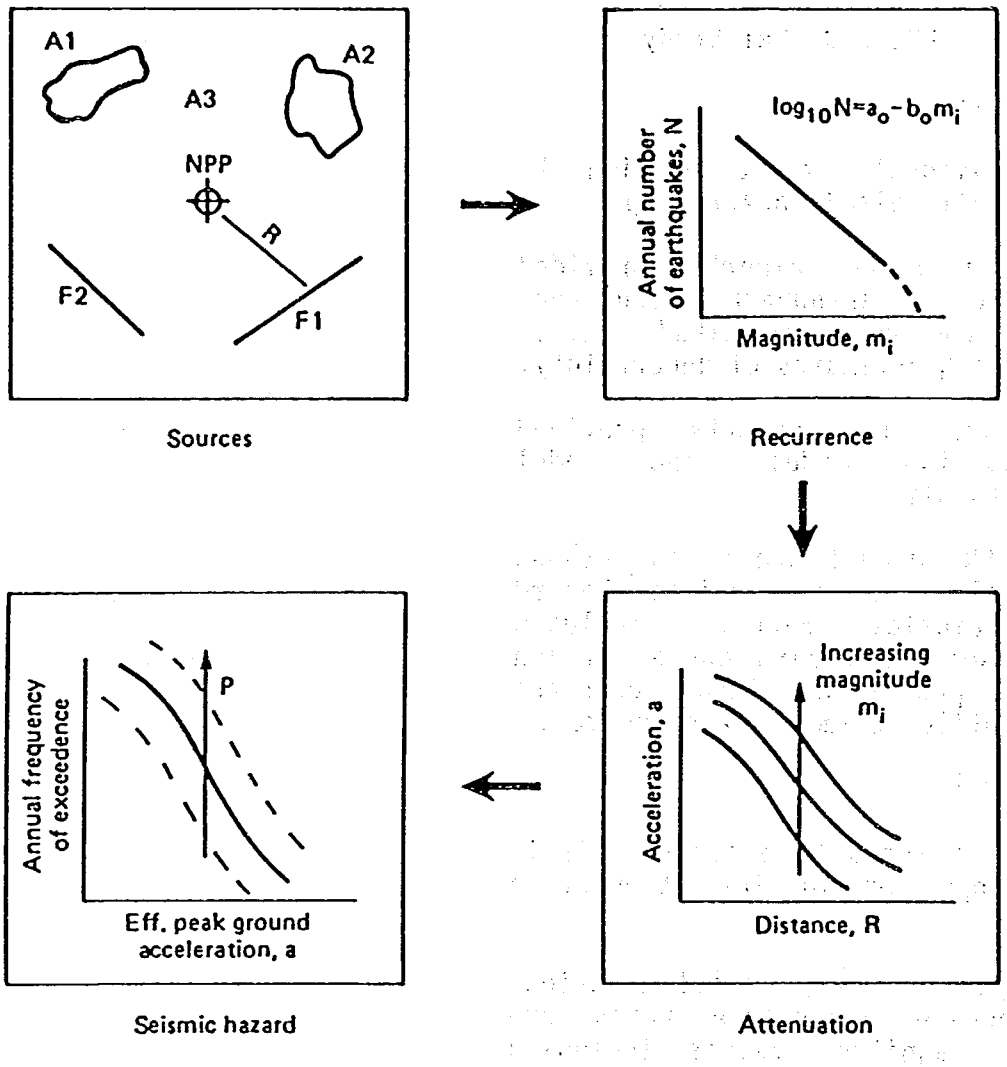


Figure 1

Four steps of a probabilistic seismic hazard analysis: Step 1 Identification of seismic sources; Step 2 Develop earthquake recurrence relationship for each seismic source; Step 3 Develop ground motion attenuation model; Step 4 Use an analytical model to calculate seismic hazard curve including uncertainty.

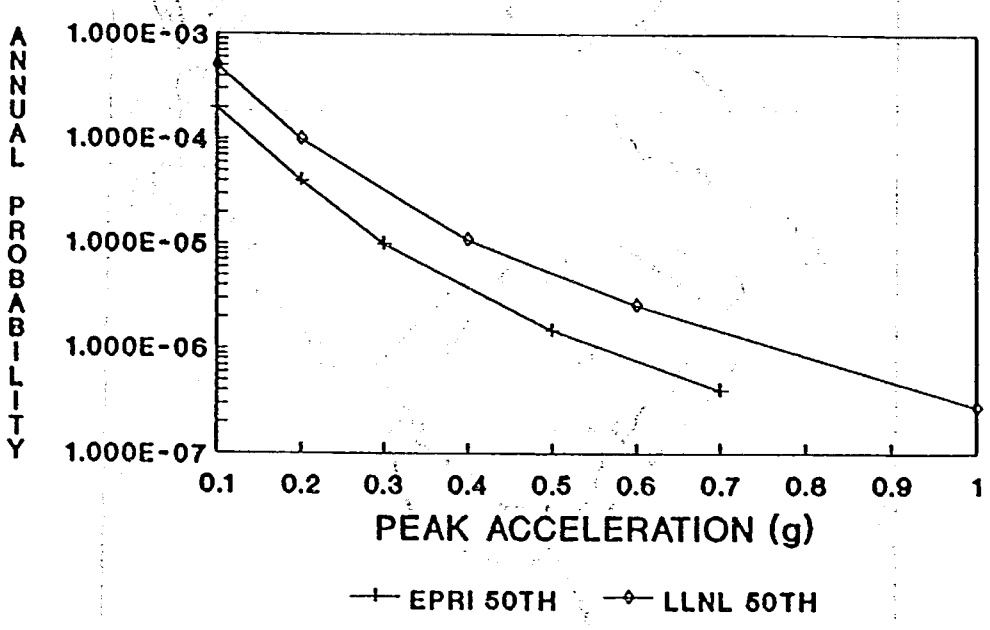


Figure 2a

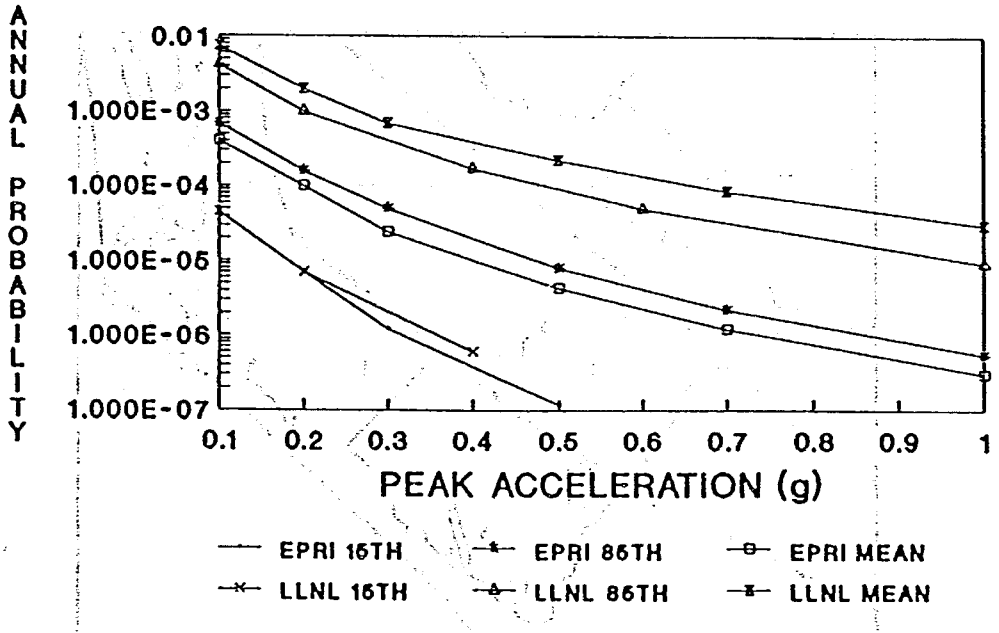


Figure 2b

Figure 2

Probabilistic seismic hazard curves for the DOE Savannah River, South Carolina, site. Figure 2a displays the LLNL and EPRI median seismic hazard curves; Figure 2b displays the LLNL and EPRI 15th percentile, 85th percentile and mean seismic hazard curves

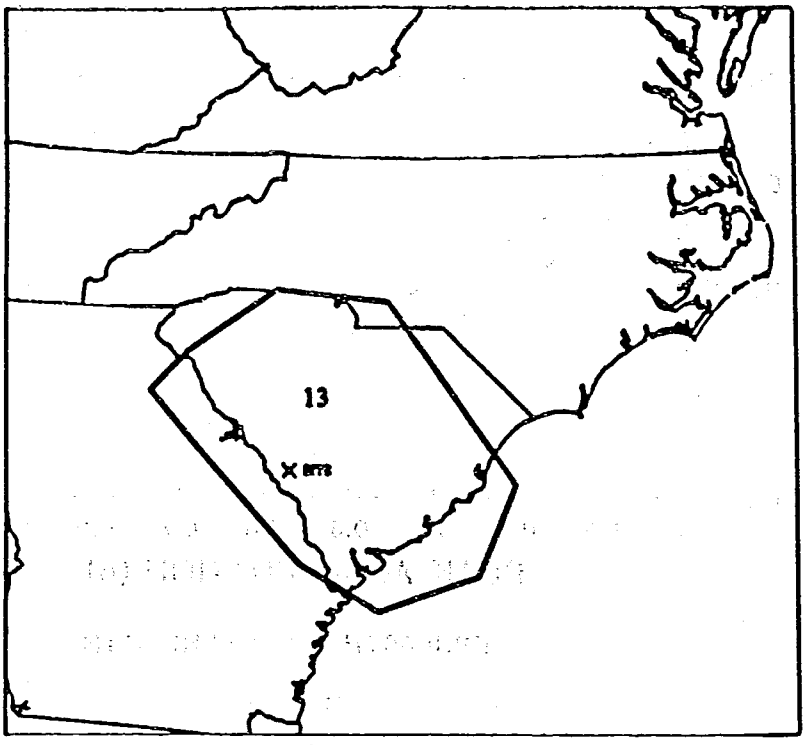


Figure 3a

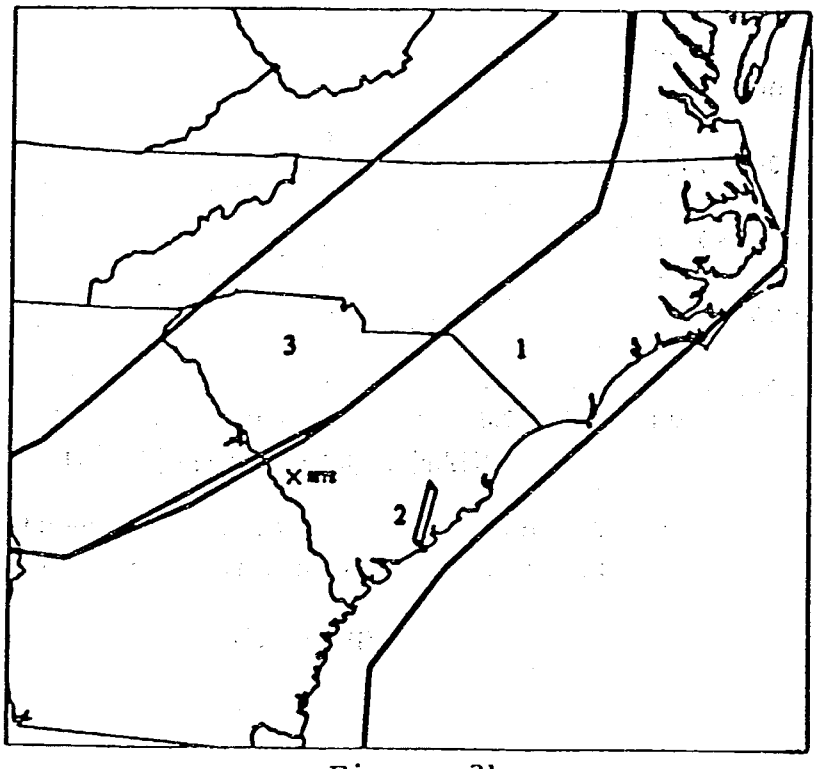


Figure 3b

Figure 3

Seismic source maps for the Southeastern United States from the LLNL study. Figure 3a shows a broad seismic source for the Charleston, seismic zone. Figure 3b shows a very localized seismic source for the Charleston, seismic zone.

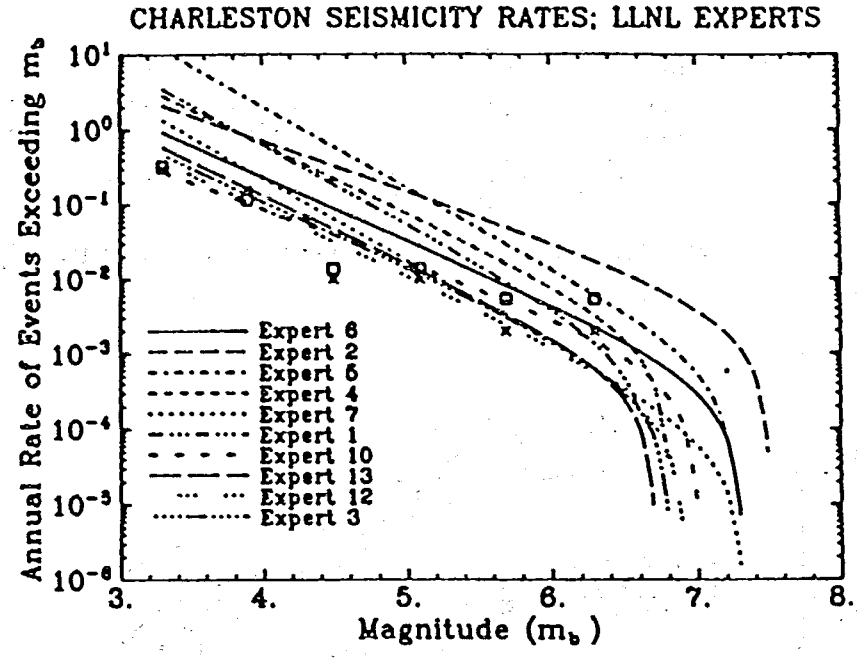


Figure 4a

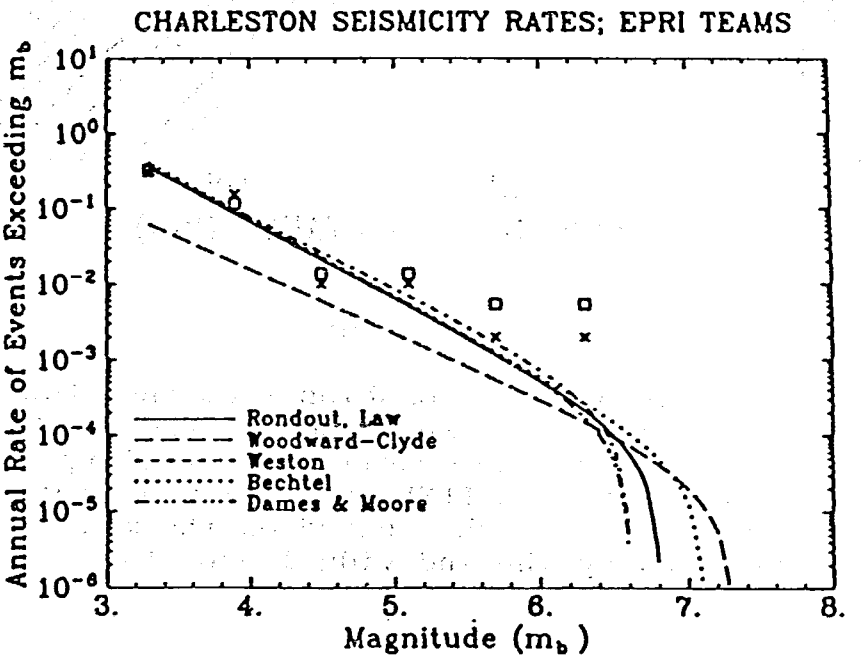


Figure 4b

Figure 4

Earthquake recurrence curves for the Charleston, South Carolina seismic source. Figure 4a displays the range of recurrence curves for the LLNL experts; Figure 4b displays the range of recurrence curves for the EPRI expert teams

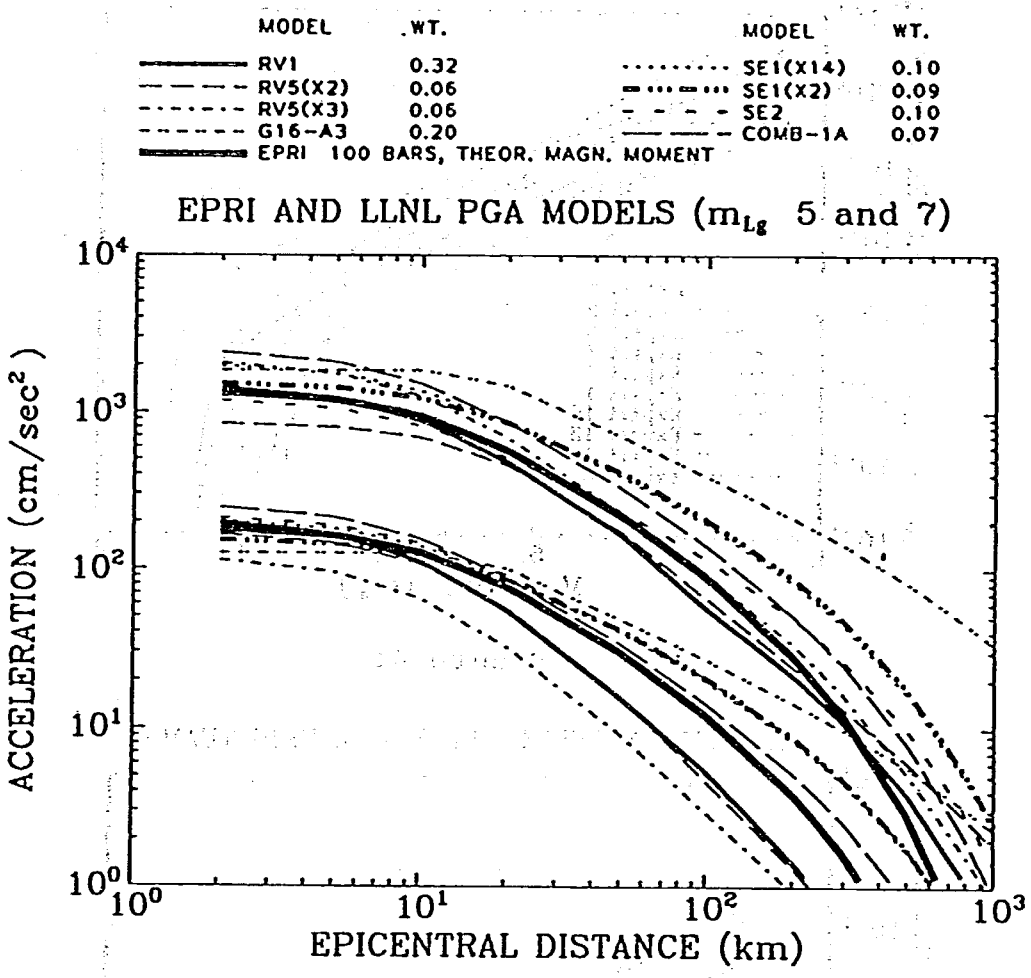


Figure 5

Attenuation relationships from the LLNL and EPRI studies for magnitudes 5 and 7. The figure shows the modeling uncertainty of the combined LLNL and EPRI models which are given weights based on the models selected by the individual experts

Earthquake Alert System Feasibility Study

by

Philip E. Harben*

ABSTRACT

An Earthquake Alert System (EAS) could give several seconds to several tens of seconds warning before the strong motion from a large earthquake arrives. Such a system would include a large network of sensors distributed within an earthquake-prone region. The sensors closest to the epicenter of a particular earthquake would transmit data at the speed of light to a central processing center, which would broadcast an area-wide alarm in advance of the spreading elastic wave energy from the earthquake. This is possible because seismic energy travels relatively slowly (3–6 km/s) when compared with the speed of light. Utilities, public and private institutions, businesses, and the general public could benefit from an EAS.

KEYWORDS: earthquake alert system; microseismic; real time; strong motion

1. INTRODUCTION

The concept of an EAS is not new. It was first proposed in an editorial by J. D. Cooper published in the *San Francisco Daily Evening Bulletin* on November 3, 1868. Although the basic concept for the system is simple, the implementation of a modern EAS requires the use of diverse and fairly sophisticated existing technology in addition to new research and development on a number of operational issues, such as reliability, accuracy, and survivability.

The Japanese have employed a seismic shutdown system for many of their railway lines since the early 1960s and have recently (1990) deployed a five-station Urgent Earthquake Detection and Alarm System (UrEDAS) as part of an earthquake disaster prevention program (Nakamura, 1989). A temporary EAS was deployed by the United States Geological Survey near the epicenter of the Loma Prieta earthquake to give warning to workers on the collapsed Cypress overpass structure in Oakland, California (Bakun, 1990).

The EAS concept discussed here consists of a distributed network of remote seismic stations that measure weak and strong earth motion and transmit the data in real time to a central facility. This facility processes the data and issues warning broadcasts in the form of information packets, containing estimates of earthquake location, zero time (the time the earthquake began), magnitude, and reliability of the predictions. Users of the warning broadcasts have a dedicated receiver that monitors the warning broadcast frequency. The user also has preprogrammed responses that are automatically executed when the warning information packets contain location and magnitude estimates above a facility's tolerance.

The alert signal broadcast is an area-wide transmission and is probably sent via FM radio or television subcarrier communications. Ensuring the survivability of the EAS is addressed with several strategies: hardening the remote stations for strong motion; using redundant widely separated central stations, communications, and warning broadcast transmitters; and noting that strong motion is emergent (i.e., it builds up over a period of several seconds), allowing some period of time in the early evolution of the earthquake, as accelerations build to damaging levels, during which a vulnerable system continues to function.

A low-cost prototype EAS could be developed in some areas by making use of existing dense networks of microseismic monitoring stations. These stations could be inexpensively

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upgraded to measure strong motion. When excess communications bandwidth is not available, a strong-motion upgrade could be inexpensively accomplished using the existing communication channel and a switching unit that passes the strong-motion signal during strong-motion conditions and passes the seismometer signal otherwise.

We designed a prototype switching unit and tested it at a surface station in the near-field of an underground nuclear explosion to simulate strong ground motion from an earthquake. This successful test demonstrated that the strong-motion record could be switched into the data stream within 30 ms and that the entire strong-motion record could be reliably switched into the real-time data stream.

About 50 stations distributed relatively uniformly along the major faults of the San Francisco Bay Area could form an effective EAS prototype network. Such a subset of stations could be selected from the U.S. Geological Survey (USGS) CALNET microseismic monitoring network. By upgrading these stations to measure strong motion and by installing a parallel real-time processor at the microseismic central processing facility, a low cost prototype EAS could be developed. The experience to be gained from an operational prototype is very important to help address issues such as false alarm rates, false alarm mechanisms, reliability, earthquake parameter estimation accuracy, and user alert-signal analysis.

In this report, we examine the feasibility of an EAS and attempt to outline some of the features, trade-offs, and technical considerations in implementing such a system. The focus of this report is on the features and design of a low-cost EAS prototype that, to the extent possible, makes use of existing seismic stations, communications, and broadcast facilities.

2. EAS USES

There are numerous potential users who would benefit from an EAS. Table 1 is a partial list of EAS applications; however, automated shutdown and activation are the most common functions. Depending on the application, these two functions vary dramatically in their consequences if action is taken as the result of a false alarm. An activation function such as opening a fire station door has minimal consequences if the door is opened due to an EAS false alarm. An automated shutdown of a nuclear power plant could disrupt the power grid of an entire city and result in expensive power-up procedures to reestablish full-power operation. An EAS false alarm shutdown cannot be tolerated for such an application. Although we will discuss user processing of the EAS alert signal in a later section, it is important to recognize that users of the alert signal have different applications and consequences of their actions. Therefore, the EAS must provide an optimal summary of the current "state of knowledge" for the earthquake parameters transmitted in the alert information so that diverse users can weigh the most recent "state of knowledge" against the consequences of action.

The real-time location and magnitude estimation that an EAS can produce would be very useful in the immediate aftermath of a large damaging earthquake. Emergency services such as fire, ambulance, and rescue could be deployed much more effectively if magnitude and epicenter estimates were immediately available. The communications media would also be much more effective in determining unfolding events related to an earthquake aftermath if estimates of epicenter and magnitude were available. Finally, the strong-motion records obtained from the EAS after a large earthquake would help in fundamental strong-ground-motion seismology investigations. Such investigations could help improve the fundamental understanding of site response and soil amplification during strong ground motion and, thereby, improve acceleration prediction, structural building codes, and structural damage assessment.

3. EAS OVERVIEW

The EAS discussed here was first advanced by Heaton (1985) and is shown in Fig. 1. Strong-motion and seismometer signals from a distributed network of remote seismic stations are transmitted in real time to the EAS processor at the central acquisition facility. The EAS processor estimates earthquake parameters such as location, origin time, magnitude, and the reliability of this information. If the earthquake magnitude estimate exceeds a given threshold, a warning signal is broadcast over an area-wide transmitter. This warning signal contains the earthquake parameter estimates. Each user of the warning signal has a dedicated receiver and processor with preprogrammed responses based on earthquake parameter values. The user's system continuously monitors the warning-signal frequency band. When a user receives a warning signal that has location and magnitude estimates in excess of the user's facility tolerances, the facility's emergency responses are automatically implemented.

3.1 Alert Information Packet

The signal transmitted by an EAS to alert an area must provide estimates of the earthquake's location and magnitude, and an estimate of the reliability of the information since many users of the warning signal would incur significant costs by taking any action based on the alert message. In general, an EAS should produce more reliable estimates of earthquake location and magnitude as the earthquake evolves and as more stations in the network transmit data that can be used in calculating the location and magnitude estimates. It is therefore important for alert information packets to be rapidly updated so that the new, more reliable estimates based on the most recent data can be transmitted to the users. The packet's update rate, however, is limited by the amount of information each packet contains, the communication method employed, and the central processor and alert algorithms used.

An initial alert signal can give the greatest lead time between the warning and the onset of strong motion. By applying a minimal warning algorithm at the central station, an alert signal can be issued shortly after only a few stations have detected strong motion from an earthquake. A very simple algorithm could be used that requires two EAS stations to measure ground accelerations above a predetermined threshold within a specified time window before the initial alert signal is issued.

After the initial alert signal is transmitted, the information transmitted in the subsequent alert packets should contain all of the salient parameters that form the input to any user's preprogrammed response algorithm. An example of a possible alert packet is shown in Fig. 2. This alert packet consists of the following information:

- Update time
- Zero-time estimate
- Location estimate
- Magnitude estimate
- Maximum acceleration
- Number of stations

Including the update time in the packet allows the user to identify the most recent information packet and to determine the time that has elapsed between the most recent information packet transmission and real time. The zero-time estimate refers to the time the earthquake began and is calculated by the earthquake-location algorithm. The location estimate gives the hypocenter coordinates for the earthquake. The magnitude estimate can be an estimate of body wave magnitude, maximum accelerations expected, earthquake moment, or a combination of these. The maximum acceleration gives the value of the maximum seen by any station (or an average of several of the stations reporting the highest accelerations) up to the time that the alert packet is transmitted. It provides the user with a direct measurement (as opposed to processed estimates), which can be compared to the magnitude estimate. Finally, only the

stations reporting strong ground motion above a given threshold are included in the number of stations. This provides the user with some indication of the reliability of the estimates. The more stations reporting strong motion (and therefore included in the estimates), the more likely that the event is a large earthquake.

3.2 Central Processor and Algorithms

The central EAS station acquires the incoming strong- and weak-motion network data for real-time analysis by the central processor. A detector algorithm can monitor the level of strong-motion signals as the data are buffered in memory. When a threshold level is exceeded, an algorithm to pick the time of *P*-wave arrival is repeatedly applied to all weak-motion data channels. When at least four stations have an established first arrival, a location algorithm is repeatedly applied to the first arrival times. As the earthquake evolves, more *P*-arrival times are available to be used in the location algorithm and, therefore, the estimates produced should be more accurate.

In parallel, magnitude and reliability estimates can be made for the data from the strong-motion channels. Several algorithms can be applied to magnitude estimation. Crude estimates can be made by using the maximum accelerations at the stations nearest the estimated earthquake location. The moment estimation method (Toksoz et. al., 1990) uses a relatively dense linear arrangement of sensors along faults to estimate the fault slippage and the length of the rupture in real time. By assuming a fault depth, the moment can then be estimated. The Japanese UrEDAS system estimates magnitude from the predominant period of the initial *P*-wave motion (Nakamura, 1989).

To the extent possible, all magnitude estimation algorithms should be applied to the data from the strong-motion channels to increase the reliability of the estimate. The arrangement of stations forming an EAS could preclude the use of the moment estimation method for earthquakes on some faults or fault segments because of a lack of instrument density.

3.3 User Processing

The extent and specifics of alert information analysis that a particular user may require before executing alert or shutdown functions depends on the user's application. Potential user applications, such as opening a fire station door, do not have a significant cost associated with a false alarm, so minimal or limited processing may be desirable to maximize lead time. However, the shutdown of a nuclear power plant using the alert signal would be high in cost. Consequently, maximum alert information analysis that trades off the time left to execute a safe shutdown against the probability that a shutdown is unnecessary may be desired.

In general, the user's preprogrammed response algorithm can contain a table of precalculated estimates of the ground accelerations expected at their site from earthquakes in a range of magnitudes and at a range of locations on the major faults in the area. When the earthquake location and magnitude estimates provided by the EAS alert packet indicate site accelerations greater than a given shutdown or alert threshold and when the alert packet reliability is sufficient, action is taken. The zero-time estimate of the earthquake can be folded into the user's decision process. A time-of-arrival estimate of strong motion is made by the user's algorithm based on the estimated earthquake location, estimated zero time, and the current time. When there is more lead-time than required for the user to perform alert actions, the user may elect to wait and receive additional alert packets to increase assurance that the estimates are stable and the reliability is high.

3.4 Alert Signal Transmission

A relatively new area-wide communication scheme that has low cost potential is FM radio and television subcarrier communication

(Communications Alternatives, 1991). The information is broadcast on a subcarrier frequency of the main transmission frequency and does not interfere with the main broadcast. Transmission coverage can be up to 100 miles in radius from the transmitter, depending on local terrain. The advantages of FM radio and television subcarrier communications are:

- Low start-up costs
- Wide area coverage
- Low user receiver costs
- Acceptable digital data transmission rate
- High reliability

The start-up equipment cost for a broadcaster to modulate and transmit on a subcarrier is between $5000 and $20,000, depending on existing equipment. The wide-area coverage of a single broadcast station should be reinforced by dual-redundant widely separated broadcast stations to improve area-wide reception owing to terrain limitations and to assure survivability of a transmitter during a large earthquake. User receivers should cost less than $100.

3.5 Survivability

Although a detailed analysis of EAS survivability is beyond the scope of this study, a few comments on this issue are in order. The survivability of an EAS will be driven by the weakest link in the individual components that make up the entire system (i.e., seismic stations, communications system, and the central station and warning transmitter). In general, individual free-field stations should survive during strong motion if proper care is taken in the station design and during station installation. A seismic station running directly on commercial ac power would be vulnerable to the power interruptions likely during a large earthquake. Powering the station by battery and charging the battery with ac power or solar panels assures independence from ac power interruptions. Proper bolting of sensors to a concrete pier, attachment of voltage-controlled oscillators and amplifiers to a rigid structure bolted to a pier, and secure mounting of batteries and charging units will help ensure proper operation during strong motion.

The survival of the EAS communications between the stations and the central processor is a potential weak link. If satellite communications were used, antenna dish alignment during strong ground motion would be a major concern. If dedicated telephone lines or radio telemetry were used, the gathering, repeaters, or central switching stations would be a concern. Direct line-of-sight radio telemetry without repeaters probably would be the least vulnerable, but it is only practical for stations within range and line-of-sight of the central station. However, it is worth noting that in the near-field of a large earthquake, strong ground motion is emergent (Shakal et. al., 1989). This means that there should be some period of time after the onset of ground motion from a large earthquake, as accelerations build to damaging levels, when a vulnerable communications system continues to function.

The survival of the central processing center/warning transmission station is also a potential weak link in an EAS. A large earthquake that occurs near the central processing center could damage or destroy the EAS before any alert warning is transmitted. Hardening all components of this EAS central processing facility may be difficult to accomplish. One possible strategy is to maintain two widely separated EAS centrals in a master-slave relationship. Using identical equipment and algorithms, the slave station issues the alert packets only when the master station fails to do so.

4. A LOW COST PROTOTYPE EAS

Although the conceptual design, technological features, and capabilities of an EAS are important, the cost is the factor that will determine if one is ever built. A California Division of Mines and Geology earthquake-warning feasibility study (Holden et al., 1989) estimates the cost of installing a dedicated 43-station EAS in southern California at $1.4 million for the remote stations, $1.9 million for the central processing facility, and $1.6 million for the annual operation and maintenance costs. Because the benefit of such a system is difficult

to quantify, the study concludes that an EAS cannot be economically justified.

A more recent study by the National Research Council (NRC, 1991) on real-time earthquake monitoring concludes that an EAS is technically feasible and recommends the installation of a prototype system. This recommendation is based, among other considerations, on forming a prototype EAS from existing microseismic monitoring stations to significantly reduce installation and operation costs.

We have examined the characteristics of existing microseismic monitoring networks and have made progress in detailing the modifications necessary to convert them into functioning EAS prototypes. We assumed that each station communicates analog data in real time to a central processing station. Two essential tasks that must be accomplished to create a prototype EAS from such a network are:

- Upgrade the remote seismic stations to measure strong motion.
- Install a real-time central processor and alert algorithms.

4.1 Remote Station Upgrade to Measure Strong Motion

Existing microseismic monitoring stations require adding a strong-motion-measurement instrument for each seismometer component. The strong-motion instruments are commercially available and are typically force-balanced accelerometers. A dramatic reduction in strong-motion instrument cost can be realized by making use of the new silicon microstructure accelerometers as strong-motion instruments. These instruments are now commercially available.

The individual seismic stations of a microseismic monitoring network typically consist of high gain vertical or three-component seismometers, amplifiers, voltage-controlled oscillators (VCOs), telephone or radio telemetry communications, and either ac power or battery power with solar panel charging.

When there is extra bandwidth on the microseismic station real-time communication channel, a strong-motion upgrade can be achieved by adding strong-motion sensors and VCOs: one for each component, and multiplexing these additional data channels on the existing communications channel. The EAS central processing facility would then receive two data streams for each station component: one containing weak-motion data from the seismometer and one containing strong-motion data from the strong-motion sensor. When spare communication bandwidth is not available, additional communication bandwidth can be added by either upgrading the existing link or by adding additional links. Unfortunately, upgrading or installing new communication links (radio or telephone) is expensive.

A low-cost solution to the strong-motion upgrade of microseismic stations that have no spare bandwidth on the existing communication channel is an instrument switch that, using only the existing communication channel, rapidly switches between seismometers and strong-motion sensors at the onset of strong motion. The EAS central processing facility would then receive only one data stream for each station component. The data stream would contain weak-motion data from the seismometer until the onset of strong motion at the station. After that time, the data stream would contain data from the strong-motion sensor. When strong motion at the station subsided, the weak-motion data from the seismometer would be passed to the data stream.

4.2 Prototype Instrument-Switching Unit

We designed a prototype instrument-switching unit and tested it. This switcher concept is shown in Fig. 3. The signals from the seismometer and from the strong-motion instrument are sent through to the instrument switcher. The strong-motion instrument signal is monitored by the switcher and, under normal weak-motion conditions, the seismometer signal is passed to the communications hardware.

When a user-determined threshold is exceeded by the strong-motion instrument (assumed to be set at a level well above the saturation amplitude of the seismometer signal), the strong-motion signal instead of the seismometer's signal is passed to the communication hardware. The switcher has a timer microchip that will perform the switching function for a fixed user-determined time period before switching back to passing the seismometer's signal. The timer chip will continue to extend the switch time by one user-set time period as long as the threshold continues to be exceeded. The unit switches back to the seismometer's signal when one complete user-defined time period has past since the last instant that the threshold was exceeded. It is worth noting that monitoring the strong-motion sensor to determine a switching condition rather than monitoring the seismometer is more reliable since the switching threshold is well within the linear regime of the strong-motion instrument. Furthermore, routine signals transmitted to the seismometers for calibration purposes cannot trigger the switch and are not affected by the prototype EAS.

We tested the prototype switcher in the near field of an underground nuclear test. A Marks Products Inc., model L-4 vertical component seismometer, a Kinemetrics FBA-11 vertical component strong-motion accelerometer, and the switcher output signal were all digitally recorded at 200 samples/s at unity gain. This prototype was set to switch-over at a threshold of 0.01 *g* and the switchover time period was set at 10 s. The results are shown in Fig. 4. The strong-motion recording is shown in the top trace in units of *g*. The middle trace is the L-4 seismometer recording in units of m/s. Note that the L-4 seismometer was physically clipping during the first 8 s or so of strong ground motion. The bottom trace is the switcher output. Since the switcher output is a mix of velocity and acceleration records, the scale was left in raw output voltage units. Clearly, the switcher output is capturing the full strong-motion record before switching back to the seismometer. The initial large negative pulse at about 15 s is actually the L-4 seismometer output. The relative scales of the two instruments is a function of the raw output voltage of the individual instruments. The relative amplitude scales could easily be modified by adding an amplification or attenuation stage to one of the instruments before input to the switcher. Finally, the dc offset evident on the strong-motion part of the switcher record was introduced from an unbalanced voltage supply that was driving the switcher. A dc voltage regulator has since been added to the switcher to eliminate this problem.

Figure 5 zooms in on the switchover time period. All traces and units are as described above. The initial L-4 seismometer signal spike is now evident on the switcher trace. Switchover occurs at about 15.1 s. The strong-motion signal contains a small transient associated with the switching relay but stabilizes quickly. The time between switchover and a stable strong-motion signal is about 30 ms. The switching is fast enough to capture the first peak of the strong ground motion from the nuclear test (Fig. 5). It should be noted that a nuclear test is a worst case test of the switcher performance because the most significant strong motion occurs in the first few cycles of the record. Strong motion from large earthquakes is generally emergent, building to peak levels over many seconds.

4.3 Central Processing Facility Modification

Most existing microseismic monitoring networks telemeter data from remote stations to the central processing facility in real time. The USGS CALNET microseismic network, for example, transmits seismic data in analog form, using radio telemetry or dedicated telephone lines, to a central processing facility. This facility digitizes the incoming data and runs an event detector on the data streams to determine if a small earthquake has occurred. In general, the data are not continuously archived; data are archived when an event is declared by the detector. Although the data are communicated to the central processing facility in real time, the central processor and detector do not run in real time. The data are

buffered in memory so that event processing and data archival can be accomplished after the event has occurred.

Assuming that the remote stations are modified to telemeter strong-motion information, either on new data channels if sufficient spare bandwidth is available, or using the instrument switcher discussed earlier, the EAS requires a real-time processor that performs real-time earthquake parameter estimation on the EAS data channels. An EAS central processing facility that is operated in parallel with the microseismic-network central-processing facility would provide the needed real-time analysis capability without impacting the routine operation of the microseismic network. Figure 6 illustrates this concept. The EAS central processing facility would be collocated with the microseismic central facility. The EAS facility consists of a dedicated real-time central processing unit, digitizers, alert algorithms, and warning communication hardware. The incoming data from the EAS stations, a subnet of the full microseismic network, would be branched off the microseismic central input for digitization and real-time analysis by the central processing facility. The microseismic central processor could also receive the data channels used by the EAS and perform routine event analysis and archiving of those channels. This would eliminate the need for archival capability from the EAS central processing facility. Archived strong-motion events could be "replayed" through the EAS system to test new algorithms or system enhancements.

4.4 A Prototype EAS in the San Francisco Bay Area

The San Francisco Bay Area is an example of a region that has an existing USGS CALNET microseismic network that may be suitable, with modifications, to also function as an EAS. One critical element is the arrangement of sensors; some of them must be located near any expected major earthquake epicenters in the area. Assuming that earthquakes occur with a uniform random distribution of epicenters only on known major faults and neglecting hypocentral depths, Heaton (1989) showed that, for a uniform distribution of n sensors along faults of total length L, the mean travel time for P-waves from the earthquake epicenters on the faults to the closest station is

$$\bar{T} = \frac{L}{4nv},$$

where v is the P-wave velocity. By assuming a fixed hypocentral depth of h for all major earthquakes and, if we also assume that a minimal warning algorithm that issues an initial alert signal would require strong ground motion at two or more stations, then the average time before P-wave arrival at the nearest two stations is

$$\bar{T} = \frac{\sqrt{\left(\frac{3L}{4n}\right)^2 + h^2}}{v}$$

Since the San Francisco Bay Area is known to have three major active faults: the San Andreas, Hayward, and the Calaveras Faults. We can apply Eq. (3) by assuming that the total length of these faults (with instrumentation) is 800 km. Assuming a P-wave velocity of 6 km/s and a hypocentral depth of 10 km (Scheimer, 1985), the mean travel time to the nearest two stations can be calculated as a function of the number of seismic stations along the total length for the three faults. Figure 7 shows the results. There is little improvement in using over 50 stations. The mean P-wave travel time using 50 stations is ~3 s. If we assume that 5 s are required to account for the small delays introduced by radio telemetry or phone line transmission and by the time it takes the alert algorithms to issue a warning and that the P-wave speed is 1.67 times the S-wave speed (Scheimer et. al., 1982), then the damaging S-wave will only have reached an area of 8 km in radius from the earthquake epicenter before the initial alert warning is received by the entire area.

It is worth noting that an EAS is most effective (i.e., gives the longest overall warning times and requires the minimum number of seismic

stations) on shallow faults, which are the sources of most of the major earthquakes in California. Known faults allow the use of a linear distribution of sensors instead of an areal distribution, effectively reducing the number of stations required for the same warning times by about a factor of 10. Shallow focus earthquakes provide the seismic ray path geometry for maximum warning to an area. Japan, for example, has major earthquakes that occur on a myriad of faults without surface expression and at relatively deep focus (Nakamura et. al., 1988), making an EAS much more difficult to implement and of less value because of the very short warning times that could be expected.

The USGS CALNET stations in the Bay Area are shown, along with the major fault traces, in Fig. 8. Clearly, a subset of 50 stations that are located on or near the major faults with an approximately uniform spacing can be selected. A large fraction of the USGS stations consist of a single vertical component seismometer. Some of these stations have fully utilized communication channels and cannot carry additional data on existing communication lines. Such stations could be upgraded to strong motion by adding the switching device described earlier and a single vertical-component strong-motion instrument. When communications are limited to a one component instrument, the vertical component is probably the best axis on which to add a strong-motion instrument since peak accelerations in the near field are generally larger on the vertical component in the early evolution of an earthquake (Shakal et. al., 1987, 1989).

5. CONCLUSIONS

An EAS would provide warning to a diverse community of users, who have applications that range from simply opening a fire station door to shutting down large manufacturing or power-production facilities. Because of the vastly different costs associated with false alarms depending on the application, an EAS must furnish a rapidly updated information packet with estimates of the earthquake's location and magnitude, and an estimate of the reliability of this information so that users can evaluate the data and make independent decisions, depending on their specific needs.

The technology currently exists to develop such an EAS; however, a dedicated system built to all the desired specifications (e.g., number of stations, locations, communications, and hardening) would be very expensive. Furthermore, some operational experience with an EAS is required to define how an optimum system should be constructed. Issues such as false alarm rate, false alarm mechanisms, reliability, earthquake parameter estimation accuracy, and user analysis of the alert information cannot be effectively addressed without the aid of information from an operational EAS. It is therefore very important to establish some prototype systems to serve as test beds for future development.

In some areas, an EAS prototype system can be developed at low cost by upgrading existing dense networks of microseismic monitoring stations so that these stations can also measure strong ground motion; once modified, they could form the nucleus of the EAS network. Strong-motion upgrades can be accomplished without installing any new or expanded communication links. For instance, strong-motion instruments can be added to existing communication links (when unused bandwidth is available) or the instrument switching device that we developed and tested can be added (when spare bandwidth is unavailable). Furthermore, EAS station maintenance can be accomplished during normally scheduled microseismic maintenance visits.

6. ACKNOWLEDGMENTS

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Table 1. A partial list of potential EAS applications.

Shutdown computers	Reroute electrical power
Shutdown disk drives	Shutdown high precision facilities
Shutdown airport operations	Shutdown manufacturing facilities
Stop trains	Shutdown high energy facilities
Shutdown gas distribution	Alert hospital operating rooms
Open fire station doors	Start emergency generators
Stop elevators in a safe position	Shutoff oil pipelines
Issue audio alarms	Shutdown refineries
Shutdown nuclear power plants	Shutoff water pipelines
Maintain safe-state in nuclear facilities	

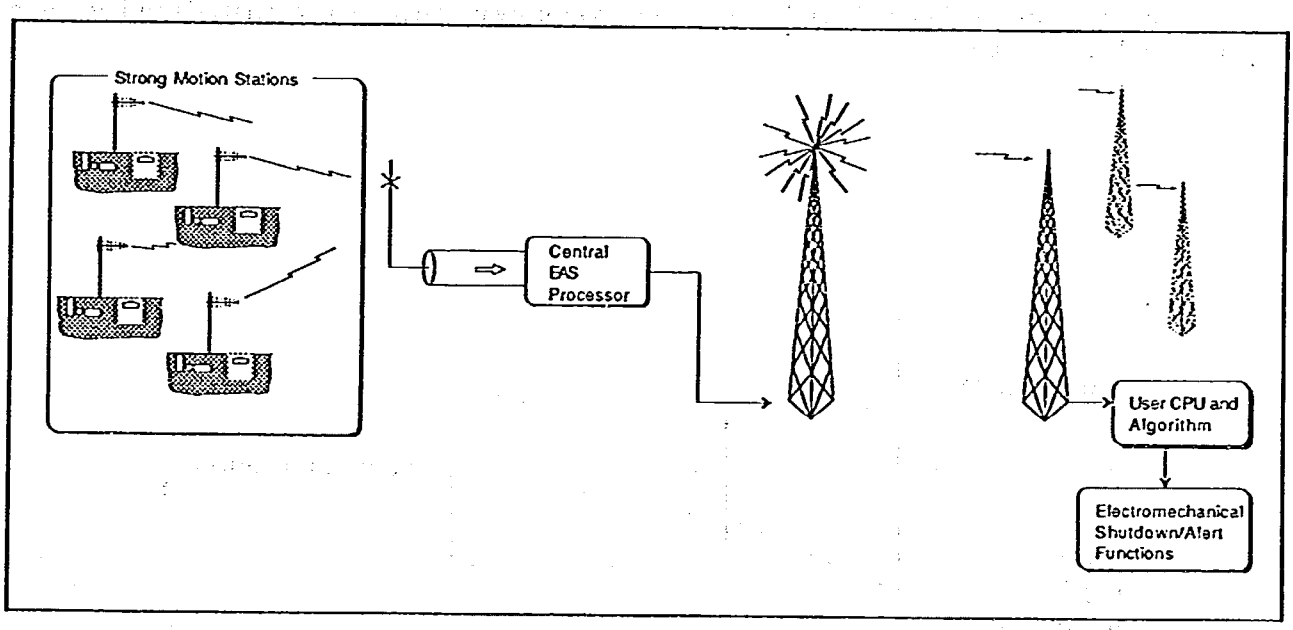


Figure 1. The EAS consists of an area-wide strong-motion network that telemeters real-time data to a central processor. The processor estimates earthquake parameters and issues an alert packet containing these estimated parameters. The user facilities receive the warning packet and execute alert or shutdown functions based on their specific preprogrammed response.

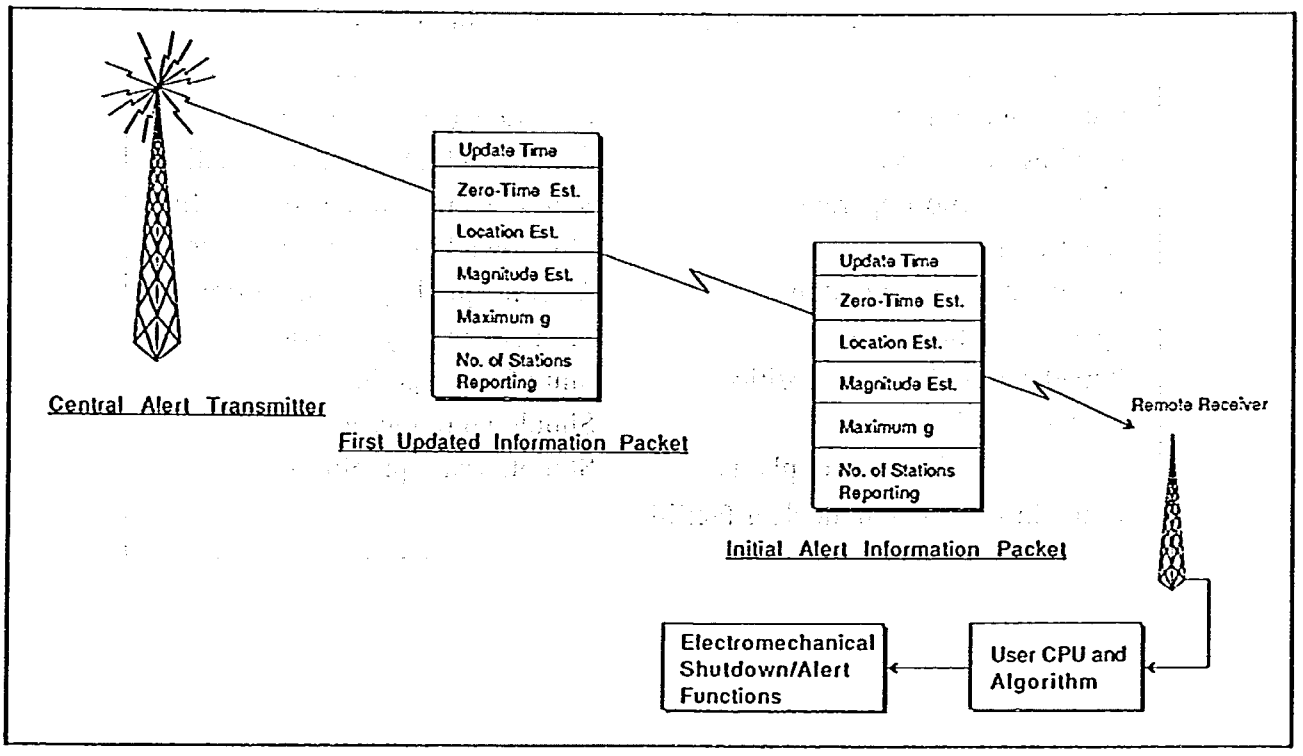


Figure 2. The alert information packet is broadcast and is rapidly updated for user analysis and possible shutdown and alert functions.

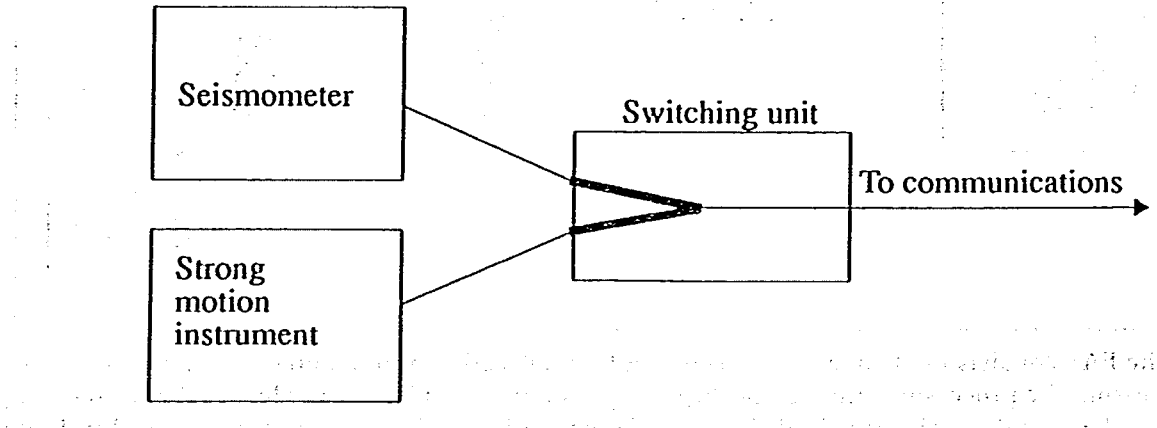


Figure 3. The switching unit receives input data from the strong-motion instrument and the seismometer. The seismometer signal is passed to the station communications under normal seismic conditions. During strong ground motion, the strong-motion instrument data, rather than the seismometer signals, are passed to the station communications.

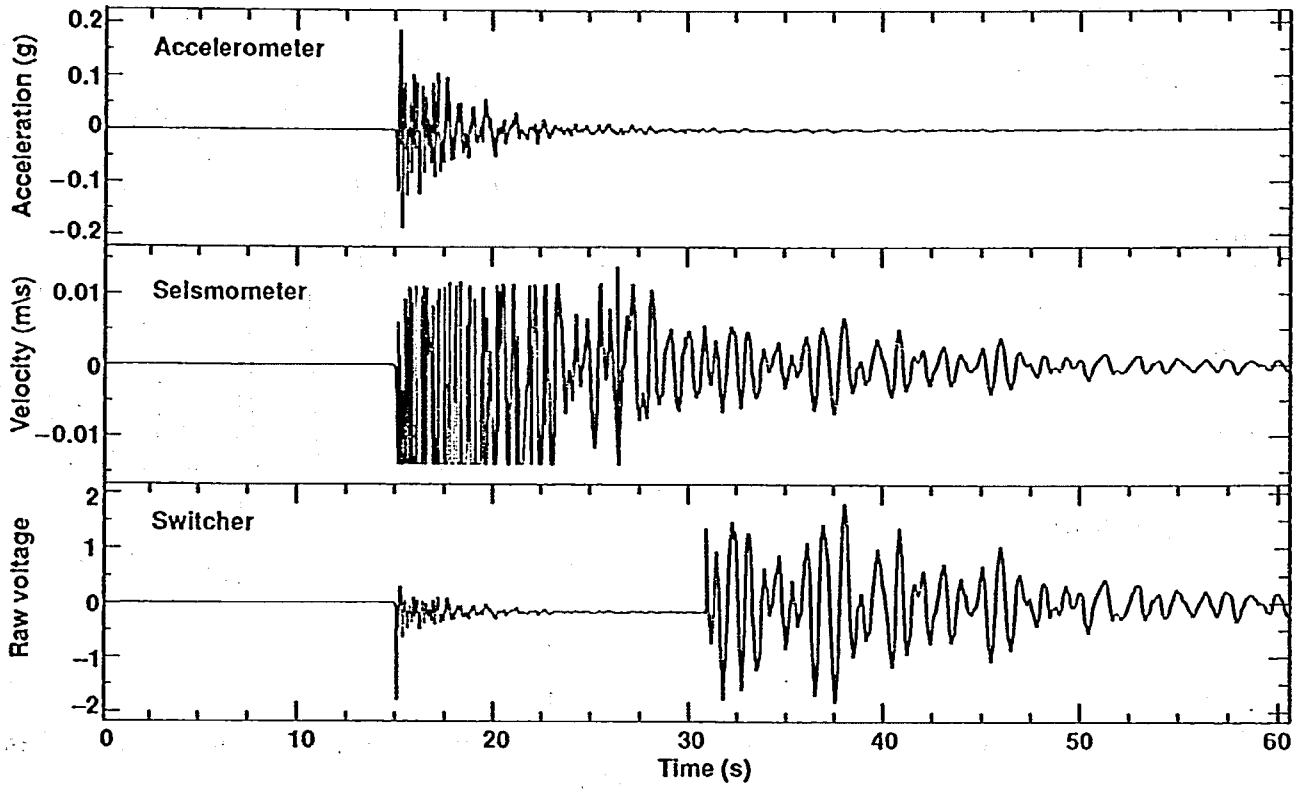


Figure 4. The surface recordings in the near field of an underground nuclear test. The top trace is the strong-motion accelerometer, the middle trace is the seismometer, and the bottom trace is the switcher output.

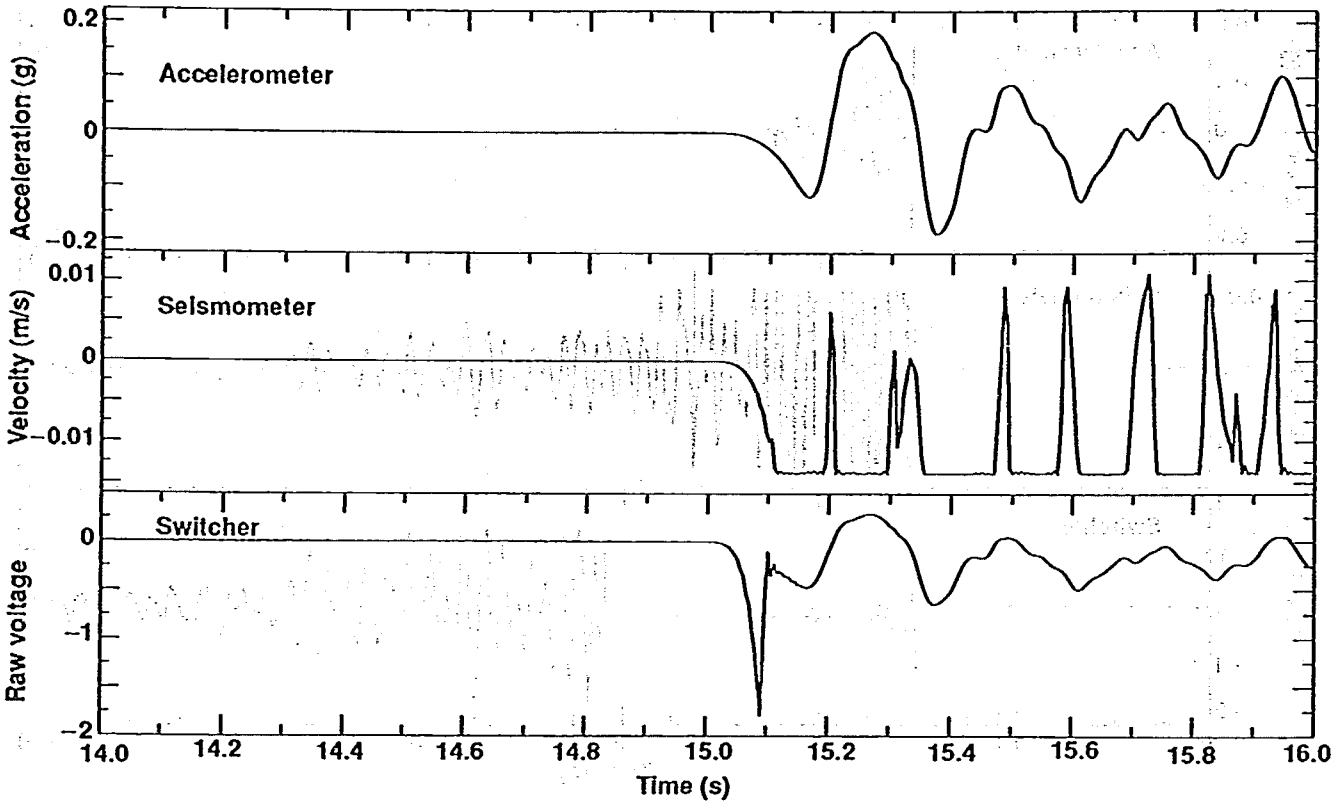


Figure 5. A zoom-in on Fig. 4 at switchover. The switchover time takes place at ~15.1 s, and the stabilization time is ~30 ms. The first major peak of the strong-motion signal is captured in the switched signal.

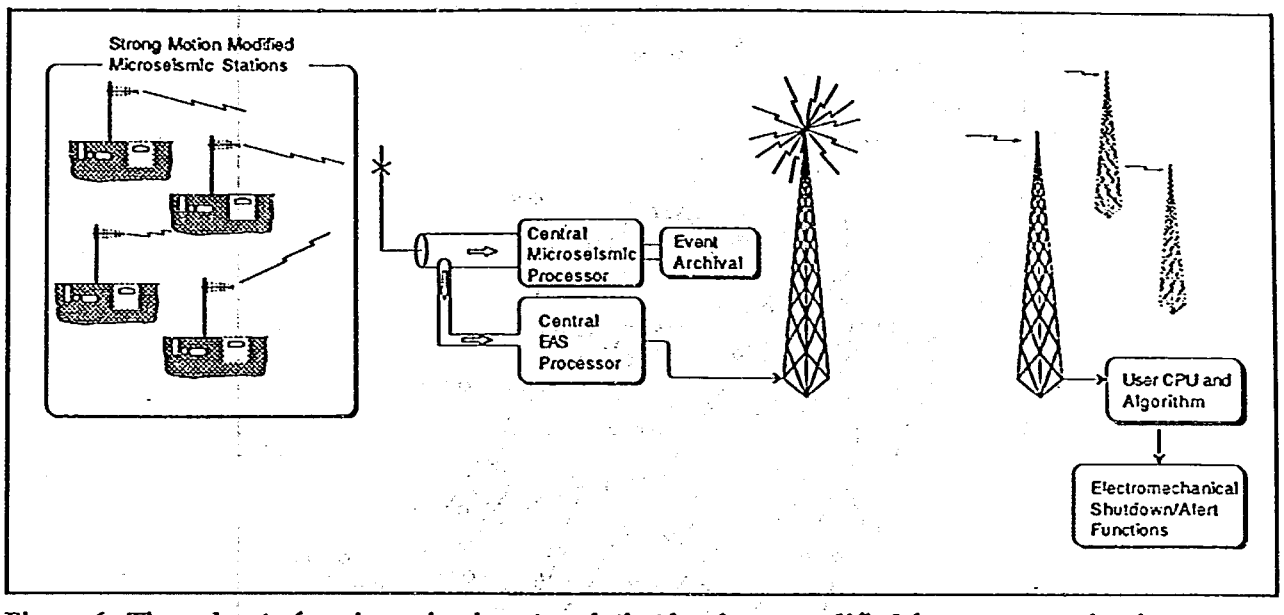


Figure 6. The subnet of a microseismic network that has been modified for strong motion is event-processed by the microseismic processor as usual and is streamed through the EAS central processor for real-time estimation of earthquake parameters and warning broadcasts.

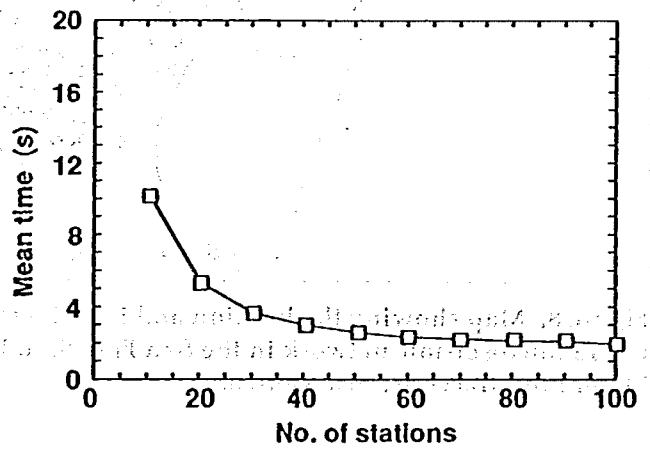


Figure 7. Plot showing the mean *P*-wave travel time to the nearest two stations of a uniformly distributed network of stations along the major faults in the San Francisco Bay Area as a function of the number of stations.

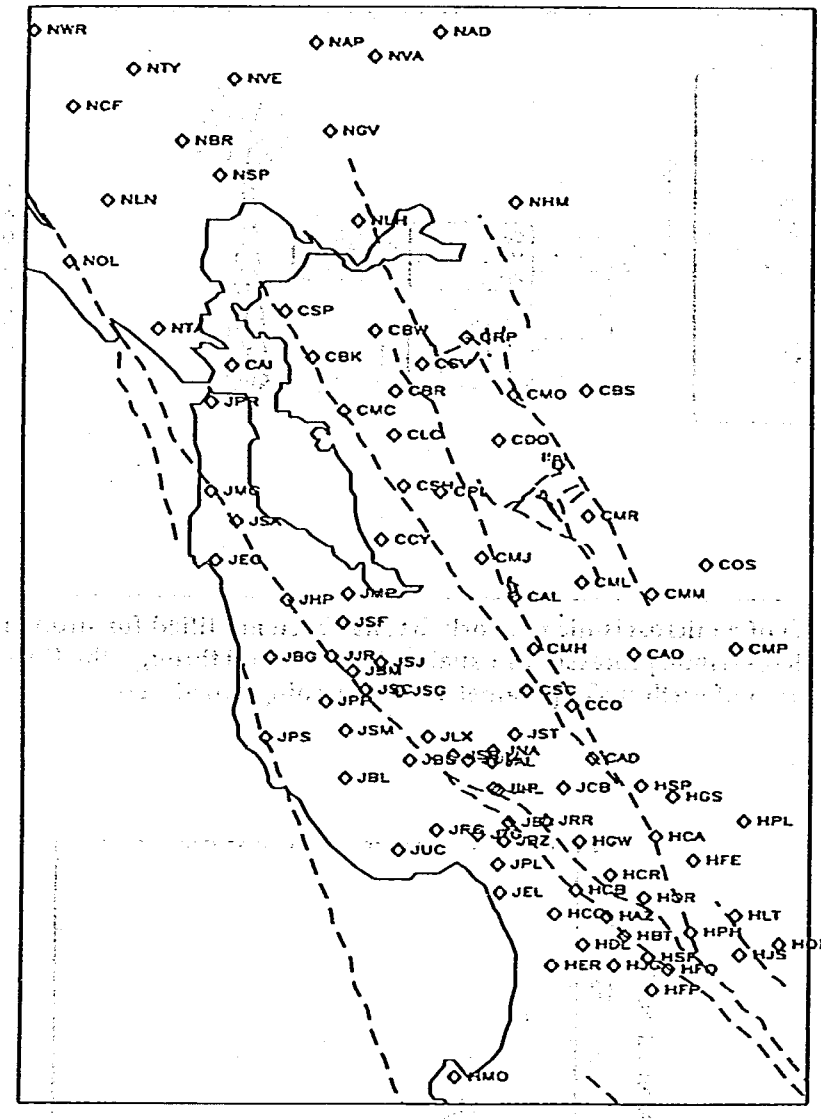


Figure 8. Map showing the location and identifiers of the USGS microseismic network in the San Francisco Bay Area. Major fault traces are also shown.

大規模地下構造物の耐震設計法ガイドライン（案）

朝倉　肇[1]、川島一彦[2]、杉田秀樹[3]

概要

建設省技術開発プロジェクト「地下空間の利用技術の開発」の一項として実施した大規模地下構造物の耐震設計技術の開発の最終成果である「大規模地下構造物の耐震設計法ガイドライン（案）」の概要を報告するものである。

キイワード：地下構造物、地中構造物、耐震設計、応答変位法、シールドトンネル、地震防災システム

1　はじめに

東京等大都市圏においては、用地の確保が困難なことから地下の利用が重要となっている。このようなニーズに答えるため、建設省は総合技術開発プロジェクト制度を用いて「地下空間の利用技術の開発」に関する研究を昭和６２年度から平成３年度までの５か年間に行った。この中では、建設省土木研究所、建築研究所、国土地理院が参加して、地下の有効利用に関する技術開発が行われた。本研究の一項として、大規模地下構造物の耐震性に関する研究が行われた。

総合技術開発プロジェクト「地下空間の利用技術の開発」の実施に際しては、民間における最新の技術を活用するため、別途、建設省土木研究所と（財）先端建設技術センターの共同研究が行われた。この中に一つが大規模地下構造物の耐震性に関する共同研究で、建設省土木研究所耐震研究室と先端技術開発センター、民間の８社（フジタ、鹿島建設、大成建設、飛島建設、間組、奥村組、鴻池組、清水建設）の間で共同研究が行われた。

こうした共同研究の成果も取り入れて、地下空間の利用技術の開発に関する研究の最終報告書が平成４年３月にまとめられた。このうち、大規模地下構造物の耐震設計に関する研究の成果は「大規模地下構造物の耐震設計法ガイドライン（案）」として取りまとめられた[1]。これは、現在までに作成されている石油パイプライン[2]、上水道[3]等小口径の管路や共同溝[4]等よりもさらに断面の大きい地下構造物の耐震設計法を示したものである。ここでは、「大規模地下構造物の耐震設計法ガイドライン（案）」の概要について紹介する。

2　ガイドライン（案）の性格と構成

このガイドライン（案）は、都市部の軟弱な地盤中に建設される大規模な地下構造物の耐震設計法を示したものである。振動特性の基づき地下構造物を分類すると、図－１のように次の３種類になる。

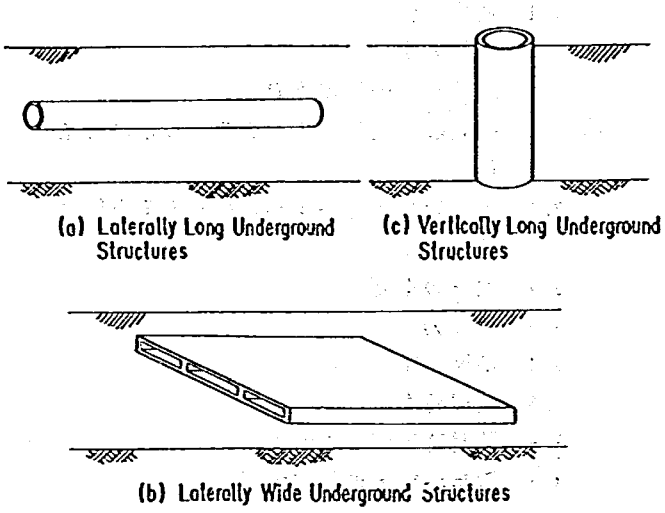


Fig.1　Classification of Underground Structures

1)線状地下構造物

地表面に沿って細長く建設される地下構造物で、構造物の長手方向の地震時断面力が支配的となる地下構造物

2)幅広地下構造物

地表面に沿って面的に広い範囲に建設される地下構造物で、鉛直面内の地震時の地盤のせん断変形が支配的な地下構造物

3)鉛直地下構造物

地中に深く建設される地下構造物で、鉛直軸方向の地震時断面力が支配的な地下構造物

本ガイドライン（案）では、対象構造物として、一応、大深度地下道路、地下駐車場、地下共同溝等を想定したが、特に構造物の用途種別、重要性は特定せず、上記の３種類の地下構造物に対して、それぞれ一般的に耐震設計法を記述することとした。研究開発の重点は、線状地下構造物の中で近年建設実績の多いシールドトンネルの耐震設計法に置いた。

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ただし、本ガイドライン（案）は、研究の成果をガイドラインふうにまとめたもので、行政上の示方書や指針ではなく、単なる研究成果である。

本ガイドライン（案）の構成は、以下に示す通りである。

第１章　総則
　1.1　適用の範囲
　1.2　用語の定義
第２章　耐震設計の基本方針
第３章　耐震設計上考慮すべき荷重及び設計条件
　3.1　耐震設計上考慮すべき荷重とその組合せ
　3.2　地震の影響
　3.3　慣性力
　3.4　地震時地盤変位
　3.5　地震時土圧
　3.6　地盤時周面せん断力
　3.7　耐震設計上の地盤種別
　3.8　動的解析に用いる入力地震動
　3.9　地震時地盤ばね定数
第４章　大断面シールドトンネルの耐震設計法
　4.1　一般
　4.2　シールドトンネル長手方向の耐震設計法
　　4.2.1　耐震設計の基本方針
　　4.2.2　応答変位法による耐震計算
　　4.2.3　二次覆工がない場合のシールドトンネルの等価軸剛性
　　4.2.4　二次覆工を有する場合の覆工の等価剛性
　　4.2.5　耐震性の照査
　　4.2.6　動的解析による照査
　　4.2.7　二次覆工の長手方向最小鉄筋量
　4.3　トンネル横断方向の耐震設計法
　　4.3.1　耐震設計の基本方針
　　4.3.2　応答変位法による耐震計算
　　4.3.3　動的解析による照査
　　4.3.4　耐震性の照査
第５章　幅広地下構造物の耐震設計
　5.1　耐震設計の基本方針
　5.2　応答変位法による耐震計算
　5.3　動的解析による照査
第６章　鉛直地下構造物の耐震設計法
　6.1　耐震設計の基本方針
　6.2　応答変位法による耐震計算
　6.3　動的解析による照査
　6.4　耐震性の照査
第７章　地震の影響の低減を期待する構造
　7.1　一般
　7.2　可とう性継手を配置する構造
　7.3　シールドトンネルのリング継手剛性を低減させる構造
　7.4　免震材を用いる構造
第８章　大規模地下道路の地震防災システム
　8.1　一般
　8.2　地震時に地下道路利用者が抱きやすい不安感
　8.3　地震時に地下道路利用者に不安感を与える要因
　8.4　地震防災システムの基本方策

3　耐震設計の基本方針

耐震計算には、原則として応答変位法[6)]を用いる。応答変位法による耐震計算に際しては、地下構造物の振動特性を考慮して、適切なモデル化を行わなければならない。応答変位法によって算出した地震時増分断面力を、常時の断面力と重ね合わせて地震時断面力を算出する。

許容応力度法によって耐震計算する場合にには、地震時断面力によって部材に生じる応力度が許労応力度以下に、また、地震時保有水平耐力法によって耐震計算する場合には地震時断面力が構造系の地震時保有水平耐力以下になるように、断面を定める。

応答変位法により耐震設計した地下構造物のうち、地震時の挙動が複雑なものについては、動的解析により安全性を照査する。

4　耐震設計の考慮する荷重

地下構造物の耐震設計に当たっては、死荷重、土圧、水圧、浮力、地盤変位、地震の影響を考慮する。荷重は、最も不利な応力、変位が生じるように組み合わせて作用させる。地震の影響としては、以下の荷重を考慮する。

(1)慣性力

設計水平震度は、次式によって算出する。

$$k_h = c_z \cdot c_G \cdot c_U \cdot k_{h0} \qquad (1)$$

ここで、

k_h:設計水平震度

c_z:地域別補正係数（図－２参照）

c_G:地盤別補正係数（表－１参照）

cu:震度別補正係数で、次式で算出する。

$$c_u = 1.0 - 0.015z \quad (2)$$

z:地表面下らの深さ(m)

k_{h0}:標準設計水平震度(=0.2)

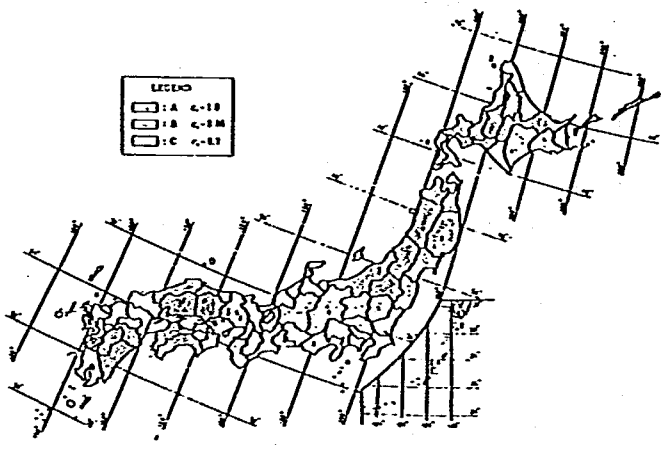


Fig.2 Seismic Zoning Map and Modification Coefficient c_z

Table 1 Classification of Ground Condition

GROUND CONDITION	DEFINITION	APPROXIMATE ESTIMATION
GROUND I	$T_G < 0.2$SEC	TERTIARY OR OLDER
GROUND II	$0.2 \leq T_G < 0.6$SEC	ALLUVIUM AND DILUVIUM
GROUND III	0.6SEC$\leq T_G$	SOFT ALLUVIUM

(2)地震時地盤変位

地震時地盤変位は、図－３に示すように、地表面からの深さ及び水平面内の位置に応じて与え、次式で算出する。

$$u(z,x) = \frac{4}{\pi^2} S_V \cdot T_s \cdot \cos\left(\frac{\pi z}{2H}\right) \cdot \sin\left(\frac{\pi x}{2L}\right) \quad (3)$$

ここで、

u(z, x):深さz(m)、水平方向にx(m)の位置における地震時地盤変位(m)

Sv:基盤面における速度応答スペクトル(m/sec)で、次式により算出する。

$$S_V = c_z \cdot S_{V0} \quad (4)$$

cz：地域別補正係数

Sv0：標準速度応答スペクトル(m/sec)で、許容応力度法による場合には、図－４に、また、地震時保有水平耐力法による場合には図－５による。

Ts:表層地盤の固有周期で、次式で算出する。

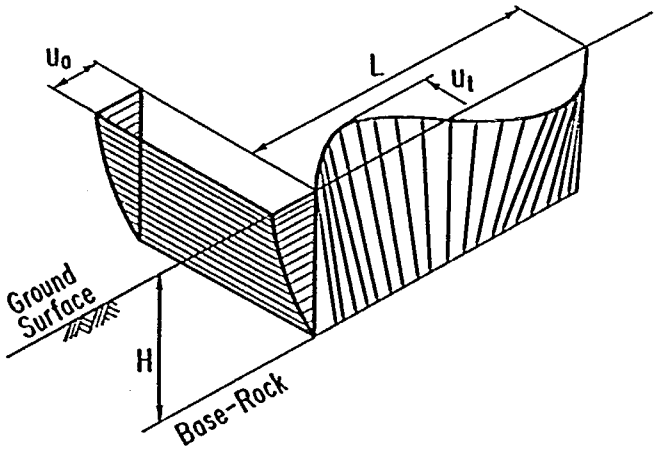


Fig.3 Deformation of Subsurface Ground Assumed in Seismic Deformation Method

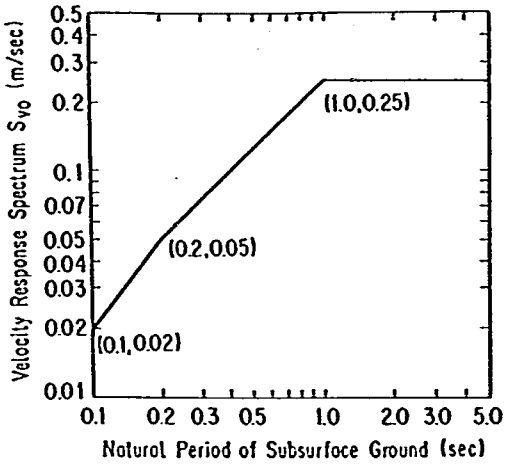


Fig.4 Standard Velocity Response Spectrum (Allowable Stress Design Method)

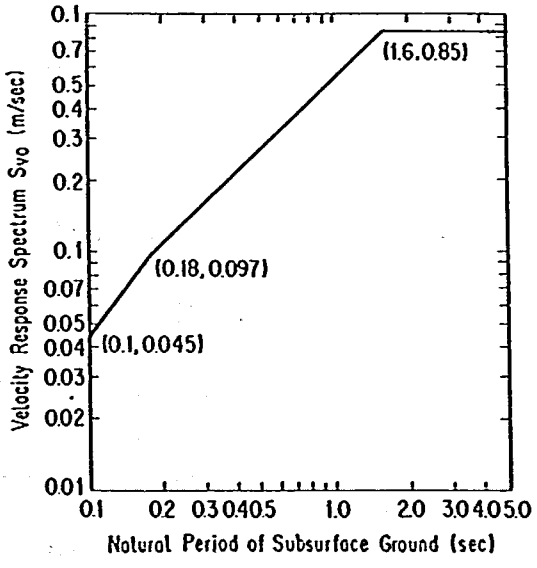


Fig.5 Standard Velocity Response Spectrum (Bearing Capacity Design Method)

$$T_S = 1.25 \cdot T_G \quad (5)$$

$$T_G = 4\Sigma \frac{H_i}{V_{Si}} \quad (6)$$

H_i：i番目の地層の厚さ(m)

V_{Si}：i番目の地層の標準せん断弾性波速度(m/sec)

L：地盤振動の波長(m)で、次式で算出する。

$$L = \frac{2L_1L_2}{L_1+L_2} \quad (7)$$

L_1、L_2：表層地盤及び基盤の地盤振動の波長で、次式で算出する。

$$L_1 = T_S \cdot V_{SD} \quad (8)$$

$$L_2 = T_S \cdot V_{SDB} \quad (9)$$

V_{SD}：表層地盤の平均せん断弾性波速度(m/sec)

V_{SDB}：基盤のせん断弾性波速度(m/sec)

(3)地震時周面せん断力

地盤と接する地下構造物の周面には、図－6に示すように式(10)による地震時周面せん断力を作用させる。ただし、式(10)による地震時周面せん断力が地下構造物に接する地盤のせん断強度を上回る場合には、地震時周面せん断力は地下構造物に接する地盤のせん断強度とする。

$$\tau = \frac{G_D}{\pi H} S_V \cdot T_S \cdot \sin\left(\frac{\pi z}{2H}\right) \quad (10)$$

ここで、

τ：地表面から深さz(m)における単位面積当りの地震時周面せん断力(tf/m^2)

S_V：基盤面における速度応答スペクトルで、式(4)による

G_D：表層地盤の動的せん断変形係数(tf/m^2)

T_S：表層地盤の固有周期で、式(5)による

H：表層地盤の厚さ(m)

(4)地震時土圧

地盤と接する地下構造物の側面には、図－7に示すように式(11)による地震時土圧を考慮する。

$$p(z) = k_H \cdot \{u(z) - u(z_B)\} \quad (11)$$

ここで、

$p(z)$：地表面から深さz(m)における単位面積当りの地震時土圧(tf/m^2)

k_H：単位面積当りの地震時地盤ばね定数(tf/m^2)

$u(z)$：地表面下らの深さz(m)における地震時地盤変位で、式(3)により算出する

z_B：地表面から地下構造物底面までの深さ(m)

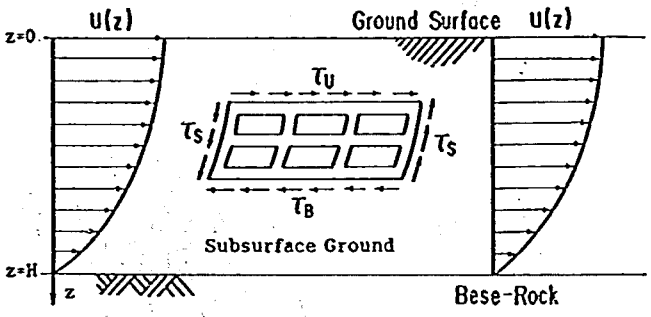


Fig.6 Design Shear Stress around Surface

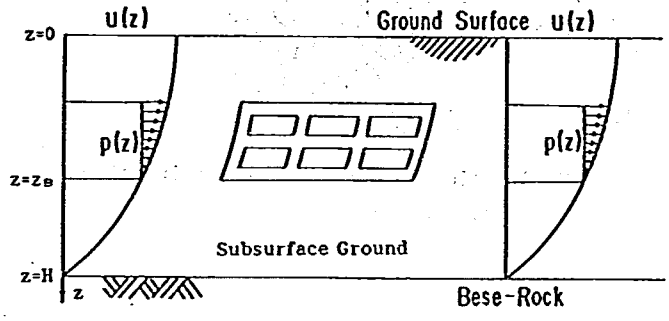


Fig.7 Dynamic Earth Pressure

(5)動的解析に用いる入力地震動

応答スペクトル法に用いる加速度応答スペクトルは、基盤において与え、原則として式(12)により算出する。

$$S = c_Z \cdot c_D \cdot S_V \quad (12)$$

ここで、

S：応答スペクトル法に用いる加速度応答スペクトル(cm/sec^2)

c_Z：地域別補正係数（図－2参照）

c_D：減衰定数別補正係数で、次式により算出する

$$c_D = \frac{1.5}{40h_i+1} + 0.5 \quad (13)$$

h_i：モード減衰定数

S_V：標準加速度応答スペクトルで、許容応力度法に用いる場合には図－8の値を、また、地震時保有水平耐力法を用いる場合には図－9の値を用いる。

時刻歴応答解析には、式(12)に近いスペクトル特性を有する強震記録、または、式(12)にスペクトル特性を合わせた記録を用いる。

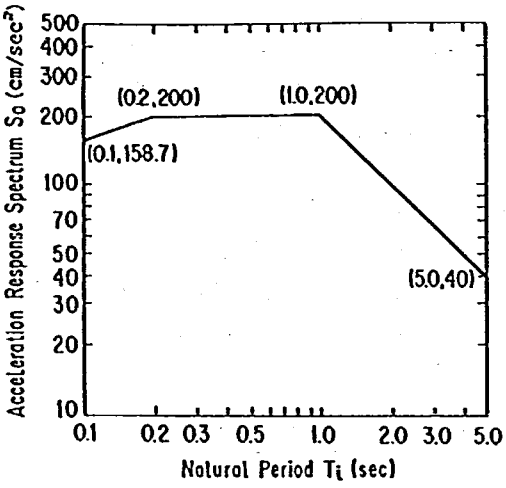


Fig.8 Standard Acceleration Response Spectrum (Allowable Stress Design Method)

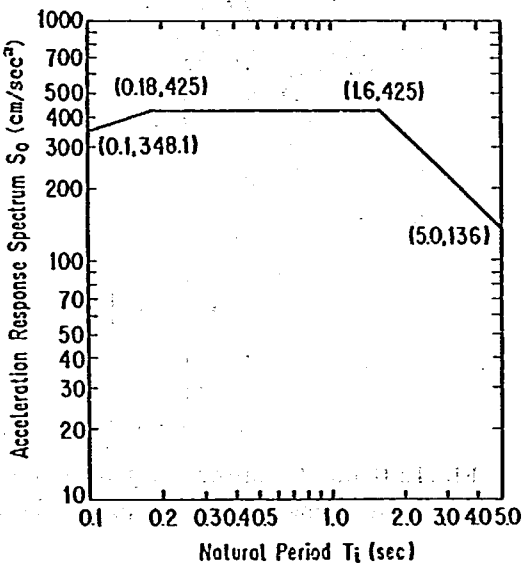


Fig.9 Standard Acceleration Response Spectrum (Bearing Capacity Design Method)

5 大断面シールドトンネルの耐震設計法

5－1 長手方向の耐震設計法

(1)応答変位法による耐震計算

シールドトンネルは、図－１０に示すように、多数のセグメントを継手で結合して建設される。シールドトンネルのモデル化には図－１１に示すようにいろいろなものがあるが、耐震設計では、剛性が等価なはりとしてモデル化することが適切である。

シールドトンネルに発生する長手方向の断面力は、図－１２に示すようにシールドトンネルを等価な剛性を有する弾性床上の梁と見なし、弾性床に式(3)で与えられる地震時地盤変形を与えて、原則として式(14)～式(21)により算出する[7]。

$$P_h^f = \alpha^f \frac{\pi u_h}{L} (EA)_{eq}^S \quad (14)$$

$$P_h^I = \alpha^I \frac{\pi u_h}{L} (EA)_{eq}^I \quad (15)$$

$$P_v^S = \alpha^f \frac{\pi (u_h + u_v)}{L} (EA)_{eq}^S \quad (16)$$

$$P_v^I = \alpha^I \frac{\pi (u_h + u_v)}{L} (EA)_{eq}^I \quad (17)$$

$$M_h = \alpha_2 \frac{4\pi u_h}{L^2} (EI)_{eq} \quad (18)$$

$$M_v = \alpha_3 \frac{4\pi u_v}{L^2} (EI)_{eq} \quad (19)$$

$$Q_h = \alpha_2 \frac{8\pi u_h}{L^3} (EI)_{eq} \quad (20)$$

$$Q_v = \alpha_3 \frac{8\pi u_v}{L^3} (EI)_{eq} \quad (21)$$

ここで、

P_h^S, P_h^I：水平面内の地盤振動による圧縮軸力、引張軸力(tf)

P_v^S, P_v^I：鉛直面内の地盤振動による圧縮軸力、引張軸力(tf)

M_h, M_v：水平面内および鉛直面内の地盤振動による曲げモーメント(tf･m)

Q_h, Q_v：水平面内および鉛直面内の地盤振動によるせん断力(tf)

$(EA)_{eq}^S$, $(EA)_{eq}^I$：等価軸圧縮剛性および等価軸引張剛性(tf)で、式(31)及び式(32)により算出する。

$(EI)_{eq}$：等価軸曲げ剛性(tf)で、式(35)により算出する。

u_h：トンネル中心軸深さにおける地震時水平地盤変位(m)で、式(3)により算出する。

u_v：トンネル中心軸深さにおける地震時鉛直地盤変位(m)で、地震時水平地盤変位u_hの1/2とする。

L：地盤振動の波長(m)で、式(7)により算出する。

α^f, α^I, α_2, α_3：地盤ひずみの構造物への伝達率で式(22)～式(25)により算出する。

$$\alpha^f = \frac{1}{1+(2\pi/\lambda^f L_i)^2} \quad (22)$$

$$\alpha^I = \frac{1}{1+(2\pi/\lambda^I L_i)^2} \quad (23)$$

$$\alpha_2 = \frac{1}{1+(2\pi/\lambda_2 L)^4} \quad (24)$$

$$\alpha_3=\frac{1}{1+(2\pi/\lambda_3 L)^4} \tag{25}$$

ここに、

$$\lambda_1=\sqrt{\frac{K_{y1}}{(EA)^S_{eq}}} \tag{26}$$

$$\lambda_1=\sqrt{\frac{K_{y1}}{(EA)^I_{eq}}} \tag{27}$$

$$\lambda_2=\sqrt[4]{\frac{K_{y2}}{(EI)_{eq}}} \tag{28}$$

$$\lambda_3=\sqrt[4]{\frac{K_{y3}}{(EI)_{eq}}} \tag{29}$$

$$L_i=\sqrt{2}\cdot L \tag{30}$$

K_{y1}, K_{y2}, K_{y3}：トンネル長手方向と、これに直交する水平面および鉛直面内の地盤ばね定数(tf/m^2)。

(2)二次覆工がない場合の等価軸剛性

1)シールドトンネルの等価軸剛性は、図－１３に示すように、引張方向と圧縮方向で異なる。等価軸剛性は、原則として式(31)及び式(32)で算出する。

$$(EA)^S_{eq}=E_S A_S \tag{31}$$

$$(EA)^I_{eq}=\frac{1}{(K_S/K_J)+1}\cdot E_S A_S \tag{32}$$

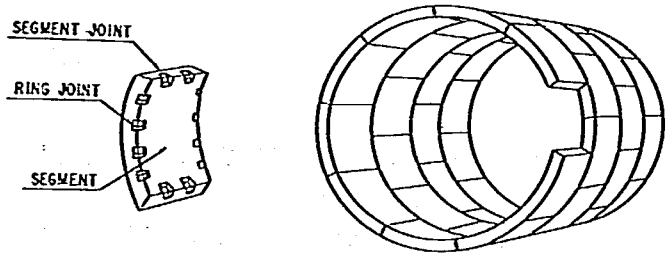
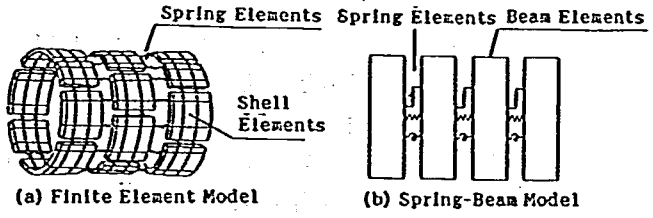


Fig.10 Segments and Joints of Shield Tunnel Lining



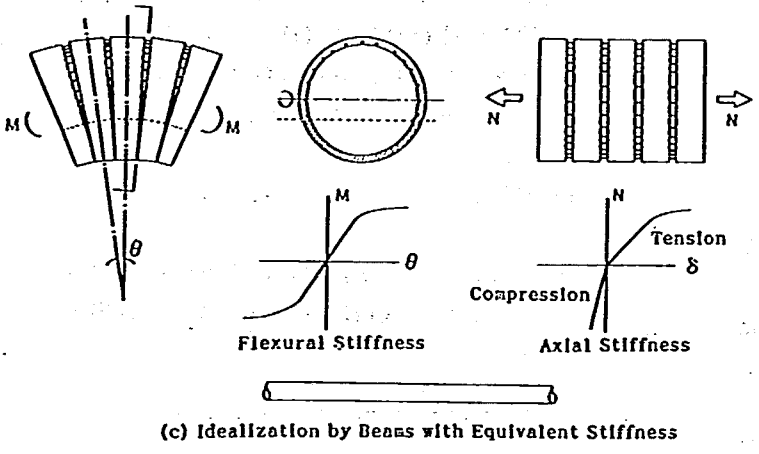


Fig.11 Idealization of Shield Tunnel

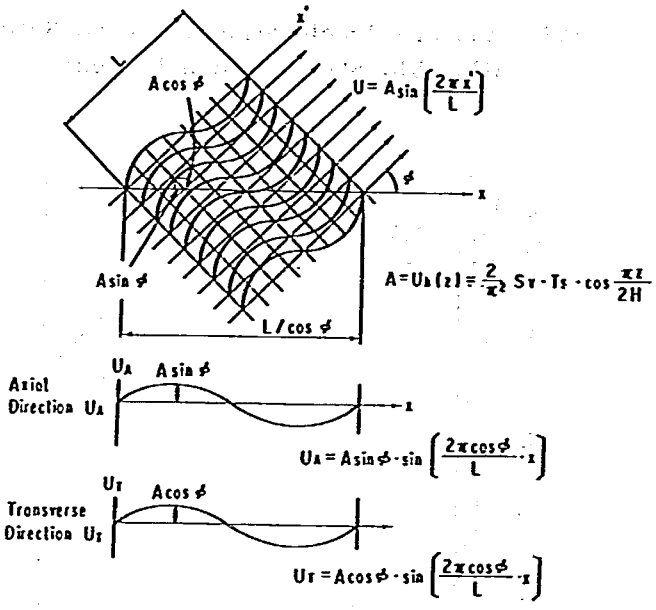


Fig.12 Ground Displacement Assumed in Seismic Deformation Method

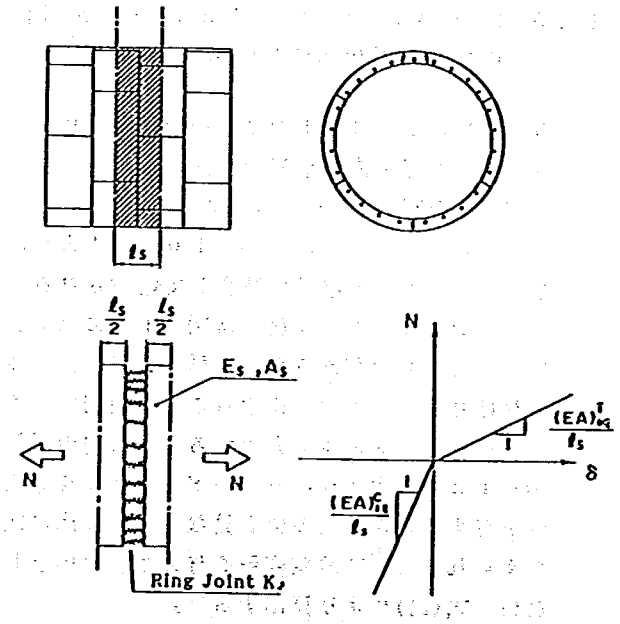


Fig.13 Axial Stiffness of Segments-Ring Joints

ここに、

$(EA)^{S}_{eq}$：等価軸圧縮剛性(tf)

$(EA)^{I}_{eq}$：等価軸引張剛性(tf)

Es：セグメントの弾性係数(tf/m^2)

As：セグメントリングの断面積(m^2)

ℓs：セグメント幅（リング長）(m)

Ks：セグメントリングの軸剛性(tf/m)で、式(33)で算出する。

$$Ks=\frac{EsAs}{\ell s} \qquad (33)$$

Kj：リング継手全体の軸剛性(tf/m)で、式(34)で算出する。

$$Kj=n\cdot kj \qquad (34)$$

kj：リング継手１箇所当りの軸剛性(tf/m)

n：リング接合部あたりのリング継手個数

2)シールドトンネルの等価軸曲げ剛性は、図－１４に示すモデルから、原則として式(35)により算出する[8]。

$$(EI)_{eq}=\frac{\cos^3\phi}{\cos\phi+(\pi/2+\phi)\sin\phi}\cdot Es\cdot Is \qquad (35)$$

ここに、

$(EI)_{eq}$：等価軸曲げ剛性(tfm^2)

Es：セグメントの弾性係数(tf/m^2)

Is：セグメントの断面２次モーメント(m^4)

ϕ：トンネル断面中心軸から、中立軸までの角度で、式(36)により算出する。

$$\phi+\cot\phi=\pi\cdot\frac{1}{2}+\frac{Kj}{EsAs/\ell s} \qquad (36)$$

(3)二次覆工を有する場合の覆工の等価剛性

二次覆工を有する場合の覆工全体の長手方向剛性は、二次覆工コンクリートにひびわれが生じた後の鉄筋のばね作用の影響を受けるため、その評価が難しい。土木研究所における覆工の動的載荷実験によれば、長手方向の圧縮力に対しては、セグメントリングと二次覆工コンクリートは並列ばねとして抵抗するが、図－１５に示すように、引張力に対しては二次覆工の損傷に応じて表－２のように４ステップに変化する[9][10]。

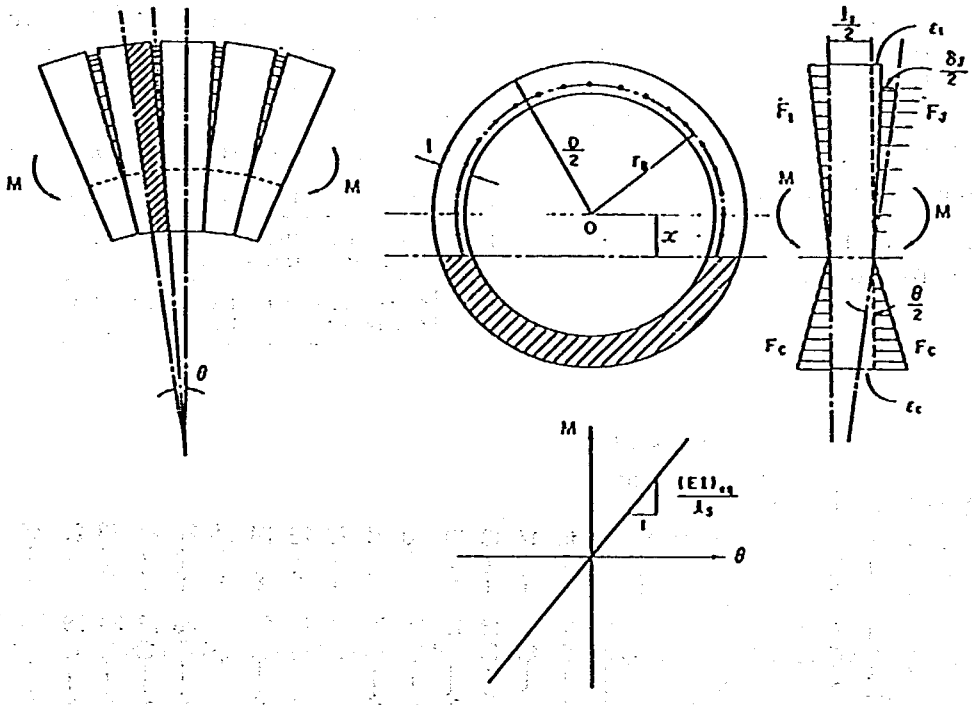
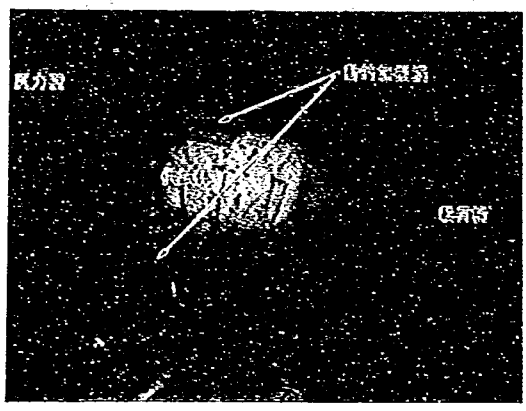
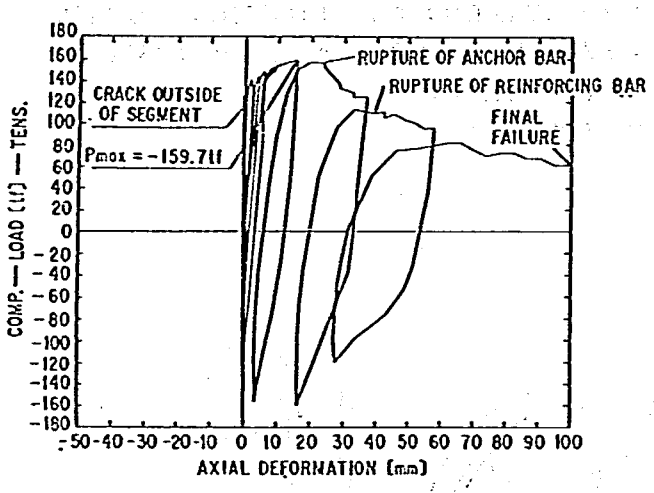


Fig.14 Flexural Stiffness of Shield Tunnel



(a) Experimental Set-up



(b) Loading Hysteresis Loop

Fig.15 Effect of Second Lining for Ductility of Flexural Deformation

(4)動的解析による照査

動的解析では、図－１６に示すように周辺地盤は地盤の非線形性を等価剛性で表した有限要素法もしくは適切に地盤の振動特性をモデル化したばね・質点系モデルによって表わし、シールドトンネルは等価引張軸剛性を有するはりにモデル化する。入力地震動は、基盤面において与えるが、地震動の箇所別の違いが地下構造物の応答に大きな影響を与えるため、これをを適切に考慮することが重要である。

(5)二次覆工の長手方向最小鉄筋量

シールドトンネルに二次覆工を設ける場合には、図－１７に示すように、特定のリング接合面のみに損傷が集中することを防止するために、二次覆工の長手方向に、式(37)で算出される以上の鉄筋を配置しなければならない。

$$A_s = (A_t \cdot \sigma_t - P_E)/\sigma_{ts} \qquad (37)$$

ここに、

A_s：二次覆工の長手方向最小鉄筋量(cm^2)

A_t：二次覆工コンクリートの断面積(cm^2)

σ_t：二次覆工コンクリートの引張強度(kgf/cm^2)

Table 2 Damage Degree of Shield Tunnel with Second Lining

Step	Damage Degree
I	All sections in 1st and 2nd linings are in elastic. No damage.
II	Cracks in 2nd lining around ring only at ring joints.
III	Cracks in 2nd lining around ring occur not only at ring joint but at other sections.
IV	Contact between the 1st lining and 2nd lining is lost. Final failure is to be developed.

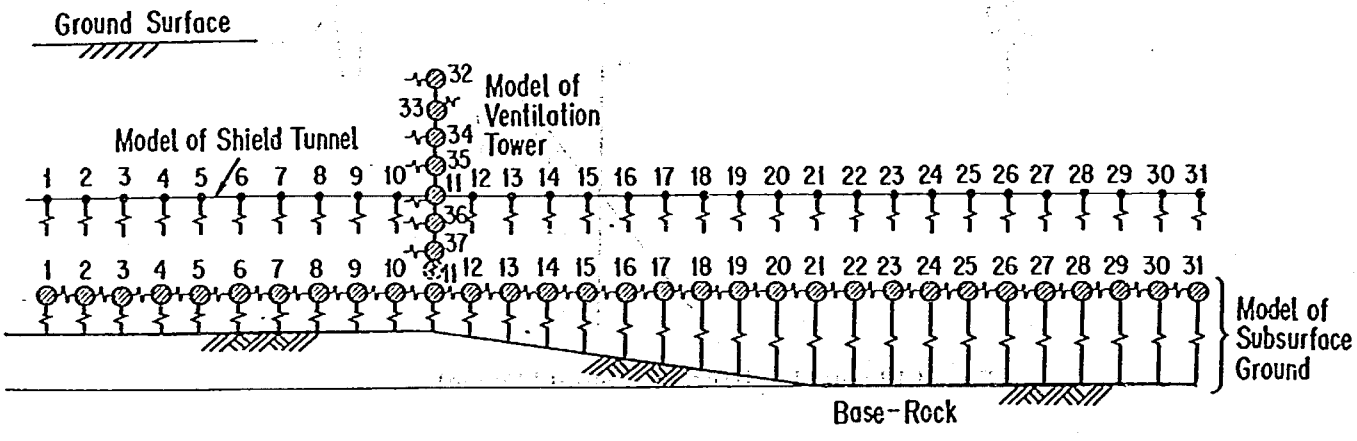


Fig.16 Model of Shield Tunnel, Ventilation Tower and Subsurface ground

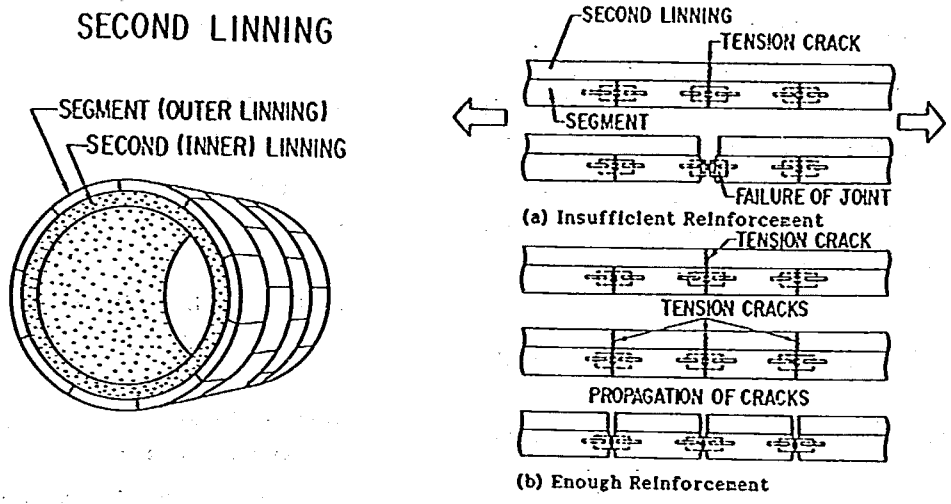


Fig.17 Minimum Reinforcement Required in Second Lining for Preventing Concentration of Damage at only Selected Ring Joints

σ_{ts}：二次覆工鉄筋の引張破断応力度(kgf/cm^2)

P_E：リング継手の引張破断強度(kgf)で、式(38)～式(39)で算出されるP_A、P_Eのうちのいずれか小さい方の値とする。

$$P_A = n \cdot p_A \quad (38)$$

$$P_E = m \cdot p_E \quad (39)$$

ここに、

P_A：アンカー鉄筋の引張破断強度(kgf)

P_E：リング継ぎボルトの引張破断強度(kgf)

n：1リング接合面あたりのアンカー鉄筋の本数（本）

m：1リング接合面あたりのリング継ぎボルトの本数（本）

p_A：アンカー鉄筋1本あたりの引張破断強度(kgf)

p_E：リング継ぎボルト1本あたりの引張破断強度(kgf)

5－2　横断方向の耐震設計法

応答変位法による耐震計算では、以下のように地震時増分断面力を算出する。

1)トンネル横断面を剛性一様なリングにモデル化する。

2)リングの周囲には、トンネルと地盤との間の相互作用を評価するため地震時地盤ばねを設ける。

3)地震力として、トンネル横断面の図心位置に沿う地震時地盤変位、地震時地盤応力、および加速度を求める。均質・整形地盤が水平方向に振動する場合には、この応力は地震時せん断応力となる。なお、地震時地盤応力が覆工に接する地盤のせん断強度を上回る場合には、地震時地盤応力は覆工に接する地盤のせん断強度とする。

4)地震力のうち、地震時地盤内応力および慣性力は、リングの法線方向と接線方向にわけ、リングに直接載荷する。地震時地盤変位もリングの法線方向成分と接線方向成分とにわけ、それぞれの方向の地震時地盤ばねを介してリングに作用させる。なお、土被りが十分にある場合は、覆工の自重に起因する慣性力を無視してもよい。

5)以上の構造モデルと地震荷重を用いて構造計算を行い、リングに生じる地震時増分断面力を算出する。

6)常時の断面力と地震時の増分断面力を重ね合わせて地震時断面力を算出する。

なお、地震時断面力の算出に際しては、トンネル及び地盤に以下の仮定及び条件を設ける。

1)表層地盤は均質である。

2)表層地盤は基本固有振動モードで振動する。

3)表層地盤および覆工の挙動は線形である。

4)覆工の慣性力の影響を無視する。

5)土被りがトンネル口径以上あり、かつ、トンネル底点が基盤面に接近していない。

6)地震時地盤応力のせん断強度での頭打ちは考慮しない。

7)地震時地盤ばねは、無限弾性体中の円孔表面に載荷した場合の力と変位の関係に基づいて、以下のように算出する。すなわち、図－18に示すように

均質な無限弾性体中の円孔を考え、この円孔表面に載荷したときの荷重と変位の関係を弾性論によって導いて地震時地盤ばねをモデル化する。荷重とそれによって生じる変位との関係は、それらを円周方向にフーリエ級数展開したときのモードごとに定式化され、地震時地盤ばね定数は式(40)及び式(41)のように与えられる。

$$k_n=\frac{2G_D}{R}\cdot C_n \tag{40}$$

$$k_n'=\frac{2G_D}{R}\cdot C_n' \tag{41}$$

$$C_n=\begin{cases}\frac{1}{2} & :n=0\\ \frac{1}{2} & :n=1\\ \frac{2n+1-2\nu_D(n+1)}{3-4\nu_D} & :n\geqq 2\end{cases} \tag{42}$$

$$C_n'=\begin{cases}0 & :n=0,1\\ \frac{n+2-2\nu_D(n+1)}{3-4\nu_D} & :n\geqq 2\end{cases} \tag{43}$$

ここに、

k_n :法線方向（接線方向）の荷重または変位に対して法線方向（接線方向）に作用する地震時ばね定数(tf/m³)

k_n':法線方向（接線方向）の荷重または変位に対して接線方向（法線方向）に作用する地震時ばね定数(tf/m³)

G_D :表層地盤の動的せん断弾性係数(tf/m²)

R :覆工の図心半径(m)

ν_D:表層地盤の動的ポアソン比

n :フーリエ次数で表したトンネルの変形モード

覆工に生じる地震時増分断面力は、原則として、式(42)～(44)によって算出する。

$$M=\beta\cdot\frac{3\pi\cdot EI}{2R\cdot H}\cdot U_h\cdot\sin\left(\frac{\pi\cdot H_0}{2H}\right)\cdot C\cdot\sin 2\theta \tag{44}$$

$$Q=-\beta\cdot\frac{3\pi\cdot EI}{R^2\cdot H}\cdot U_h\cdot\sin\left(\frac{\pi\cdot H_0}{2H}\right)\cdot C\cdot\cos 2\theta \tag{45}$$

$$N=-\beta\cdot\frac{3\pi\cdot EI}{R^2\cdot H}\cdot U_h\cdot\sin\left(\frac{\pi\cdot H_0}{2H}\right)\cdot\left(1+\frac{G_D\cdot R^3}{6EI}\right)\cdot C\cdot\sin 2\theta \tag{46}$$

$$C=\frac{4(1-\nu_D)\cdot G_D\cdot R^3}{(3-2\nu_D)\cdot G_D\cdot R^3+6(3-4\nu_D)\cdot EI} \tag{47}$$

ここに、

M:単位延長当りの覆工に生じる曲げモーメント(tf·m/m)

Q:単位延長当りの覆工に生じるせん断力(tf/m)

N:単位延長当りの覆工に生じる軸力(tf/m)

θ :覆工頂点からの角度(rad)

H:表層地盤の層厚(m)

H_0:トンネル中心の深度(m)

R:覆工の図心半径(m)

EI:覆工の曲げ剛性(tf·m²/m)(単位延長当り)

G_D:表層地盤の動的せん断弾性係数(tf/m²)

ν_D:表層地盤の動的ポアソン比

U_h:地表面での地震時水平地盤変位(m)で、式(3)より算出する。

β :計算式の精度を補う係数（1.3とする）

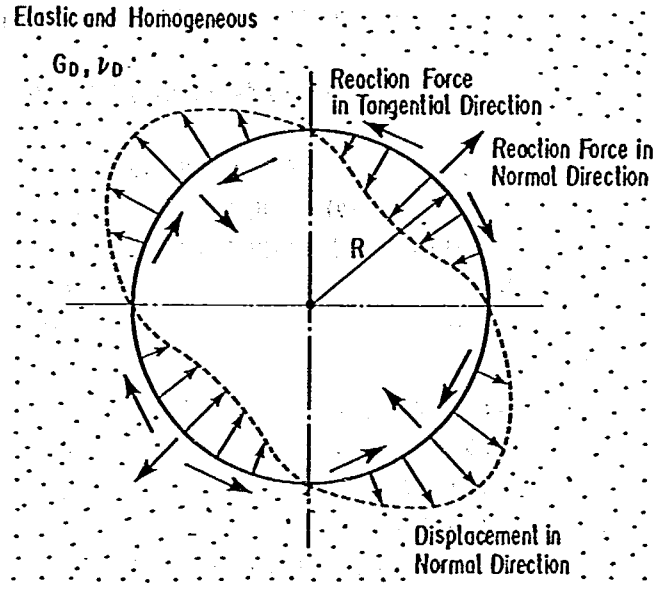


Fig.18 2nd Model Reformation of Fourier Serries assumed for Tunnel Ring

6 幅広地下構造物の耐震設計法

幅広地下構造物の耐震計算では、地震時の地盤のせん断変形の影響が支配的であり、したがって、地震時周面せん断力が地震外力として重要である。このため、幅広地下構造物の短手方向及び長手方向の両方向に対して、図－１９に示すように地震時周面せん断力、地震時土圧、慣性力を作用させて地震時増分断面力を算出する。

動的解析により安全性を照査するのが望ましい条件は、以下の通りである。

1)地盤条件が、深さ方向や水平方向に著しく異なり、このため式(3)に示したような地盤が水平方向の成層をなすと仮定して算出する地震時地盤変位が不十分な場合。また、地表面や基盤が大きく傾斜している

等、地盤の不整形性が著しい場合。このような場合には、図－２０に示すように幅広地下構造物と周辺地盤を２次元平面ひずみモデルでモデル化する場合が多い。

2)幅が非常に広く、幅広地下構造物全体が板のように面外に変形する影響が無視できない場合。また、地盤条件が平面的に著しく変化する場合。このような場合には、図－２１に示すように、深さ方向には表層地盤を１層もしくは２層程度に離散化するモデル化が有効である。幅広地下構造物は、解析目的に応じて板要素もしくは３次元のシェル要素として、モデル化する。

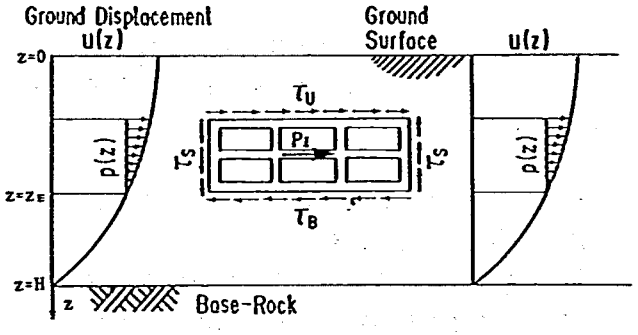


Fig.19 Shear Stress around Surface, Dynamic Earth Pressure and Inertia Force for Seismic Design of Laterally Wide Underground Structures

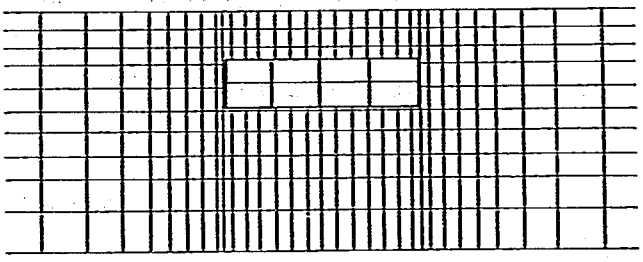


Fig.20 Two Dimensional Finite Element Idealization

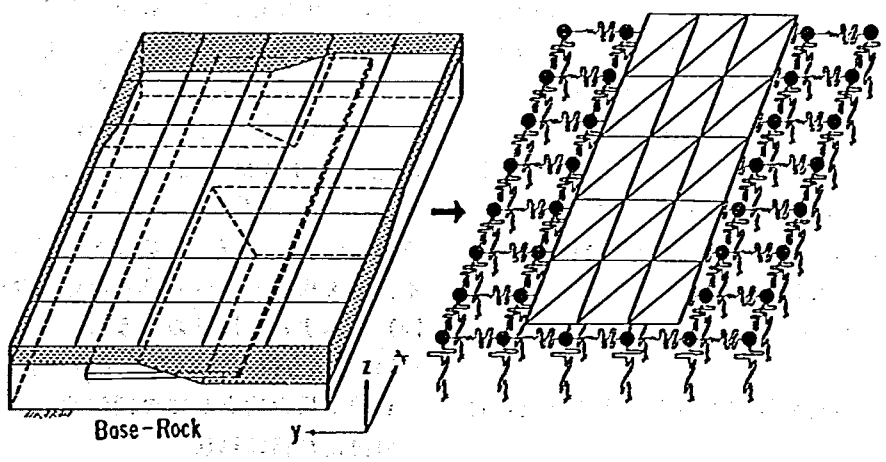


Fig.21 Idealization by Plate Elements and Shell Elements

7　鉛直地下構造物の耐震設計法

鉛直地下構造物では、地震時地盤変位及び地震時周面せん断力の影響が支配的となるため、これらの深さ方向の分布を正確に評価して耐震計算しなければならない。図－２２は、地震荷重の作用のさせ方を示したものである。用途によって異なるが、平面的にも大きい鉛直地下構造の中には平面内の変形も含めて３次元的に変形するものもある。このような場合には、シェルもしくはソリッド要素を用いて鉛直地下構造物をモデル化しなければならない。

8　地震防災システム

8－1　一般

地下道路の建設では構造的な耐震性を確保するために耐震設計を行うが、このようにしても地震に対して不安を感じる道路利用者が存在する。道路利用者が漠然とした不安を持っているところに地震が発生すると、わずかな不安が大きく増幅されて思わぬ二次災害が発生する危険性もある。このため、耐震設計による構造的な耐震性の確保が基本であることはもち論であるが、これに加えて地下道路利用者が抱きやすい地震に対する不安感を可能な限り低減し、いざという場合に二次災害が発生しないようにしておく必要がある。

8－2　地震時に地下道路利用者が抱きやすい不安感

ＶＥ（Value Engineering）法を用いて、道路利用者が「地下道路」と「地震」を合わせてと聞くと抱きやすい不安感にはどのようなものがあるかを示すと、地下道路に対して日常から抱いている不安感と、地震時に発生する可能性があると考える事象に対し

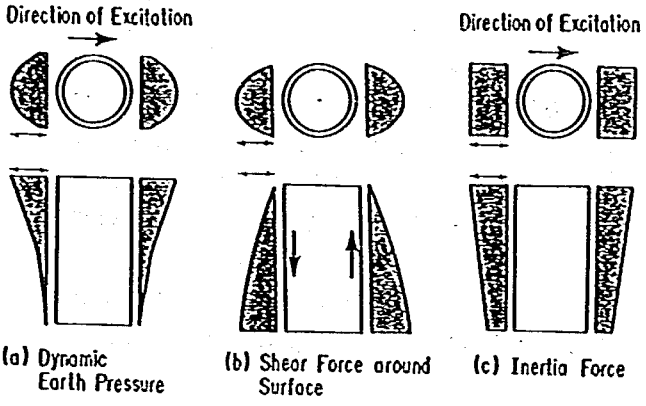


Fig.22 Seismic Effects to Vertically Long Underground Structures

て抱く不安感の２つがある。これらの大部分は、専門家からみれば単なるき憂に過ぎないものが多いが、「地下」と「地震」と聞くとどのような状態を道路利用者が連想するかを正しく知っておくことが重要である。

(1)日常から抱いている不安感

a)潜在的な不安感

地下道路に対して漠然と抱いている不安で、地上の道路と比較して、暗いのではないか、狭いのではないか、汚いのではないか、といったものがある。また、地下道路の運転に慣れていない、地下道路は親しみにくいのではないか、時間感覚や方向感覚を用いにくいのではないかといったものもある。

b)車両走行に係わる不安感

地下道路を走行することに対して抱いている不安で、例えば、地上の道路と比較すると、地下では路線や位置がわかりにくいのではないか、視界や交通マナーも悪いのではないかといったものである。

c)環境に係わる不安感

地上の道路と比較して地下道路の環境整備が十分でないのではないかといったことに対して抱いている不安で、例えば、付帯施設やトイレを含む生活施設の整備が不十分ではないのか、他施設と連絡しにくいのではないか、といったものである。また、環境の変化が乏しく、環境の管理が不十分ではないか、といったものもある。さらに、地上の道路に比較すると利用者が少ないのではないかということから不安を感じるという場合もある。

(2)地震時に発生する可能性があると考える事象に対して抱く不安感

a)被害に係わる不安感

構造物本体や内部施設、あるいは、車両等の被災により、自分に危険が及ぶのではないかという不安である。例えば、地震被害にともない地下道路に浸水があったり、自動車事故によって火災が発生するのではないかというものである。パニックが発生したり、停電したり、被害を目撃することも不安感に含まれる。

b)地震時の対応に係わる不安感

緊急時の対応方法がわからないことに対する不安である。例えば、防災設備がどこにあるのか、その使用方法がわからない、災害に遭った経験がない、といったものである。

c)避難に係わる不安感

緊急時にうまく避難できないかもしれないことに対する不安である。例えば、避難路や避難場所がわからないのではないか、避難に時間がかかるのではないか、避難誘導が十分でないのではないか、人が殺到するのではないか、といったものである。

d)被災・避難情報に係わる不安感

緊急時に必要な情報が得られないのではないかということに対する不安である。例えば、地震規模やこれによる地上の被災状況が地下ではわからないのではないか、情報が得られてもその信頼性が低いのではないか、といったものである。

e)交通事故に係わる不安感

これは、地下道路に特有の不安ではないが、地震にともなって車両が蛇行[11]したり、急ブレーキをかけたりして、交通事故が発生するのではないかという不安である。

8－3　地震時に地下道路利用者に不安感を与える要因

地下道路利用者がなぜ地震に対して不安感を覚えるかの理由には様々なものがある。また、ある不安感に対して複数の理由があったり、お互いに関連しあっていたりする。このような関係の関連付けは非常に難しいが、ＦＳＭ（Fuzzy Structural Modeling）法及びＩＳＭ（Interpretive Structural Modeling）法で解析した一例が、図－２３、図－２４である。このような解析に基づき地下道路利用者が抱く不安感を構成する最も最も根源的な要因（最下層要因）を示すと、以下のようになる。

(1)日常から抱いている不安感を生じさせる要因

a)地下構造物としての閉鎖性

b)複雑な路線形状

c)歩行者の混在

d)不十分な交通教育
e)不十分な環境整備
f)他施設との連絡困難

(2)地震時に発生する可能性があると考える事象に対して抱く不安感を生じさせる要因
a)付帯施設の耐災性

利用者が地震に対して抱く不安感を低減するためには、以下の方策を検討することが有効である。
(1)地上環境の模擬
地下であることを感じさせない空間造りが基本である。空間を大きく取って圧迫感を小さくする事が基本である。また、太陽光を取り入れたりする工夫も有効である。

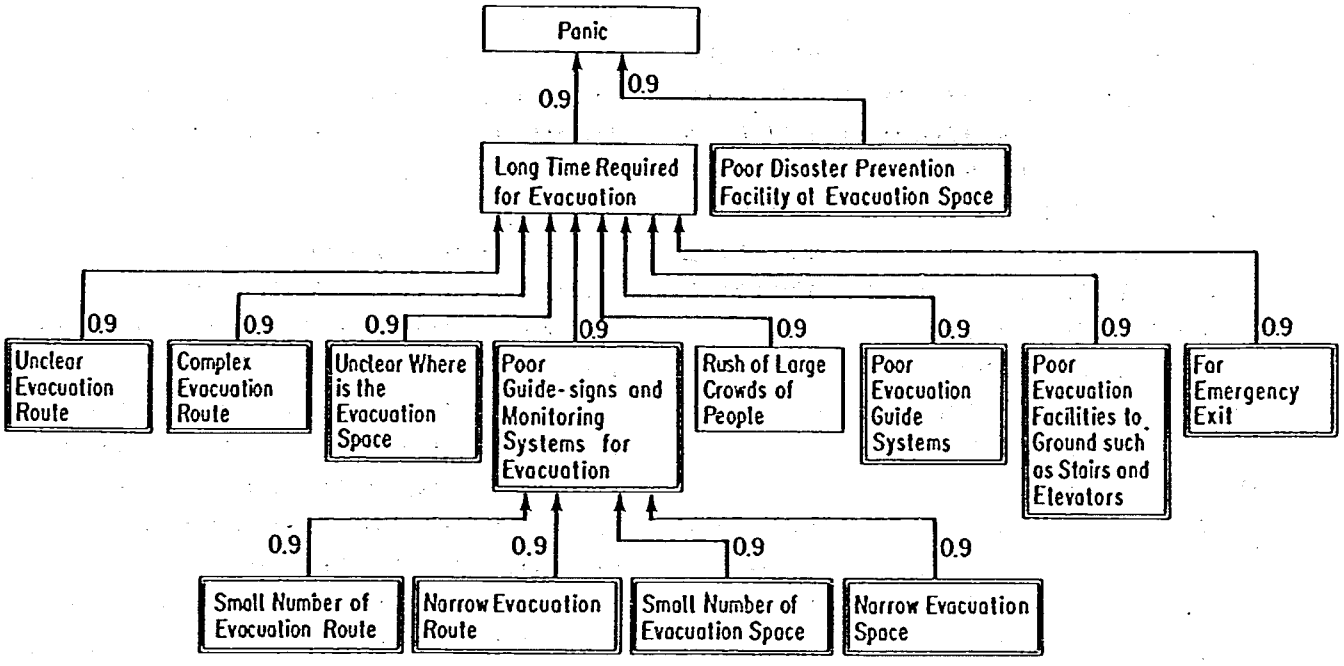


Fig.23 Structural Model predicted by FSM for Fear and Anxiety against Evacuation

b)車両の耐災性
c)震災の非日常性
d)不十分な避難設備
e)不十分な避難誘導体制
f)不十分な情報連絡体制
g)路上障害物の発生
h)トンネルの振動

8－4　地震防災システムの基本方策

地下道路利用者が抱く不安感を低減するためには、不安感を生む最下層要因を解消できる方策を実施することが基本的に最も有効である。ただし、最下層要因といっても、不安感を生む程度は異なるため、地震防災対策としてはそれらのうち支配的な要因に対する対策を講じる必要がある。

このため、8－3に示した最下層要因のうち支配的な要因をＡＨＰ（Analytical Hierarchy Process）法によって定量的に評価した結果に基づけば、道路

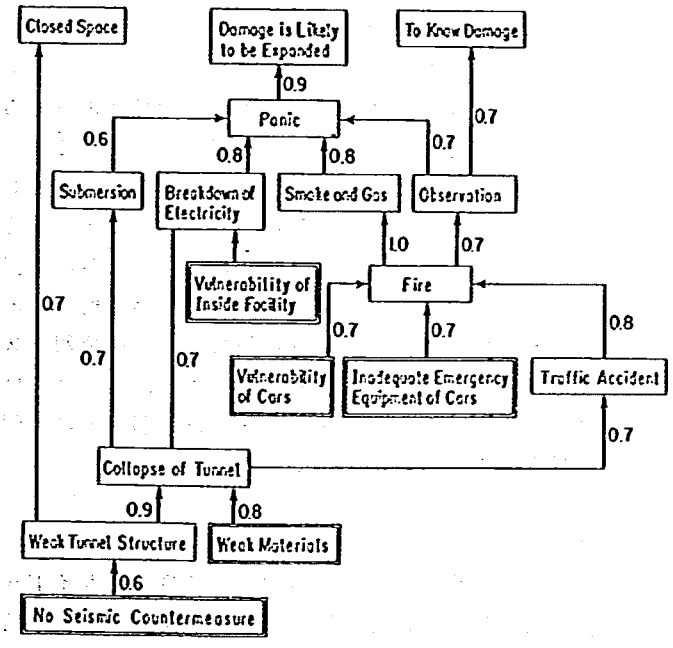


Fig.24 Structural Model Predicted by FSM for Possible Influence of Damage

(2)地下特性の積極的な利用

地下であることの良さを積極的に強調するものである。地下は地上に比較して振動しにくく、したがって、地上に比較して耐震的に優れていることを積極的にアピールすることが有効である。地震時には、むやみに地上に出ようとすることの方が危険であることを知らせていくことが有効である。

(3)防災教育、防災訓練の充実

不安感の中には、地下施設ではどこに防災施設があるのかわからないから始まって、防災施設全体に対する知識不足によるものが多い。したがって、地下道路の防災システムを分かりやすく解説したパンフレットの用意や、防災センターをガラス張りにし、内部が見えるような工夫が有効である。一般に、防災センターは目立たない場所に設置されている場合が多いが、積極的のこうしたセンターの役割と活動を紹介することが有効である。

(4)交通制御システムの充実

地震発生と同時に地下道路への流入を規制するシステムの整備が有効である。

(5)非常用設備の整備

いざという場合に地震後に確実に利用できる地上へのアクセスの確保を図ることが有効である。ただし、(2)とも関連するが、地上へ脱出が最善とは言えない場合がある。むしろ地下に安全な防災空間として強固な防災シェルターを設け、ここに避難しさえすれば、万全なシステムが整えられているようにすることも有効と考えられる。

(6)情報連絡体制の整備

地下道路内で、どのような地震が発生し、地上ではどのような状況になっているかを画像により知らせることができるシステムの設置が有効である。地上のよく知られている街路等の町並みをモニターしておけば、地上の状況を常に知ることができ、安心感につながりやすい。文字や音声だけによる情報提供は、情報の量が少なく、かえって道路利用者の不安感を増幅することになりかねない。画像情報システムは、２４時間利用することにより、常時においては地上の状況をモニターでき、道路利用者に時間感覚を持たせるのに有効である。

9　まとめ

以上、「大規模地下構造物の耐震設計法ガイドライン（案）」の概要を紹介した。今後、地上の土地利用や各種の規制によっては、道路、駐車場等にも大規模に地下利用を進めなければならないことが予想される。本ガイドライン（案）が、こうした構造物の耐震設計に役立てば幸いである。

最後に、本ガイドライン（案）の取りまとめに際しては、共同研究に参加した関係各位の多大なご協力を賜った。また、研究の実施に際しては、京都大学防災研究所の土岐憲三教授のご指導を賜った。ここに記して厚くお礼申し上げる次第である。

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Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States—An Overview

by

William S. Bivins

ABSTRACT

The Federal Emergency Management Agency (FEMA) initiated a project to assess the seismic vulnerability and impact of disruption of lifeline systems nationwide. The purpose of the project was to develop a better understanding of the impact of disruption of lifelines from earthquakes and to assist in the identification and prioritization of hazard mitigation measures and policies. FEMA plans to utilize results from the project to promote national awareness of the importance of protecting lifeline systems from earthquakes, and assuring reliability and continued serviceability of lifelines. The project was undertaken by the Applied Technology Council (ATC) and was conducted in several phases. Phase I, which provides the bases for this paper, provides a national overview of lifeline seismic vulnerability and impact of disruption. Lifelines considered include electric systems, water systems, transportation systems, gas and liquid fuel supply systems, and emergency service facilities. The vulnerability estimates and impacts developed are presented in terms of estimated direct damage losses and indirect economic losses. The losses are considered to represent *a first approximation* because of the assumptions and methodology utilized, because several lifelines are not included, and because, in some cases, the available lifeline inventory data lack critical information. This paper is an overview of the project drawn from the full report, *Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States — ATC-25*, Applied Technology Council, (ATC, 1991).

KEYWORDS: lifelines, earthquake, damage, impacts

1. INTRODUCTION

Lifeline is a term denoting those systems necessary for contemporary life and urban function, without which large urban regions cannot exist. Lifelines basically convey food, water, fuel, energy, information, and other material necessary for modern human existence from the production areas to the consuming areas. Also included are, of course, the transportation of the modern human.

With the advent of more and more advanced technology, the United States has increasingly become dependent on the reliable supply of lifeline-related commodities, such as electric power, fuel, and water. Earthquakes are probably the most likely natural disaster that would lead to major lifeline disruption. Extended disruption of lifelines serving a city or major urban region would lead to major economic losses, deteriorated public health and probable population migration. It clearly became necessary to identify the potential for major disruption to these lifelines, especially at the regional level.

FEMA initiated the study to better understand the impact of disruption of lifelines from earthquakes and to assist in the identification and prioritization of hazard mitigation measures and policies. The study report, Rojahn (1991), from which this paper is extracted, will improve national awareness of the importance of protecting lifeline systems from earthquakes, and of assuring lifeline reliability and continued serviceability.

Lifelines of critical importance at the U.S. national level have been analyzed to estimate overall seismic vulnerability and to identify those lifelines having the greatest economic impact, given large credible earthquakes. The lifelines examined include electric systems; water, gas and oil pipelines; highways and bridges; airports; railroads; ports; and emergency service facilities. The vulnerability estimates and impacts developed are presented in terms of estimated direct damage losses and indirect economic losses. These losses are considered to *represent a first approximation* because of the assumptions and

[1]Federal Emergency Management Agency, Washington, DC 20472

methodology utilized, because several lifelines are not included, and because, in some cases, the available lifeline inventory data lack critical capacity information.

Four basic steps were followed to estimate lifeline damage and subsequent economic disruption for given earthquake scenarios. The steps were:

- Develop a national lifeline inventory database,
- Develop seismic vulnerability functions for each lifeline component/system,
- Characterize and quantify the seismic hazard nationwide, and
- Develop direct damage estimates and indirect economic loss estimates for each scenario earthquake.

Several problems were encountered that could not be resolved because of technical difficulties and lack of available data. For example, telecommunication systems, nuclear and fossil-fuel power plants, dams, and certain water, electric, and transportation facility types at the regional transmission level were excluded from consideration in this project because of the unavailability of inventory data or the need for more in-depth studies.

Interaction effects between lifelines, secondary economic effects, and damage resulting from landslide were also not considered in developing the project report. These, and other limitations, tend to overestimate the losses. Lack of capacity information for most lifelines was also a definite limitation. In aggregate, due primarily to the exclusion of certain systems, the estimates of losses are believed to be quite conservative.

2. NATIONAL LIFELINE INVENTORY

Development of the inventory for all major lifelines in the United States was a major task. Sufficient detail was needed for lifeline seismic vulnerability assessment and impact of disruption at the national level. This required that the inventory be compiled electronically in digital form and dictated the inclusion of lifelines only at the transmission level.

The investigators initially contacted a number of government, utility, trade and professional organizations, and individuals to identify nationwide databases, especially electronic databases. In most cases, these organizations referred the project back to FEMA, since they had either previously furnished the information to FEMA, or know that the data had been furnished to FEMA by others. FEMA's database became a major source of data for several of the lifelines. Unfortunately, much of the inventory data generally date from about 1966.

The network inventory contained in the database is generally at the higher transmission levels, as opposed to lower distribution levels. As an analogy, the inventory contains only the national *arterial* level, and neglects the *capillary* system. For example, all federal and state highways are inventoried, but county and local roads are not.

2.1 Inventory Overview. The inventory data have been compiled into an electronic database consisting of:

- digitized location and type of facility for single-site lifeline facilities (e.g., ports), and
- digitized right-of-way and limited information on facility attributes for network lifelines (e.g., longitude and latitude of each end of a pipeline segment with diameter).

The inventory is only partial, in that important information on a number of facility attributes (e.g., number or length of spans for highway bridges) was unavailable in the FEMA database.

The inventory data include information only on the conterminous United States because lifelines in other regions would not be affected by the scenario earthquakes considered in this study.

2.2 Inventory Summary. The specific lifelines that have been inventoried are:

- Transportation
  - Highways (489,890 km of highways; 144,785 bridges)
  - Railroads (270,611 km of right-of-way)
  - Airports (17,161 civil and general aviation airports)

- Energy
  - Electric Power Transmission (4,551 substations; 441,981 km of transmission lines)
  - Gas and Liquid Fuel Transmission (77,109 km of crude oil pipelines; 85,461 km of refined oil pipelines; 67,898 km of natural gas pipelines)
- Emergency Service Facilities
  - Emergency Broadcast Facilities (29,586 stations)
  - Hospitals (6,973 medical care centers)
- Water Aqueducts and Supply (3,575 km of aqueduct, excluding Utah for which data were unavailable).

Telecommunication, an important lifeline, was excluded because of the unavailability of data. In addition, data on nuclear reactors and dams are excluded because it was believed that such facilities should be the subject of special studies because of the existing regulations relating to seismic safety in many regions and the expected complexity of the performance and impact of these facility types. Thus the losses established by this study will be underestimated to the extent that these facility types are not included.

Also excluded from the inventory, but included in the analysis, are distribution systems at the local level. The number of facility types (water, highway, and electrical systems and police and fire stations) with in each 25-km by 25-km grid cell was estimated on the basis of proxy by population.

3. LIFELINE VULNERABILITY FUNCTIONS

The second step in the project was the development of lifeline vulnerability functions. These functions which describe the expected or assumed earthquake performance characteristics of each lifeline as well as the time required to restore damaged facilities to their pre-earthquake capacity. The development of the vulnerability function for each lifeline considered are too lengthy to detail here but consisted of the following components:

- General information, which comprises (1) a *description* of the structure and its main components, (2) *typical seismic damage* in qualitative terms, and (3) *seismically resistant design* characteristics for the facility and its components. This information was included to define the assumed characteristics and expected performance of each facility and to make the functions more widely applicable.
- Direct damage information, which consists of (1) a description of its basis in terms of structure type and quality of construction, (2) default estimates of the quality of construction for present conditions and corresponding *motion damage curves*, (3) default estimates of the quality of construction for upgraded conditions, and (4) *restoration curves*.

These functions reflect the general consensus among practicing structural engineers that, with few exceptions, only California and portions of Alaska and the Puget Sound region have had seismic requirements incorporated into the design of local facilities for any significant period of time. For all other areas of the United States, present facilities are assumed to have seismic resistance less than or equal to that of equivalent facilities in California.

The two key quantitative vulnerability-function relationships developed under this project—*motion-damage curves* and *restoration curves*—define expected lifeline performance for each of the study regions and for the heart of the quantitative vulnerability analysis. The curves are based on the data and methodology developed on the basis of expert opinion in the ATC-13 project (ATC 1985). The data and methodology of ATC-13 are applicable for California structures only and the data were revised and reformatted to reflect differences in seismic design and construction practices nationwide.

Motion-damage curves developed define estimated lifeline direct damage as a function of seismic intensity (i.e., Modified Mercalli Intensity); direct damage is estimated in terms of repair costs expressed as a fraction or percentage of value. Curves are provided for each study region.

Restoration curves developed for this project define the fraction of initial capacity of the lifeline as a function of elapsed time since the earthquake. Again, curves are defined for each region.

4. SEISMIC HAZARD

Seismic hazard, as used in the study, is the expectation of earthquake effects. It is usually defined in terms of ground shaking parameters (e.g., peak ground acceleration, Modified Mercalli Intensity, peak ground velocity) but, broadly speaking, can include fault rupture, ground failure, or other phenomena resulting from an earthquake. Seismic hazard is a function of the size, or magnitude of an earthquake, distance from the earthquake, local soils, and other factors, and is independent of the buildings or other items of value that could be damaged.

The technical approach for evaluating the seismic hazard of lifeline structures are defined in detail. The approach involves identifying (1) the most appropriate descriptor (parameters) for describing the seismic hazard, (2) regions of high seismic activity, (3) representative potentially damaging, or catastrophic, earthquakes within each of these regions that could be used as scenario events for the investigation of lifeline loss estimation and disruption, and (4) a model for estimating the seismic hazard for each of these scenario events.

Following a review of available parameters for characterizing seismic hazard, the Modified Mercalli Intensity (MMI) Scale was elected for use. The MMI scale is subjective; it is dependent on personal interpretations and is affected by the quality of construction in the affected area. Even though it has these limitations, it is still useful as a general descriptor of damage, especially at the regional level.

The study presents an overview of the seismicity of the United States and representative earthquakes that were selected for the study.

In order to estimate the seismic hazard of the scenario events over the affected area, a model of earthquake magnitude, attenuation, and local site effects is required. For this study the Evernden model (Evernden and Thomson-1985) was selected. The advantages included: (1) verification via comparison with historical events, (2) incorporation of local soil effects and ready availability of a nationwide geologic database, and (3) ready availability of closed-form attenuation relations. An important additional attribute was that the Evernden model would estimate the distribution and intensity of seismic shaking in terms of MMI, the shaking characterization used in the ATC-13 study and the basic parameter, in this study, for lifeline vulnerability functions.

Based on the review of representative earthquakes scenario events were selected for this investigation. These events, presented by Region, Event Name, and Magnitude, are:

- Northeastern, Cape Ann, 7
- Southeastern, Charleston, 7.5
- Central, New Madrid, 7 and 8
- Western Mountain, Wasatch Front, 7.5
- Northwestern, Puget Sound, 7.5
- Southern California, Fort Tejon, 8
- Northern California, Hayward, 7.5.

With the exception of the Cape Ann, Charleston, and Hayward events, all magnitudes are reflective of the representative earthquake for the region. The scenario events for Cape Ann, Charleston, and Hayward have magnitudes one-half unit higher than the representative event. These magnitudes are interpreted as maximum credible for these locations.

5. ESTIMATES OF DIRECT DAMAGE

The analysis of seismic vulnerability of lifeline systems and the economic impact of disruption is based on an assessment of three factors:

- Seismic hazard,
- Lifeline inventory, and
- Vulnerability functions.

In this study these factors are used to quantify vulnerability and impact of disruption in terms of (1) direct damage and (2) economic losses resulting from direct damage and loss of function of damaged facilities.

Direct damage is defined as damage resulting directly from ground shaking or other collateral loss-causes such as liquefaction. For each facility, it is expressed in terms of cost of repair divided by replacement cost and varies from 0 to 1.0 (0% to 100%). It is estimated using (1) estimates of ground shaking intensity provided by the seismic hazard model, (2) inventory data specifying the location and type of facilities affected, and (3) vulnerability functions that

relate seismic intensity and site conditions to expected damage.

The approach to estimate direct damage considers both damage resulting from ground shaking as well as damage resulting from liquefaction. Damage due to other collateral loss causes, such landslide and fire, are not included because of the unavailability of inventory information and the lack of available models for estimating the losses nationwide.

The analysis approach for computing direct damage due to ground shaking proceeded as follows. For each earthquake scenario, MMI levels were assigned to each 25-km grid cell in the affected region, using the Evernden MMI model, assigned magnitude, and assigned fault rupture location. Damage states were then estimated for each affected lifeline component in each grid cell, using the *motion-damage curves.*

Dollar loss estimates are based on the available inventory data, cost per facility assumptions, and other models and assumptions with are described in the report. As a result, the accuracy of these estimates may vary from lifeline to lifeline. Data from the various lifelines are combined to estimate the total direct damage dollar losses for the eight scenario earthquakes as follows:

Earthquake	Direct Dollar Loss (in Billions, 1991)
Cape Ann	$4.2
Charleston	$4.9
Fort Tejon	$4.9
Hayward	$4.6
New Madrid, M=8.0	$11.8
New Madrid, M=7.0	$3.4
Puget Sound	$4.4
Wasatch Front	$1.5

6. ESTIMATION OF INDIRECT ECONOMIC EFFECTS

Earthquakes produce both direct and indirect economic effects. The direct effects, such as loss due to collapsed structures, are obvious and dramatic. However, the indirect effects that these disruptions have on the ability of otherwise undamaged enterprises to conduct business may be quite significant. However, there is very little literature on indirect economic losses.

This study provides a first approximation of the indirect economic effects of lifeline interruption due to earthquakes. A methodology was developed to relate lifeline interruption estimates to economic effects of lifeline interruption in each economic sector. This required a two-step process: (1) development of estimates of interruption of lifelines as a result of direct damage and (2) development of estimates of economic loss as a result of lifeline interruption.

Lifeline interruption resulting from direct damage is quantified in the study in residual capacity plots that define percent of function restored as a function of time. The curves are estimated for each lifeline type and scenario earthquake using (1) the time-to-restoration curves, (2) estimates of ground shaking intensity provided by the seismic hazard model, and (3) inventory data specifying the location and type of facilities affected.

For site-specific systems (i.e., lifelines consisting of individual sited or point facilities) the time-to-restoration curves are used directly. Extended regional networks required special analyses procedures consisting of connectivity analyses and serviceability analyses.

The extensive analyses of the study yields the following estimates of total indict economic losses for the eight scenario earthquakes as follows:

Earthquake	Indirect Loss (in Billions, 1991)
Cape Ann	$9.1
Charleston	$10.2
Fort Tejon	$11.7
Hayward	$11.1
New Madrid, M=8.0	$14.6
New Madrid, M=7.0	$4.9
Puget Sound	$6.1
Wasatch Front	$3.9

7. HAZARD MITIGATION OF CRITICAL LIFELINES

Based on the combined direct and indirect economic losses presented in the study and with consideration

of the assumptions and limitations expressed throughout the report the following relative ranking of the criticality of different lifelines in terms of the estimated impact of damage and disruption:

Rank 1 . . . . . Electric Systems
Rank 2 . . . . . Highways
Rank 3 . . . . . Water Systems
Rank 4 . . . . . Ports
Rank 5 . . . . . Crude Oil.

The study also provides extensive recommendations on measures to reduce the vulnerability of lifeline systems. The recommendations fall into the following categories:

- Damage reduction measures (e.g., strengthening a building, densifying the soil beneath a structure, component improvements)
- Provision of system redundancy
- Operational improvements

9. RECOMMENDATIONS FOR FURTHER WORK

The ATC-25 project has raised a number of questions and indicated areas in which knowledge is inadequate or nonexistent with respect to the impact of lifeline disruption due to earthquake. Following are recommendations for further research and other efforts.

- Lifeline Inventory. Appropriate organizations are encouraged to develop standards for complete lifeline inventories, and coordinate the acquisition of the needed additional updated data from various lifeline owners.
- Lifeline Component Vulnerability. A major effort should be initiated to acquire data on lifeline seismic performance and damage, and conduct analysis towards the development of improved component vulnerability functions. This effort should also investigate lifeline recovery data, and incorporate the extensive experience realized during the October 17, 1989 Loma Prieta, California, earthquake, as well as from other damaging earthquakes.
- Seismic Hazard Data. The U.S. Geological Survey should develop a series of digitized soil/geologic databases.
- Economic Analysis and Impacts Data and Methodology. Further research is recommended in economic areas such as:
  - Economic impacts associated with lifeline disruption,
  - Second-order economic effects (e.g., interaction between lifelines),
  - Elasticities of demand, or substitution of lesser disrupted lifeline for a more disrupted lifeline,
  - Inter-regional impacts, and
  - So-called "benefits," such as increased economic activity associated with repair or replacement of older equipment with new technology.

It should be noted that this study did not address the environmental consequences associated with lifeline disruption, especially the potential for oil spills from broken pipelines in the nations' waterways following, for example, a New Madrid event.

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Design Criteria for Base Isolation of Buildings

by

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ABSTRACT

Presented are the results an investigation in which the performance of "code-designed" fixed-base and base-isolated structures were compared.. Two frames, a steel concentrically braced frame and a steel special moment resisting frame were designed as both fixed-base and base-isolated in accordance with the 1991 Uniform Building Code. To establish comparable performance, the base-isolated frames were designed to varying fractions of the code recommended base shear: 100%, 50% and 25%. Non-linear time history analyses were conducted, for three ensembles of earthquakes, to assess the effect of design force level and base-fixity on response and damage level. In general, the base-isolated frames designed to 100% of the code recommended base shear out-performed the fixed-base frames. Based on average values of a number of response quantities, comparable performance could be achieved at design forces lower than currently specified by code for the base-isolated frames.

KEYWORDS: base-isolation; building code; dynamic response; relative performance; steel frame.

1. INTRODUCTION

This paper presents the results of an analytical investigation in which the performance of "code-designed" fixed-base and base-isolated structures are compared. The objective of the study is to determine approximately the design force level for a base-isolated structure which will result in performance comparable to that of a fixed-base structure.

Two simple planar frames, a steel concentrically-braced frame (SBF) and a steel special moment-resisting frame (SMF) have been designed using the static lateral load procedure, in accordance with the 1991 Uniform Building Code(UBC). The frames have been designed as both fixed-base and base-isolated. Design parameters for the frames have been selected to represent a large part of the building inventory: frames are 4-story, 3-bay structures, assumed to be located in Seismic Zone 4 (PGA=0.4g), and founded on dense or stiff soil (UBC soil profile S_2). The base-isolated buildings are assumed to be supported on laminated elastomeric isolation bearings, designed to provide an isolated period of 2.5 s, with equivalent viscous damping equal to 15% of critical.

To establish comparable performance, each base-isolated frame has been designed to 100%, 50% and 25% of the code recommended base shear. The response of the fixed-base design on the isolation system has also been considered (this is equivalent to designing an isolated structure to greater than 100% of the code recommended base shear). To facilitate reference to the various design cases, the following nomenclature has been adopted:

FRAME TYPE - DESIGN CASE

where FRAME TYPE is SBF or SMF, and DESIGN CASE is FB (fixed-base), BI.0 (fixed-base design on isolators), BI.1 (base-isolated, designed to 100% of the code recommended base shear), BI.2 (base-isolated, 50% of code) and BI.3 (base-isolated, 25% of code). Thus, SMF-BI.2 refers to the steel moment frame designed to 50% of the code recommended lateral force.

To evaluate the performance of each design, nonlinear time history analyses have been con-

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ducted, for 3 ensembles of earthquakes, using the computer program DRAIN-2D (Kanaan and Powell, 1973). A number of response quantities have been examined, including, the number of frames that exhibit yielding or collapse, the number of yielded superstructure elements, the maximum total and relative roof displacement of the superstructure, first story drift and the maximum displacement of the base-isolation bearing. Statistical analyses of these quantities have been conducted to determine the approximate design force level for the base-isolated structure which would provide performance comparable to that of the fixed-base structure.

2. FRAME DESIGNS

2.1 Building Configuration

The frames under consideration are assumed to be part of a hypothetical four-story office building, which is rectangular in plan, measuring nominally 45.7 m by 22.9 m with 7.62 m bays in both directions. Story heights are 3.66 m for the first story, and 3.35 m for the second, third, and fourth stories. The roof and floors are assumed to be rigid and to act as diaphragms in the horizontal direction. Alternating frames are designed to carry the lateral load; hence, the tributary width for lateral load is 15.3 m and for gravity load is 7.62 m. The plan and a typical section of the building are shown in Figure 1.

2.2 Dead and Live Load

Frames are designed for a roof dead load of 2.75 kPa and a floor dead load of 3.92 kPa. The corresponding live loads are 0.9 kPa and 2.40 kPa, respectively. The total seismic dead weight (W) is 5190 kN for the fixed base frames and 6570 kN for the base-isolated frames.

2.3 Base Shear, Fixed-Base Design

The design base shear, V, for a fixed-base frame is given by (UBC 1991),

$$V = \frac{ZIC}{R_w} W \qquad (1)$$

in which Z is the seismic zone factor, I is the importance factor, R_W is a reduction factor which is measure of the ability of a structure to sustain inelastic deformations without collapse, W is the seismic dead weight, and C is given by,

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad (2)$$

where S is the site soil coefficient, and T is the fundamental period of the structure, in seconds. T is approximated by

$$T = C_t (h_n)^{3/4} \qquad (3)$$

where h_n is the height of the building in feet and C_t = 0.02 for the braced frame and 0.035 for the moment frame.

For the fixed-base frames: Z=0.4, I=1, S=1.2, and R_W = 12 for the moment frame and R_W = 8 for the braced frame.

2.4 Base Shear, Base-Isolated Design

The design base shear for elements above the isolation plane is given by (UBC, 1991)

$$V = \frac{K_{max} D}{R_{WI}} \qquad (5)$$

in which D is the design displacement of the isolation system, K_{max} is the maximum effective stiffness of the isolation system at the design displacement D, as normally determined by experiment, and R_{WI} is similar to R_W, although not equal in value. The design displacement D, in inches, of the isolation system is given by

$$D = \frac{10\, Z\, N\, S_i\, T_i}{B} \qquad (6)$$

in which Z is the seismic zone factor; N is a coefficient related to the proximity of the building to the nearest active fault; S_i is the site soil coefficient for isolated structures; B is a coefficient related to the effective damping of the isolation system, and T_i is the fundamental period of the isolated structure. The fundamental

period of the isolated structure is related to K_{min}, the minimum effective stiffness of the isolation system, by

$$T_i = 2\pi\sqrt{\frac{W}{K_{min}g}} \qquad (6)$$

In this investigation, the isolation bearings are modeled as bi-linear hysteretic elements as shown in Fig. 2. The bi-linear model is fairly general and represents sufficiently well the experimentally observed cyclic load-deformation behavior for many types of bearings. The model is defined by initial stiffness K_1, post-yield stiffness K_2 and yield force F_1. Assuming yielding of the isolation system occurs at 5%W and the post-yield stiffness is equal to 10% of the pre-yield stiffness

$$K_2 = K_E - 0.045\frac{W}{D} \qquad (7)$$

where K_E is the effective stiffness of the isolation system at the design displacement D (Fig-.2). Note that with the assumed bi-linear model, $K_{max}=K_{min}=K_E$ (UBC, 1991).

For the base-isolated frames: Z=0.4, S_i=1.5, N=1, T_i=2.5 s, B=1.35, and R_{WI}=3 for both the braced frame and moment frame. This results in a design displacement for the isolation system of 280 mm.

2.5 Final Designs

A summary of the design specified or computed fundamental period and base shears for the various design cases are listed in Table 1. Note, under current guidelines, the moderate 43% reduction in base shear for the base-isolated braced frame, and the mere 14% reduction for the base-isolated moment frame.

Presented in Table 2 are member sizes for the braced frames, and in Table 3 for the special moment frames. All members are standard structural sections, per the American Institute of Steel Construction. Note, due to a limitation on minimum member sizes, no reduction in member size was achieved for the braced frame designed to 25% of the code recommended base shear, hence, there are only 3 distinct isolated designs for the concentric braced system (BI.0, BI.1 and BI.2).

3. EARTHQUAKE TIME HISTORIES

For the dynamic analysis, 18 recorded earthquakes have been selected from the original 30 used by Seed et al. (1976) to develop the S_2 design spectrum of the design provisions. The 18 have peak ground accelerations ranging from 0.11g to 0.28g, and peak ground velocities ranging from 120 mm/s to 360 mm/s. The original 18 form the "As-recorded" ensemble. To conform to the zone 4 design spectrum PGA of 0.4g, a second ensemble has been created by scaling each record to a peak ground acceleration of 0.4g. This is referred to as the "PGA" ensemble. The influence of velocity is examined by scaling each record to a peak velocity of 500 mm/sec, which is the average PGV of the ensemble after it has been scaled to 0.4g. This is referred to as the PGV ensemble.

4. METHOD OF ANALYSIS

Models were developed for the SBF and SMF for analysis using DRAIN-2D (Kanaan and Powell, 1973). In all cases, nodes are located at the geometric beam-column centerline intersection, seismic mass is lumped at the beam-column joint, joints are assumed to be infinitely rigid and a single element is used to model each story column and each girder span.

In all cases, beams and columns are modeled using a beam/column element with typical A36 steel properties (E=200 GPa, F_y=248 MPa) and nominal section properties. Beams and columns are assumed to exhibit an elastic-plastic moment curvature relation, with no hardening in the SBF, and 2% hardening in the SMF. Braces are modeled using buckling truss elements (Jain and Goel 1978) with typical A46 steel properties (E=200 GPa, F_y=317 MPa). Isolators are modeled using bilinear, elastic-plastic springs located at the base of each first story column. The stiffness of the individual isolators are a fraction of the overall isolation system properties, based on the proportion of dead load carried by each

column. A schematic of the DRAIN-2D model for the SBF is shown in Fig. 3, and for the SMF in Fig. 4.

In the time history analyses, P-Δ effects are considered approximately, the dynamic analysis is performed from the deformed static equilibrium position and original elastic stiffness proportional damping, corresponding to 2% in the fundamental mode, is assumed. Analyses have been conducted for the first 30 seconds of each strong motion record in the 3 ensembles, for the 9 design cases. A total of 486 time history analyses have been conducted.

5. RESULTS

Of the 3 ensembles used in the investigation, the PGA ensemble was the most "damaging"; only the results for the PGA ensemble are presented herein. A compilation of all the results may be found in Lin and Shenton (1992). A summary of response statistics for the steel braced frames are presented in Table 4, and for the steel moment frames in Table 5.

5.1 Braced Frame Designs

The "code-designed" base-isolated frame (BI.1) clearly out-performed the fixed-base frame (FB) in almost every response category, as noted in Table 4. This includes the number of frames which exhibit yield (a measure of the incidence of superstructure yielding for a given frame and ensemble), average number of yielded superstructure elements (a measure of the extent of superstructure yielding), time of first superstructure yield and average relative roof displacement. In only one response category did the SBF-BI.1 not perform better than the SBF-FB, that being first story drift ductility (defined as the first story drift divided by the story drift required to cause first yield in the brace). The average first story drift for SBF-BI.1 was 31 mm, while that for the SBF-FB was 28 mm, resulting in a comparable, but slightly larger drift ductility for the BI.1.

As would be expected, the absolute roof displacement for the isolated structures is greater than that of the fixed base design. The total roof displacement is also seen to be relatively insensitive to design force level. Note that the average relative roof displacement for SBF-BI.1 is roughly 1/3 the total roof displacement, indicating near rigid body behavior in the superstructure.

The results in Table 4 are presented in an alternate format in Fig. 5. Plotted in the figure is the isolated response quantity normalized by the fixed-base response quantity, versus the fraction of code recommended base shear. Comparable performance in this figure corresponds to an ordinate value of 1.0. It is obvious from this figure that the design force level required to provide comparable performance varies with performance criterion. The isolated design BI.2 out-performed the fixed-base design when based on incidence and extent of yield; a design force of approximately 85% of code would be required to provide comparable performance when based on relative roof displacement; the code-designed fixed-base frame performed slightly better than the code-designed base-isolated frame when based on drift ductility.

The average peak isolator displacement for the PGA ensemble was 224 mm, 173 mm and 138 mm, for SBF-BI.0, SBF-BI.1 and SBF-BI.2, respectively. In every case, the average is below the design displacement of 280 mm; however, the design displacement was exceeded in 10 of 54 analyses conducted for the isolated braced frame designs. The largest isolator displacement for SBF-BI.1 and the PGA ensemble was 538 mm, almost twice the design displacement.

5.2 Moment Frame Designs

The "code-designed" base-isolated moment frame (SMF-BI.1) out-performed the fixed-base frame (SMF-FB) in a number of categories; however, performance was only comparable in others. The SMF-BI.1 out-performed the SMF-FB when based on average number of yielded superstructure elements, relative roof displacement and first story drift ductility, as noted in Table 5. The two performed comparably when based on incidence of collapse (defined by an

extreme roof displacement and excessive number of yielded members) and extent of superstructure yielding.

It is interesting to note that in this case the average total roof displacement of SMF-BI.1 is slightly lower than that of the fixed-base design. Also, the average relative roof displacement for SMF-BI.1 is roughly 2/3 the total roof displacement. This, in conjunction with the incidence and extent of superstructure yielding for SMF-BI.1, indicates superstructure behavior which is far from rigid or purely elastic, a common assumption in the analysis of isolated structures.

The results in Table 5 are presented in the alternate normalized format in Fig. 6. In this case it is clear that comparable performance is achieved, for almost all response criteria with the PGA ensemble, with an isolated moment frame designed to between 50% and 75% of the code recommended base shear.

The average peak isolator displacement for the PGA ensemble was 150 mm, 139 mm, 100 mm and 76 mm, for SMF-BI.0, SBF-BI.1, SBF-BI.2 and SMF-BI.3, respectively. Again, the average is below the design displacement of 280 mm; however, the design displacement was exceeded in 9 of 72 analyses conducted for the isolated moment frame designs. The largest isolator displacement for SMF-BI.1 and the PGA ensemble was 394 mm, just 1.4 times the design displacement.

6. CONCLUSIONS

In this study the relative performance of "code-designed" fixed-base and base-isolated steel frames have been compared. Results presented indicate that, based on a number of performance criteria, code-designed base-isolated structures perform better than conventional fixed-base structures. Comparable performance can be achieved with an isolated structure designed to a lateral force level less then that required by the 1991 UBC provisions.

Comparable performance in a braced frame can be achieved with a design force level between 50% and 100% of the code recommended value: the actual level of design force varies with response criterion. Comparable performance in a moment frame can be achieved with a design force level between 50% and 100% of the code recommended value. A lateral force corresponding to 50% of the code specified value provides comparable performance in many cases.

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Table 4. Summary of Steel Braced Frame Results for PGA Ensemble

Design	# of frames that exhibit yield	Average # of Yielded Elements	Average Time of first yield (s)	Average Roof displacement with respect to ground (mm)	Average Roof displacement with respect to isolators (mm)	Average First floor ductility ratio
FB	18	7.3	5.1	106	106	2.2
BI.0	3	1.1	15.4	253	38	1.4
BI.1	6	2.4	9.0	257	89	2.4
BI.2	12	5.1	6.9	246	159	9.4

Table 5. Summary of Steel Moment Frame Results for PGA Ensemble

Design	# of frames that collapse	# of frames that exhibit yield	Average # of Yielded Elements	Average Time of first yield (s)	Average Roof displacement with respect to ground (mm)	Average Roof displacement with respect to isolators (mm)	Average First floor ductility ratio
FB	0	17	34.1	5.1	149	149	2.1
BI.0	0	14	13.1	7.6	129	78	1.0
BI.1	0	17	21.7	5.3	145	96	1.2
BI.2	2	18	33.2	6.1	175	137	2.6
BI.3	5	18	33.5	6.1	184	163	4.3

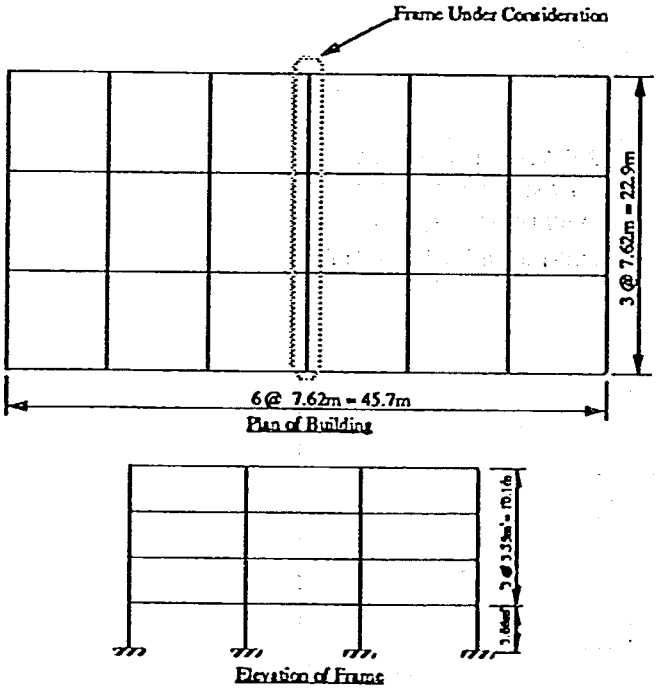


Figure 1. Building Plan and Frame Elevation

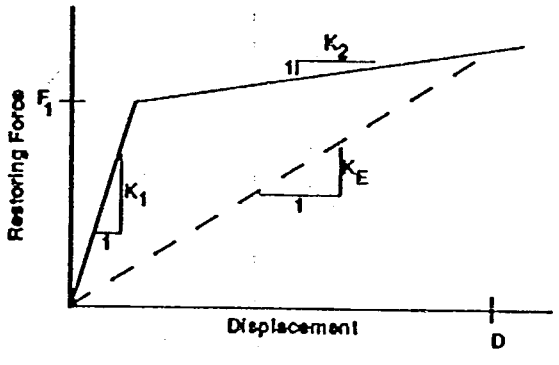


Figure 2. Isolation Bearing Bi-Linear Model

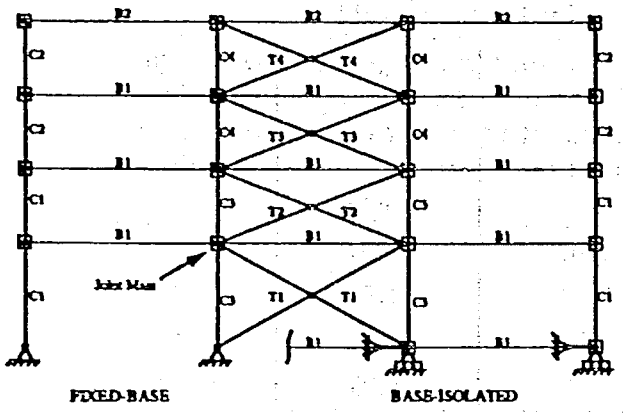


Figure 3. DRAIN-2D Model of Steel Concentric Braced Frame

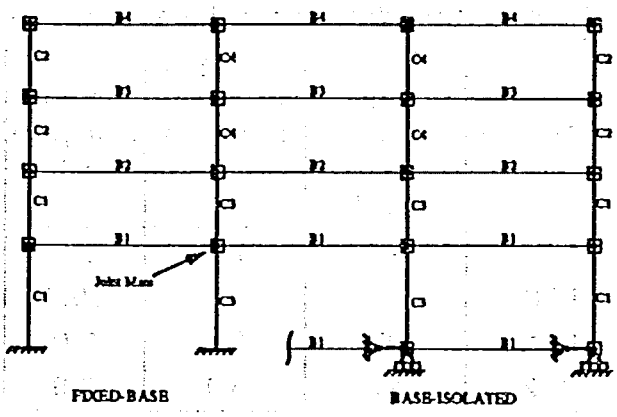


Figure 4. DRAIN-2D Model of Steel Moment Frame

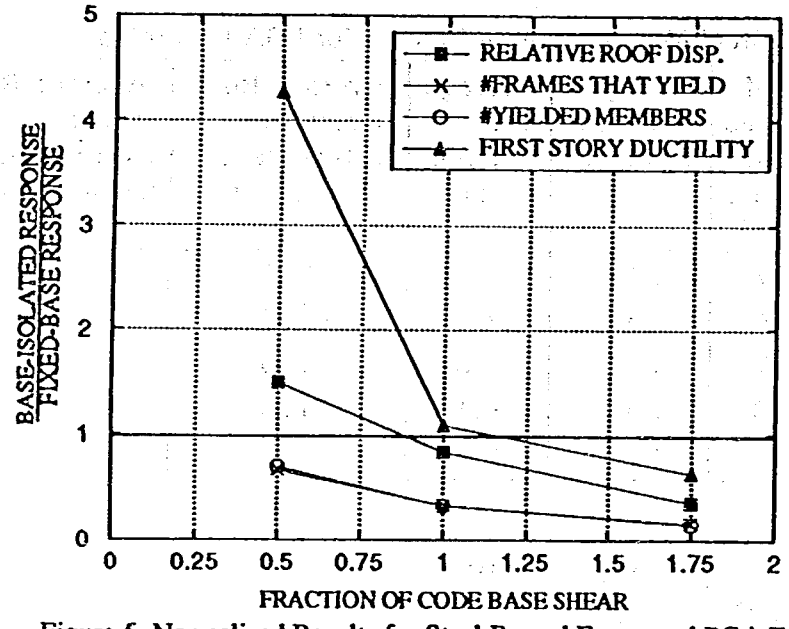


Figure 5. Normalized Results for Steel Braced Frame and PGA Ensemble

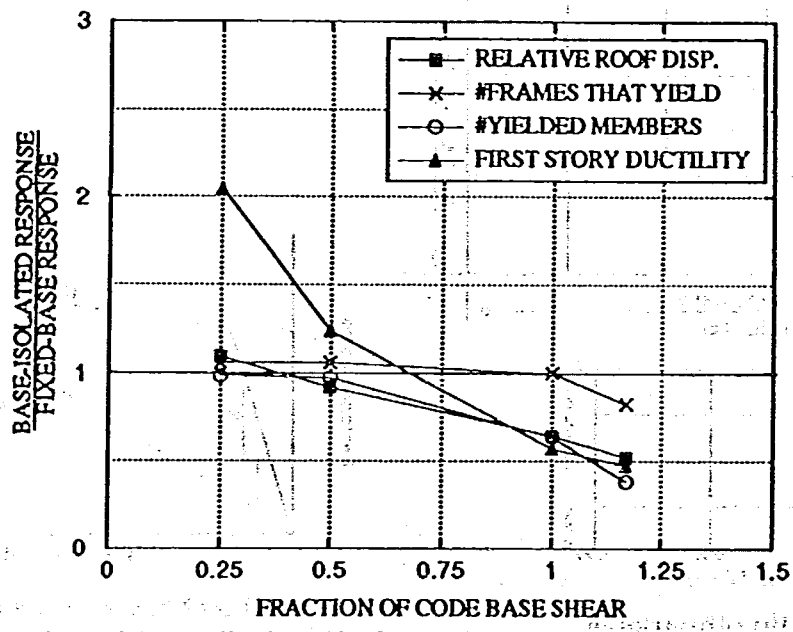


Figure 6. Normalized Results for Steel Moment Frame and PGA Ensemble

道路橋の免震設計法マニュアル（案）

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要　約

建設省土木研究所と民間２６グループ２８社は、平成元年度から平成３年度まで「道路橋の免震構造システムの開発に関する共同研究」を行った。３か年全体の研究成果を「道路橋の免震設計法マニュアル（案）」として取りまとめたので、その概要を報告する。

キーワード

免震設計、道路橋、共同研究、免震設計法マニュアル

１．まえがき

建設省土木研究所と民間２６グループ２８社は、平成元年度から平成３年度まで「道路橋の免震構造システムの開発に関する共同研究」を行った。これは、特殊な長大橋ではなく一般橋クラスの橋を対象として、長周期化とエネルギー吸収を基本とする免震設計システムを開発することを目的と実施したものである。共同研究では、表－１に示すように、１）道路橋への適用に優れた免震装置の開発、２）免震橋に適用できる落橋防止装置及び伸縮装置の開発、３）簡便で合理的な免震設計法の開発、４）免震設計の適用性に優れた橋梁タイプの検討の４項目を開発目標とした。

１）の免震装置の開発では、図－１に示すように１０種類の免震支承を開発した。２）の落橋防止装置・伸縮装置の開発では、写真－１に示すノックオフ橋台の設計法を検討するとともに、写真－２に示す橋軸直角方向にも移動可能な伸縮装置を開発した。３）の免震設計法の開発の中では、大型模型を用いた免震橋の模型振動実験を行い、写真－３に示す免震橋の振動特性を検討するとともに、解析方法の検証を行うことのできる実験データの提供を行った。４）の免震設計の適用性の検討では、橋長１ｋｍ級の超多径間連続橋の建設が可能となること、免震設計を用いた既存橋の耐震補強、桁連結等を検討した。写真－４は、既存橋の桁連結に免震支承を用いた例である。

毎年の検討内容は共同研究報告書としてまとめられているが、３か年全体の研究成果を「道路橋の免震設計法マニュアル（案）」として取りまとめたので、その概要を報告する。

２．マニュアル（案）の性格と構成

このマニュアル（案）は、免震装置を用いることにより、道路橋示方書Ⅴ耐震設計編の「８章　地震の影響の低減を期待する構造」に規定される「慣性力の低減を期待する構造」を採用する場合の耐震設計法及び構造細目を示すものである。また、既存橋の耐震補強や桁連結による多径間連続化についても示している。ただし、本マニュアル（案）は、共同研究の成果をマニュアルふうにまとめたもので、行政上の示方書や指針ではなく、単なる研究成果である。

マニュアル（案）の構成は、以下に示す通りである。

１．総則
　１．１　適用の範囲
　１．２　用語の定義
２．免震設計の基本方針
３．免震設計
　３．１　一般
　３．２　震度法に用いる設計水平震度
　３．３　地震時保有水平耐力法に用いる設計水平震度及び等価設計水平震度
　３．４　免震装置の特性値
　３．５　橋の固有周期の算出方法
　３．６　橋の減衰定数の算出方法
４．免震装置の設計
　４．１　設計の基本方針
　４．２　積層ゴム支承の設計
　４．３　鉛プラグ入り積層ゴム支承の設計
　４．４　高減衰積層ゴム支承の設計
　４．５　鋼製ダンパーの設計
　４．６　摩擦ダンパーの設計
　４．７　粘性ダンパーの設計
　４．８　ローラー型免震支承の設計
５．動的解析による照査
　５．１　一般
　５．２　動的解析手法
　５．３　解析モデル
　５．４　動的解析に用いる地震入力
　５．５　安全性の照査
６．免震装置を用いる場合の構造細目
　６．１　一般
　６．２　主要構造物間の遊間
　６．３　落橋防止構造
　６．４　ノックオフ装置
　６．５　伸縮装置の構造細目
７．免震装置の性能確認試験方法
　７．１　一般
　７．２　免震装置の動的特性
　７．３　免震装置の静的特性
８．免震設計を用いた耐震補強
９．免震設計を用いた単純桁の連続化

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３）建設省関東地方建設局企画部技術調査課

参考資料－1　道路橋の免震設計の手順
参考資料－2　道路橋用の免震支承
参考資料－3　動的解析に係わる検討例
参考資料－4　免震橋架用伸縮装置・落橋防止構造
参考資料－5　ＲＣ橋に対する免震設計の適用性
参考資料－6　鋼橋に対する免震設計の適用性
参考資料－7　超多径間連続橋に対する免震設計の適用性
参考資料－9　耐震補強に対する免震設計の適用性
参考資料－8　単純桁多径間連続橋に対する適用性
参考資料－10　時刻歴応答解析（地震時保有水平耐力法）に用いる標準地震入力例

3．免震設計の基本方針

免震設計の採用の適否は、橋の固有周期およびエネルギー吸収性能を増大させる効果を常時及び地震時の両面から検討した上で判断しなければならない。　免震設計が適している橋の条件は、一般に以下の通りである。

1)地盤が健硬で、基礎周辺地盤が地震時に安定している場合

2)下部構造の剛性が高く、橋の固有周期が特に長くない場合

これらは、いずれも橋を長周期化することにより橋に作用する地震力の低減を期待しやすい条件である。下部構造の剛性の目安としては、免震装置を用いた場合の橋の固有周期が、免震装置を用いない場合の橋の固有周期の1.5倍程度以上になる場合が望ましい。免震化された上部構造の固有周期と下部構造の固有周期がこの程度に離れていれば、上部構造と下部構造の連成振動の影響は小さく、橋に生じる変位は主として免震装置に集中する。

道路橋に免震設計を適用する際に、一番問題となるのはどの程度まで長周期化するかという点である。わが国では、地盤条件が極めて軟質であり、また、長周期地震動の成分が卓越しやすい大規模地震が発生すること等の条件から、設計地震力が固有周期の2/3乗に反比例して低下する固有周期は、Ⅰ種地盤では1.1秒、Ⅱ種地盤では1.3秒、Ⅲ種地盤では1.5秒と米国、ニュージーランド等の諸外国に比較すると長く設定されている。このため、こうした固有周期よりもさらに固有周期を長くとらないと長周期化による慣性力の低減は見込めない。

長周期化すると桁に生じる慣性力は低下するが、桁の変位は増大する。このため、桁と桁、あるいは、桁と橋台間の遊間を大きく取らないと、桁と桁あるいは桁と橋台が衝突することになる。しかし、遊間を大きくすると伸縮継手も大きくしなければならない。こうすると、ここで車両の走行による大きな騒音や振動が生じるだけでなく、維持管理上の問題も大きい。

このため、道路橋では桁端の処理の問題からあまり長周期化することは好ましくない。したがって、道路橋の免震設計の基本的な考え方としては、長周期化ではなく、減衰性能の向上と分散省としての地震力の分散に重点をおかなければならない。すなわち、

1）免震支承の剛性を変えることにより、下部構造ごとに望み通りの地震力の分散を図りつつ、免震支承の導入により減衰性能を向上させて地震力の低減も図る。

2）無理に長周期化するのではなく、地震力の配分とのバランスをとりつつ、地盤との共振を避けるように固有周期を調節する。

これは、BaseをIsolateする免震設計ではなく、減衰性能の向上により橋の振動の低減を目指した設計法である。したがって、ここでは、これをMenshin Designと呼ぶ。Menshinとは、地震力の低減という意味である。

橋本体は、免震装置をこれと同じ等価剛性を有する等価線形ばねによってモデル化し、この部材の減衰定数を等価減衰定数h_Bとして、静的フレーム法もしくは動的解析により設計する。静的フレーム法では、橋の固有周期Tは設計振動単位ごとに、原則として式(1)により算出する。

$$T = 2.01\sqrt{\delta} \qquad (1)$$

$$\delta = \frac{\int w_i \cdot u_i^2}{\int w_i \cdot u_i} \qquad (2)$$

ここに、

T　：設計振動単位ごとの固有周期（秒）

w_i：上部構造および下部構造の節点ｉの重量（tf）

u_i：耐震設計上の地盤面より上の上部構造および下部構造の重量を水平方向に作用させた場合の節点ｉにおける変位(m)

また、橋の減衰定数hは、原則として式(3)により算出する。

$$h = \frac{\Sigma K_{Bi} \cdot u_{Bi}^2 \cdot c_{hi}}{\Sigma K_{Bi} \cdot u_{Bi}^2 \cdot c_i} \qquad (3)$$

$$c_{hi} = h_{Bi} + \frac{h_{Pi}}{K_{Pi}} + \frac{h_{Fui}}{K_{Fui}} + \frac{h_{F\theta i} \cdot H^2}{K_{F\theta i}}$$

$$c_i = 1 + \frac{K_{Bi}}{K_{Pi}} + \frac{K_{Bi}}{K_{Fui}} + \frac{K_{Bi} \cdot H^2}{K_{F\theta i}}$$

ここで、

h　　：橋の減衰定数

h_{Bi}　：ｉ番目の免震装置の等価減衰定数

h_{Pi}　：ｉ番目の下部構造の減衰定数

h_{Fui}　：ｉ番目の基礎の併進の減衰定数

$h_{F\theta i}$：ｉ番目の基礎の回転の減衰定数

K_{Bi}　：ｉ番目の免震装置の等価剛性(tf/m)

K_{Pi}　：ｉ番目の下部構造の等価剛性(tf/m)

K_{Fui}　：ｉ番目の基礎の併進ばね定数(tf/m)

$K_{F\theta i}$：ｉ番目の基礎の回転ばね定数(tf/radian)

u_{Bi} ：i番目の免震装置の設計変位(m)

H ：フーチング上面から上部構造の慣性力の作用位置までの高さ(m)

ただし、式(3)は、基礎の併進及び回転は上部構造の慣性力によってのみ定まり、橋脚躯体の影響はないと仮定した場合に、略算される減衰定数を与えるものである。橋脚躯体の慣性力の影響や橋の振動モードを正しく取り入れて解析する場合には、式(4)により橋の減衰定数を算出するのがよい。

$$h = \frac{\sum_{j=1}^{n} \phi_{ij}^{T} \cdot h_j \cdot K_j \cdot \phi_{ij}}{\phi_i^{T} \cdot K \cdot \phi_i} \qquad (4)$$

ここで、

ϕ_{ij} ：i次振動モードの要素jのモードベクトル

h_j ：要素jの減衰定数

K_j ：要素jの剛性マトリックス

ϕ_i ：i次振動モードのモードベクトル

K ：構造全体の剛性マトリックス

なお、式(3)及び式(4)により橋の減衰定数を算出する場合の各要素の減衰定数としては、表-1の値を参考にするのがよい。

4．免震設計に用いる設計地震力

免震設計では、免震装置の設計は道路橋示方書に規定される震度法と地震時保有水平耐力法の2つにより行う。また、橋の設計は震度法によるほか、RC橋脚の地震時保有水平耐力を照査する。

震度法に用いる設計水平震度k_hは、式(5)により求める。

$$k_h = c_Z \cdot c_G \cdot c_I \cdot c_T \cdot c_E \cdot k_{h0} \quad \geq 0.1$$

ただし、

$$c_I \cdot c_E \geq 0.8 \qquad (5)$$

ここで、c_Z：地域別補正係数（図－2）、c_G：地盤条件別補正係数（表－3）、c_I：重要度別補正係数（表－4）、c_T：固有周期別補正係数（表－5）、k_{h0}：標準設計水平震度（＝0.2）で、これらはいずれも道路橋示方書に規定される通りである。c_Eは、免震装置のエネルギー吸収特性に基づく補正係数で、橋の1次のモード減衰定数hに応じて表－6の値を用いる。

地震時保有水平耐力法に用いる設計水平震度k_{hc}及び等価水平震度k_{he}は、式(6)、(7)により求める。

$$k_{hc} = c_Z \cdot c_I \cdot c_R \cdot c_E \cdot k_{hc0} \geq 0.3 \qquad (6)$$

$$k_{he} = k_{hc} / \sqrt{2\mu - 1} \qquad (7)$$

ここで、c_Z：地域別補正係数（図－2）、c_I：重要度別補正係数（表－4）、c_R：振動特性別補正係数（表－7）、k_{hc0}：地震時保有水平耐力法に用いる標準設計水平震度（＝1.0）、μ：RC橋脚の許容じん性率で、これらはいずれも道路橋示方書に規定される通りである。c_Eは、免震装置のエネルギー吸収特性に基づく補正係数で、橋の1次のモード減衰定数hに応じて表－8の値を用いる。

5．免震装置の設計

5．1 免震装置の特性値

免震設計では、免震装置に生じる地震時の変位（免震装置の設計変位と呼ぶ）u_B、免震装置の等価剛性K_B及び免震装置の等価減衰定数h_Bが重要である。これらは、以下のように定める。

（1）免震装置の設計変位

免震装置の設計変位u_Bは、式(8)により算出するものとする。

$$u_B = \frac{k_h \cdot W_u}{K_B} \quad \text{〔震度法の場合〕} \qquad (8)$$

$$u_B = \frac{k_{hc} \cdot W_u}{K_B} \quad \text{〔地震時保有水平耐力法の場合〕} \qquad (9)$$

ここに、

u_B ：免震装置の設計変位(m)

k_h ：式(5)による震度法に用いる設計水平震度

k_{hc} ：式(6)による地震時保有水平耐力法に用いる設計水平震度

K_B ：免震装置の等価剛性(tf/m)

W_u ：免震装置が水平力を分担する上部構造部分の重量(tf)

（2）免震装置の等価剛性及び等価減衰定数

免震装置の等価剛性および等価減衰定数は、原則として式(2)および式(3)により算出する。

$$K_B = \frac{F(u_{BE}) - F(-u_{BE})}{2\,u_{BE}} \qquad (10)$$

$$h_B = \frac{\Delta W}{2\pi W} \qquad (11)$$

ここで、

K_B ：免震装置の等価剛性(tf/m)

h_B ：免震装置の等価減衰定数

u_{BE}：免震装置の有効設計変位(m)で、次式で算出する

$$u_{BE} = c_B \cdot u_B \qquad (12)$$

u_B ：式(8)もしくは式(9)による免震装置の設計変

位(m)

c_B ：慣性力の非定常性を表わす補正係数（0.7とする）

$F(u_B)$：免震装置に変位uを与えるのに必要な水平力(tf)

W ：免震装置の弾性エネルギーで、図－3に示す三角形の面積(tf・m)

△W：免震装置が吸収するエネルギーの合計で、図－3に示す水平変位と水平荷重の履歴曲線の面積(tf・m)

5．2 免震装置の動的特性

（1）免震装置は、その等価剛性K_B及び等価減衰定数h_B等の平均値が設計値に対して±20%以内にあるように設計、製作しなければならない。ここで免震装置の等価剛性及び等価粘性減衰定数の平均値とは、式(12)による免震装置の有効設計変位u_{BE}による10回の正負くり返し載荷から求められた4回目から10回目までの載荷における平均的な等価剛性及び等価粘性減衰定数をいう。これは、免震装置の等価剛性及び等価減数定数が設計値に対して±20%以内に収まっていれば、上部構造の加速度、変位応答等の変動幅が設計上特に問題になることはないためである。ただし、特性値のバラツキが大きく、安定した履歴曲線が得られない免震装置は使用してはならない。

（2）免震装置は、式(9)による免震装置の設計変位u_Bによる50回の正負連続くり返し載荷に対して、安定でなければならない。くり返し載荷回数を50回としたのは、1回の大地震による主要動の繰り返し回数を30回と見込み、これに余裕を見てその1.5倍をとったものである。

（3）地震中に免震装置が非線形域に入る大きな変位を生じたとしても地震後の橋の走行性、その他の観点から免震装置はできる限り原位置に復帰することが望ましい。このため、式(9)による免震装置の設計変位u_Bを与えた状態から自然に免震装置を解放した状態で、免震装置に残る最大残留変位u_{BR}は、式(13)を満足しなければならない。

$$u_{BR} \leq 0.10 \times u_B \qquad (13)$$

ここで、

u_{BR}：免震装置の残留変位(cm)

u_B ：式(9)による免震装置の設計変位(cm)

（4）免震装置は、常時荷重の変動、温度変化等環境の変化もしくは地震時のくり返し載荷等の影響に対して等価剛性K_B及び等価減衰定数h_Bが安定したものでなければならない。安定性に関して検討すべき事項として以下のものが考えられる。

1) 温度変化による桁の伸縮や活荷重による振動のくり返し
2) 履歴経験
3) 鉛直荷重の変化
4) 振動数および振動速度の変化
5) クリープや乾燥収縮等による静的与変位
6) 振動方向
7) 温度変化

5．3 免震装置の静的特性

（1）免震装置は、材料、機構などの面から長期的に安定して使用できるものでなければならない。また、温度変化に伴う橋の伸縮の影響を受けるので、これによるくり返し変位に対して安定でなければならない。

（2）免震装置は、上部構造の死荷重により生じる局部せん断ひずみに対して安定した構造としなければならない。局部せん断ひずみの算定式は、道路橋支承便覧「3.6 ゴム支承の設計」の規定による。

（3）上部構造の死荷重による免震装置のクリープによる圧縮変形量は、ゴム総厚の5%以下でなければならない。

（4）温度が－10℃の時と40℃の時の免震装置の等価剛性の比は、1.5 以下でなければならない。ただし、この温度変化の範囲は気候が普通の地方の場合であり、寒冷な地方においては実状に応じて温度変化の範囲を定めなければならない。

（5）免震装置は、風荷重等の地震の影響以外の水平荷重に対して有害な変位を生じないように十分な初期剛性を有するものでなければならない。

6．免震装置の性能確認試験法

6．1 一般

免震装置は、免震設計の重要な要素となるものであるから、設計で設定した特性が確実に発揮できるものでなければならない。免震装置のエネルギー吸収機構にはいろいろなものがあるが、主要なものを挙げると以下の通りである。

(a)変位依存型免震装置

(b)摩擦型免震装置

(c)速度依存型免震装置

したがって、本マニュアル（案）では、こうした3つの構造特性に分けて性能確認試験方法を示している。

6．2 免震装置の動的特性

5．2に示した免震装置の動的特性を確保するために、以下の試験を行うのがよい。

(1)設計に用いた等価剛性及び等価減衰定数を確認するための試験

(2)地震時保有水平耐力法に用いる免震装置の設計変位u_Bによる50回の正負連続くり返し載荷試験

(3)免震装置に生じる残留変位を確認するための試験

(4)常時荷重の変動、温度変化等の環境条件に対する安定性を確認するための試験

1)くり返し載荷に対する安定性

免震装置の中には、地震によりくり返し載荷をうけると、等価剛性や等価減衰定数などの特性値が変化す

るものがある。これらが、地震力を受けている間に大きく変化すると、設計で想定した状態とは異なった振動を引き起こす恐れがあるため、これらの変化の大きさに制限を加える必要がある。

2)履歴経験に対する安定性

免震装置の中には、同じような変位を連続して受けても、最初の特性値と一度その変位を経験したあとの特性値が異なるものがある。免震装置の特性値が、地震力を受ける間に変化すると、設計で想定した状態と異なった振動を引き起こす恐れがあるため、この変化の大きさに制限を加える必要がある。

3)支圧応力度の変化に対する安定性

免震装置に作用する鉛直荷重が、架設誤差や下部構造の不等沈下等の影響で、設計で想定した値と異なる場合がある。このような条件下でも、免震装置が安定して機能することを確認するものである。

4)変形速度の変化に対する安定性

地震時には免震装置はいろいろな振動速度の変形が生じるため、これらに対して安定して機能することを確認するものである。

5)静的予変位に対する安定性

免震装置は、一般に温度変化の影響による静的な変位振幅をくり返し受ける状態で使用される。静的予変位の大きさによって、免震装置の中には動的特性が変化するものがある。この変化が大きいと、設計で想定した状態とは異なった挙動を生じる場合があるため、この変化の大きさに対する検討をするものである。

6)温度変化に対する安定性

免震装置周辺の温度は、季節的に装置の供用期間内にくり返し変化する。このような環境条件下においても、免震装置の特性が安定して機能することを確認するための試験である。

6．3　免震装置の静的特性の確認試験方法

5．3に示した免震装置の静的特性を確認するために、以下の試験を行なうのがよい。

(1)温度変化にともなう常時のくり返し変位に対する耐久性を確認するための試験

(2)クリープによる圧縮変形量を確認するために試験

(3)温度変化が-10℃の時と40℃の時の免震装置の等価剛性を確認するための試験

6．4　性能確認試験の例

マニュアル（案）では、上記6．2及び6．3に対する試験方法が、変位依存型免震装置、摩擦型免震装置、速度依存型免震装置に分けて詳細に示されている。ここでは、こうした性能確認試験法の例として、動的特性の中から、くり返し載荷に対する安定性の確認試験方法と変形速度の変化に対する安定性の確認試験方法を、変位依存型免震装置を例に示す。

（1）くり返し載荷に対する安定性の試験

温度を＋20℃とし、設計支圧応力度に相当する鉛直荷重を作用させた条件下で、式(9)による免震装置の設計変位u_Bおよびそれよりも小さい変位で、振動数0.5Hzの水平力を免震装置に10回くり返し作用させた場合の免震装置の等価剛性および等価減衰定数は、原則として式(14)および式(15)を満足しなければならない。

$$R_{KC}=\frac{|K_{Bj}-K_{Bm}|}{K_{Bm}}\quad\begin{matrix}\leqq 0.3\ (j=1,2,3)\\ \leqq 0.1\ (j=4,5\cdots10)\end{matrix}\qquad(14)$$

$$R_{hc}=\frac{|h_{Bj}-h_{Bm}|}{h_{Bm}}\quad\begin{matrix}\leqq 0.3\ (j=1,2,3)\\ \leqq 0.1\ (j=4,5\cdots10)\end{matrix}\qquad(15)$$

ここで、

R_{KC}：免震装置の等価剛性の変化率

R_{hc}：免震装置の等価減衰定数の変化率

K_{Bj}：第ｊ回目の載荷時の免震装置の等価剛性(tf/m)

K_{Bm}：免震装置の平均等価剛性(tf/m)で、式(16)により算出する

h_{Bj}：第ｊ回目の載荷時の免震装置の等価減衰定数

h_{Bm}：免震装置の平均等価減衰定数で、式(17)により算出する。

$$K_{Bm}=\frac{1}{7}\sum_{j=4}^{10}K_{Bj}\qquad(16)$$

$$h_{Bm}=\frac{1}{7}\sum_{j=4}^{10}h_{Bj}\qquad(17)$$

なお、性能確認試験においては、免震装置の等価剛性K_Bおよび等価減衰定数h_Bは、式(18)および式(19)により算出する。

$$K_B=\frac{F(u_{max})-F(u_{min})}{u_{max}-u_{min}}\qquad(18)$$

$$h_B=\frac{\Delta W}{2\pi W}\qquad(19)$$

ここで、

K_B：免震装置の等価剛性(tf/m)

h_B：免震装置の等価減衰定数

$F(u_{max})$：免震装置に水平変位ｕを与えるのに必要な水平力(tf)

u_{max}：免震装置に与えた水平変位ｕの最大値(m)

u_{min}：免震装置に与えた水平変位ｕの最小値(m)

ΔW：免震装置が１サイクル間に吸収するエネルギーで図－４に示す水平変位の履歴曲線の面積

(tf·m)

W ：免震装置の弾性エネルギーで、図－４に示す三角形の面積(tf·m)

（２）変形速度の変化に対する安定性

温度を＋20℃とし、設計支圧応力度に相当する鉛直荷重の作用下で、ひずみ振幅が±100%、または、変位振幅が±15cmで、振動数0.1Hz、0.5Hz、1.0Hzの水平力を免震装置に各10回くり返し作用させた場合の免震装置の平均等価剛性および平均等価減衰定数は、原則として式(20)および式(21)を満足しなければならない。

$$R_{Kf} = \frac{|K_{Bm1} - K_{Bm2}|}{K_{Bm1}} \leqq 0.2 \qquad (20)$$

$$R_{hf} = \frac{|h_{Bm1} - h_{Bm2}|}{h_{Bm1}} \leqq 0.2 \qquad (21)$$

ここで、

R_{Kf} ：変形速度の変化に対する免震装置の平均等価剛性の変化率

R_{hf} ：変形速度の変化に対する免震装置の平均等価減衰定数の変化率

K_{Bm1}, K_{Bm2}, K_{Bm3} ：加振振動数がそれぞれ0.5Hz、0.1Hz、1.0Hzの場合の免震装置の平均等価剛性(tf/m)で、式(16)により算定する。

h_{Bm1}, h_{Bm2}, h_{Bm3} ：加振振動数がそれぞれ0.5Hz、0.1Hz、1.0Hzの場合の免震装置の平均等価減衰定数で、式(17)により算定する。

7．あとがき

一般橋クラスの道路橋では、諸外国のように長周期化による設計地震力の低減は１種地盤等ごく特別な場合をのぞいてできないことから、減衰性能と慣性力の分散を中心とした日本向けの免震設計法を提案した。これはBaseをIsolationする免震設計ではなく、減衰性能の向上により橋の振動低減を目指した設計法である。以上のようにした上で、従来の道路橋示方書に規定される地震力を減衰性能の度合に応じて震度法では最大２０%、地震時保有水平耐力法では３０%低減した値を用いることを提案としてまとめた。

謝辞

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Table 1 Research Theme and Organizations of Joint Research Program between the Public Works Research Institute and 28 Private Firms for Developing Menshin Design Method of Highway Bridges

Research Theme	P	Ka	Si	Ob	Ku	Tn	H	Ni	Su	M	G	Ok	Ti	I	Nk	Ko	Ns	Ot	Y.	To	Bs	Bb	Se	Sh	Pc	J	N	Chief	Sub-Chief
1. Development of Device for Isolation																													
1.1 High Energy Absorbing Rubber Bearing								○											○	○	○			○					
1.2 Friction Damper													○									○	○						
1.3 Steel Damper								○							○													Ol	Bs
1.4 Link Bearing Develop of																	○												
1.5 Viscous Damper																		○											
1.6 Test Method	○							○							○		○	◎	○	○	○	○	○	○					
2. Development of Expansion Joint and Falling-off Prevention Device for Isolated Bridge																												Tc	—
2.1 Expansion Joint				○									○																
2.2 Falling-off Prevention Device	○												○					○											
3. Development of Design Method for Isolated Bridge																													
3.1 Design Philosophy	○	○	○	○		○						○	◎					○							○	○	○		
3.2 Dynamic Responce Analysis Method	○		○		○	◎		○			○			○													Ka	Pc
3.3 Design Method of Device for Isolation	○						○	○				◎	○		○		○	○	○	○	○	○	○	○					
3.4 Simplified Design Method	○	○	◎		○	○					○	○																	
3.5 Design Method of Expansion Joint and Falling-off Prevention Device	○			○			○								○			○							◎				
4. Application of Base Isolation to Bridge																													
4.1 Application to Prestressed Concrete Bridge	○	◎							○	○			○														○		
4.2 Application to Steel Bridge	○													○	○	○											◎	Ob	J
4.3 Application to Multiple Super-long Bridge	○			◎					○							○									○	○	○		
4.4 Application to Seismic Retrofit	○									○				○				○								◎			

P: Public Works Research Institute, Ka: Kajima, Si: Shimizu, Ob: Ohbayashi, Ku: Kumagai, Tn: Takenaka Doboku + Takenaka, H: Hazama, Ni: Nishimatsu, Su: Sumitomo, M: Mitsui, G: Goyoh, Ok: Okumura, Ti: Taisei + Tokyo Fablic + Nippon Chuzo, I: Ishikawajima Harima, Nk: NKK + Nippon Chuzo, Ko: Kobe Steel, Ns: Nippon Seiko, Oe: Oiles, Y: Yokohama Rubber, To: Toyo Rubber, Bs: Bridgestone, Bb: BBM, Se: Seibu Polymer, Sh: Showa Densen, Pc: Pacific Consultants, J: Japan Engineering Consultant, N: New Structural Engineering Consultants.

Table 2 Damping Ratio Recommended for Structural Components

Structural Components	Steel	Concrete
Superstructures	0.02～0.03	0.03～0.05
Menshin Device	Damping Ratio by Eq.(11)	
Pier/Columns	0.03～0.05	0.05～0.1
Footing	0.1～0.3	

Table 3 Modification Factor for Ground Condition c_G

Ground Group	I	II	III
c_G	0.8	1.0	1.2

Table 4 Modification Factor for Importance c_I

Group	c_I	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road. Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 5 Modification Factor for Structural Response c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$ $c_T = 2.69T^{1/3} \geqq 1.00$	$0.1 \leqq T \leqq 1.1$ $c_T = 1.25$	$1.1 < T$ $c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$ $c_T = 2.15T^{1/3} \geqq 1.00$	$0.2 \leqq T \leqq 1.3$ $c_T = 1.25$	$1.3 < T$ $c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$ $c_T = 1.80T^{1/3} \geqq 1.00$	$0.34 \leqq T \leqq 1.5$ $c_T = 1.25$	$1.5 < T$ $c_T = 1.64T^{-2/3}$

Table 6 Modification Factor for Energy Dissipation Capability c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$h \geqq 0.1$	0.9

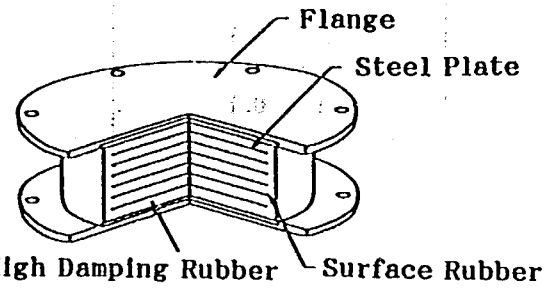
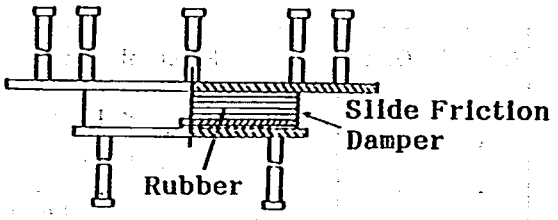
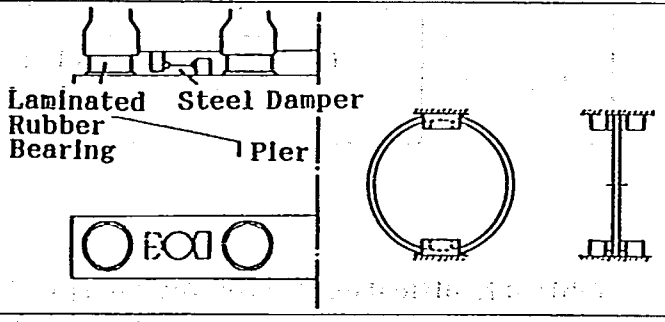
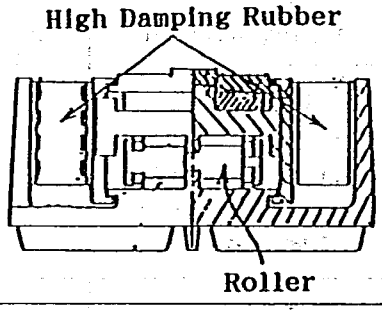
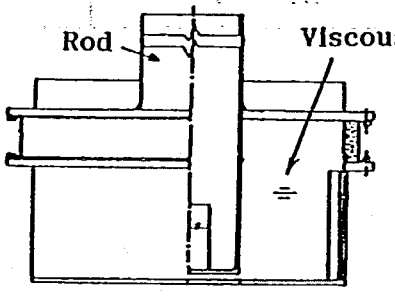
Table 7 Modification factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leqq 1.4$ $c_R = 0.7$		$1.4 < T_{EQ}$ $c_R = 0.876T_{EQ}^{-2/3}$
Group II	$T_{EQ} < 0.18$ $c_R = 1.51\ T_{EQ}^{1/3} \geqq 0.7$	$0.18 \leqq T_{EQ} \leqq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51T_{EQ}^{1/3} \geqq 0.7$	$0.29 \leqq T_{EQ} \leqq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59T_{EQ}^{-2/3}$

Table 8 Modification Factor for Energy Dissipating Capability

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$0.1 \leqq h < 0.12$	0.9
$0.12 \leqq h < 0.15$	0.8
$0.15 \leqq h$	0.7

Fig. 1 Menshin Devices Developed

TYPE	OUTLINE OF MENSHIN DEVICE
HIGH DAMPING RUBBER BEARING	
SLIDE FRICTION -RUBBER BEARING	
STEEL DAMPER	
ROLLER MENSHIN BEARING	
VISCOUS	

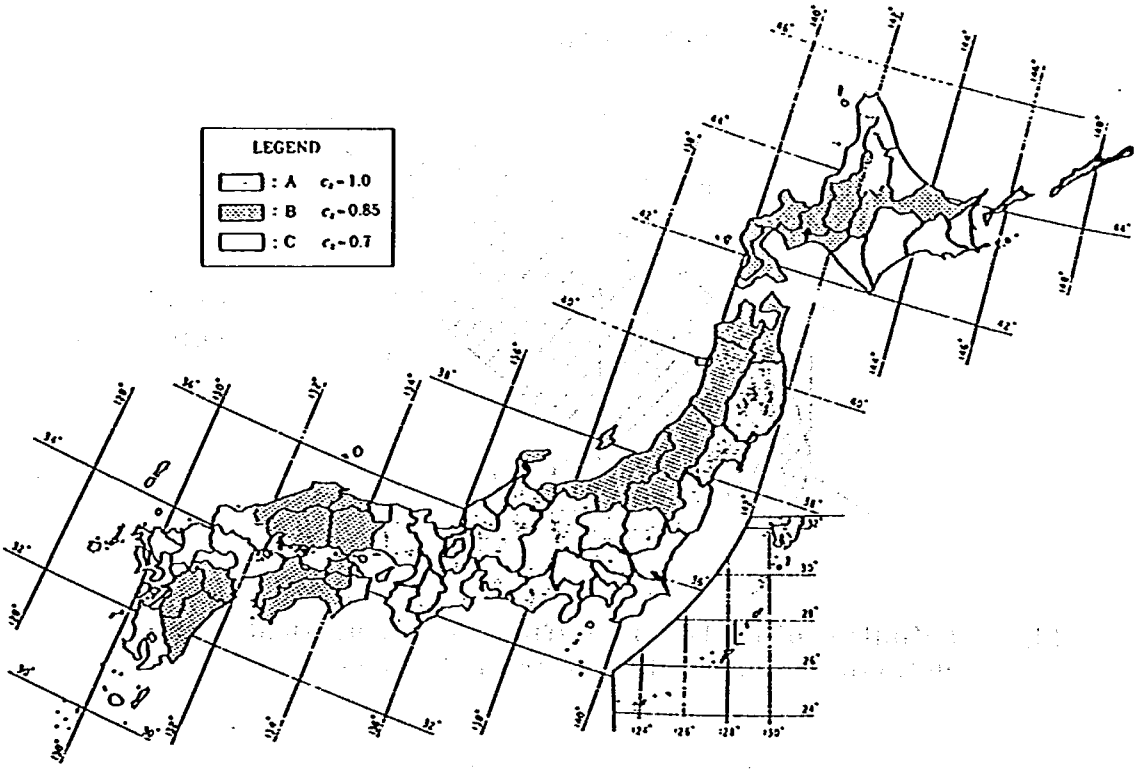


Fig. 2 Modification Factor for Zone c_z

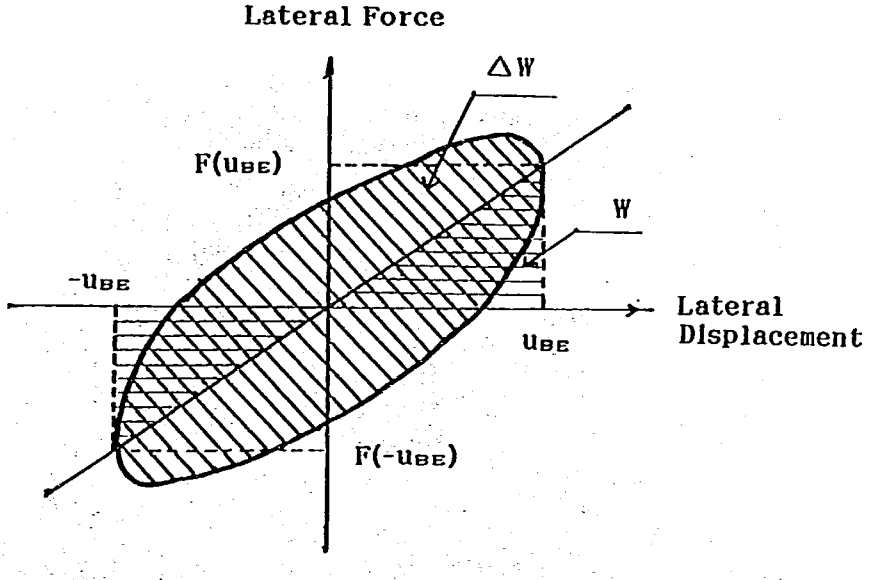


Fig. 3 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menshin Device for Design of Menshin Device

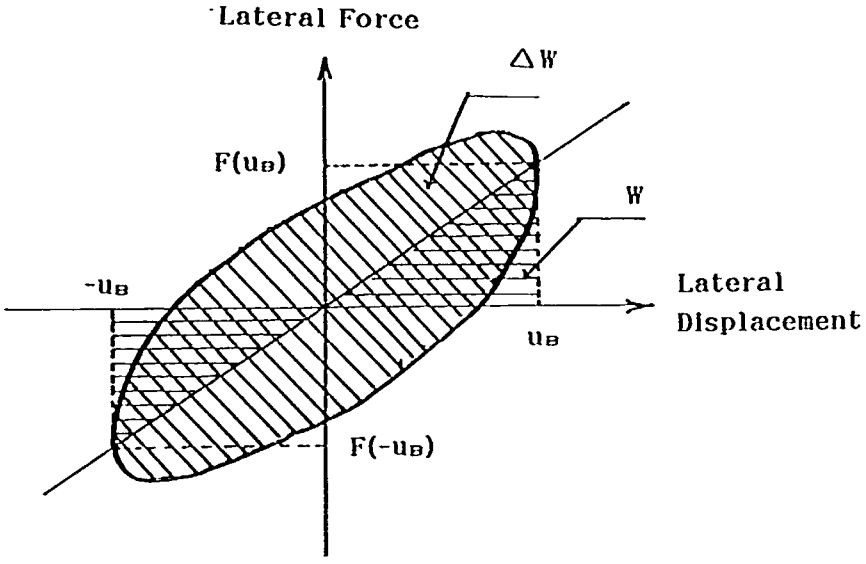


Fig. 4 Definition of Equivalent Stiffness and Equivalent Damping Ratio of Menshin Device for Approving Tests

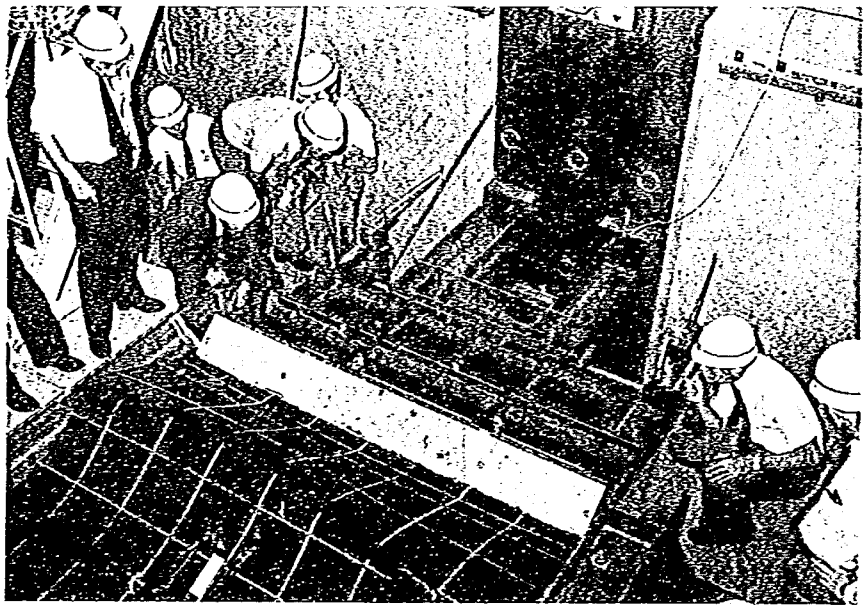


Photo 1 Knock-off Abutment

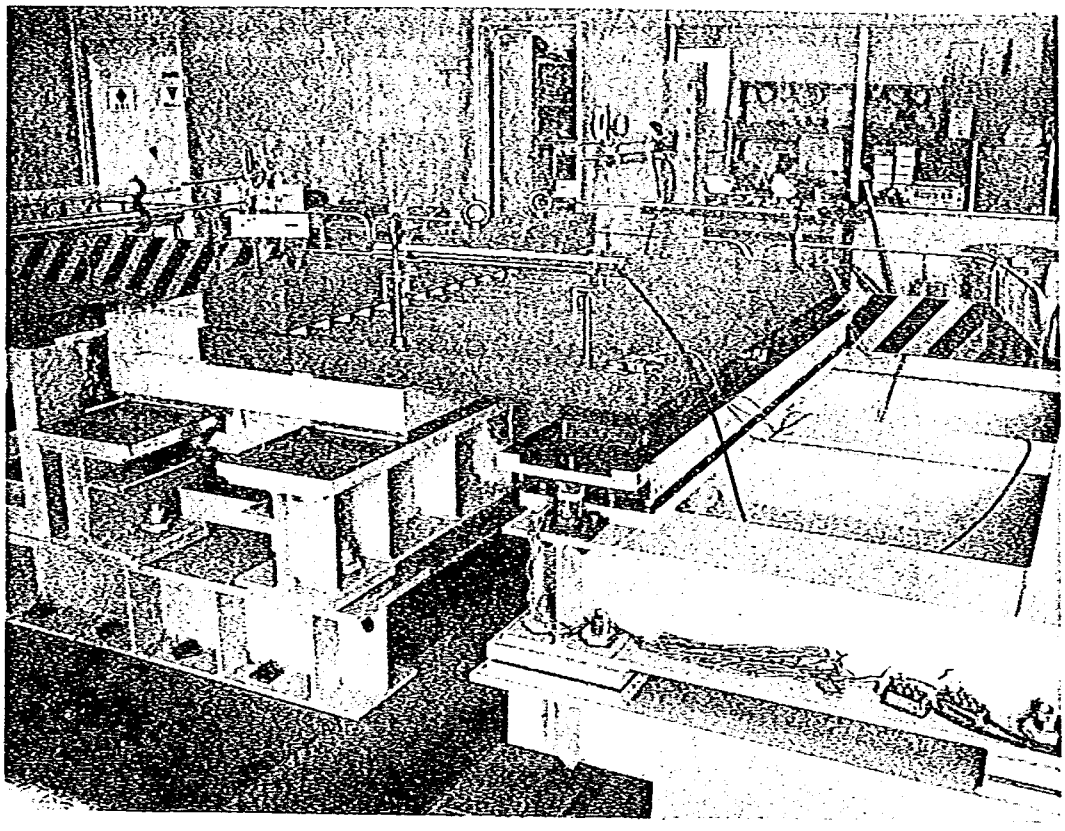


Photo 2 Expansion Joint Movable in Longitudinal and Transverse Directions

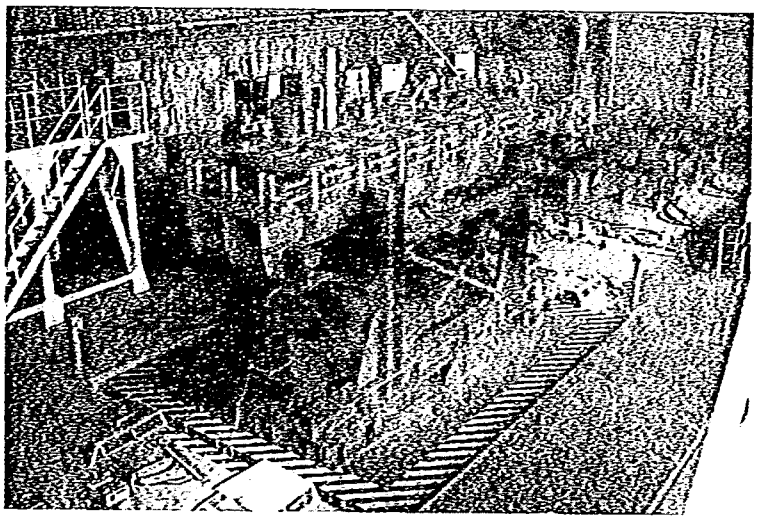
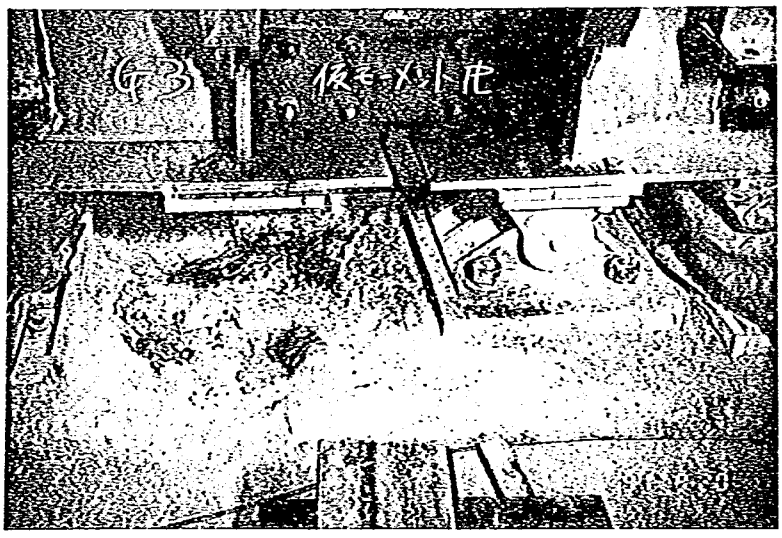


Photo 3 Shaking Table Tests for Studying Structural Response of Menshin Bridges



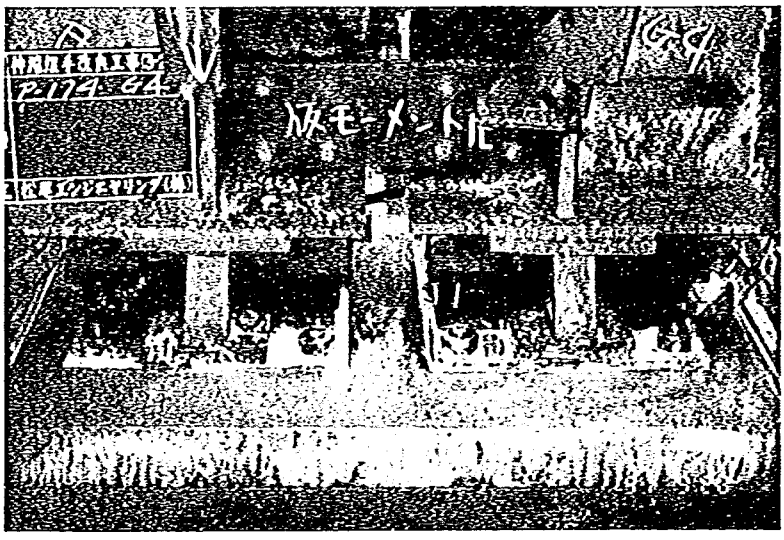


Photo 4 Deck Connection of Existing Bridge

Friction Controllable Bearings for Sliding Base Isolation Systems

by

Maria Qing Feng and Masanobu Shinozuka*

ABSTRACT

Recently, sliding base isolation systems began to be utilized widely in civil and architectural engineering as exemplified by the recent decision on the part of US Federal Government (General Services Administration) to deploy 250-300 spherical sliders to base-isolate the 100 year old, 5-story and 60,000 ton Court of Appeals Building in San Francisco. While the building survived the 1906 San Francisco Earthquake, it was damaged by the Loma Prieta earthquake. Sliding base isolation systems, it is generally perceived, have disadvantages in that they are not effective for small to medium earthquakes, and may not be able to maintain the same value of friction coefficient under a prolonged inactivity and that they may suffer from excessive displacements under strong earthquakes. This paper presents the results of a recent experimental and analytical study jointly performed by Princeton University and Taisei Corporation (Refs. 1 and 2) demonstrating that the use of friciton controllable bearings can not only neutralize the disadvantages indicated above but also make it possible to take full advantage of the beneficial features of sliding base isolation; its inherent effectiveness in limiting the transfer of the ground acceleration to the structure.

KEY WORDS

Hybrid sliding isolation, Friction controllable sliding bearings, Earthquakes, Instantaneous optimal control, Shaking table

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1 INTRODUCTION

The purpose of this research is to physically develop a sliding bearing that can actively control the friction force on the sliding interface and to demonstrate by shaking table experiments and numerical simulation that the difficulties associated with sliding isolation systems can be significantly alleviated, if not totally eliminated, by such a device. More specifically, the purpose is:

1. to propose and physically develop a friction controllable sliding bearing and a hybrid sliding isolation system using such bearings,

2. to develop control algorithms for controlling the friction force which has nonlinear characteristics, and

3. to analytically and experimentally demonstrate that such a hybrid isolation system can indeed intelligently control the friction force on the sliding interface so as to confine the sliding displacement in an acceptable range, to reduce the residual displacement, and at the same time, to minimize the transfer of seismic force to the structure.

The hybrid isolation system using friction controllable bearings (FCB's) is conceptually depicted in Figs. 1 with a bridge structure resting on the bearings. Each bearing has a fluid chamber which is connected to a pressure control system composed of a servo valve, an accumulator and a computer. The friction on the interface between the bearing and the ground is controlled by adjusting the fluid pressure in the chamber. The computer calculates an appropriate signal to control the fluid pressure based on the observed

structural response, such as response acceleration and sliding displacement, and sends it to the pressure control device as shown in Figs. 1.

The idealized section view of the friction controllable sliding bearing is given in Fig. 2. The bearing made of steel is of disk shape containing a fluid chamber inside which is sealed by a rubber O-ring around the circular perimeter just inside the sliding surface of the bearing. A sliding material such as PTFE plate is placed on the sliding surface.

The word "HYBRID" is used for this system for the following reason. The system can be a paasive sliding isolation system as long as the pressure of the bearing chamber, and thus the friction is kept at a constant value. At the same time, the system can be an active system as long as the pressure is controlled so that the friction is controlled. In the analytical formulatin involving such a hybrid system, the control force does not appear in the equation of motion as an independent term like the force from an actuator. The active control in this case is implemented through the friction term in the equation, and in that sense the system may also be interpreted as "Semi-Active". From the view point of the energy requirement to control the isolation device, the system has a significant advantage since the pressure control does not need as much energy as the common active structural control by actuators would require.

2 ANALYTICAL MODEL

A rigid structure supported by the friction controllable sliding bearings is considered. The motion of the structure can be modeled by a single-degree-of-freedom (SDOF) model as shown in Fig. 3. The equations of motion of the structure under earthquake excitation can then be written as follows.

1. Sticking Phase — Phase I

$$\dot{x} = 0, \quad x = const. \tag{1}$$

2. Sliding Phase — Phase II

$$\ddot{x} = -\ddot{z} - f\ \mathrm{sgn}(\dot{x}), \tag{2}$$

3. Criteria for transition from Phase I to Phase II

$$|\ddot{z}| > f \tag{3}$$

4. Criteria for transition from Phase II to Phase I

$$\dot{x} = 0 \tag{4}$$

$$|\ddot{z}| < 2f \tag{5}$$

where x is the sliding displacement of the mass relative to the ground; $\ddot{z}$ is the input earthquake acceleration; μ is the coefficient of friction on the sliding interface; f is the normalized friction force defined as $f = \mu g$.

In the sticking phase, Eq. 1 governs the motion of the structure until the Eq. 3 becomes true. As soon as the condition in Eq. 3 is met, the sliding phase starts and Eq. 2 governs the sliding motion. During the sliding phase, whenever $\dot{x}$ becomes zero, the criterion, Eq. 4, is checked to determine the subsequent behavior. Validity of the inequality given by Eq. 4 is the condition for entering the sticking phase. That is, if the inequality holds, the structure will stick to the ground and Eq. 1 apply. If the criterion given by Eq. 4 is not satisfied, Eq. 1 will continue to govern the the subsequent sliding motion.

On the other hand, the normalized friction force f on the sliding interface between the structure and the ground is controlled by changing the fluid pressure in the bearing chamber through a pressure control system consisting of a computer, servo valve, amplifier, etc. The dynamic characteristics of the pressure control system are assumed to follow the first order time delay model:

$$T\dot{p} + p = u \tag{6}$$

where, p is the pressure in the fluid chamber of bearing; u is the pressure control signal from computer; T is the time constant

The normalized friction force f is negatively proprtional to the pressure p in the bearing chambe:

$$f = -c_1 p + c_2 \tag{7}$$

where c_1 and c_2 are constants.

3 INSTANTANEOUS OPTIMAL CONTROL

Control theory has not been well developed for the systems in which the control force has the nonlinearity, unique to the friction controllable sliding isolation device. For this reason, an optimal control algorithm for control of the nonlinear friction system is developed on the basis of the instantaneous optimal control theory originally proposed by Yang et al. (Ref. 3).

The optimal pressure control signal $u(t)$ is determined by minimizing the following time dependent objective function $J(t)$ at every time instant t for the entire duration of an earthquake.

$$J(t) = q_d x^2(t) + q_f f^2(t) + r u^2(t) \qquad (8)$$

in which the normalized friction force f equivalently represents the amount of response acceleration and also serves as a measure of the transfer of seismic force to the structure. The weighting coefficient q_d and q_f are non-negative and r is positive. They indicate the relative importance in the control objectives of the sliding displacement, response acceleration and pressure control signal, respectively. The basic objectives of the control is to make the structure slide as much as possible within an acceptable range and at the same time to ensure the transfer of seismic force to a minimum.

With this theory, the following control algorithm is derived under the assumption that the structural motion is always in the sliding phase. The equation of motion given by Eq. 2 should be used as a constraint when minimizing the objective function $J(t)$. The first order time delay relationship between the control singal and the pressure described in Eq. 6, as well as the linear relationship between the friction and the pressure shown in Eq. 7, are also constraints. In the present formulation, however, these equations will be solved numerically using the Newmark's β method with $\beta = 1/6$ as shown below and these numerical solutions will be used as constraints:

$$\begin{aligned} x(t) &= x(t-\Delta t) + \dot{x}(t-\Delta t)\,\Delta t \\ &\quad + \ddot{x}(t-\Delta t)\frac{\Delta t^2}{2} \\ &\quad + [\ddot{x}(t) - \ddot{x}(t-\Delta t)]\frac{\Delta t^2}{6} \end{aligned} \qquad (9)$$

$$f(t) = f(t-\Delta t) + \dot{f}(t-\Delta t)\,\Delta t + [\dot{f}(t) - \dot{f}(t-\Delta t)]\frac{\Delta t}{2} \qquad (10)$$

Furthermore,

$$x(t) = a\, f(t)\,\mathrm{sgn}(\dot{x}(t)) + b\ddot{z}(t) + d_1(t-\Delta t) \qquad (11)$$

$$f(t) = -c\, u(t) + d_2(t-\Delta t) \qquad (12)$$

where

$$a = b = -\frac{\Delta t^2}{6}, \quad c = \frac{c_1 \Delta t}{2T + \Delta t} \qquad (13)$$

$$\begin{aligned} d_1(t-\Delta t) &= x(t-\Delta t) + \dot{x}(t-\Delta t)\Delta t \\ &\quad + \frac{1}{3}\ddot{x}(t-\Delta t)\,\Delta t^2 \end{aligned} \qquad (14)$$

$$\begin{aligned} d_2(t-\Delta t) &= \frac{2T}{2T+\Delta t}\left(\frac{c_2\,\Delta t}{2T} + f(t-\Delta t)\right. \\ &\quad \left. + \frac{1}{2}\dot{f}(t-\Delta t)\Delta t\right) \end{aligned} \qquad (15)$$

Thus, the following generalized objective function is established by introducing the Lagrangian multipliers λ_1 and λ_2:

$$\begin{aligned} H(t) &= q_d\, x^2(t) + q_f\, f^2(t) + r\, u^2(t) \\ &\quad + \lambda_1[x(t) - a\, f(t)\,\mathrm{sgn}(\dot{x}(t)) \\ &\quad - b\,\ddot{z}(t) - d_1\,(t-\Delta t)] \\ &\quad + \lambda_2[f(t) + c\, u(t) - d_2\,(t-\Delta t)] \end{aligned} \qquad (16)$$

The necessary conditions for minimizing the objective function $J(t)$ are:

$$\frac{\partial H}{\partial x} = 0, \quad \frac{\partial H}{\partial f} = 0, \quad \frac{\partial H}{\partial u} = 0, \quad \frac{\partial H}{\partial \lambda_1} = 0, \quad \frac{\partial H}{\partial \lambda_2} = 0, \qquad (17)$$

Substituting Eq. 16 into Eq. 17 yields the optimal pressure control signal:

$$u(t) = F_f\, f(t) + F_d\, x(t)\,\mathrm{sgn}(\dot{x}(t)) \qquad (18)$$

where, the control feedback gains F_f and F_d are calculated by

$$F_f = \frac{c\, q_f}{r}, \quad F_d = \frac{c\, a\, q_d}{r} \qquad (19)$$

and the displacement $x(t)$ and friction $f(t)$ are used for feedback purpose. Again, notice that $\mathrm{sgn}(\dot{x})$ can be obtained from the displacement signal without the need to measure the velociy. The friction is difficult to measure by a sensor, but the signal from the acceleration sensor $(\ddot{x}(t)+\ddot{z}(t))$ can be used instead of f. Therefore, the control signal becomes:

$$u(t) = F_f\ |\ddot{x}(t)+\ddot{z}(t)| + F_d\ x(t)\ \mathrm{sgn}(\dot{x}(t)) \quad (20)$$

In the development of the optimal control algorithm shown above, the time delay of control device shown in Eq. 6 has been incorporated. In this case, the control is referred to as "instantaneous optimal control with time delay".

If the response of the control device is so fast that the time delay can be ignored, a similar procedure to the above can be shown to result in the following control signal:

$$u(t) = F + F_d\ x(t)\ \mathrm{sgn}(\dot{x}(t)) \quad (21)$$

where

$$F = \frac{c_1 c_2 q_f}{r + q_f {c_1}^2}, \quad F_d = \frac{c_1 a q_d}{r + q_f {c_1}^2} \quad (22)$$

In this case, only the sliding displacement $x(t)$ needs to be measured and fed back.

4 SHAKING TABLE TEST

4.1 Structure Model and Isolation Device

The structure model used for experiments is shown in Fig. 4. The model, representing a rigid structure, consists of a steel frame and steel weights. The total weight of the model is 12 tonf. The model is supported equally by four friction controllable sliding bearings on the shaking table. The bearing developed for the experiments is shown in Fig. 5. Figure 6 is a photograph of the bearing being used in the experiments. The bearing, with a brass sheet of 1 mm thickness attached to be used as sliding surface, slides on a stainless steel plate fixed on steel I-bars bolted down on the shaking table, as shown in Fig. 4. Furthermore, a rubber O-ring of 5.7 mm in diameter acts as seal for the fluid in the fluid chamber. The area of the sliding surface is 86.0 cm ^{2}, and the vertically projected area of the fluid chamber is 57.7 cm^2. No restoring force device is used in order to study the effect of friction force only. A servo valve is located at the center of the experimental structure from which the pressurized fluid is distributed to each sliding bearings as described in Fig. 4.

4.2 Passive and Hybrid Isolation

Typical experimental results are obtained under the El Centro (NS, 1940) ground acceleration. In the passive isolation results as shown in Fig. 7(a), the pressure in the bearing chamber is kept at 30 kgf/cm^2, corresponding to the coefficient of friction 5.3%. A large residual sliding displacement of 62 mm after the earthquake is observed.

Figure 7(b) shows the results of hybrid isolation experiments using instantaneous optimal control with time delay, in which the feedback gains are set to: F_f = 16.5 kgf/cm^2 and F_d = -16.4 kgf/cm^3 with the pressure control signal confined between u_{max} = 45 kgf/cm^2 and u_{min} = 10 kgf/cm^2.

The performance of the hybrid isolation system is compared to that of the passive isolation system. Figure 8 shows the maximum response acceleration, maximum sliding displacement, and the residual displacement of the structural model with passive or hybrid isolation system under linearly scaled El Centro records of different peak ground acceleration. Hybrid isolation results shown in these figures are those under instantaneous optimal control algorithm without time delay.

In the passive isolation, if a small friction coefficient, for example, 1.6%, is used, a high level of isolation performance is expected since the response acceleration is reduced to a small level. In this case, however, the maximum displacement becomes excessive very rapidly as the input earthquake becomes more intense. On the other hand, if a large friction coefficient, such as 10.2%,

is used, the sliding displacement can be confined within a relatively small range. However the isolation performance in this case is limited in the sense that the acceleration can not be satisfactorily reduced particularly for small to medium earthquakes.

The hybrid isolation system can alleviate these problems associated with passive isolation system; the response acceleration can be considerably reduced even for small to medium earthquakes, while excessive sliding displacement can be prevented for large earthquakes. Another advantage of the hybrid system is clearly seen in Fig. 8 where the residual sliding displacement can be maintained almost at zero level.

In summary, the hybrid isolation system performs better than the passive system in the sense that a reduction of response acceleration is achieved for small to medium earthquakes, and at the same time, the excessive sliding displacement under large earthquakes is prevented. Furthermore, the residual displacement is reduced to zero in most cases.

5 CONCLUSIONS

A systematic study on a hybrid sliding seismic isolation system using friction controllable bearings has been presented in order to significantly improve the base isolation characteristics of sliding systems. For this purpose, a hybrid sliding isolation system using friction controllable bearings has been physically developed, and shaking table experiments were performed on a rigid structural model equipped with such a hybrid system. A computer code has been developed for real-time on-line control implementation. The dynamic characteristics of the control system between bearing pressure and sliding friction have been identified. The results of hybrid sliding isolation experiments were compared with those of passive isolation.

The following conclusions are obtained:

1. Significant advantage of the proposed hybrid sliding isolation system has been demonstrated: (1) for the small to medium earthquakes, the coefficient of friction is kept at the minimum value to reduce the transfer of the seismic force to the structure to a minimum; (2) As the input earthquake becomes more intense, the friction is controlled to confine the sliding displacement of the structure to an acceptable range, while at the same time to keep the transfer of seismic force as small as possible. Such intelligent features of the hybrid system can not be achieved by the passive sliding isolation system.

2. Control algorithms developed for control of nonlinear friction force proved to be effective in achieving the desired control performances. In addition, they are practical and easy for real-time on-ling control operations.

3. The analytical model of dynamic characteristics between the bearing pressure control signal and the friction on the sliding interface has been identified. Computer simulation results excellently match the experiments. This implies that the analytical model represents the actual system very well, showing the possibility of utilizing the model to perform analytical study on other types of real structures equipped with the hybrid isolation system under different earthquake conditions.

4. The hybrid sliding isolation system proved to be quite robust, demonstrating the high potential for the application of the system to actual structures.

Items 3 and 4 above have not been discussed in this paper. The reader is refered to Ref. 4 for the detail.

6 ACKNOWLEDGEMENT

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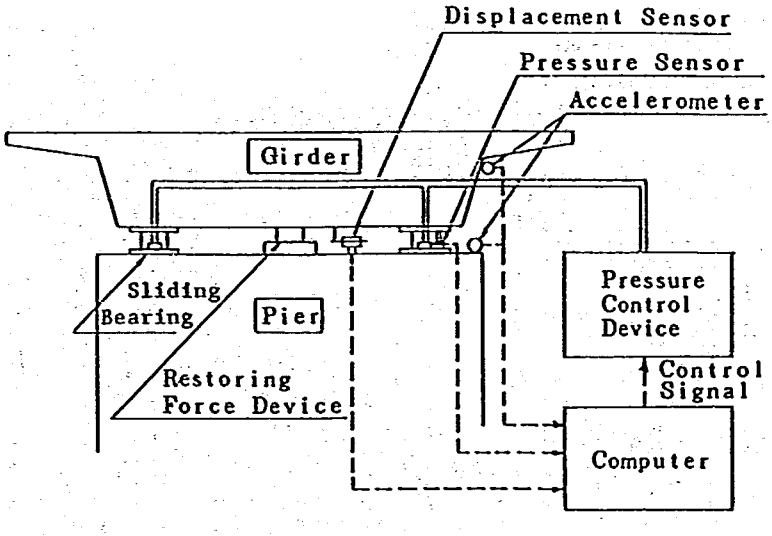


Figure 1: Concept of Hybrid Sliding Isolation System for Bridges

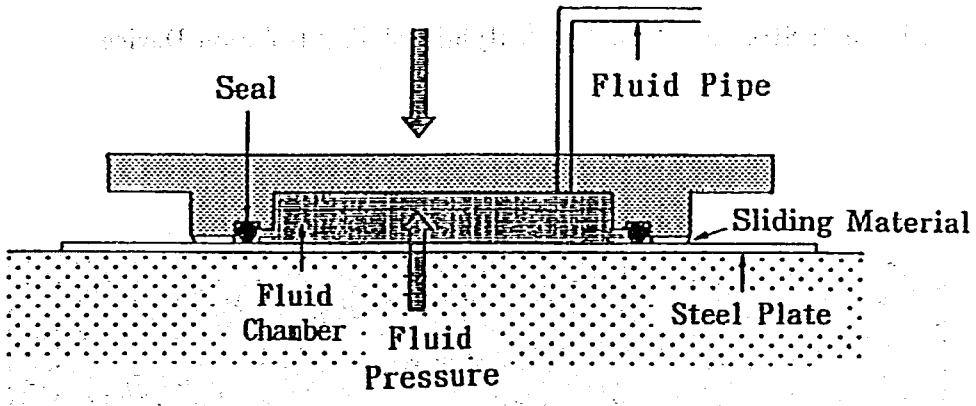


Figure 2: Idealized View of Friction Controllable Sliding Bearing

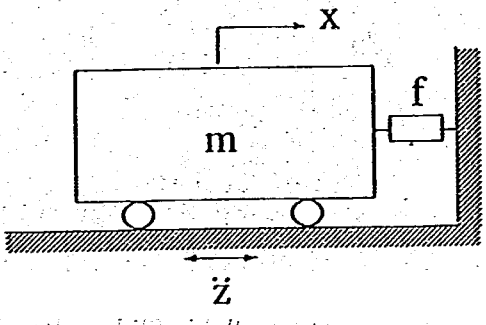


Figure 3: Analytical Model

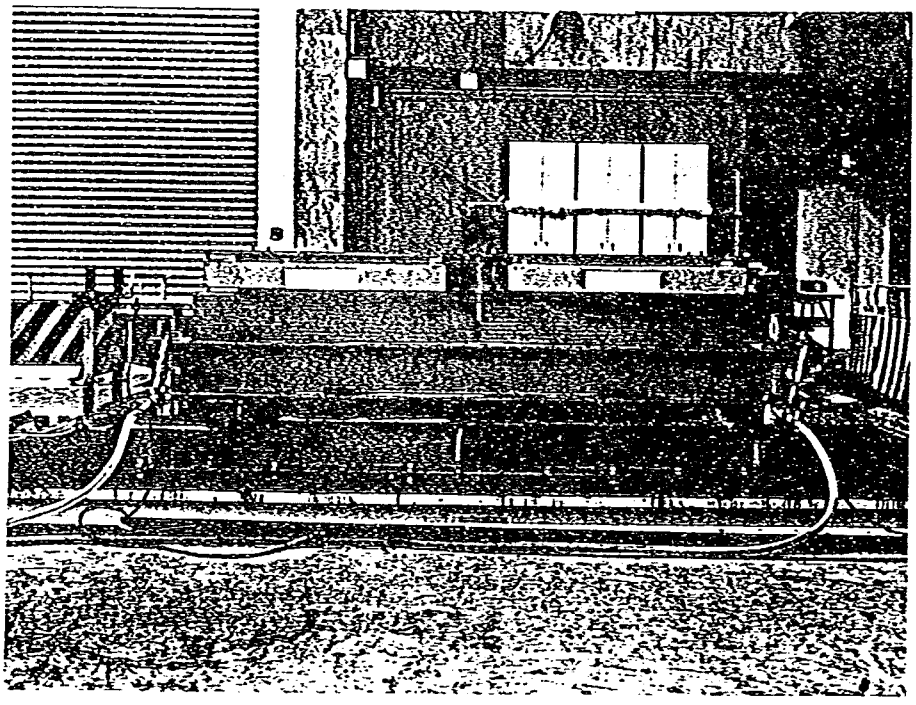


Figure 4: Structure Model with Hybrid Sliding Isolation Device

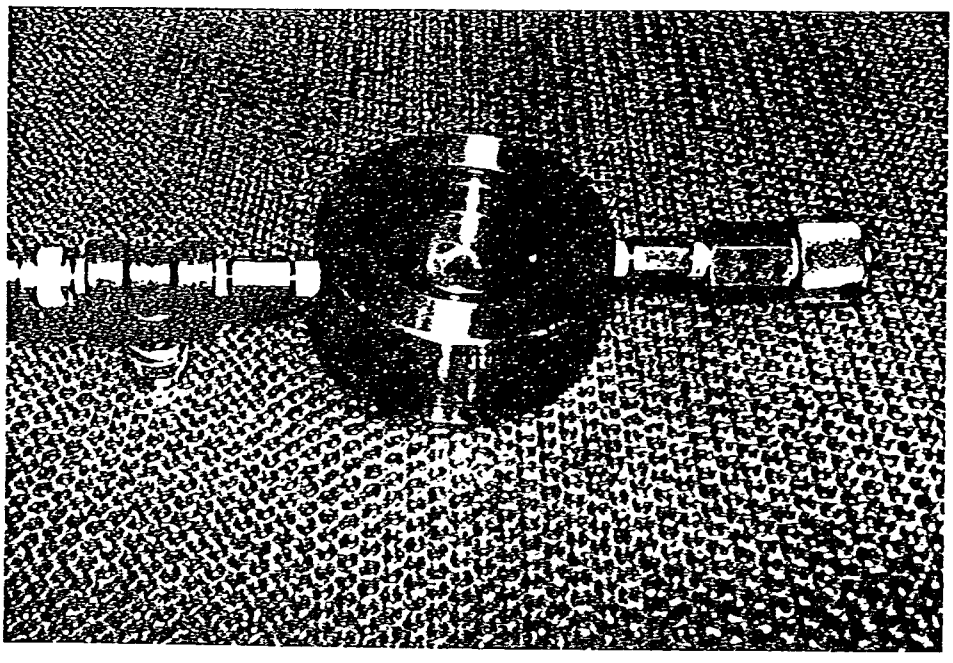


Figure 5: Friction Controllable Sliding Bearing

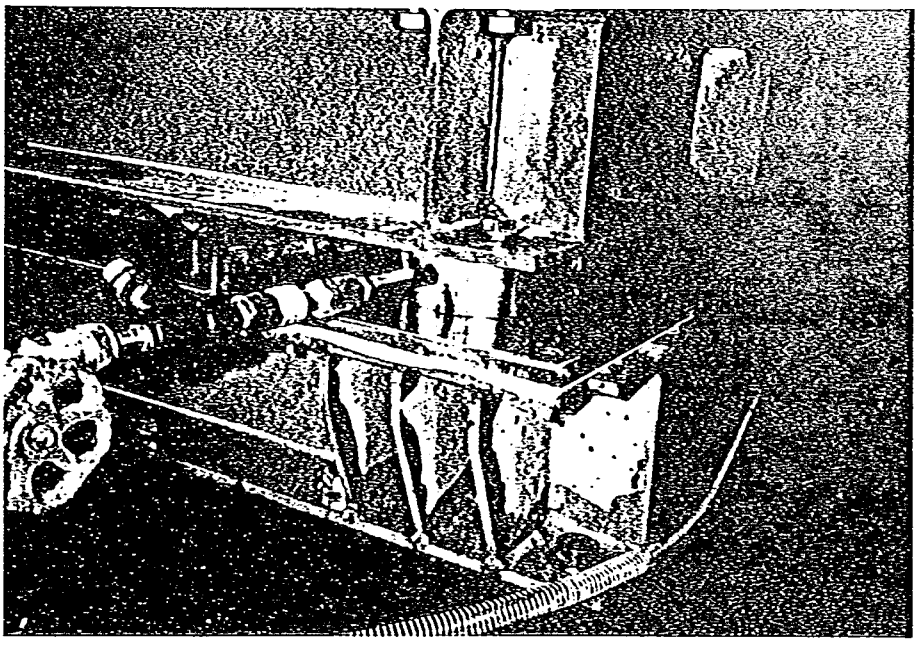


Figure 6: Friction Controllable Sliding Bearing for Experiment

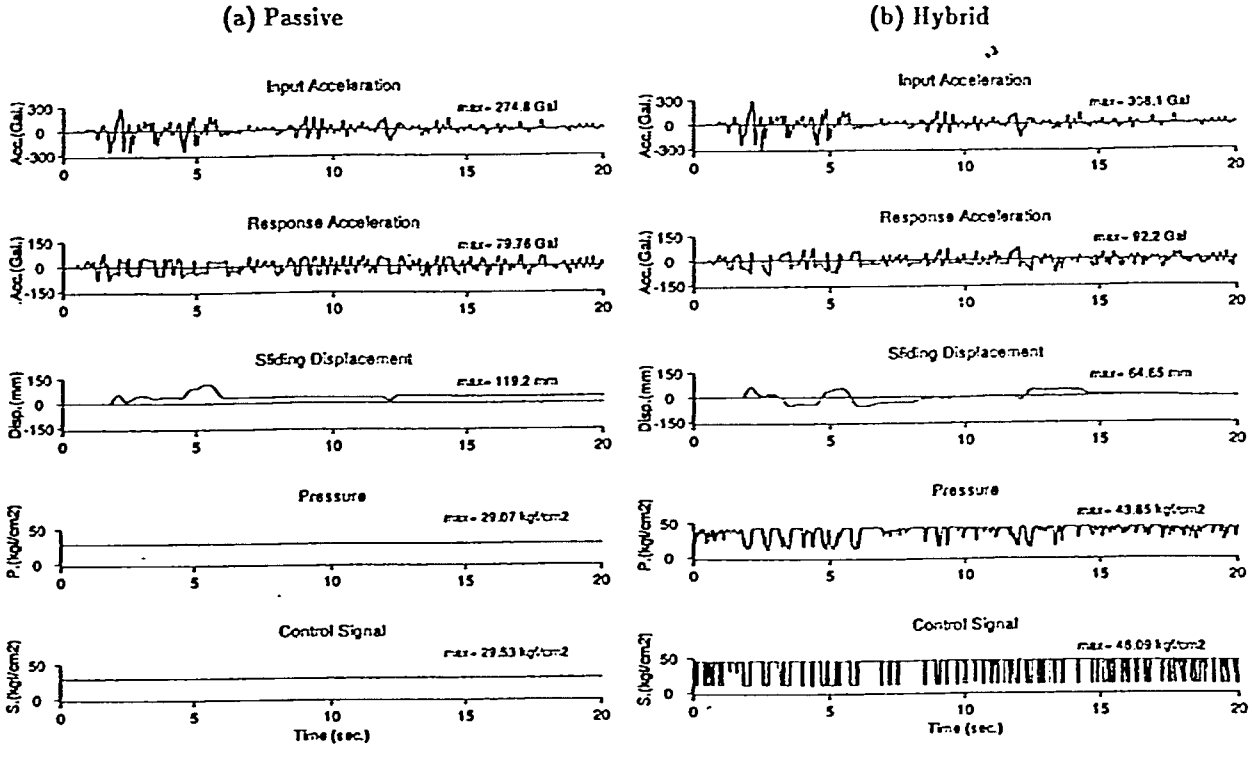


Figure 7: Experimental Results

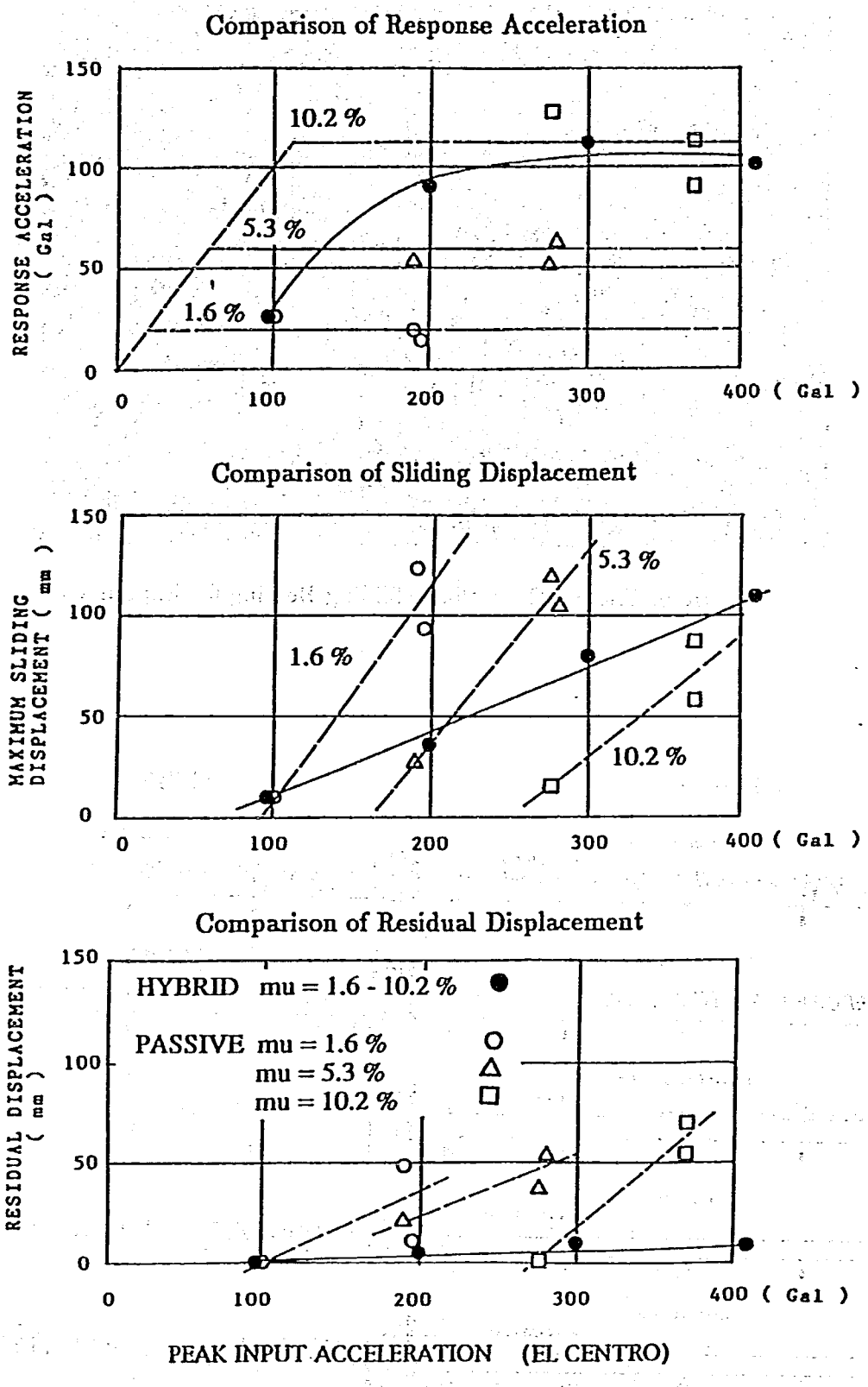


Figure 8: Comparison of Passive and Hybrid Isolation

日本における制振機構を有する建築構造物の現状とその動向

北川良和*1，緑川光正*2

【要約】
本論では我が国における制振構造，特にアクティブ，ハイブリード方式による制振構造の現状を紹介している．まず制振構造システムの分類を行い，引き続き制御理論と要求性能との関係及び制振機構を有する建築構造物の事例，結果の概要について述べている．

【キーワード】
制振構造システムの分類，構成，アクティブ・ハイブリッド制振構造，制御理論，要求性能，事例

1．はじめに

我が国は太平洋側で起こる大規模地震域，日本海岸側で起こる中規模地震域および内陸に発生する浅発生中規模地震域の真っ只中に存在するという地震活動の特殊性を有しており，建物の地震対策は不可欠となっている．一方最近の建物には，地震や強風に対する高度の安全性保持や機能維持，さらにより快適な居住性などが要求されている．この社会的要求を経済的に実現する有効な手段として，建物の制振構造化がある．その一部はパッシブ（受動的）制振として免震構造の形で実用化され[1・2]，現在種々の制振装置に関する技術開発が行われている．

行政的にもこの種の新技術，特にアクティブ（能動的）制振構造，ハイブリッド制振構造の健全な発展のために，制振構造技術を用いた建物の設計基準について社会的コンセンサスを得る必要がある．このような背景のもとで建設省では，振動制御技術を用いた建物の安全性評価法の検討を行うこと，建物の振動制御の実用化システムの開発に資することを目的として，官民連帯共同研究「建築構造物の実用化技術の開発」が平成2年度より3か年計画で開始された．

本論では，これまでの知見をもとに，制振構造のうちアクティブ制振・ハイブリッド制振を対象とした制振構造システムの分類構成，制御理論と要求性能，我が国における制振建物の現状，実在及び試験体制振建物の実例について概観する．

2．制振構造システムの分類と構成

制振構造は応答の抑制・制御に外部エネルギーを必要とするか否かによって分類される．外部エネルギーを必要とするアクティブ（能動的）方式には外乱に直接抵抗するために制御力としてエネルギーを供給するフルアクティブ方式，外乱に直接抵抗するのではなく，建築構造物の特性（パラメータ）を変化させるためにエネルギーを供給するセミアクティブ方式がある．外部エネルギーを必要としないパッシブ（受動的）方式には免震方式，TMD方式などがある．さらに両者を直列的または並列的に組み合わせること（異種混合）によってパッシブ制御の効果を高めるためにエネルギーを供給するハイブリッド方式がある．

これら制振構造のうちアクティブ方式は，建物の内部または外部にセンサ（計測器）を設置し，そこで得られる外乱情報(信号)により各種の制御を行うものである．センサが建物の外にあり，外乱を事前に感知し各種制御を行うフィードフォワード制御，センサが建物内にあり，建物の応答を感知し各種の制御を行うフィードバック制御がある．制振システムの一例をFig. 1に示す．

一般に地震・強風等の動的荷重が作用するとき建物の挙動は，以下のようになる．

$$m\ddot{\chi} + c\dot{\chi} + k\chi = f + p$$

ここで，m：振動系の質量
c：速度に依存する粘性減衰係数
k：変位に依存する復原力
f：系に作用する外乱（風による外力，地震による外力など）
$\ddot{\chi}$：地盤に対する相対応答加速度
$\dot{\chi}$：地盤に対する相対応答速度
χ：地盤に対する相対応答変位
p：制御力

これら各種記号はマトリックスまたはベクトル表示である．このうち制御構造により建物の応答を抑制または制御（一般に「低減」）する量として，応答加速度（$\ddot{\chi}$），速度（$\dot{\chi}$），変位（χ）がある．そのため質量m，減衰c，復元力k，外力fを制御・調整するか，または制御力pを導入することとなる．

このような動力学の基本原則に従って制振構造を分類すると，質量を制御・調整するものとして質量付加型.(剛体質量利用，液体質量利用など），減衰を制御・調整するものとして減衰調整型（減衰可変機構，隣棟間設置機構など），復原力を調整するものとして剛性調整型（剛体可変機構，脚部柔性機構など），制御力を導入する外力付加型（噴射，反力壁利用など）などに大別される．

制御力を導入する方式を除けば，制振構造独自のものではない．従来からの耐震・耐風構造においてもま

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ったく同様である．その意味では，制振構造と従来の耐震・耐風とは本質的に異なるものではなく，同一の原理上のものである．異なる点は，従来の耐震・耐風構造では，減衰・復原力などの調整を柱・梁・壁・筋かいなど従来からの構造部材の特性を調整することで行っていたのに対し，制振構造ではそれらの調整を，従来の構造部材とは異なるある種の装置・機構により行う点にある．

3．制御理論と要求性能[3),4)]

現在，主として制振に用いられている方法は，外乱によってゆれ始めた建物を元の無振状態に戻すこと（居住性の確保が中心）を目標とした加速度・速度・変位量などをフィードバックさせるフィードバック制御法である．

古典制御理論では，制御システムの設計段階で，固有振動数を外乱の卓越振動数とずらしたり減衰率を高めたりする簡単な制御則で振動効果を得ようとしている．一方，対象物の状態量を多数フィードバックするために現代制御理論が用いられている．これは制御対象の可制御性および可観測性に問題がある．これらの問題が解決されれば，制御システムの振動特性を自由に変化させることができる．さらに，現在は，最適レギュレータを含む最適制御が中心に行われている．

制御理論体系を階層制御システム理論に基づき分類すると，全体の目標を達成するための制御システムの構成を決定する自己組織型，与えられた構造の枠内で最適化すべき評価関数およびパラメータを適応的に決定する適応制御型，決められた評価関数を最適化するための操作量を決定する最適制御型，プロセス変数を設定値に一致させる操作量を直接求める直接型の４段階がある．これらの段階に内在する基準としては，対象とするシステムの複雑さ（単純さ），扱う情報の曖昧さ（確実さ）が考えられる．これら２つの要因を座標軸としたシステム制御理論のマッピング例をFig. 2に示す．

制振構造システムの設計では，対象建物の振動特性を十分に把握することが重要である．また制御システム内部にフェールセーフ機構等を兼ね備えていることが信頼性向上のために不可欠であり，環境に応じて制御に必要なパラメータを変化させる適応制御を用いた適応型フェールセーフシステムの開発が望まれている．

建物の場合，構造・材質・施工・強度の経年変化など未知変数が多く，制御システムを構築するのが難しい．また地震動を制御する場合，地震動をセンサでとらえてから，制御回路に入力信号が届きアクチュエータを作動させるまでのシステムレスポンスが速いことが必要となる．しかし，地震波動の伝播方向やセンサの設置状況などにより，入力信号にはすでにさまざまな不確定要素が含まれている．このように曖昧で不確定性の高い問題を扱うものに知的制御（ファジイ理論，ニューラルネットワークなどを用いたもの）が最適であり，これからの発展と応用化が期待される．構造安全性評価上適用可能な要求される事項と制御理論との関係の一例をFig. 3に，ファジイ最適化制御手法を用いた制御システム例をFig. 4に示す．このシステムの最大の特徴は，ファジイ理論を応用してリアルタイムの地震動予測を行い，リアルタイムの構造同定と居住性，安全性を満たす目標応答，経済性，技術性を満たす目標制御変数とを合わせて建物応答関数を求め，ファジイ最大化決定手法により最適制御変数を決定するところにある．これはＰＩＤ（Proportional, Integral and Differential）制御的なファジイ制御手法と根本的に異なる．

4．制振機構を有する建築構造物の事例

建物における制振技術（アクティブ，パッシブ）の実用への適用をTable 1に示す[5)]．

アクティブ，ハイブリッド制御技術は研究開発中のものが殆どであり，実建物への適用例は現段階で４例である．このうちＫＳビル（1989）については，日本建築センター内に設置された特別評定委員会で制振装置の付加に伴う構造安全性及び維持管理について技術的検討が行われた．

Table 1に示されたアクティブ，ハイブリッド制振建物，および実用化段階にある代表的な模型試験体の事例を以下に示す．

4.1 ＫＳビル(Ref. 6, Figs5, 6 and 7)

１１階Ｓ造制振建物は事務所棟として東京都中央区に1989年建設された．比較的頻度の高い中小地震（震度Ⅳ程度）と強風に対して，建物の並進・捩れ応答を低減させ，居住性，使用性を向上させることを目的としている．

制振機構はＡＭＤ（Active Mass Damper）方式で，付加質量は建物重量の約１％，制御効果は最大加速度と最大変位を無制御時の0.5～0.7程度となることを目標としている．なお，過大入力時には制御装置は停止する．これまでに自由振動実験，強制振動実験が実施され，現在地震・風観測が実施されており，貴重な記録が数多く得られている．

4.2 Ｋ研究実験棟(Ref. 7, Figs8, 9, and 10)

３階建Ｓ造制振建物は実験棟として東京都調布市に1990年建設された．制振機構はＡＶＳ（Active Variable Stiffness）方式である．これは鉄骨フレームと

鉄骨ブレースとの接合部にシリンダーロック装置を介在させたもので，コンピュータ制御により装置を固定・解放させることにより，建物の剛性・周期を変えることが出来る．その結果大地震（250gal程度）に対する建物の共振を避けることが出来る．これまでに強制振動実験が実施され，現在地震観測が実施されている．

4.3 Sビル(Ref. 8, Figs11, 12, and 13)

１１階建てS造制振建物は事務所棟として東京都渋谷区に1991年建設された．比較的頻度の高い中小地震（100gal程度）と強風に対して建物の並進・捩れ応答を低減し，居住性を向上させることを目的としている．制振機構はAMD方式である．付加質量は建物重量の1.1%で，制御効果は最大変位を無制御時の0.4～0.8程度となることを目標としている．なお，過大入力時には制御装置は停止する．

4.4 MMタワー(Ref. 9, Figs14, 15, and 16)

70階建S／SRC造制振建物は事務所・ホテル棟として神奈川県横浜市に建設中で，1993年に完成される．年間１～２回程度起こる強風に対して建物上層部にあるホテルの居住性を改善することを目的としている．制振機構は多段振り子式でハイブリッド方式（Tuned Active Damper）である．付加質量は建物重量の0.13%で，制御効果は最大加速度を無制御時の0.4 程度となっていることを目標としている．なお，過大入力時には制御装置は停止し，装置はパッシブ化される．

4.5 実大試験体(Ref. 10, Figs17, 18 and 19)

６階建S造実大試験体は東京都江東区に1989年建設された．制振機構はAMD方式で，付加質量は建物重量の１%，制御効果は最大変位を無制御時の0.3～0.5程度となることを目標としている．これまでにAMD装置の振動台実験，常時微動測定，強制振動実験，地震・風観測が実施されており，貴重な記録が数多く集録されている．なお，このシステムは実際の制振建物に用いられている（3）Sビル参照）．

4.6 実大試験体(Ref. 11, Figs 20, 21 and 22)

７階建S造実大試験体は東京都江東区に1991年建設された．制振機構はHMD（Hybrid Mass Damer）方式で，多段積層ゴムを用いたパッシブ型TMDにACサーボモータによりアクティブに制御力を発生させるものである．付加質量は建物重量の約１．２%，制御効果は最大加速度を無制御時の 0.5程度となることを目標としている．これまでにHMDの装置の振動台実験，強制振動実験，地震・風観測が実施されており，貴重な記録が数多く得られている．なお，このシステムは強風に対して建物の並進・捩れ応答を低減し，居住性を向上することを目的として，50階建S造実大建物（大阪市弁天町，ORC２００シンボルタワー，賃貸住宅・ホテル棟）に適用される予定である．

4.7 実大試験体(Ref. 12, Figs 23, 24 and 25)

６階建S造実大試験体は茨城県つくば市に1991年建設された．制振機構はAMD／TMD（Active and Passire Mass Damper）方式である．付加質量は建物重量の約１%，制御効果は最大加速度を無制御時の0.5程度となることを目的としている．制御装置は中小地震，強風に対してAMD方式，大地震に対してはTMD方式となるよう自動的に方式が切り替わり制振効果が継される．又停電時にはTMD方式となる．これまでに強制振動実験，地震・風観測が実施されており貴重な記録が数多く得られている．

4.8 実大試験体(Ref. 13, Figs 26, 27 and 28)

６階建S造実大試験体は茨城県つくば市に1991年建設された．制振機構はTMD(Tuned Roller-Pendulum Damper)方式である．これまでに自由振動実験，強制振動実験，地震・風観測が実施されており，貴重な記録が得られている．

一方可変剛性と可変減衰機構を組み合わせたセミアクティブ方式による可変抵抗力ダンパー（Variable Resistanse Damper）の振動台実験が実施されており，貴重な記録が得られている．基礎免震構造建物への適応が検討されている．

4.9 8階建S造縮小試験体(Ref. 14, Figs 29, 30 and 31)

制振機構はAMD方式で，付加質量は試験体重量の0.4%，制御効果は最大変位，最大加速度を無制御時の0.4～0.8程度となることを目標としている．

これまでにスピルオーバ，ロバスト性モデルの低次元化を考慮した振動台実験等，実験的，解析的検討が実施されており，貴重な記録が数多く得られている．

4.10 4階建S造縮小試験体(Ref. 15, Figs 32, 33 and 34)

制振機構はAMDおよびATMD(Active Tuned Mass Damper)方式で，付加質量は試験体の約１%，制御効果は最大応答変位，加速度を無制御時の0.6程度となることを目標としている．これまでに振動台実験が実施されており，貴重な記録が得られている．その他ジャイロ機構を用いた制振装置，可変摩擦支承の開発が行われており，実用化へ向けて検討されている．

4.11 4階建S造縮小試験体(Ref. 16, Figs 35, 36 and 37)

制振機構はAMD方式で，付加質量は試験体の約１%，制御効果は最大変位を無制御時の0.3～0.5程度となることを目標としている．なお，過大入力，停電時

には，付加質量である回転体が振り子作用によって，パッシブ型制振構造となる．これまでに振動台実験が実施されており，貴重な記録が得られている．

4.12 10階建S造縮小試験体(Ref. 17, Figs 38, 39 and 40)

制振機構はHTMMD(Hybrid Tuned Mulli-Masses Damper)方式である．試験体の上層2階を免震構造とし，8層と9層都の間にある免震層にコンピュータ制御による補助付梃子機構を設計することによってTMMD機構のハイブリッド化を行い，制振効果を高めている．TMMD構造部及び補助質量は各々試験体重量の約25,5%で最大変位を無制御時の0.5～0.8程度となることを目標としている．これまでにHTMMD方式による振動台実験が実施されており，貴重な記録が得られている．

5．おわりに

本論では日本における制振，特にアクティブ，ハイブリッド制振建物の現状を紹介した．この種制振構造は一部実際の建物へ実用化されているものの，居住性改善の観点から中小地震，強風といった外乱対策として主として民間企業を中心に，実用化開発が行われている．実務という観点から主として駆動装置の開発が中心となり，これによって制振構造システムの性能が決まるといっても過言ではない．動作信頼性の観点から構造安全性に対して社会的コンセンサスが得られたとき新しい構造設計体系が可能となる．このとき制振構造システムを含む建物全体，すなわち建物系，機械系，環境系を総合したトータルシステムとしての構造安全性にかかわる動作信頼性を確保するが肝要といえよう．

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Table 1 List of Seismic Response Control Building Structure (Active/Passive Control Method)[1]

Building Name	Date Completion	Building Structure Story		Building Uses	Location	Designer/ Construction	Type of Seismic Control Device
Chiba Port Tower	1986	S	(125m)	Observatory	Chiba-shi, Chiba	Nikken Sekkei/ Takenaka	Tuned Mass Damper (TMD)
Yokohama Marine Tower	1987	S	30F	Observatory	Yokohama-shi, Kanagawa	Shimizu/ Shimizu	Tuned Liquid Damper(TLD)
Sonic City	1988	S SRC	31F4B	Office	Oniya-shi, Saitama	Nikkeen Sekkei/ Fujita	Friction Damper
Cold Tower	1988	S	(136m)	Observatory	Utazu-cyo, Kagawa	Mitui/ Mitui	Tuned Liquid Damper(TLD)
Higashiyama Sky Tower	1988	S	7F (134m)	Observatory	Nagoya-shi, Aichi	Nagoya City & Nihon Sogo Arch./ Kajima, Taisei, Tokuso	Tuned Mass Damper (TMD)
Asahi Beer Tower	1989	S	22F2B	Office	Taito-ku, Tokyo	Nikken Sekkei/ Kumagai	Friction Damper
Fukuoka Observatory	1989	S	(148m)	Observatory	Fukuoka-shi, Fukuoka	Nikken Sekkei/ Taisei, Takenaka, Kajima, Shimizu, Obayashi	Tuned Mass Damper (TMD)
Kajima Corp. KI-Building	1989	SRC	5F(A) 9F(B) 1B	Office	Minato-ku, Tokyo	Kajima/ Kajima	Joint Damper(Ball Damper)
Kyobashi Seiwa Building (IS Building)	1989	S SRC	11F1B	Office	Chuo-ku, Tokyo	I & I Architects & Associates Kajima/ Kajima	Active Mass Driver(AMD)
Sky Tower	1989	S	(135m)	Observatory	Nagasaki-shi, Nagasaki	Mitsubishi Heavy Industry/ Mitsubishi Heavy Industry	Tuned Mass Damper (TMD)
Crystal Tower	1990	S SRC RC	37F2B	Office	Osaka-shi, Osaka	Takenaka/ Takenaka	Tuned Mass Damper (TMD)
Control Bldg. of Shaking Table Facility in Kajima Ins. of Construction Technology(K Lab Building)	1990	S	3F	Office	Cyofu-shi, Tokyo	Kajmai/ Kajima	Active Variable Stiffness (AVS)
Fujita Corp. Head Office Bldg.	1990	S SRC	20F4B	Office	Shibuya-ku, Tokyo	Fujita/ Fujita	Lead Damper
Oujiseishi Headquarters Bldg.	1991	S	15F4B	Office	Chuo-ku, Tokyo	Ouji-Kajima JV Design Kaca/ Kajima: Takenaka	Honeycomb Damper
Shimizu Head Office Bldg.	1991	S SRC	24F2B	Office	Minato-ku, Tokyo	Shimizu/ Shimizu	Viscous Damper
Hibikiryokuchi Sky Tower	1991	S	(135m)	Observatory	Kitakyushu-shi, Fukuoka	Mitsubishi Heavy Industry/ Mitsubishi Heavy Industry	Tuned Mass Damper (TMD)
Sendagaya INTES (S Building)	1991	S	11F1B (45m) 1B	Office	Shibuya-ku, Tokyo	Takenaka/ Takenaka	Active Mass Driver(AMD)
MM21 Land Mark Tower (MM Tower Building)	1993	S SRC	70F (296m)	Hotel & Office	Yokohama-shi, Kanagawa	Mitsubishi Estate Co./constructor JV	Tuned Active Damper

F: Floor, B: Basement, H: Height(m), — : Active Seismic Control Device

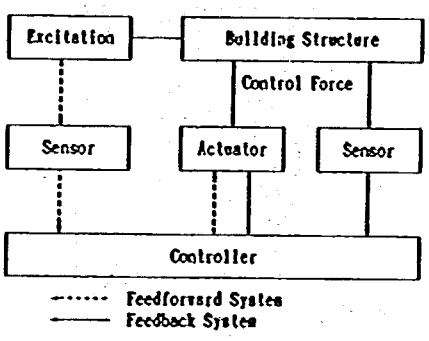


Fig. 1 Flowchart of An Active Structural Control System

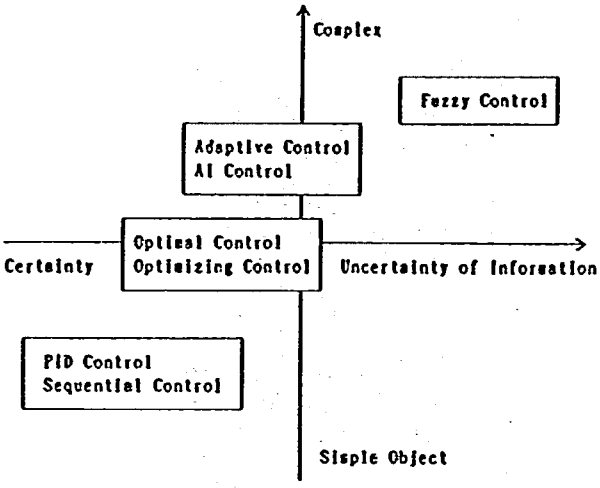


Fig. 2 Mapping of Control Theory Classification

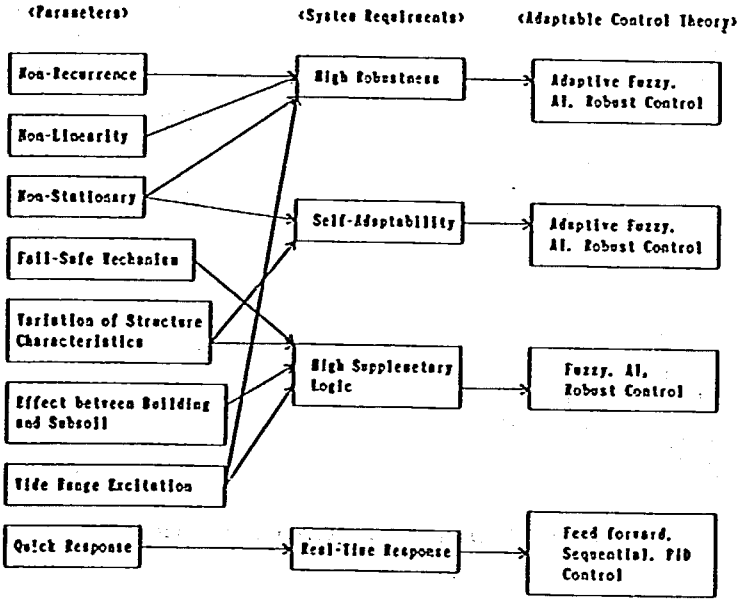


Fig.3 Relationship between Control Theory and Requirement

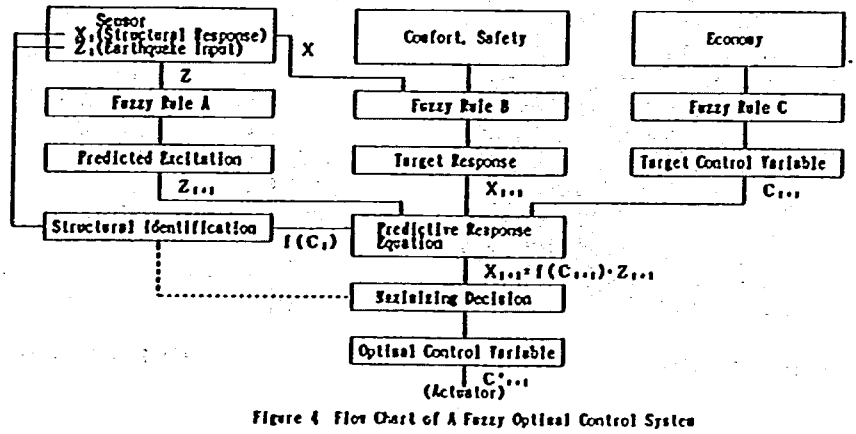


Figure 4 Flow Chart of A Fuzzy Optimal Control System

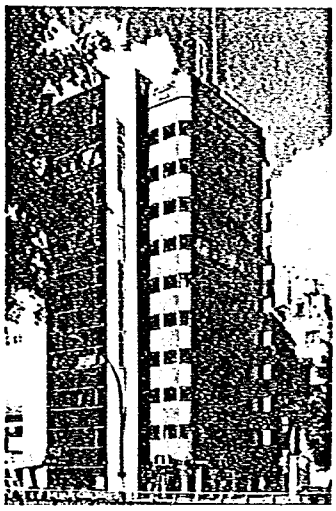
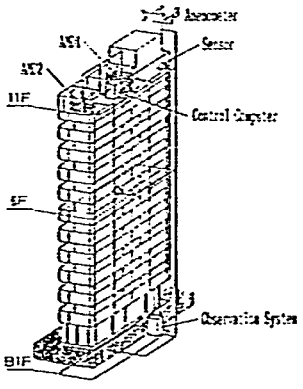


Fig. 5 Entire View of Building



Fig. 6 AMD Device

Realization of Active Mass Driver(AMD) System in 1989 (Kyobashi Seiwa Building in Tokyo)



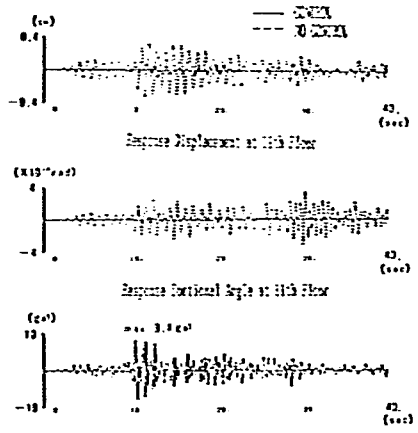


Fig. 7 AMD System and Earthquake Observation

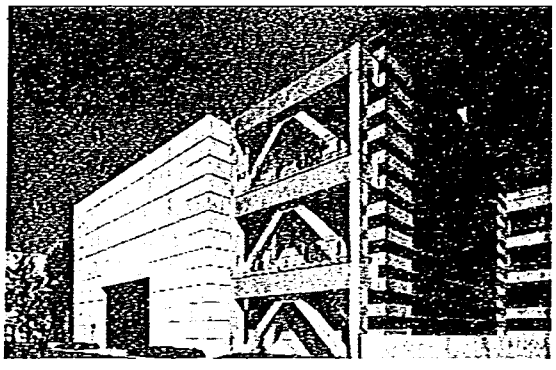


Fig. 3 Entire View of Building[1]

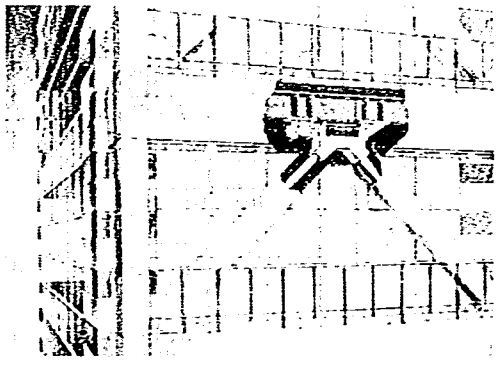


Fig. 9 AVS System

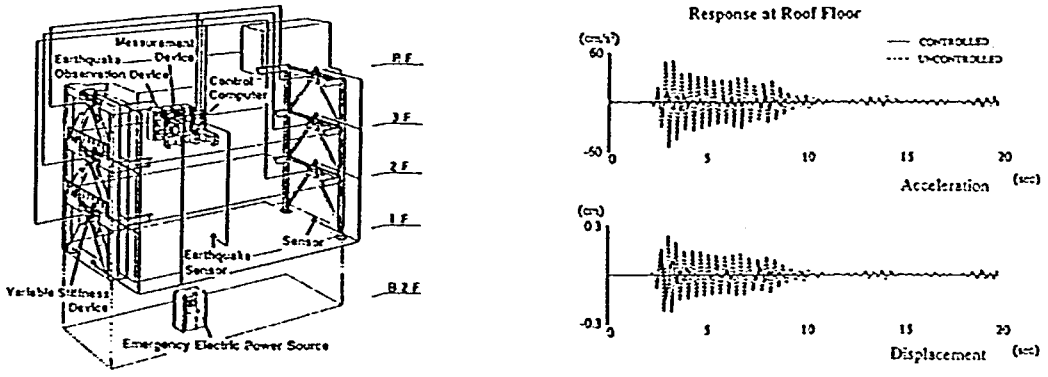


Fig. 10 AVS System and Comparison of Response

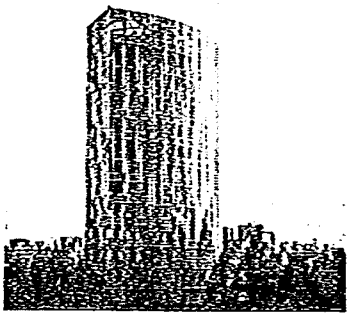
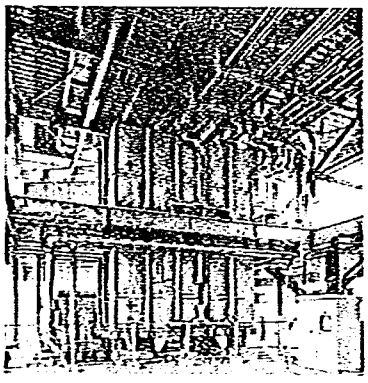


Fig. 11 Entire View of Building



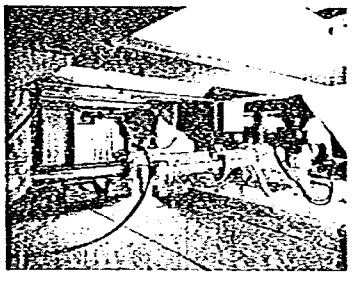


Fig. 12 AMD Device

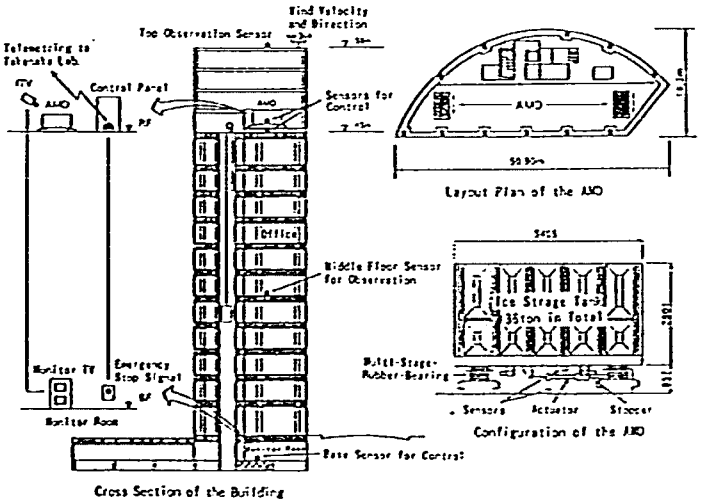


Fig. 13 Outline of AMD System

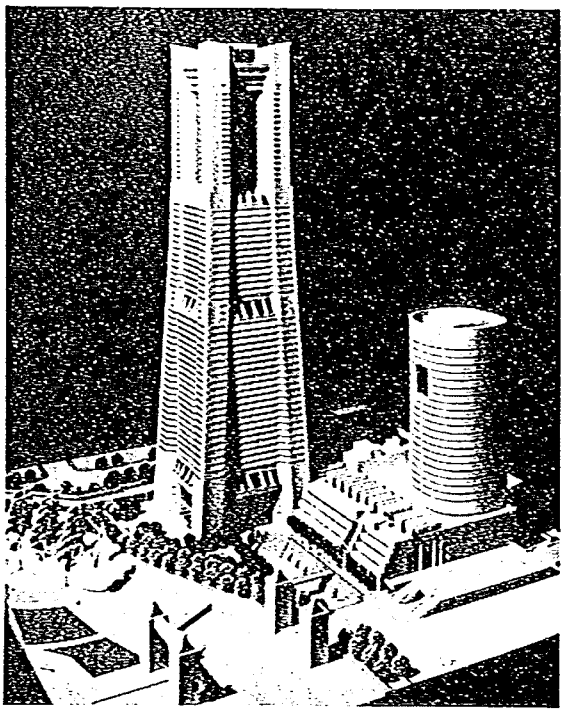
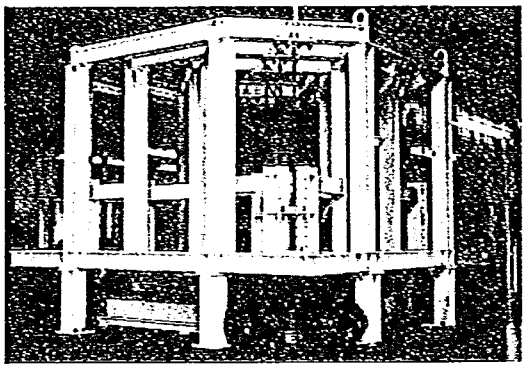


Fig. 14 Entire View of Building Model



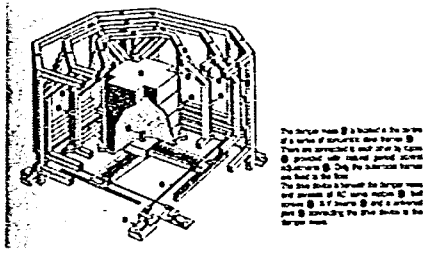
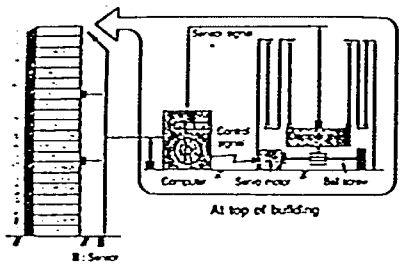


Fig. 15 TAD Device



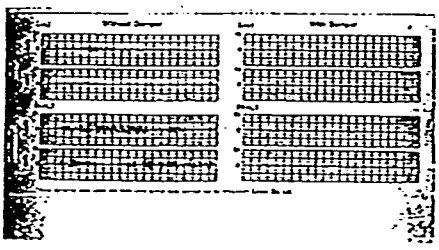


Fig. 16 TAD System and Comparison of Response

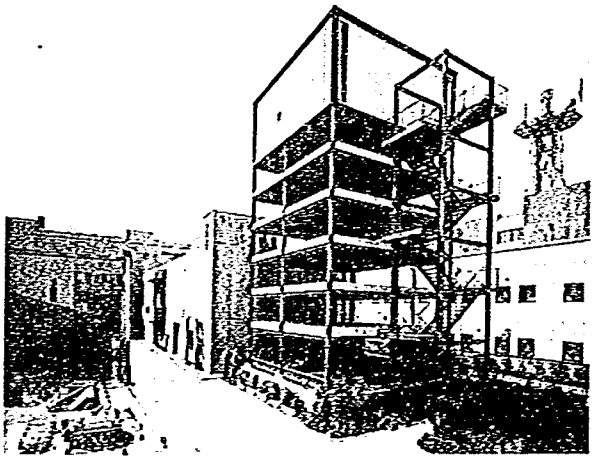


Fig. 17 Entire View of Actual Model

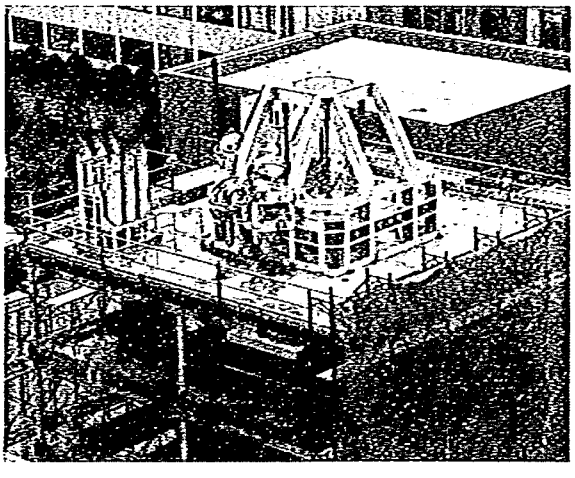
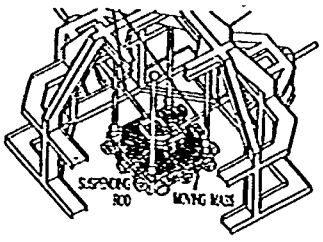
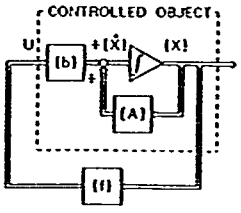


Fig. 18 AMD Device





$$\{\dot{X}\} = [A]\{X\} + \{b\}U$$

$$U = -\{f\}^T\{X\}$$

$$\{X\} = \begin{Bmatrix} X \\ \dot{X} \end{Bmatrix}$$

{X} : STATE VECTOR

{f} : FEEDBACK COEFFICIENT VECTOR

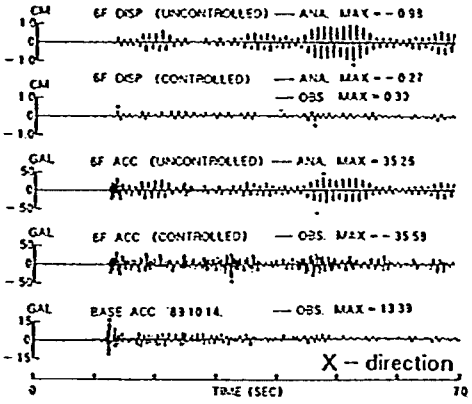


Fig. 19 AMD System and Comparison of Response

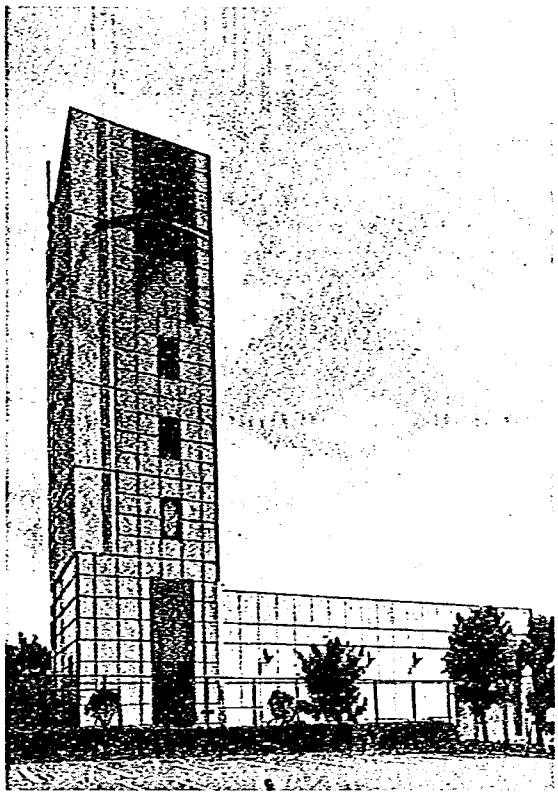
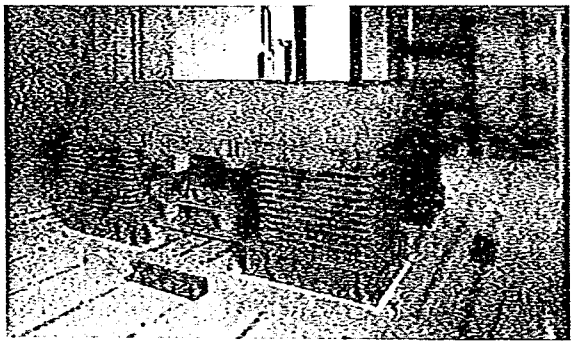


Fig. 20 Entire View of Actual Model



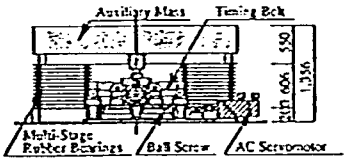
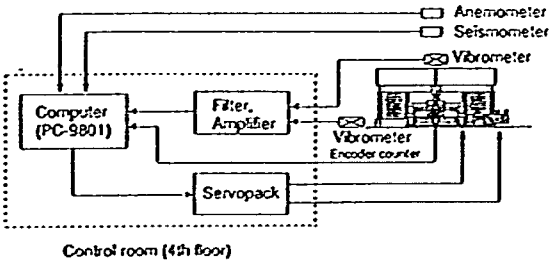


Fig. 21 AMD Device



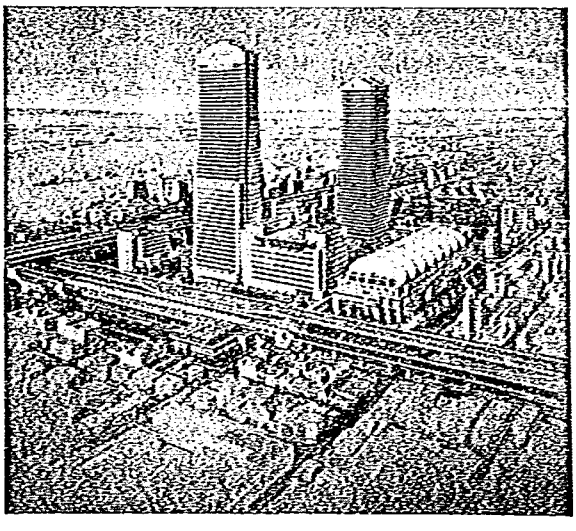


Fig. 22 AMD System and Entire View of Building in Future

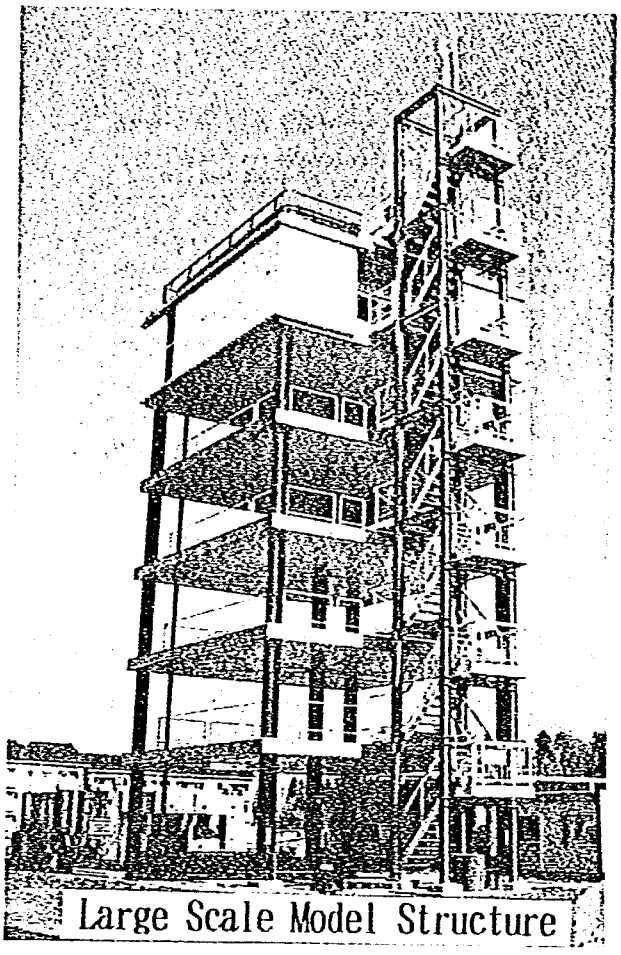


Fig. 23 Entire View of Actual Model

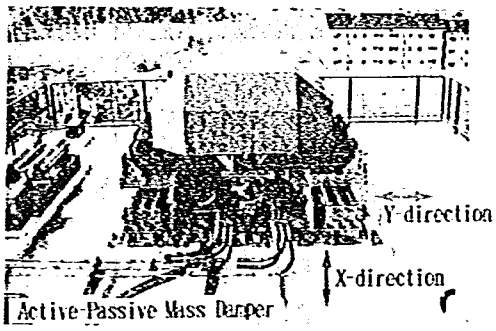


Fig. 24 Active Passive Mass Damper

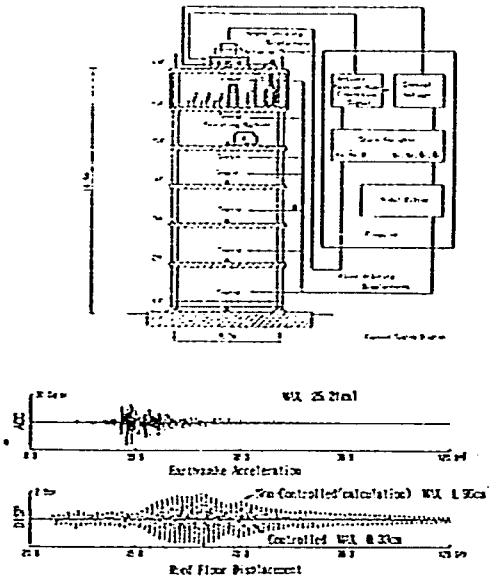


Fig. 25 APMD System and Comparison of Response

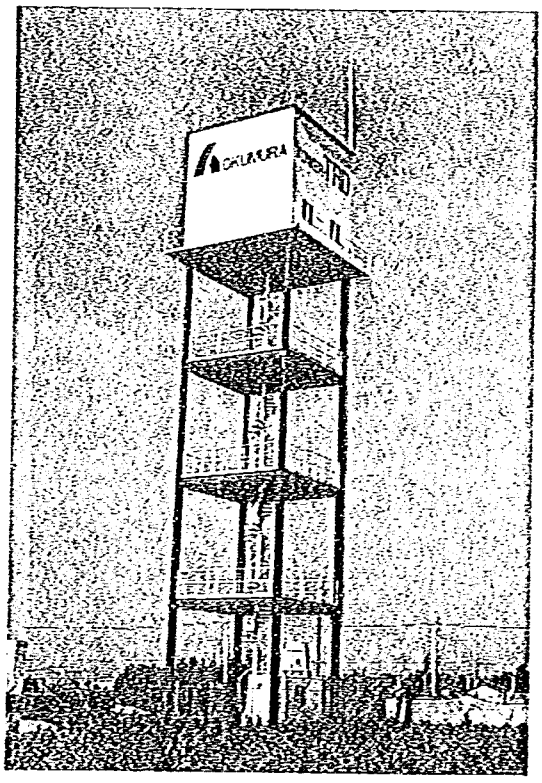
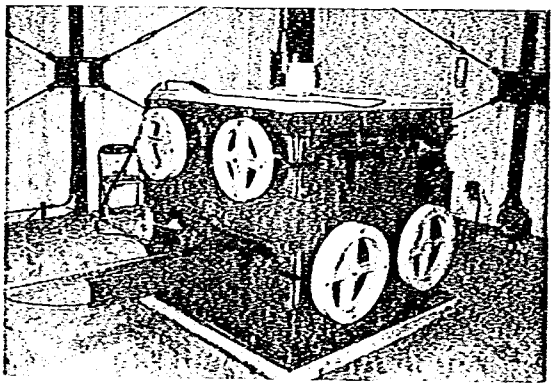


Fig. 26 Entire View of Actual Model



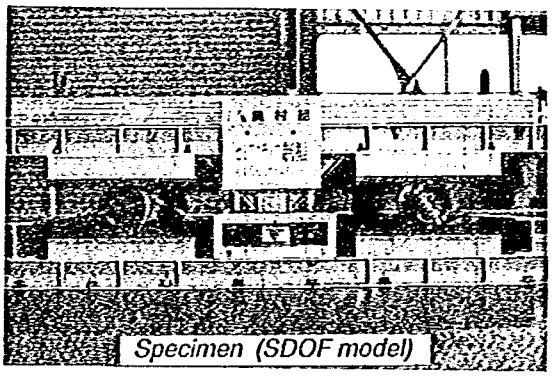


Fig. 27 TMD, VED Divices

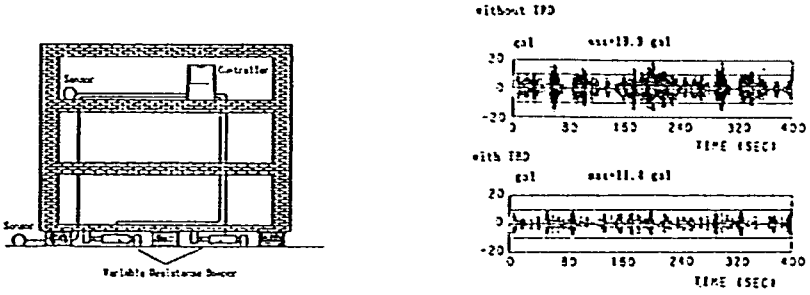


Fig. 28 Application of Semi-Active Vibration Control System and Acceleration Response of IMD System

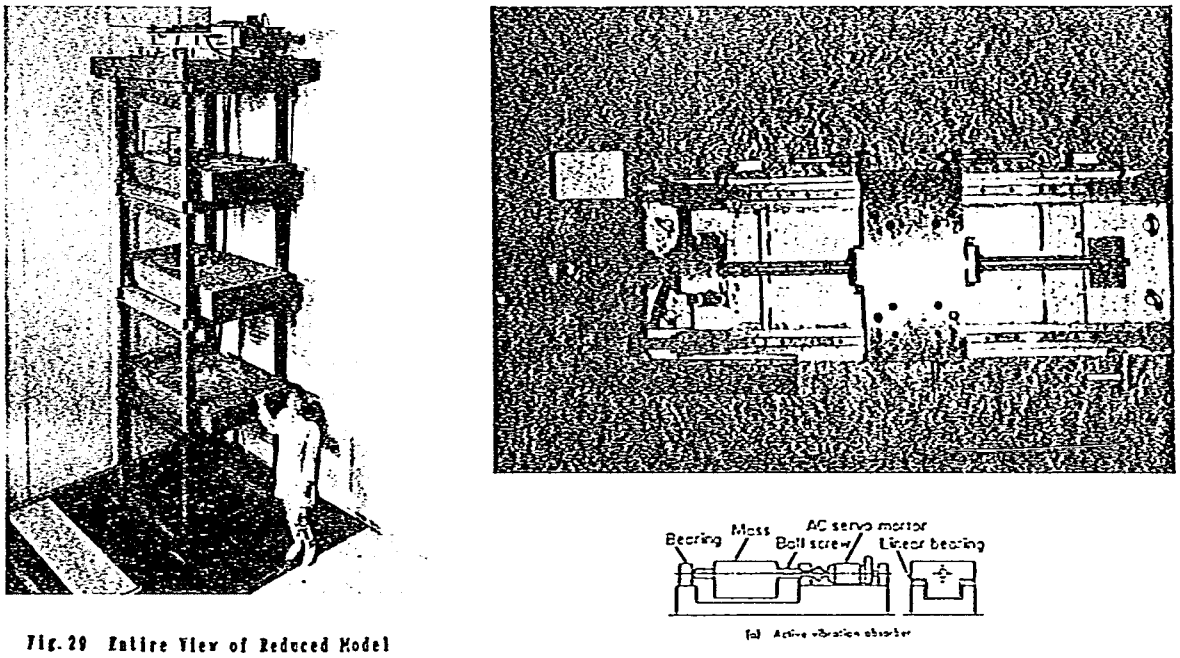


Fig. 29 Entire View of Reduced Model

Fig. 30 AMD Device

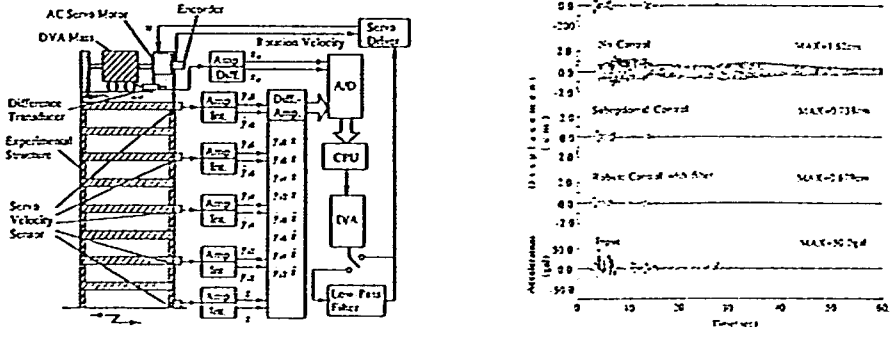


Fig. 31 AMD System and Comparison of Response

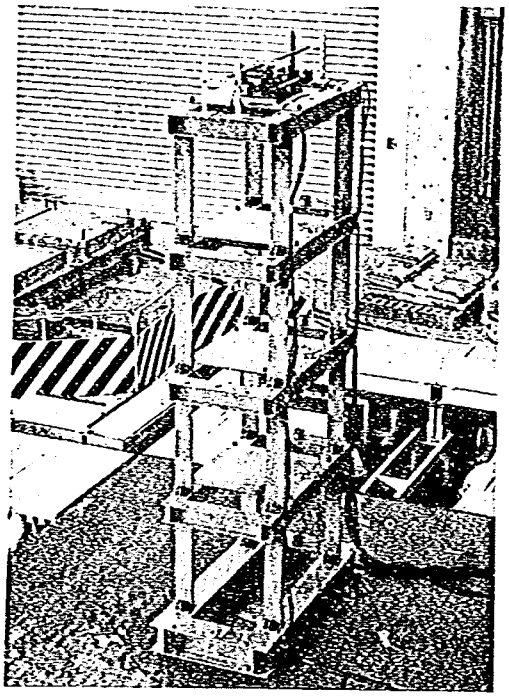


Fig. 32 Entire View of Reduced Model

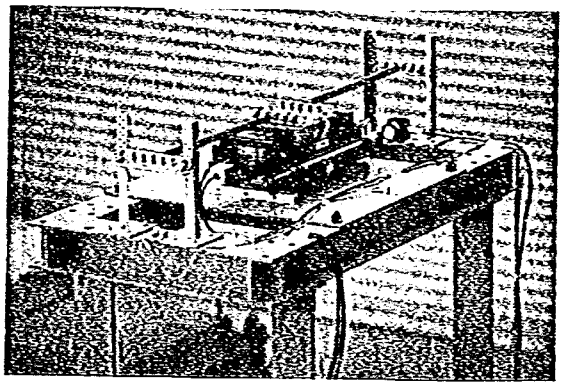


Fig. 33 AMD Device

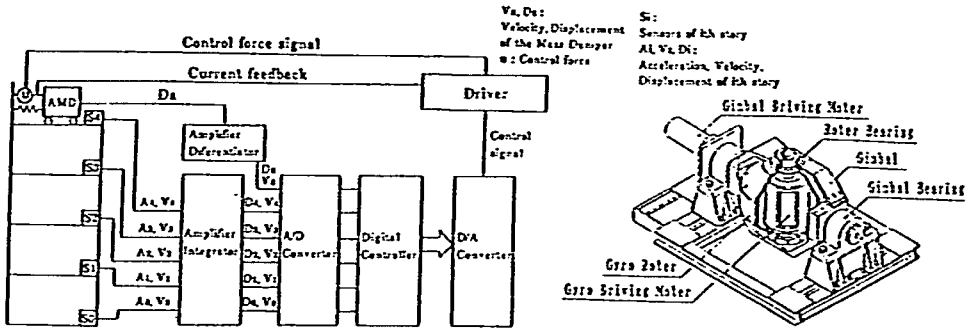


Fig. 34 AMD System and AMD Device using Gyrostabilizer

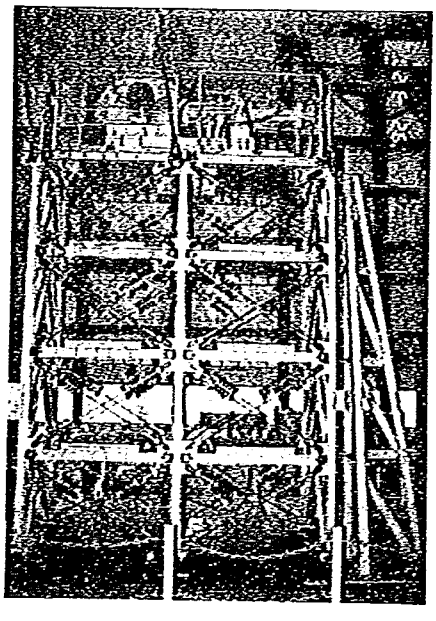


Fig. 35 Entire View of Reduced Model

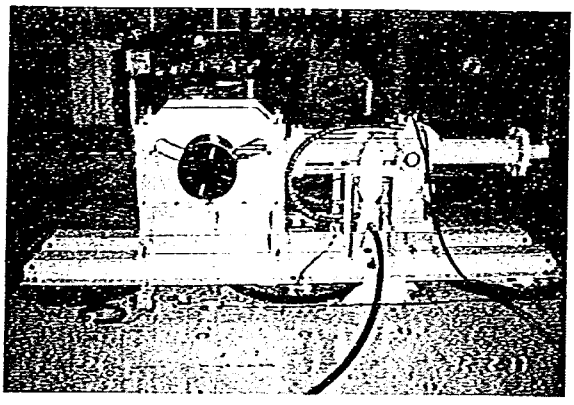


Fig. 36 AMD Device

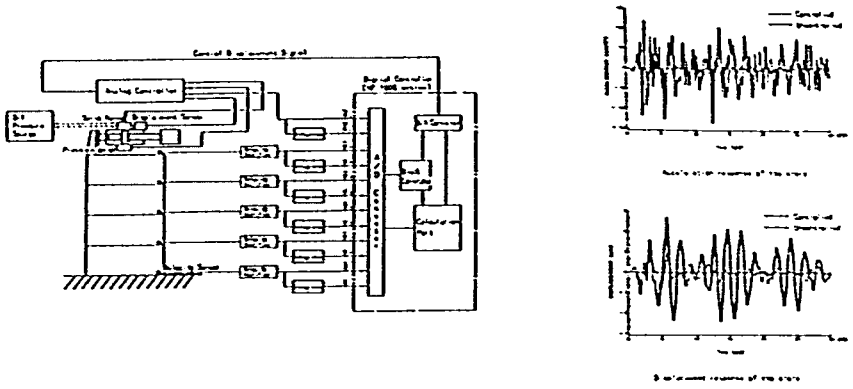


Fig.37 AMD System and Comparison of Control

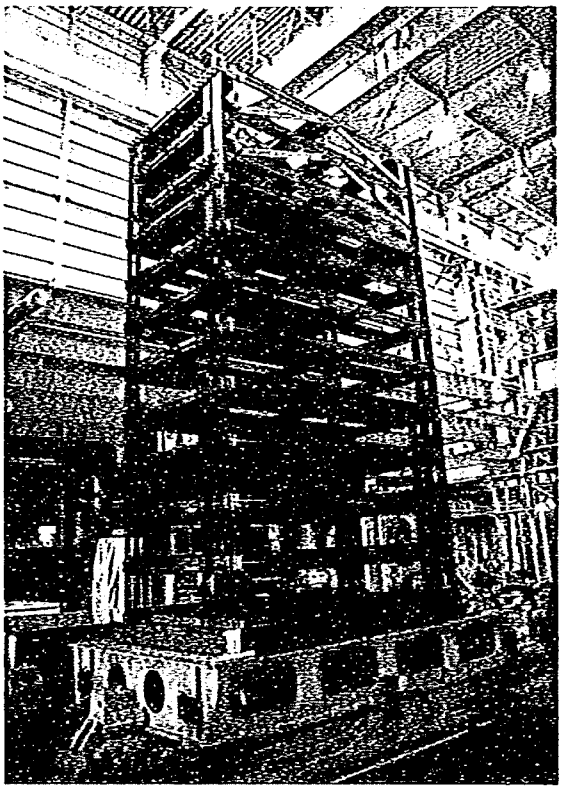


Fig.38 Inline View of Reduced Model

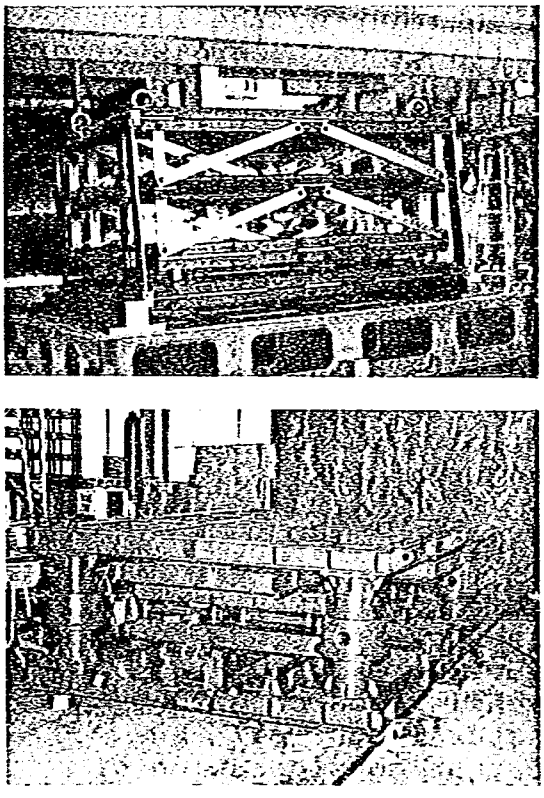


Fig.39 HTMMD Device

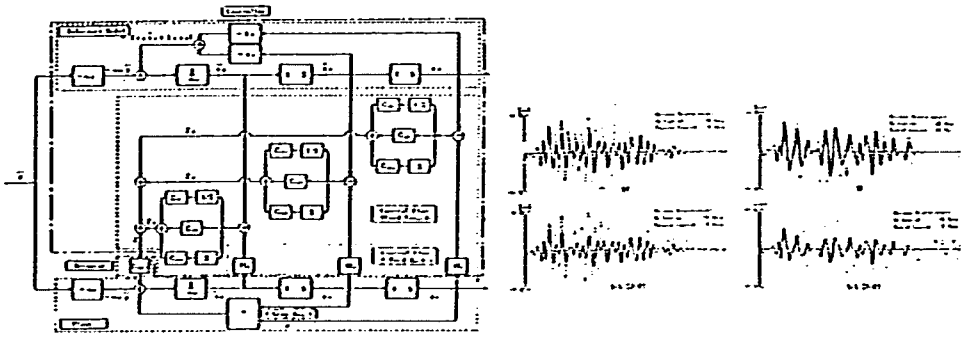


Fig.40 Boick Chart and Comparisn of Control

Status of NSF Structural Control Research Programs

by

S. C. Liu[1], H. J. Lagorio[1], and K. P. Chong[1]

ABSTRACT

This paper reports the status of a major research initiative in Structural Control launched in 1991 by NSF. Proposals submitted during the first year (in Fiscal Year 1992) of a 5-year program are analyzed. The proposal review process is described. Technical implications of intelligent control systems and smart materials as applied to civil infrastructure systems are discussed.

KEYWORDS: Structural control, infrastructure systems, smart materials, intelligent structures.

1. INTRODUCTION

Starting from 1989, an intensive effort has been made to develop a long-term national research program in structural control. An important consideration of such a program is to foster, on a coordinated basis, multidisciplinary research and development of passive, active and hybrid control technology for application to structures and nonstructural building systems under dynamic loads. Collaborative efforts among researchers in interdisciplinary areas involving academics, industrial experts and professional practitioners are considered extremely essential.

The important technical events taken place in this program development effort are:

* Establishment of the U.S. Panel on Structural Control Research, 1989.
* International Workshop on Intelligent Structures, Taipei, Taiwan, July 1990.
* NSF-EPRI Joint Workshop on Intelligent Control Systems, October 1990.
* U.S. National Workshop on Structural Control Research, University of Southern California, October 1990.
* NSF Workshop on Sensors and Signal Processing for Structural Control, February 1991.

PROGRAM INITIATION

Following these events, a firm research program base is established. In the summer of 1991, NSF officially announced the "NSF Research Initiative on Structural Control for Safety, Performance, and

[1] Program Directors, Engineering Directorate, National Science Foundation, Washington, DC 20550, USA

Hazard Mitigation" - a collaborative effort with the Strategic Highway Research Program (SHRP) of the National Research Council (NRC).

The fundamental objectives of the program initiative are:

- encourage innovations in passive, active, and hybrid control systems.
- advance sensor and actuator technology and signal processing techniques and their applications.
- develop intelligent robots, other devices and computer systems for structural safety.
- develop innovative systems for energy absorption, added damping, and variable stiffness.
- study the robustness and system reliability of control systems.
- develop guidelines, standards and practical approaches and techniques for design, fabrication and field installation.
- investigate emerging technology subjects in innovative structural systems, and smart materials, sensors, and devices for detection, sensing, monitoring, and diagnosis.

In line with the above objectives, the following key research problems are identified:

- New materials.
- Performance evaluation, analytical and experimental verification.
- Standardized test procedures and performance criteria.
- Integrated structural and architectural systems design techniques and standards.
- Control algorithm development, robustness, and reliability, adaptivity, and optimization.
- Sensors, actuators and monitoring system development and optimal deployment.
- Advanced computer technology incorporating artificial intelligence, neural network, fuzzy logic, etc. for on-line targeting, detection, monitoring, diagnosis, and control.
- Intelligent robots and devices.
- Modelling, signal processing, and computer simulations.
- Practical methods and techniques for design, manufacturing, fabrication, and implementation.
- Laboratory and field experiments of systems and devices under actual or simulated dynamic excitations.
- Design and demonstration projects of control systems as applied to new or

existing structures.

- Innovations in base isolation, energy dissipation devices, and other passive techniques.

To carry out the above described research, collaboration among researchers, practicing engineers, and manufacturers is encouraged. It is expected that the research will generate new knowledge, techniques, and a better understanding of the working principles as well as practical limitations inherent in various control systems so that conditions of applicability and design standards and limits of each class of systems can be established and evaluated.

The ultimate goal of the research program is to bring about technological advances through system integration and development innovation so that control technology can be realistically implemented in a timely fashion to provide safety resistance to protect structures such as buildings, bridges, tunnels, and other critical lifeline systems against earthquakes, winds, and other natural or manmade hazard environments, in addition to improving the quality of life.

PROPOSAL EVALUATION

Upon announcing the above-mentioned research initiative, sixty seven (67) proposals were received by the specific proposal submission deadline, i.e., October 30, 1990. Since these proposals cover a wide spectrum of diverse technical subjects, special considerations were given in the review process of such proposals - following the traditional NSF peer review system - to ensure all proposals are evaluated by their technical/scientific merits, relevant factors such as educational and practical values, collaborations between academic and industrial sectors, interfaces among cross-disciplinary subjects, etc.

As a result, a 2-stage review process was adopted by which

(1) Each and every proposal was sent out by mail to peers for disciplinary reviews,

(2) A cross-disciplinary review panel consisting of experts in such areas as control systems, earthquake and wind engineering, sensors and actuator technology, materials and transportation systems was convened to evaluate and rank these proposals.

The panel, after a 1 1/2 day deliberation ranked the proposal into three groups: (1) Highly Recommended, (2) Fundable if Funds are Available and (3) Not Fundable.

Based on the Panel's recommendations, NSF will select by Summer 1992 approximately 7 to 10 proposals for support.

PROPOSAL ANALYSIS

Among proposals submitted, 40 deals with control of earthquake responses of structures in various modes, vs. passive, active, or hybrid. Twenty four (24) proposals study the wind response control and 15 proposals address problems in control of general structural responses and vibrations including one dealing with waves in periodic structures having imperfection and another dealing with random, chaotic motions by active techniques. Among the active control techniques, one proposal investigates the application of active structural members for earthquake and wind resistance.

Several proposals (Table 2) involve development and application of fiber optical sensors in structural control- 4 against earthquake loads, one against winds, and 3 others for general structural motion control. Among proposals which contain the study of actuators and actuation, 6 address earthquake problems, 5 wind problems, and 3 general vibration problems. Types of actuators proposed in these studies include active mass dampers (or drivers), active bracing systems, piezoelectric and electromagnetic actuators, regeneratic actuators, screw jackets, shock absorbers, controlled friction, viscous, or other types of dampers, connectors, or energy dissipators. In terms of processors (i.e., control algorithms, computer software and hardware systems), 3 proposals focus on optimal, robust control against earthquakes, 1 against winds, adaptive control techniques, 6 proposals address earthquake problems, 3 wind problems, and 2 general motion control. Several other proposals employ or involve neural networks techniques in control algorithms and models, 3 for earthquakes and one each for winds and general motion control. Proposals involve material innovations and applications (i.e., high-performance or smart materials) are relatively few. Among them are investigations in using shape memory alloys to provide variable damping and actuation, and self-healing materials for repairing of concrete crack.

Among the proposals described in Tables 1 and 2, there are a few dealing specifically with the systems aspects of the control systems themselves or the structural systems which are under control. These aspects include performance evaluation/requirements/ standards, control systems which are designed for monitoring, detection, and maintenance purposes, and methods for system identification, modelling, and reliability issues involved (Table 3).

These proposals can also be looked at from the application points of view (Table 4). Almost 50% (30 proposals) of the total proposals submitted are attempted to applied to earthquake protection of

various types of buildings structures or their components. Seventeen (17) proposals involve wind-resistant applications and 14 others for other types of motion or response control. In addition, there are a few proposals for applications to non-building types of structures, i.e., bridges (and their components), lifeline systems, industry facilities, and other civil infrastructures. Among these one proposes to investigate control of marine vehicles from impacting to bridge piers, another proposes to use passive approach to control the earthquake damage to wharf structures. It is also interesting to note that while most proposals are formulated for engineering analysis/ design/construction applications in general terms, a few are prepared for more specific purposes, for example, to retrofit a structure under various environmental load conditions, and to retrofit bridges using a vibration isolator controlled in real time.

ENGINEERING RESEARCH IN INTELLIGENT CONTROL SYSTEMS AND SMART MATERIALS

The above analysis indicates an enthusiastic response by the U.S. research community for research in developing and applying intelligent control systems (ICS) to civil engineering structures. It is clear that considerable advance can be made to improve structural safety through research in structural, earthquake, and wind engineering through use of control technology. The possibility of applying control systems to mitigate the structural damage against earthquakes, winds, and other loads open up a wide range of challenging research opportunities for multidisciplinary investigators in the field. In this connection, it is interesting to also note some current engineering research in smart materials under way.

Smart materials is a class of materials which have their own sensors (nervous system), actuators (muscular systems) and control (brain system), mimicking biological systems. Current research activities aim at understanding, developing and synthesizing material systems which behave like biological systems. The U.S., Japan, and industrialized Western European countries are leading in the fundamental research on smart materials and smart systems. To this point, no smart materials, except some that have been biologically synthesized, have been successfully produced beyond laboratories. The Japanese remains at a stage of concept formulation. The U.S. efforts have been focused on development of smart biomaterials and smart systems for control of space structures and mechanical vibrations. Sensors include: optical fibers, strain gages, corrosion sensors (e.g., one developed at Lehigh Engineering Research Center) and other environmental sensors, sensing particles

(e.g., magnetic particles). Examples of actuators are: shape memory alloys (which would return to their original shape), hydraulic systems, piezoelectric devices (which move in an electric field). Microchips, computer software and hardware systems constitute the control aspects.

Smart materials may even heal themselves when cracked, for example, self-healing concrete being investigated in universities of Illinois and Michigan. One idea is to place hollow fibers filled with crack-sealing material into concrete, which if cracked would break the fiber releasing the sealant. Hollow beads filled with sealant (for self-healing), and smart paints, which will release red dye (contained in capsules) when cracked are also being investigated. Concrete may also heal its own cracks under pressure with sufficient humidity. Optical fibers which change in light transmission due to stresses are useful sensors. They can be embedded in concrete or attached to existing structures. Optical fiber sensor systems have been applied for on-line, and real-time monitoring of critical components of structural systems (such as bridges) for detection and warning of imminent structural failures. Fundamentals and dynamics of embedded optical fibers in concrete, electro-rheological (ER) fluids for the vibration control of structures have been investigated. Such ER fluids can stiffen up very rapidly (changing elastic and damping properties) in an electric field.

Several such smart materials are studied to provide sensoring and actuating capabilities, i.e., shape memory alloys, and surface superelastic microalloying as sensors and microactuators; and magnetostrictive active vibration control.

CONCLUDING REMARKS

It is clear that smart materials, smart structures, and control technologies, when sufficiently developed and broadly implemented, possess an excellent potential to impact the safety design and construction of new structures in the future. Furthermore, the dynamic control and intelligent technology also offers a viable, alternative solution to the nation's enormous problems in the construction of new civil infrastructure and rehabilitation of existing structures which are deficient in their functionality, serviceability and in their seismic, wind, or other resistant capacities.

As such it is generally agreed that dynamic control and intelligence technology holds considerable promises in the design applications of civil engineering structures, new and existing. By its interdisciplinary nature, such technology also has the potential impact to the future development of a diverse field of scientific or technical subjects, such as material

science, vibration, biotechnology, neural network, and new devices.

Control Technique	No. of Proposals				
	Earthquake	Wind	General Vibration	Waves	Random Chaotic Motion
Passive	12	8	4	1	
Active	18	12	8		1
Hybrid	10	4	1		
Total No. Proposals	40	24	13	1	1

Table 1. Proposal Distribution by Control Techniques

Control Element	No. of Proposals		
	Earthquake	Wind	General Vibration
Sensors	4	1	3
Actuators	6	5	3
Processors (Algorithms, Modelling, etc.)	13	4	3
Materials	2	1	2
Total No. Proposals	25	11	11

Table 2. Proposal Distribution by Control Elements

	Earthquakes	Winds	General Vibrations	Applied to Highway Systems
Performance Evaluation	2	1	2	1
Monitoring Detection Maintenance	4	1	3	4
System Modelling & Identification	1		1	
Reliability	3		2	
Total No. Proposals	10	2	8	5

Table 3. Proposals to Study System Characteristics, Modelling, and Analysis

		Earthquakes	Winds	Others
Elements/Systems Types of Structures	Buildings	30	17	14
	Bridges	4	1	1
	Lifelines, Infrastructures	3	1	
	Wharves	1		
	Traffic Signal Structures		1	
	Claddings	2		
	Nonstructural Elements, Secondary Systems	2		
Special Purposes	Retrofit	3	2	1
	Diagnosis, Monitoring	1	1	4
	NDE			2

Table 4. Proposal Distribution by Applications

免震用積層ゴム支承の標準化のための諸検討

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［要約］
建設省総合技術開発プロジェクトにおいて実施している免震用積層ゴム支承の研究の内容及び日本において使用されている免震用積層ゴムの概要とその製造法等を紹介している。

［キーワード］
免震構造、積層ゴム、標準化、
建設省総合技術開発プロジェクト

1．はじめに

日本では、建設省総合技術開発プロジェクト「建設事業への新素材・新材料利用技術の開発」を実施している。そこでは、新素材・新材料のうち金属系複合材料の1つとして建築物の免震構造用の積層ゴムをとりあげ、その利用技術指針を作成することとしている。

建築物の免震構造用積層ゴムについては、すでに多くの研究がされ、材料特性、免震装置としての積層ゴム支承の特性、積層ゴムを利用した免震建築物の設計・解析・地震応答観測等に関して日本建築学会等で成果が発表されている。また、50棟前後の免震構造建築物が建設されているか、建設中である。さらに、日本建築学会では「免震構造設計指針」が刊行されている。

本プロジェクトにおける積層ゴムに関する研究は、上記のようなこれまでの研究の状況を勘案して、それらの研究成果・積層ゴムメーカーでの実験結果等を参考に、免震構造建築物が健全なかたちで建設され、その建設促進に寄与させるための利用技術指針を確立しようとしているものである。

そのため、積層ゴム及び積層ゴム支承自身の特性、とくに建築物への使用にとって構造設計者が把握することが望ましいと思われる特性等についての技術資料を収集し、それらを整理してその結果を技術指針としてまとめ、免震積層ゴムの標準化に向けての資料を整備する予定である。

本研究を行うため、（社）日本ゴム協会に「免震ゴム利用技術特別委員会」を設置し同協会と建設省建築研究所が協同研究を締結している。本論文は、それらの研究の内容を紹介し、日本における免震積層ゴムの概要及び製造法等を紹介するものである。

日本は、地震多発地域に位置し、多くの被害を受けてきた。そのため、日本の建築構造技術にとって地震被害防止は、古くからの大課題であった。日本の建築構造設計法は、地震力などに抵抗できるよう、構造部材を適切に配置し、部材の強度・じん性・剛性を増加させる考え方に基づいてきた。一方、地盤と建築物の間ですべりが生じるようにして、建築物に大きな地震力が作用しないようにする工夫が、古くから考えられてきた。こうした考え方は、免震と呼ばれてきた。そして、少数ではあるが明治時代から免震研究が行なわれ、昭和初期には、免震構造建築物が建設された。しかし、技術の未発達のため、その後は、免震構造は実現しなかった。

第2次大戦後、地震応答解析技術が進歩し地震時の建築物の挙動が把握できるようになった。積層ゴム支承が開発され、これを使用して固有周期を延長して地震時の構造物の応答を低減させる免震構造が注目されるようになった。

固有周期を延長し、地震などの振動の主要周期と構造物の固有周期を離して応答を低減する技術は建築物だけでなく、機械・機器などでも可能であり、実績もある。本研究は、建築物の免震を扱い、積層ゴム支承の力学的的特性等を明らかにし、性能試験法等を示し、建築構造設計者を対象に、積層ゴム支承という新しい材料・装置が免震構造建築物に適切に使用されるための資料を提供しようとするものである。

2．免震の原理と効果

積層ゴム支承を使用した免震構造は、建築物の最下階と基礎との間の柱直下に水平方向の剛性が低く鉛直荷重を支持できる積層ゴム支承を設置して固有周期を延ばし、ダンパーを併用して減衰を増加させ地震応答加速度を減少させるものである。

積層ゴム支承は、薄いゴムシートと鋼板を相互に重合わせ、加硫接着させたもので、大きな鉛直方向剛性と耐力を持ち上部構造を支持し、小さな水平方向剛性により建築物の固有周期を長くする。ダンパーは、粘性体の抵抗や金属の塑性変形等により振動エネルギーを吸収して振幅を減少させる。

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免震構造建築物の地震観測の結果によると、建築物基礎部の加速度に比べ、上部構造の加速度応答は 1/3 程度まで低減されることがある。従来の建築物の加速度応答倍率は通常２倍程度以上であることと比較すると、免震の効果がわかる。

３．積層ゴム及びダンパーの種類と特性

３．１積層ゴムの概要

積層ゴム支承は、鉛直方向には建築物重量を支えるために高い耐力と剛性が求められる。鉛直剛性が低いと、上下方向の地震動に対する応答が大きくなったり、ロッキング振動を生じたり、重量分布の不均一などのため建築物の傾斜が生じる恐れもでてくる。一方、建築物の水平方向の振動固有周期を長期化するため水平方向の剛性は低く、大きな変形能力が必要になる。

積層ゴム支承は、鉛直方向はゴムの横変形が鋼板で拘束され高い剛性と耐力がえられる。水平方向は薄いゴム部分のせん断変形は拘束されず、ゴムの柔軟性により低い剛性と大きな変形能力を持つ。（図１）

積層ゴムは、一般には偏平な円柱形である。積層ゴムの鉛直支持荷重、鉛直及び水平方向の剛性、限界水平変位等は、形状、寸法、ゴムの層厚及び層数などで異なる。ゴムが薄く層数が少ないほど、水平方向剛性に対する鉛直方向剛性の比率が高くなる。通常用いられる積層ゴムの水平方向剛性は、鉛直方向剛性に比べて1/1000程度と極端に小さい。又、積層ゴムは、形状の偏平度によって水平荷重－水平変形特性に違いが出てくる。偏平度を表わす指標として２次形状係数S_2（＝直径／ゴム総厚）が用いられる。一般に積層ゴムの２次形状係数は２以上のものが用いられる。２次形状係数が大きい場合、すなわち積層ゴムが偏平な場合には、水平荷重により生じる積層ゴムの水平変形にはせん断変形の占める割合が多く、水平荷重－水平変形関係は鉛直荷重の影響をあまり受けず安定した特性を示す。一方、２次形状係数が小さいほど積層ゴムは細長くなり、水平変形に占める曲げ変形の割合が増え、水平荷重－水平変形係数は鉛直荷重の影響を受け易くなる。

積層ゴム支承におけるゴムの役割は、高い弾性と低い剛性を利用して支承の水平方向の剛性を低く変形能力を高く復元性を保持させることである。鋼板の役割は、ゴムの横変形の拘束による鉛直方向の剛性と支持力の上昇である。（図２）

３．２　積層ゴムの種類と特徴

積層ゴム支承には、大きく次の４種類がある。

①天然ゴム系積層ゴム
②高減衰ゴム系積層ゴム
③鉛入り積層ゴム
④弾性すべり系積層ゴム

①天然ゴム系積層ゴム

天然ゴム系積層ゴムは、現在まで最も多く使用されている。天然ゴムをゴムの原材料とする。これ自体の減衰性は少ない。水平荷重－水平変形特性は、一般にゴムのせん断歪が約 200％程度まではほぼ直線関係を示し、それ以上変形関係が進むと徐々に剛性が大きくなるハードニング現象を示す。ゴムが破断する時のせん断歪は約 400％程度以上である。

②高減衰積層ゴム

高減衰積層ゴムは、ゴム自身に粘弾性の特性を持たせた減衰性のある合成ゴムを使用するものである。

③鉛入り積層ゴム（図３）

鉛入り積層ゴムは、積層ゴム中心部に円柱状の鉛プラグを埋込んだもので、積層ゴムの水平変形に伴って生じる鉛プラグの塑性履歴吸収エネルギーによって減衰性能を高めている。

④弾性すべり系積層ゴム（図４）

弾性すべり支承系積層ゴムは、積層ゴム支承を建築物の基礎等に固定せずにすべり板の上に設置する。小地震の場合には、摩擦のためすべらず、積層ゴムの作用により地震力を低減するが、大きな地震の場合にはすべり、上部構造の地震力が増加しない。ただし、すべった場合には、弾性すべり支承は単独では、すべった後に元位置には戻らない。そのため、地震後に元位置に戻すためばねなどが併用される。

②③の積層ゴム支承は、ともに部材単体で、復元力性能とエネルギー吸収性能を持つ。そのため、通常は、これらの積層ゴム支承を使用する場合には、ダンパーは併用されない。①の天然ゴム系の積層ゴムは、エネルギー吸収性能はほとんど持たないため、免震構造とする際には、別にダンパーを併用する。

いずれの積層ゴムも、積層されるゴムの劣化、鋼板の腐食等の防止のため外周部分に被覆ゴムがまかれる。頂部と底部には上部構造及び基礎への固定のためにフランジ鋼板が配置される。弾性すべり支承の場合には底部にはフッソ樹脂などのすべり材が接合されている。

３．３積層ゴムの設計法

1) 積層ゴムの設計法

積層ゴム支承には、主に次のような力学的性能を持つことが必要である。

(a) 所要の鉛直荷重（上部構造の固定荷重及び積載荷

重）を支持する。
(b) 設定した水平方向及び鉛直方向の固有振動周期を実現するばね定数を有する。
(c) 地震力よって予想される水平変位に対して必要な水平変形能力を有する。

積層ゴム支承は、上記の3つの事項を主な設計条件として、次のような手順で積層ゴム支承を設計する。

①設計条件の設定（鉛直荷重支持力、ばね定数、水平変形能力）
②ゴム材料及び定数の決定（諸弾性率、破断ひずみ等）
③積層ゴム支承の全体形状の設定（直径、ゴム総厚）
④ゴム層構造の決定（層数、ゴムシート厚、内部鋼板厚、フランジ鋼板形状）
⑤設計条件を満足していることの確認

3．4ダンパーの概要

振動エネルギーを消費させる「ダンパー」装置がある。積層ゴムに大きな減衰性がない場合、ダンパーが併用される。ダンパーには、粘性体等の粘性抵抗を利用したものや鋼材等の履歴ループを利用したものなどがある。エネルギーを消費させる原理や使用材料、メカニズムなどに応じて、ダンパーには次のようなものがある。

①粘性体等の抵抗を利用したもの（図5）
②弾塑性材料の履歴ループを利用したもの（図6）
③摩擦力を利用したもの（図7）

4．積層ゴムを使用した免震構造物の構造設計

免震構造建築物の地震に対する設計の基本的な流れは、次のようなものである。

①対象地震と安全レベル、その他の性能の目標設定
②地盤と地震動特性の検討
③周期設定と免震装置の基本特性の検討
④予備解析による応答等に基づく構造設計
⑤地震応答解析と安全性、その他の目標の達成の検証
⑥詳細設計

免震構造は、特殊な構造であり、建築基準法が予想していないものであるため、建設にあたっては、日本建築センターの免震構造評定と建設大臣による建築基準法第38条の規定に基づく認定が必要になる。

これらの免震建築物では、静的な設計用地震力に対する構造計算に加えて、地震の加速度記録に基づき、「動的解析」が行われる。このとき、入力地震動としては、「El Centro」などのいくつかの標準的と考えられる過去の地震動記録、建築物の建設予定地域に近い場所で観測された地震記録などの複数の記録を用いる。このとき、基本的には、2つのレベルの地震動の大きさを考える。レベル1の地震動は、最大速度25cm/sec、レベル2の地震動は最大速度50cm/sec程度に規準化して用いている。通常、レベル1の地震動に対する応答はほぼ弾性範囲内、レベル2の地震動に対する応答は適切な変形以内としている。

5．積層ゴムの一般的特性

これまでに建設された免震構造建築物のデータを参考にして、上記の基本的条件について目安となる値を示すと、次のようなものである。

・重量支持能力（長期面圧）－ 20～90kg/cm²程度
・固有周期（水平方向） － 1～3秒程度
・減衰（等価減衰定数） － 5～15%程度

図8～10にこれらの状況を示す。

6．おわりに

これまでに紹介したような検討を行い、免震積層ゴムの標準化への技術資料を整備する予定である。

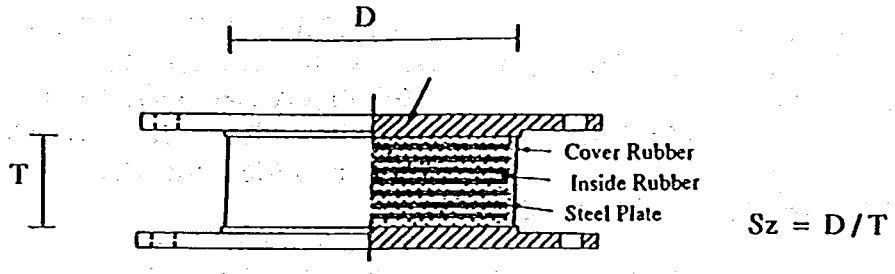
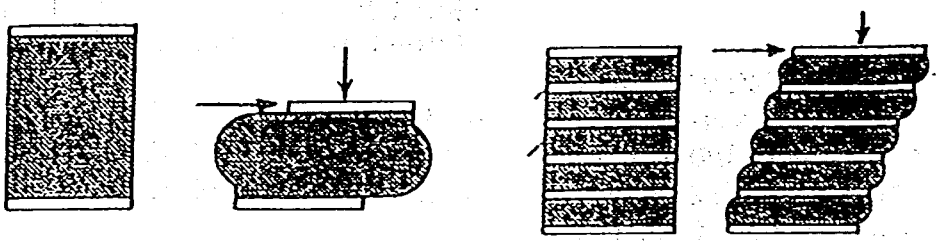


Fig. 1 Rubber bearing



① Single Rubber ② Laminated Rubber

Fig. 2-1 Single rubber and laminated rubber

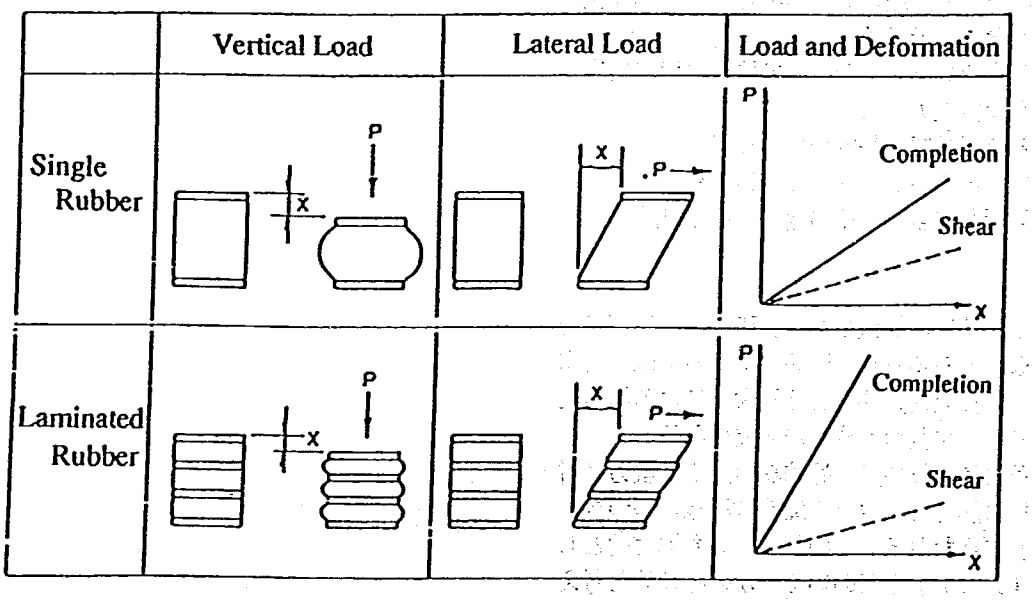


Fig. 2-2 Single rubber and laminated rubber

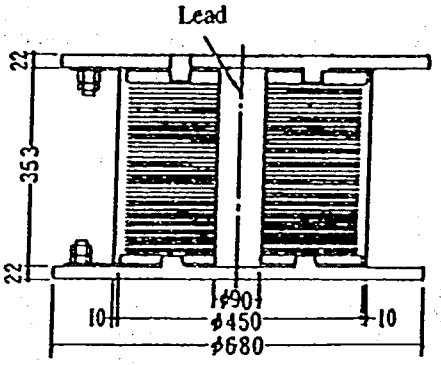
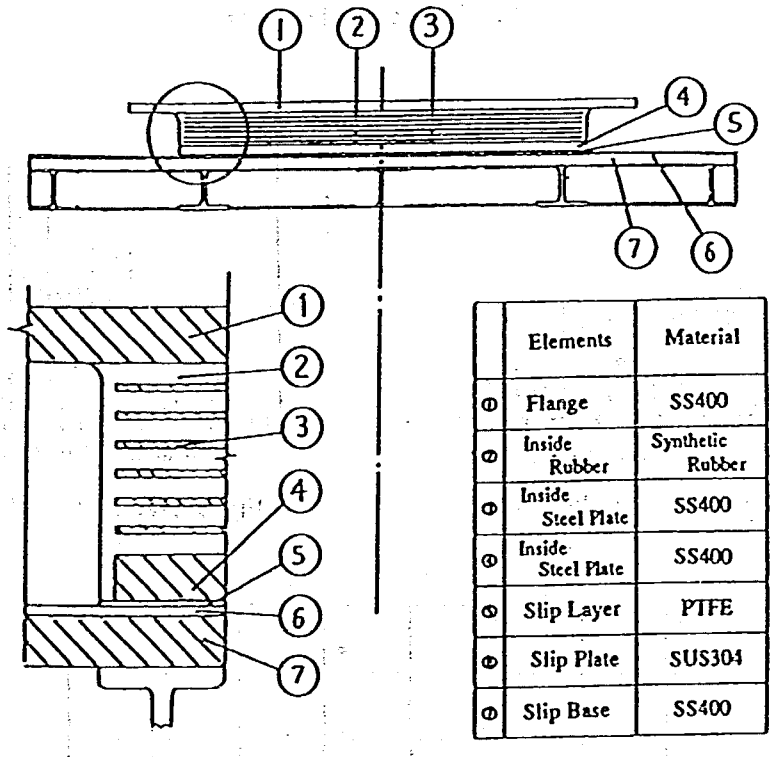


Fig. 3 Lead rubber bearing



	Elements	Material
①	Flange	SS400
②	Inside Rubber	Synthetic Rubber
③	Inside Steel Plate	SS400
④	Inside Steel Plate	SS400
⑤	Slip Layer	PTFE
⑥	Slip Plate	SUS304
⑦	Slip Base	SS400

Fig. 4 Slip layer and rubber bearing

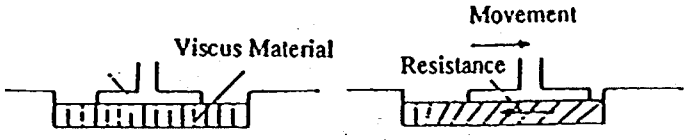


Fig. 5 Viscus damper

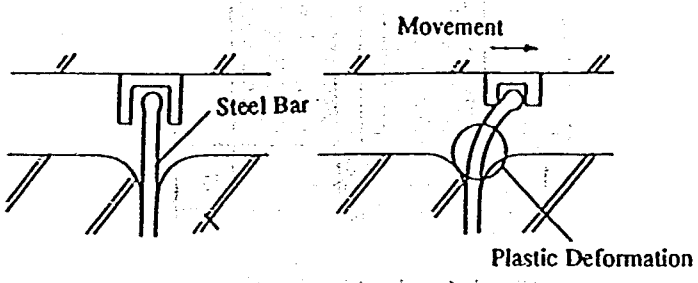


Fig. 6 Plastic damper

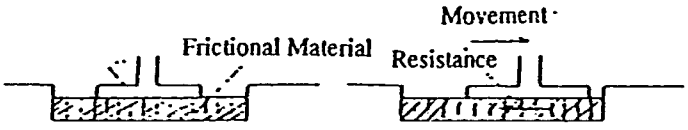


Fig. 7 Frictional damper

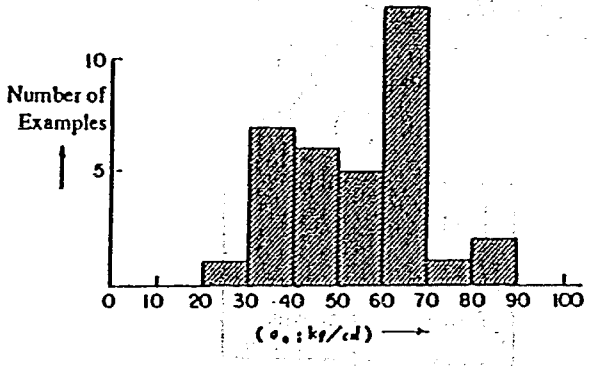


Fig. 8 Stress of bearing

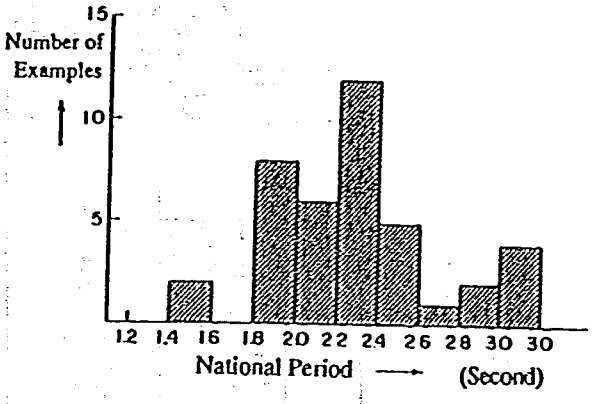


Fig. 9 Natural period of building

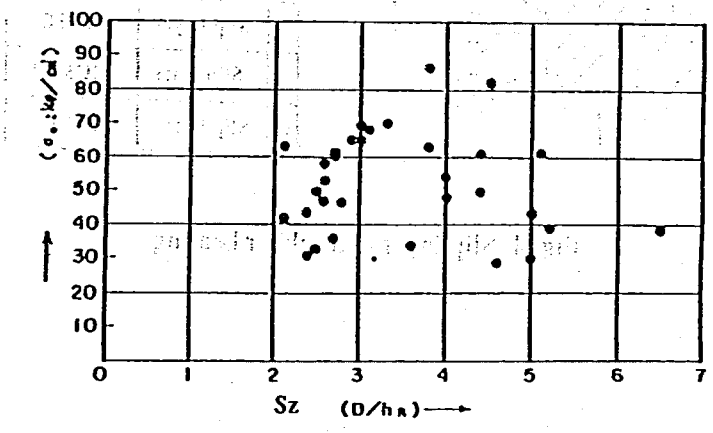


Fig. 10 Sz and stress of bearing

免震橋梁の動的挙動に関する温度の影響

中野　修[*1]、谷口秀之[*2]、西　弘明[*2]

要約

最近橋梁にも用いられ始めた免震支承が北海道のような寒冷地においても十分な免震効果を発揮できるかどうかを把握するために、実際の橋梁にあわせて製作された免震支承の実物大模型を用いた室内実験によって低温時における挙動を調べてきた。その結果、免震支承は予想以上に温度変化の影響を受けることがわかったが、温度変化により受けた免震支承の剛性の変化が橋梁全体系に与える影響については不明だった。そのため、今回動的解析を行うことによりその影響を調べたが、温度変化により橋脚基部の曲げモーメントが４０～５０%増加することがあることがわかった。

キーワード：免震支承、寒冷地、温度変化による影響、動的解析

１．はじめに

一般に、免震橋梁とは、支承部に上部工を水平方向に柔らかく支持し、かつ減衰性能を持った免震支承と呼ばれる特殊な装置を用いて、橋梁全体の固有周期を長周期化し地震力を低減することを目的とした橋梁である。

北海道開発局では、一般国道４４号根室市温根沼大橋の釧路側側径間部にこのタイプの形式を採用し、現在建設中である。

北海道のような寒冷地で免震支承の採用を考えた場合、ゴム材料の物性値（剛性、減衰定数など）の変化によって、設計通りの免震効果が得られないことが懸念されたため、温根沼大橋で採用する免震支承の設計に基づいて製作した鉛プラグ入り積層ゴム支承（ＬＲＢ）および高減衰ゴム支承（ＨＤＲ）の２タイプの実物大模型を用いて、平成元年度より、５０回繰り返しせん断試験、圧縮試験、せん断試験およびせん断履歴試験等の室内実験を温度を変化させて行ってきた[1)]。この結果、剛性に関しては免震支承自体は予想以上に温度変化の影響を受けることが分かったが、温度変化により受けた免震支承の剛性の変化が橋梁全体系に与える影響については不明だったので、今回動的解析を行うことによりその影響を調べた。

２．温根沼大橋の概要

北海道開発局は全国で８橋ある免震橋梁計画のうちの１橋として、一般国道４４号温根沼大橋の架け換えの際に免震支承の採用を予定している。なお、温根沼大橋は８橋の中で唯一寒冷地に建設されるものである。架橋地点は、根室半島の付根部分で、過去３０年の最低気温の平均は－９．１度である。現温根沼大橋は、北海道根室市の温根沼に架けられている橋長99.5mの鋼曲弦ワーレントラス＋鋼合成桁橋であり、架け換え後の新橋（図－１）は、橋長456m、側径間部が４径間連続鈑桁、中央径間部が支間140mのニールセン系ローゼ桁となり、完成予定は平成７年度である。なお、釧路側側径間部のみ免震構造になってるが、両端のＡ１およびＰ４には通常の積層ゴム支承（ＲＢ）を使うことにしている。

今年度、免震支承を採用する側径間部が架設されるので、秋に振動実験を行う予定である。

３．免震支承の概要

免震装置は、構造物を柔らかく支持するアイソレーター部とエネルギー吸収により減衰を持たせるダンパー部からなっている。アイソレーター部にはゴムと鋼板を交互に重ね合わせた積層ゴムと呼ばれるものが用いられ、ダンパー部には鉛や鋼棒などが用いられる。また、橋梁に用いられる免震装置は、一般に支承部に設置されることが多いので特に免震支承と呼ばれる。したがって、免震支承は狭い部分に設置できるようにアイソレータ部とダンパー部が一体化されたようなコンパクトなものでなければならないため、現在大きく分けて２種類の免震支承がある。１つは、鉛プラグ入り積層ゴム支承と呼ばれる積層ゴムに減衰を持たせるために鉛プラグを挿入したもので、もう１つは、高減衰ゴム支承と呼ばれる積層ゴムのゴム材料自体に減衰性能を持たせた高減衰ゴムと呼ばれる特殊なゴムを用いたものである。それぞれの免震支承を図－２に示す。

４．免震支承の温度依存性

免震支承の動的特性は、図－３に示されるようなバイリニア型バネで近似されるせん断力～せん断変位曲線より求められる等価剛性（K_e）および等価粘性減衰

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定数（h_{eq}）で表される。K_{eq}およびh_{eq}は、次式のように定義される。

$$K_e=\frac{AE}{EC}、\quad h_{eq}=\frac{1}{\pi}\cdot\frac{\Delta W}{W}$$

ここで、ΔW：□ＡＢＣＤの面積
W：△ＡＥＣの面積

上記K_eおよびh_{eq}を実際のせん断力～せん断変位曲線から得るために実物大模型（ＬＲＢおよびＨＤＲ）を用いたせん断試験を行った[2]。模型の設計変位は地震時保有水平耐力法により求められ、その値は１５ｃｍである。供試体の設計条件などの詳細については文献１）に示されている。

せん断試験は、圧縮荷重９２ｔ（死荷重相当）のもとでせん断変位量を１５０ｍｍ（設計変位）とし、周期を約４０秒として行った。また、供試体温度は＋３０℃から－２０℃まで変化させて行った。

その結果得られたそれぞれの温度における等価剛性および等価粘性減衰定数を表－１に示す。また、常温時と－２０℃時の実験から実際に得られた履歴ループを図－４に示す。

免震支承の温度変化に対する安定性に関しては、「道路橋の免震設計法ガイドライン（案）」[3]（以下ガイドラインとする）において、橋の固有周期の変化を２０％以内に抑えるために、等価剛性の比が＋３０℃と－３０℃の間で1.5以内であるよう規定されているが、表－１の値より－２０℃と＋３０℃の時の等価剛性の比は、ＬＲＢ、ＨＤＲそれぞれ２体の平均で、ＬＲＢが1.63、ＨＤＲが1.59となっている。

なお、参考までに載荷周期１００秒の場合の常温と－３０℃の時の等価剛性を示すとそれぞれＬＲＢが１３１９kg/cm、２０８９kg/cm、ＨＤＲが１８３２kg/cm、３３９０kg/cmであり、常温と－３０℃でその値を比較すると、ＬＲＢが1.58、ＨＤＲが1.85となった。

5．解析モデルおよび手法

解析は、常温時および－２０℃時の支承の相対変位および橋脚基部の最大曲げモーメントを算出し、比較することとした。この場合の相対変位とは、支承の上端と下端の変位の差である。

解析モデルは、免震支承が採用される側径間部のみを対象とし、図－５に示されるような骨組みモデルとした。免震支承のモデル化は、等価線形解析では等価剛性および等価粘性減衰定数を用いて線形モデルとし、また、非線形解析においてはバイリニアモデルとして支承部に組み入れた。（図－６）

また、橋脚の剛性は弾性域内にあるものとして算出し、地盤バネは、「道路橋示方書　Ｖ耐震設計編」[4]に示されている算定方法に従い対象地盤のＮ値より求めた。

解析方法は時刻歴応答解析法を用いたが、温根沼大橋では免震支承は橋軸方向のみ効果を持つような構造とされているため、構造研究室で開発したプログラム「ＮＬＤＡＰ」[5]を用いて、橋軸方向に関してのみ解析を行った。なお、同プログラムにおいて応答値はNewmark-β法による直接積分により求め、粘性減衰はReyleigh減衰を仮定した。

6．動的解析

6．1　固有値解析

最初に、免震橋梁の振動特性を把握するために各温度において固有値解析を行った。

常温（約２０℃）と－２０℃における設計変位１５０ｍｍの時の１橋脚当たりの免震支承の等価剛性および等価粘性減衰定数を表－２に示す。なお、表の中の値はＬＲＢおよびＨＤＲともに供試体２体ずつの平均値を支承数（６基）倍したものである。

Ａ１およびＰ４の支承部には通常の積層ゴム支承（ＲＢ）を使用するが、実験値が得られていないため、その剛性は常温においては設計値[6]の363t/mをそのまま採用し、－２０℃においては免震支承と同程度の剛性の変化があるものと仮定して設計値の1.3倍をとり、472t/mとした。

表－３に１０次モードまでの固有周期をＬＲＢ、ＨＤＲそれぞれ温度別に比較したものを示す。この表から、温度変化により免震支承の剛性が変化することによって１次モードの固有周期は１３～１４％程度異なるが、２次モード以降はほとんど変化していないことがわかる。これは、１次モードは上部工の変位が支配的であるため、免震支承の剛性の変化の影響が現れていると考えられる。図－７に常温時のＬＲＢを用いた場合の１次モード図を示す。

6．2　時刻歴応答解析結果

6．2．1　等価線形解析

現温根沼大橋では1975年に強震計が設置されて以来強震観測が行われている。今回、時刻歴応答解析に用いた地震波は、これまでに観測された記録の中から表－４に示された1978年 1月12日に観測されたデータを

用いた。その強震記録波形および加速度応答スペクトルを図－８、９に示す。

解析は、次のような条件の下で行った。

積分時間間隔：Δｔ＝0.01秒
応答時間　　：Ｔ＝１０秒
Newmark-β法：β＝1/4

橋全体の減衰定数ｈは、ガイドラインに示されている次式を用いて求めた。

$$h=\frac{\Sigma h_{Bi}K_{Bi}u_{Bi}^{2}+\Sigma\frac{h_{pi}K_{Bi}^{2}u_{Bi}^{2}}{K_{pi}}}{\Sigma K_{Bi}u_{Bi}^{2}+\Sigma\frac{K_{Bi}^{2}u_{Bi}^{2}}{K_{pi}}}$$

ここで、h_{Bi}：免震装置の減衰定数
h_{pi}：下部構造の減衰定数
K_{Bi}：免震装置の等価剛性
K_{pi}：下部構造の等価剛性

Reyleigh減衰のα、βを決定するための１次モード、２次モードの減衰定数は、それぞれ橋全体の減衰定数ｈをとるものとした。上式により求められたｈを表－５に示す。

以上のような条件のもとで解析を行った結果として、支承の相対変位および橋脚基部の曲げモーメントの常温時と-20℃時の値を比較したものを表－６、７にそれぞれ示す。これらの表から支承の相対変位に関しては、ＬＲＢの場合は-20℃時の方が若干大きめにでているが、ＨＤＲの場合は、温度が変化してもそれほど変位には差がないことが分かる。また、曲げモーメントの比較からはＬＲＢの場合、-20℃時に常温と比較して数%の増加であるが、ＨＤＲの場合は４０～５０%も増加していることが分かる。ＨＤＲの曲げモーメントが温度によって大きく異なった理由としては、等価剛性および固有周期の変化率は、ＬＲＢ、ＨＤＲの両者でそれほど異ならないため、温度によるそれらの変化率ではなく、ＬＲＢ、ＨＤＲそれぞれを採用した場合の橋全体の固有周期と入力地震波の性質との関係によるものと考えられる。したがって、温度変化が橋梁全体系の挙動に必ずしも影響を与えるわけではないことがわかった。

６．２．２　非線形解析

６．２．１では、免震支承の温度変化にポイントを置き、それが橋梁全体系に与える影響について調べた結果、温度変化が橋梁全体系の挙動に必ずしも影響を与えるわけでないことがわかった。そこで、ここでは、免震支承の温度やタイプは考えずに解析手法の影響に力点を置き、免震支承のバイリニア型特性を等価線形モデルに近似した解析（＝等価線形解析）とバイリニア型バネをそのまま用いた非線形解析との比較を行った。

バイリニア型バネにモデル化した場合の１橋脚当たりの免震支承の１次剛性、２次剛性および降伏変位（図－３参照）は、以下のものと仮定した。

１次剛性：５５５０tf/m
２次剛性：８５４tf/m
降伏変位：16.44mm

等価線形解析の場合には、免震支承の等価剛性はせん断変位量の関数となるが、設計変位が１５０mmであるので、この場合の等価剛性は１３６０tf/mである。

６．２．１で用いた地震波を入力し、それぞれの解析を行った結果得られた橋脚基部の曲げモーメントを表－８に示す。この結果、非線形解析の方が４～５倍大きな値となったが、実際の支承の変位は約２０mm程度なので変位量を１５０mmとした影響（剛性を低く見た影響）が現れた可能性もある。したがって、変位量２０mmとして等価剛性を算出した場合（＝４７１４tf/m）および１次剛性を等価剛性とした場合の解析を行い、非線形解析による値と比較した。その結果を表－９に示す。

非線形解析結果と比較して、前者は、６０～９０%、後者は５０～７０%の差があった。ただし、最大曲げモーメント発生時刻は両者で異なる。

７．まとめ

今回の解析で、明らかになったことをあげると以下のようになる。

①温度が常温から-20℃に変化することによって免震支承の等価剛性は約４０%上がるが、この影響を橋の固有周期で見るとＬＲＢ、ＨＤＲともに１４%程変化することが分かった。

②免震支承の剛性の変化が橋の固有周期に及ぼす影響は1次モードだけに顕著に現れた。

③免震支承の相対変位に関しては、等価剛性の変化率がＬＲＢおよびＨＤＲでそれほど変わらないのにも関らず、ＬＲＢでは１０%前後低温時の方が大きくなっているが、ＨＤＲでは温度が変化しても変位にはほとんど違いは見られなかった。

④橋脚基部の曲げモーメントに関してはＬＲＢでは数%、ＨＤＲにおいては４０～５０%の増加と両者で大きく異なった。

⑤等価線形解析と非線形解析による値を比較した結果、せん断変位量の仮定によっては結果に大きな影響を与えるため、等価線形解析を行う場合には十分注意を払うことが必要であることがわかった。

8．おわりに

今回は、入力地震波が１ケースのみの解析なので、規模の異なる複数の地震波を用いて線形および非線形解析を行い、両者の結果の比較およびより正確な免震橋梁の挙動を把握し、断面力の変化の傾向をつかみたい。

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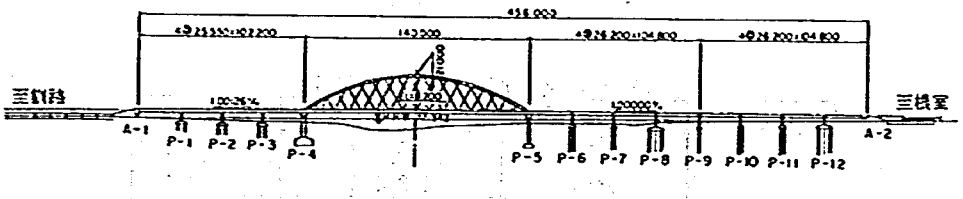
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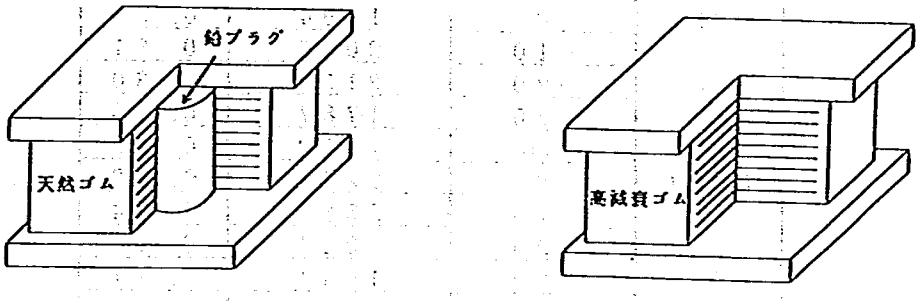
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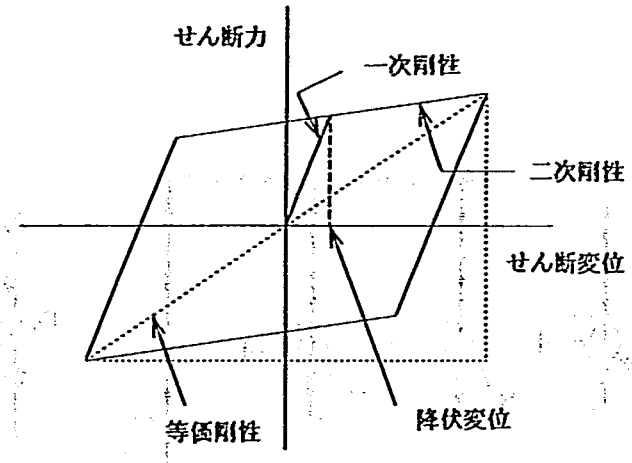


図－1　温根沼大橋

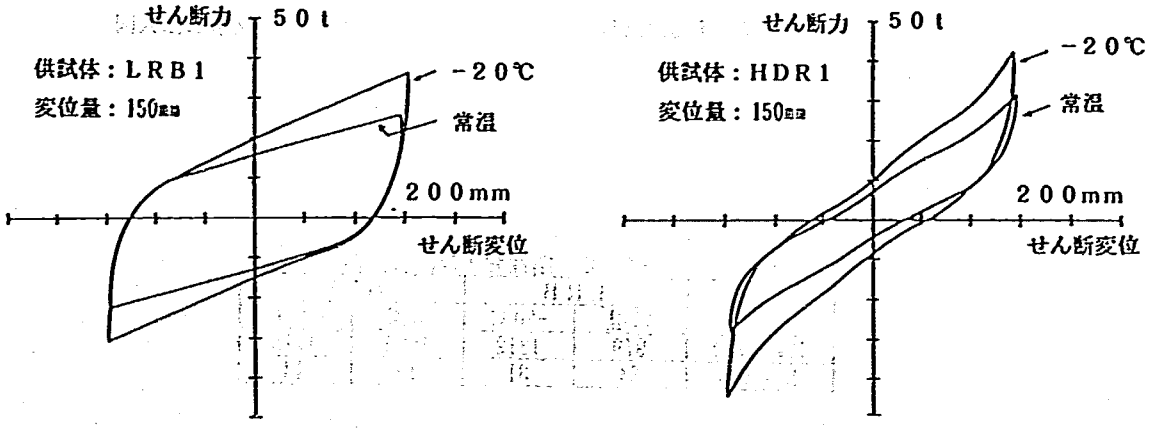


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図－2　免震支承



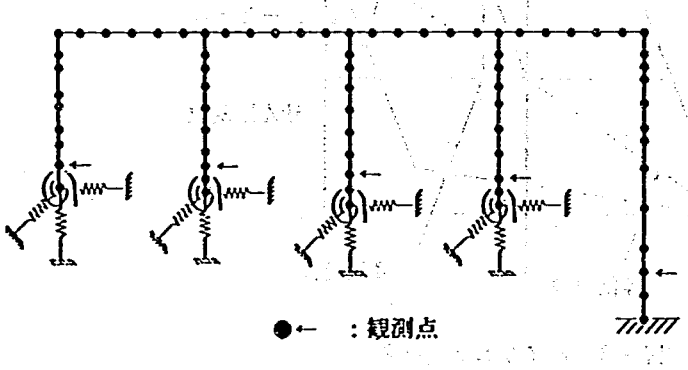
図－3　バイリニア型バネ



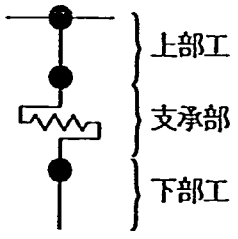
図－4　履歴ループの比較

表－1　各温度における等価剛性

供試体	温度（℃）	等価剛性(kg/cm)	減衰定数
LRB1	+30	1332	0.34
	常温	1645	0.34
	0	1832	0.31
	−10	2072	0.31
	−20	2217	0.31
LRB2	+30	1346	0.34
	常温	1619	0.34
	0	1994	0.33
	−10	2005	0.31
	−20	2155	0.30
HDR1	+30	1787	0.13
	常温	2079	0.12
	0	2249	0.12
	−10	2586	0.14
	−20	3044	0.14
HDR2	+30	1944	0.11
	常温	2136	0.12
	0	2340	0.13
	−10	2515	0.13
	−20	2874	0.13



図－5　解析モデル



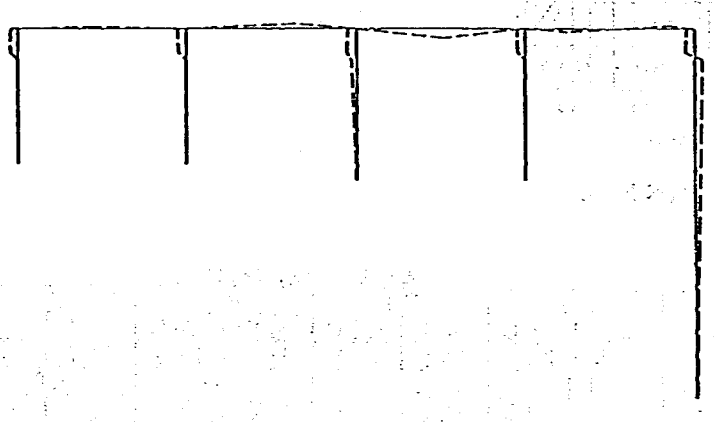
図－6　支承部拡大図

表－2　剛性および減衰定数

	LRB		HDR	
	常温	-20℃	常温	-20℃
Ke (t/m)	979	1312	1265	1775
h_{eq} (%)	34	31	12	14

表－3　固有周期（s）

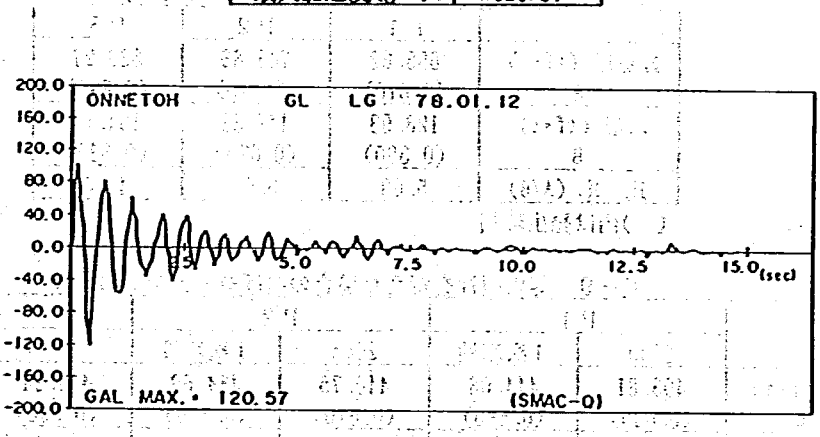
	LRB		HDR	
モード	常温	-20℃	常温	-20℃
1	1.20973	1.05605	1.09633	0.94095
2	0.50958	0.50902	0.50957	0.50902
3	0.48331	0.48258	0.48278	0.48176
4	0.39794	0.39758	0.39775	0.39728
5	0.29420	0.29407	0.29414	0.29397
6	0.23620	0.23618	0.23620	0.23617
7	0.22022	0.21838	0.21863	0.21589
8	0.18659	0.18566	0.18579	0.18440
9	0.17457	0.17385	0.17393	0.17287
10	0.15261	0.15259	0.15259	0.15256



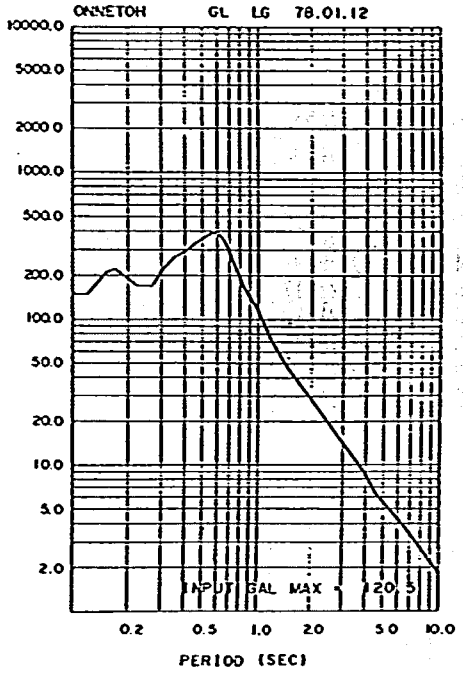
図－7　1次モード図（LRB：常温）

表－4　強震記録

年月日	1978.1.12
マグニチュード	5.1
深さ(km)	40
震央距離(km)	31
最大加速度(gal)	120.57



図－8　入力地震波



図－9　加速度応答スペクトル

表－5　橋全体の減衰定数　h

	LRB		HDR	
	常温	-20℃	常温	-20℃
h (%)	26	23	10	11

表－6　支承の相対変位

		A1		P1		P2		P3		P4	
L	温度	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃
R	δ(mm)	18.5	21.0	18.5	20.6	18.5	20.5	18.7	19.4	26.1	26.6
B	比　率	1.14		1.11		1.11		1.04		1.02	
H	温度	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃
D	δ(mm)	26.7	27.7	26.3	26.8	26.6	26.4	24.5	24.5	40.1	33.1
R	比　率	1.04		1.02		0.99		1.00		0.83	

表－7　曲げモーメントの比較

		P1		P2		P3		P4	
L	温度	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃
R	M(tf・m)	129.67	134.38	142.00	147.34	197.54	202.68	16627.0	16587.0
B	比　率	1.04		1.04		1.03		0.998	
H	温度	常温	-20℃	常温	-20℃	常温	-20℃	常温	-20℃
D	M(tf・m)	156.94	225.07	184.37	281.14	214.36	305.24	23321.0	21248.0
R	比　率	1.43		1.52		1.42		0.91	

表－8　線形および非線形解析の比較（曲げモーメント）

	P1	P2	P3
非線形（tf・m） A	655.62 (0.870)	743.86 (0.870)	823.27 (0.890)
線形（tf・m） B	128.93 (0.390)	147.38 (0.670)	194.14 (0.630)
比　率（A/B）	5.09	5.05	4.24

（　）内は発生時刻

表－9　等価剛性を変えた場合の曲げモーメントの比較

	P1		P2		P3	
	20cm	1次剛性	20cm	1次剛性	20cm	1次剛性
M（tf・m） C	403.81 (0.600)	444.08 (0.590)	440.75 (0.610)	484.69 (0.900)	434.94 (0.590)	483.2 (0.890)
比率(A/C)	1.62	1.48	1.69	1.53	1.89	1.70

（　）内は発生時刻

液状化性砂地盤上盛土の動的遠心模型実験（その２）

古賀泰之*1、古関潤一*2、高橋晃浩*3

要約

地盤が液状化した場合の盛土の地震時挙動を調べるために、加振加速度として異なる周波数の正弦波と２種類の不規則波を用いた動的遠心模型実験を行った。地盤の過剰間隙圧と盛土の沈下に及ぼす加振波形の影響と、不規則な加振波形を等価化する際の累積損傷度理論の適用性を明らかにした。

キーワード：盛土、液状化、遠心模型実験装置、
不規則波、累積損傷度理論

１．はじめに

砂地盤の液状化は、その上につくられた盛土の沈下を引き起こす。このような盛土の地震時挙動を明らかにするために、数多くの模型実験が今までに行われてきている。しかし、大きな拘束応力のもとで実験を行った例は少ない。

遠心模型実験は、縮小した模型内で実地盤と同じ応力状態を再現できる実験手法である。加振装置を用いることにより、動的な問題にも適用されてきている[1)2)3)]。

筆者らは、水平砂地盤と、その上に盛土がある場合を対象に、加振加速度として60Hzの正弦波を用いて行った動的遠心模型実験について既に報告した[4)]。ここでは、100Hzの正弦波と２種類の不規則波を用いた追加実験の結果を、上記の実験結果とあわせて報告する。

２．実験方法

長さ80cm，高さ30cm，奥行き10cmの箱型土槽の中に、図１に示す盛土モデルと水平地盤モデルを作成した。

砂層は、豊浦砂を用いて空中落下法により作成した。砂層の表面は、回転半径に応じて円弧状に成形した。盛土モデルでは、土槽前面の透明なガラスを通して砂層の変形状況を記録できるように、染色した砂を用いてメッシュを描いた。

土槽を負圧容器内に設置して、水の30倍の粘性をもつシリコンオイルで砂層を飽和させた。盛土は、豊浦砂とクレイサンドを４対１の重量比で混合し、含水比を15%に調整したものを用いて作成した。

30Gの遠心加速度を加えたのち、水平加振を行った。応答加速度、過剰間隙圧及び盛土の沈下量を図１に示す位置に設置した計器で測定した。実験条件の一覧を表１に示す。加振加速度として60Hzと100Hzの正弦波、及び２種類の不規則波を用いた。これらの加振波形を図２に示す。

３．データ整理・計算方法

計算方法は筆者らの既往の研究[5)]と同様である。

3.1 等価加振加速度

累積損傷度理論に基づいて、不規則波加振時について、等価な加振加速度を求めた。計算手順を図３に示す。

1) 動的せん断応力比の計算

間隙圧計を設置した位置（Ｐ１～Ｐ６）における有効上載圧σ_{vc}'と動的せん断応力比τ_d/σ_{vc}'を１次元的に算定した。

$$\sigma_{vc}' = \gamma' \cdot z + \gamma_t \cdot H \qquad (1)$$

ここに、γ'：有効単位体積重量
z：地盤表面からの深度
γ_t：盛土の湿潤密度
H：盛土高さ

$$\frac{\tau_d}{\sigma_{vc}'} = \frac{1}{\sigma_{vc}'} \sum_i \frac{\gamma i \cdot hi}{g} \cdot Ai \qquad (2)$$

ここに、σ_{vc}'：加振前の有効上載圧
Ai：i番目の層の応答加速度
hi：i番目の層の層厚
γi：i番目の層の砂の単位体積重量
g：重力加速度

液状化により応答加速度は増幅する場合も減少する場

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合もあるが、簡易化のために、計算上はこのような応答加速度の変化を無視した。

2) 液状化強度曲線の仮定

液状化強度曲線は、繰返しねじりせん断試験により求められた豊浦砂の液状化強度曲線を参考にして、(3)式により与えられるものと仮定した。

$\tau_d/\sigma_{mc}' \geq 0.3$ のとき；

$$\tau_d/\sigma_{mc}' - 0.3 = -1.25 \log (N_c/N')$$

$\tau_d/\sigma_{mc}' < 0.3$ のとき；

$$\tau_d/\sigma_{mc}' - 0.3 = -0.11 \log (N_c/N') \qquad (3)$$

ここに、N_c：応力比 τ_d/σ_{mc}' で繰返しせん断したときの液状化までの繰返し回数

N'：液状化強度曲線の位置を決める定数

σ_{mc}'：$K_0=0.5$ として σ_{vc}' から計算する平均有効主応力

(3)式を書き換えると次のようになる。

$$N_c/N' = f\ (\tau_d/\sigma_{mc}') \qquad (4)$$

ここに、f：応力比と繰返し回数比の関係を示す関数

3) 損傷度係数の計算

不規則なせん断応力比を半波毎に区切った場合の、i 番目の半波までの損傷度係数を次式により定義する。

$$D = \sum_i \frac{0.5}{(N_c)_i} \qquad (5)$$

ここに、$(N_c)_i$：i 番目の半波の応力比 $(\tau_d/\sigma_{mc}')_i$ で繰返しせん断したときの液状化までの繰返し回数

(4)式を(5)式に代入して得られた次式を用いて、損傷度係数の計算を行った。

$$D = \frac{1}{N'} \cdot \sum_i \frac{0.5}{f\{(\tau_d/\sigma_{mc}')_i\}} \qquad (6)$$

4) 等価加振加速度の計算

繰返し回数20回の規則的な繰返しせん断応力を、不規則波と同じ損傷度係数を与えるように作用させた場合の等価なせん断応力比 $(\tau_d/\sigma_{mc}')_{eq}$ は、次式により求めることができる。

$$D = \frac{1}{N'} \cdot \sum_i^{48} \frac{0.5}{f\{(\tau_d/\sigma_{mc}')_{eq}\}} = \frac{20}{N' \cdot f\{(\tau_d/\sigma_{mc}')_{eq}\}} \qquad (7)$$

平均有効主応力 σ_{cc}' を有効上載圧 σ_{vc}' に換算したのち、等価なせん断応力比を(2)式に代入して得られた次式を用いて、等価加振加速度 A_{eq} を求めた。

$$(\tau_d/\sigma_{vc}')_{eq} = \frac{1}{\sigma_{vc}'} \cdot A_{eq} \cdot \sum_i \frac{\gamma i \cdot hi}{g} \qquad (8)$$

等価化係数 C_{eq} を次のように定義して求めた。

$$C_{eq} = \frac{A_{eq}}{A_{max}} \qquad (9)$$

ここに、A_{max}：等価化を行う前の加振加速度の最大振幅

不規則波加振時に対する等価加振加速度 A_{eq} と盛土の沈下量の関係を整理し、正弦波加振時の関係との比較を行った。また、等価加振加速度 A_{eq} と過剰間隙圧との関係についても、同様な比較を行った。

3.2 液状化抵抗率

前節では液状化による応答加速度の変化を考慮しなかったが、本節では地盤中の動的せん断応力を詳細に評価するために、これを考慮した。

液状化強度曲線の位置を決める係数 N' は、前節における等価加振加速度 A_{eq} の計算値には影響を及ぼさなかったが、本節では地盤の液状化強度を評価するために N' の値の推定を行った。

1) 液状化強度の計算

損傷度係数Dを用いて、水平地盤モデルの液状化強度を次のようにして求めた。

仮定した N' の値が正しい場合には、$D \geq 1.0$ は液状化した状態を、$D < 1.0$ は非液状化の状態を示す。ゆえに、それぞれの状態について次式が得られる。

$$\text{液状化}\quad : N' \leq \sum_i \frac{0.5}{f\{(\tau_d/\sigma_{mc}')_i\}} \qquad (10)$$

$$\text{非液状化}: N' > \sum_i \frac{0.5}{f\{(\tau_d/\sigma_{mc}')_i\}} \qquad (11)$$

過剰間隙圧の測定値Δuが(1)式による有効上載圧の計算値σ_{vc}'に等しくなり、次式による過剰間隙圧比r_uが1.0となった状態を液状化として定義し、不規則波加振時と正弦波加振時の双方について間隙圧計の設置位置における液状化した時点の繰返し回数を算定した。

$$r_u = \frac{\Delta u}{\sigma_{vc}'} \qquad (12)$$

液状化した時点の繰返し回数までの動的せん断応力比を(9)式に、その繰返し回数の半波前までの動的せん断応力比を(10)式に代入することによって、液状化強度曲線の位置を決める定数N'の範囲を推定した。

2) 液状化抵抗率の計算

液状化抵抗率F_Lは次式で定義される。

$$F_L = \frac{R}{L} \qquad (13)$$

ここに、R：動的せん断強度比

L：地震時せん断応力比

同様に、i番目の半波までの等価な液状化抵抗率F_{Li}を次式により定義した。

$$F_{Li} = \frac{R_{20}}{L_i} \qquad (14)$$

ここに、R_{20}：規則波20波に対応する動的せん断強度比

L_i：i番目の半波までの等価なせん断応力比

推定したN'と繰返し回数$N_c=20$と(3)式に代入することにより、動的せん断強度比R_{20}を次のように算定した。

$N' \geqq 20$のとき；

$$R_{20} = 0.3 - 1.25 \cdot \log(20/N')$$

$N' < 20$のとき；

$$R_{20} = 0.3 - 0.11 \cdot \log(20/N') \qquad (15)$$

等価なせん断応力比L_iは、推定したN'を用いて、前節における$(\tau_d/\sigma_{mc}')_{eq}$の算定と同じ方法により算定した。

間隙圧計の設置位置における液状化抵抗率F_{Li}と過剰間隙圧の関係を整理し、この関係に及ぼす盛土の影響について検討した。

4．結果及び考察

4.1 測定データの時刻歴

高周波の不規則波と100Hzの正弦波を加振加速度として用いた場合の、盛土モデルの測定データの時刻歴をそれぞれ図5及び図6に示す。正弦波加振時の結果と比較した場合、不規則波加振時の特徴として以下の点が挙げられる。

(1) 過剰間隙圧(P2，P3及びP6)は、加振加速度振幅(A0)が大きくなるときに階段状に上昇する。

(2) 盛土の沈下(DV)は一定速度ではなく、加振加速度振幅が大きくなるときに沈下速度が大きくなる。

(3) 水平地盤部では、液状化により応答加速度(A1及びA2)が低下したあとで突然大きく応答する場合がある。

4.2 等価加振加速度

盛土モデルに対する不規則波を用いた1回目加振時の、等価加振加速度と等価化係数の算定結果を表2に示す。

ほとんどのケースでは、水平地盤部よりも盛土下の地盤のほうが等価化係数が大きく、また、盛土下の地盤では算定位置が浅くなるにしたがって等価化係数が若干大きくなった。これらの結果は、盛土下の地盤では有効上載圧が大きいために、動的せん断応力比が小さくなることが関与していると考えられる。

等価化係数は、高周波の不規則波のほうが低周波の不規則波よりも全般的に大きくなっている。これは、前者のほうが後者よりも大きな振幅の波数が多いためと考えられる。

以下では、等価加振加速度として、盛土下地盤の中間深さ(P5)における値を代表値として用いる。

4.3 盛土の沈下量

1回目加振時の加振加速度と盛土天端の沈下量の関係を図7に示す。ただし、加振加速度として、不規則波加振時は最大振幅を、正弦波加振時は平均振幅を用いた。同じ加振加速度に対する沈下量は、正弦波加振時のほうが不規則波加振時よりも大きかった。

一方、等価加振加速度と沈下量の関係は、図8に示すように加振波形によらずほぼ同一のものとなった。これは、不規則な加振波形を等価化するうえでの累積損傷度理論の有効性を示している。

周波数の影響に関しては、60Hzの正弦波を用いた加振では、100Hzの正弦波を用いた場合よりも大きな沈下を生じた。これは、低周波数の加振のほうが等価1波当振幅

が大きく、また、加振継続時間が大きくなることによると考えられる。筆者の実施した振動台実験[6]でも同様な結果が得られている。累積損傷度理論に基づく等価化の過程では、加速度応答特性の違いを除いて、このような周波数の影響を考慮することはできない。

4.4 地盤の過剰間隙圧

盛土モデルの1回目加振時について、地盤の最大過剰間隙圧比と等価加振加速度の関係を図9に示す。水平地盤部(P1～P3)についても、また、盛土下の地盤(P4～P6)についても、加振波形によらずほぼ同様の関係が得られた。これは、累積損傷度理論による不規則な加振波形の等価化が、盛土の沈下量だけでなく、地盤の過剰間隙圧に関しても有効であることを示している。

4.5 水平地盤の液状化強度

水平地盤モデルの液状化強度の算定結果を表3にまとめて示す。液状化強度曲線の位置を決める定数N'は1～3程度、動的せん断強度比R_{20}は0.16～0.21程度の値となり、加振波形の違いの影響は現れなかった。これは、地盤の液状化強度を算定する際に累積損傷度理論が適用可能であることを示している。

4.6 液状化抵抗率と過剰間隙圧比の関係

液状化抵抗率F_Lと過剰間隙圧比r_uの関係を、水平地盤モデルについて図10に、盛土モデルについて図11に示す。水平地盤モデルではそれぞれの位置で推定したN'を用いて液状化抵抗率を算定し、盛土モデルでは水平地盤モデルにおける代表的な値としてN'＝2を用いた。また、盛土モデルにおける盛土直下のP4での過剰間隙圧は、図5及び図6に示したように加振中は負圧となるため、データ整理の対象から除外した。これらの図より以下のことがわかる。

(1) 水平地盤の過剰間隙圧比は、浅い位置のP1及びP4ではF_L^{-2}に近く、その他の深い位置ではF_L^{-7}に近い。また、水平方向の測定位置（図10のP1～P3とP4～P6の比較）の影響は小さい。

(2) 盛土下の地盤（図11のP5及びP6）では、水平地盤部（P1～P3）よりも同じF_Lに対する過剰間隙圧比が小さい。

(3) (1)及び(2)の結果に及ぼす加振履歴（図10及び図11のステップ1とステップ2の比較）、及び加振波形の影響は小さい。

(1)及び(2)に関して、振動台実験を対象とした既往の結果[5]を図12に示す。この場合には、浅い位置のほうが深い位置よりも同じ液状化抵抗率に対する過剰間隙圧比が小さくなる傾向があり、また、水平地盤部と盛土下の地盤における液状化抵抗率と過剰間隙圧比の関係の差は小さい。今回の遠心模型実験で異なる結果が得られた理由は不明であり、今後の検討を要する。

5．まとめ

(1) 不規則波加振時には、砂地盤での過剰間隙圧の発生と、盛土の沈下は、加速度振幅が大きいときに進行した。

(2) 累積損傷度理論に基づいた不規則な加振波形の等価化は、地盤の過剰間隙圧と盛土の沈下量を評価するうえで有効である。

(3) 同じ液状化抵抗率に対する過剰間隙圧比は、盛土下の地盤よりも水平地盤部のほうが大きく、水平地盤部の地表面付近で最大になった。

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Table 1 Test Cases and Conditions

Model	Case	Type of Model Ground	Shaking Wave	Shaking Acc. (g)*						Dr of Sand Layer (%)
				1st	2nd	3rd	4th	5th	6th	
63-1	63-1-1	Embankment	Sinusoidal	1.2						***
	63-1-2		60Hz,20cycle	2.5	4.1	5.2	7.3	8.7	12.7	***
	63-1-3			3.7	5.6	7.1	8.3	12.5		***
	63-1-4			5.5						***
	63-1-5			7.3						***
	63-1-6			9.0						***
1-3	1-3-1	Horizontal Layer	Sinusoidal	2.3	3.3	4.5	5.6			64
	1-3-2		60Hz,50cycle	3.4	4.3	5.2				61
	1-3-3			4.5	5.6					59
2-1	2-1-1	Embankment	Sinusoidal	4.0	5.8	9.4				57
	2-1-2		100Hz,20cycle	5.4	8.5	8.6				62
	2-1-3			8.9	9.1					60
	2-1-4	Embankment	Irregular	6.0	8.6	10.8				60
	2-1-5		High Frequency	8.3	10.8					61
	2-1-6			4.3	6.3	7.7	10.6			63
	2-1-7	Embankment	Irregular	5.4	6.9	8.4				63
	2-1-8		Low Frequency	7.0	8.6					65
	2-1-9			8.7						66
2-2	2-2-1	Horizontal Layer	Sinusoidal	3.9	5.0	6.0				66
	2-2-2		100Hz,50cycle	3.4	4.5	5.6	6.6			68
	2-2-3	Horizontal Layer	Irregular	4.9	6.4	7.1	9.2			66
	2-2-4		High Frequency	8.1	8.9	11.1				65
	2-2-5	Horizontal Layer	Irregular	**	6.8	8.4				63
	2-2-6		Low Frequency	6.7	8.4					63

* An average amplitude is shown for sinusoidal shakings, and the maximum amplitude for irregular shakings.

** The shaking acceleration was not known due to mismeasurement.

*** The relative density of the sand layer for model 63-1 was not known due to mismeasurement. It was assumed to be almost the same as others, because the preparation method was the same.

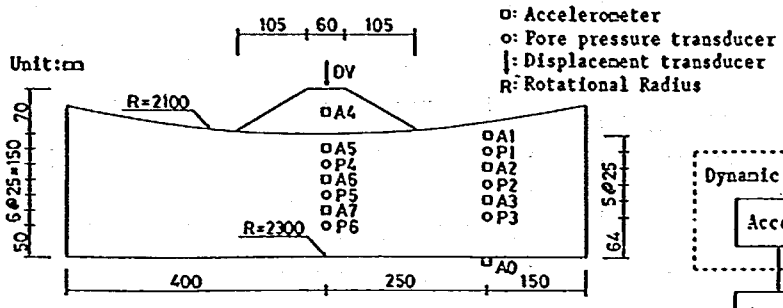
Table 2 Equivalent Shaking Accelerations and Coefficients of Equalization

Case	Shaking Wave	Equiv. Shaking Acc. A_{eq} (g) / Coef. of Equalization C_{eq}			
		Free Ground*	at P4	at P5	at P6
2-1-4	Irregular	3.0 / 0.50	3.8 / 0.64	3.6 / 0.61	3.5 / 0.59
2-1-5	High Frequency	6.8 / 0.82	4.7 / 0.57	5.6 / 0.68	6.2 / 0.75
2-1-6		2.4 / 0.55	2.6 / 0.61	2.6 / 0.61	2.5 / 0.59
2-1-7	Irregular	2.5 / 0.47	3.2 / 0.59	3.0 / 0.56	3.0 / 0.55
2-1-8	Low Frequency	2.8 / 0.40	3.7 / 0.53	3.5 / 0.49	3.3 / 0.48
2-1-9		3.7 / 0.42	4.1 / 0.48	3.9 / 0.44	3.7 / 0.42

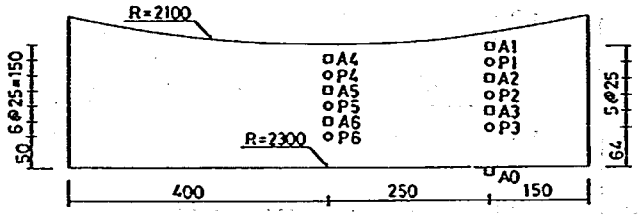
* In the free ground A_{eq} and C_{eq} were the same irrespective of the depth, because the water level was equal to the surface of the sand layer.

Table 3 Liquefaction Resistance in Free Ground

Case	Shaking Wave	Shaking Step	Point of Calculation	Liq. Resistance Ratio R_{20}	N' to Locate Liq. Resistance Curve
1-3-1	Sinusoidal 60Hz,50cycle	1st	P1	0.186	1.8
1-3-2	Sinusoidal 60Hz,50cycle	1st	P1	0.235	5.1
			P2	0.195	2.2
			P4	0.271	11.0
			P5	0.264	9.5
			P6	0.258	8.3
		2nd	P2	0.166	1.2
			P5	0.251	7.1
			P6	0.265	9.5
		3rd	P5	0.261	8.8
			P6	0.273	11.3
1-3-3	Sinusoidal 60Hz,50cycle	1st	P3	0.153	0.92
			P6	0.155	0.96
		2nd	P3	0.188	1.9
			P6	0.191	2.0
2-2-1	Sinusoidal 100Hz,50cycle	1st	P1	0.212	3.2
			P2	0.167	1.2
			P3	0.204	2.7
		2nd	P1	0.194	2.2
			P2	0.154	0.95
			P3	0.197	2.3
2-2-2	Sinusoidal 100Hz,50cycle	1st	P1	0.213	3.3
			P2	0.180	1.6
			P3	0.187	1.9
		2nd	P1	0.199	2.4
			P2	0.178	1.6
			P3	0.195	2.2
2-2-4	Irregular High Frequency	1st	P1	0.174	1.4
			P2	0.196	2.3
			P3	0.212	3.2
			P4	0.214	3.3
			P5	0.193	2.1
			P6	0.216	3.5
		2nd	P1	0.152	0.90
			P2	0.189	2.0
			P3	0.202	2.6
			P4	0.200	2.5
			P5	0.218	3.6
			P6	0.233	4.9
2-2-6	Irregular Low Frequency	1st	P1	0.200	2.5
			P2	0.147	0.81
			P3	0.176	1.5
			P4	0.266	9.9
		2nd	P1	0.207	2.8
			P2	0.168	1.3
			P3	0.194	2.2
			P4	0.231	4.7
			P5	0.235	5.2
			P6	0.211	3.1

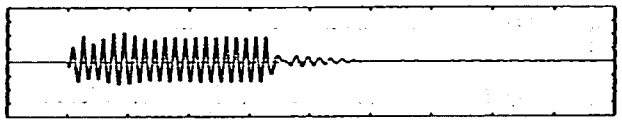


(1) Embankment Model



(2) Horizontal Layer Model

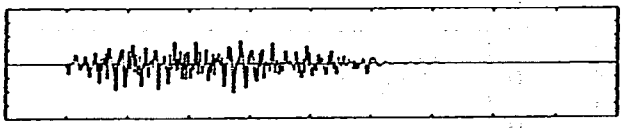
Fig.1 Test Models and Location of Transducers.



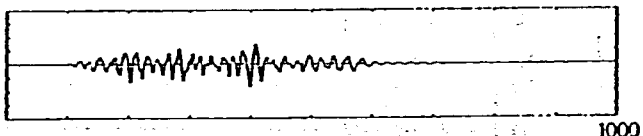
(1) Sinusoidal Wave of 60Hz (20cycle)



(2) Sinusoidal Wave of 100Hz (20cycle)



(3) Irregular wave of High Frequency



(4) Irregular wave of Low Frequency

Fig.2 Shape of Shaking Waves

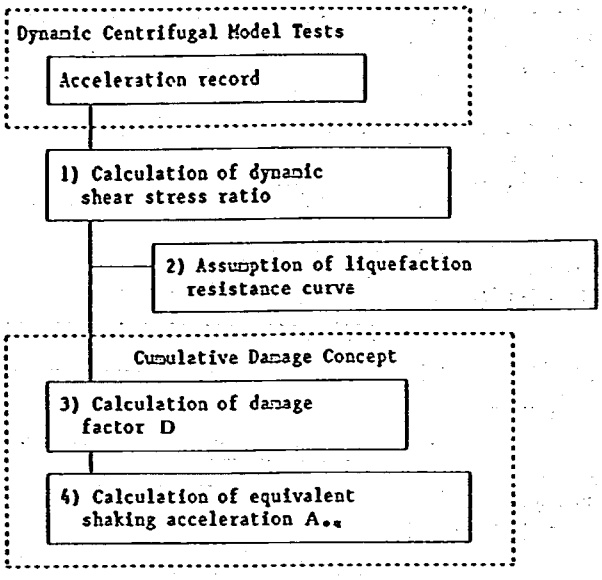
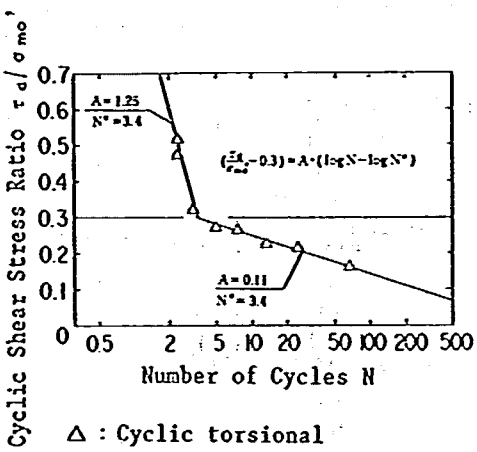


Fig.3 Procedure of Equalization



△ : Cyclic torsional shear test results
Toyoura sand, D_r= 55～60%
σ_{vo}'=1.0, σ_{ho}'=0.5(kgf/cm²)
Axial displacement confined
γ_{DA}=15%

Fig.4 Assumed Liquefaction Resistance Curve

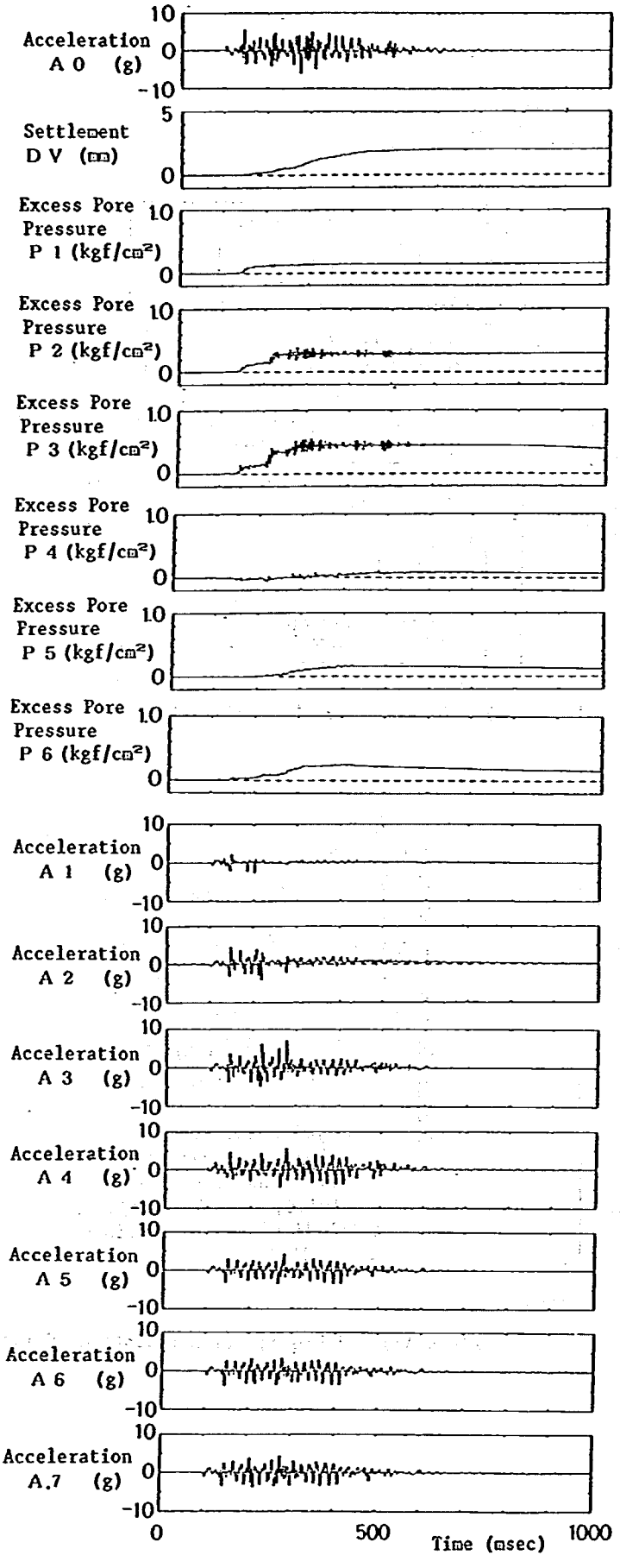


Fig.5 Measured Data of Embankment Models (Irregular Wave of High Frequency)

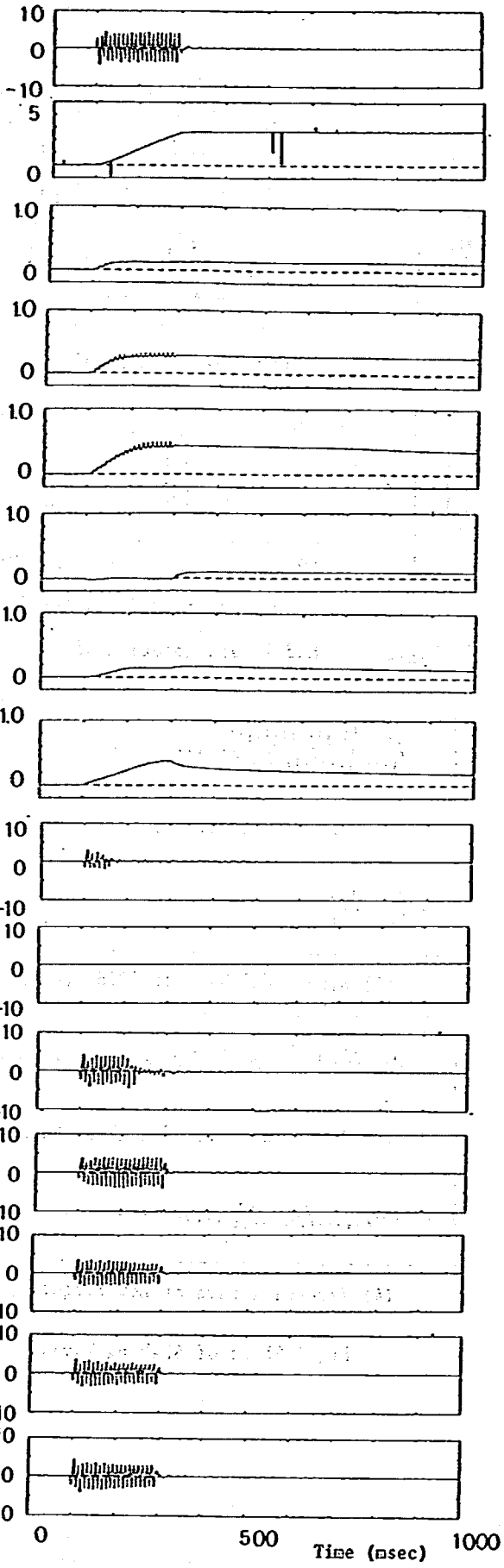


Fig.6 Measured Data of Embankment Models (Sinusoidal Wave of 100Hz)

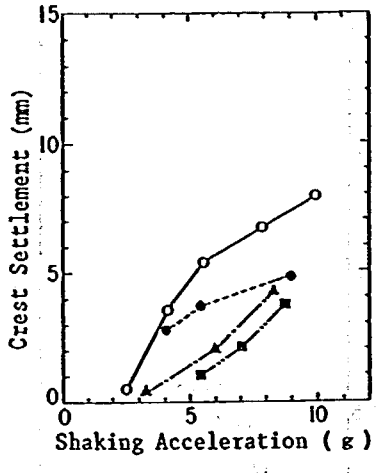


Fig.7 Shaking Acc. vs. Crest Settlement

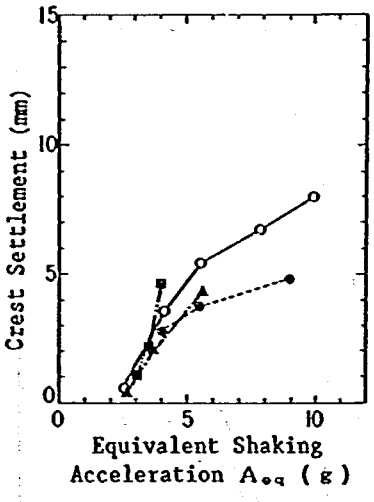
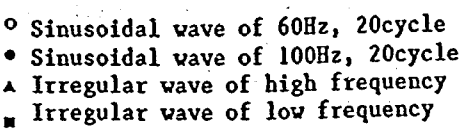
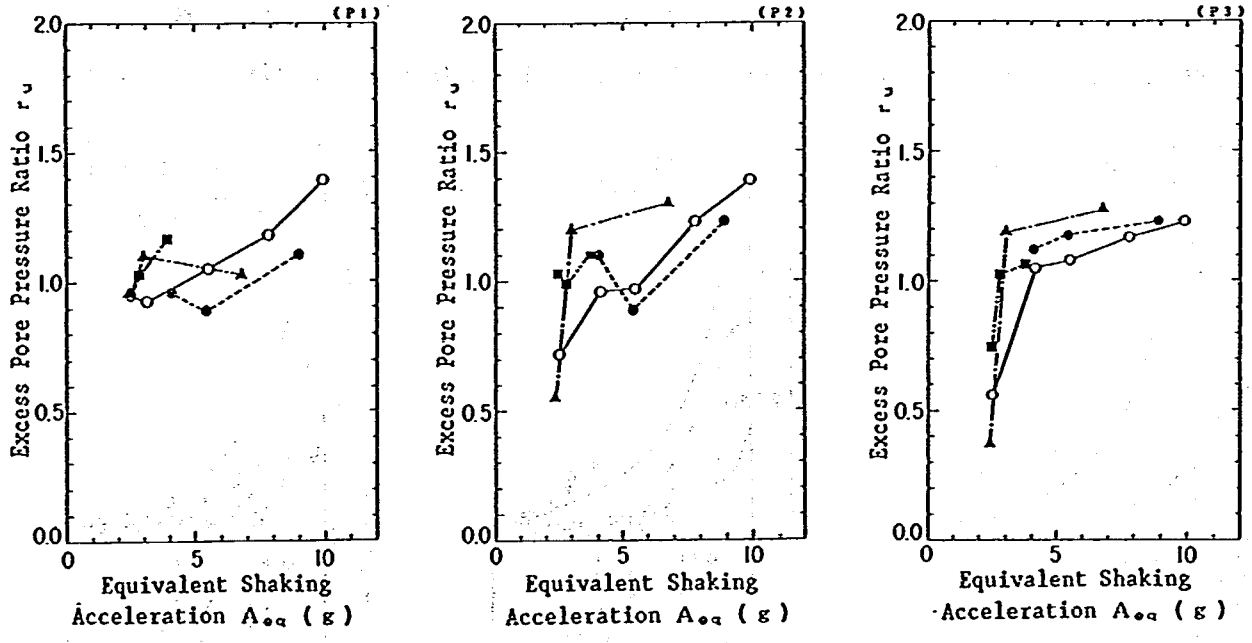


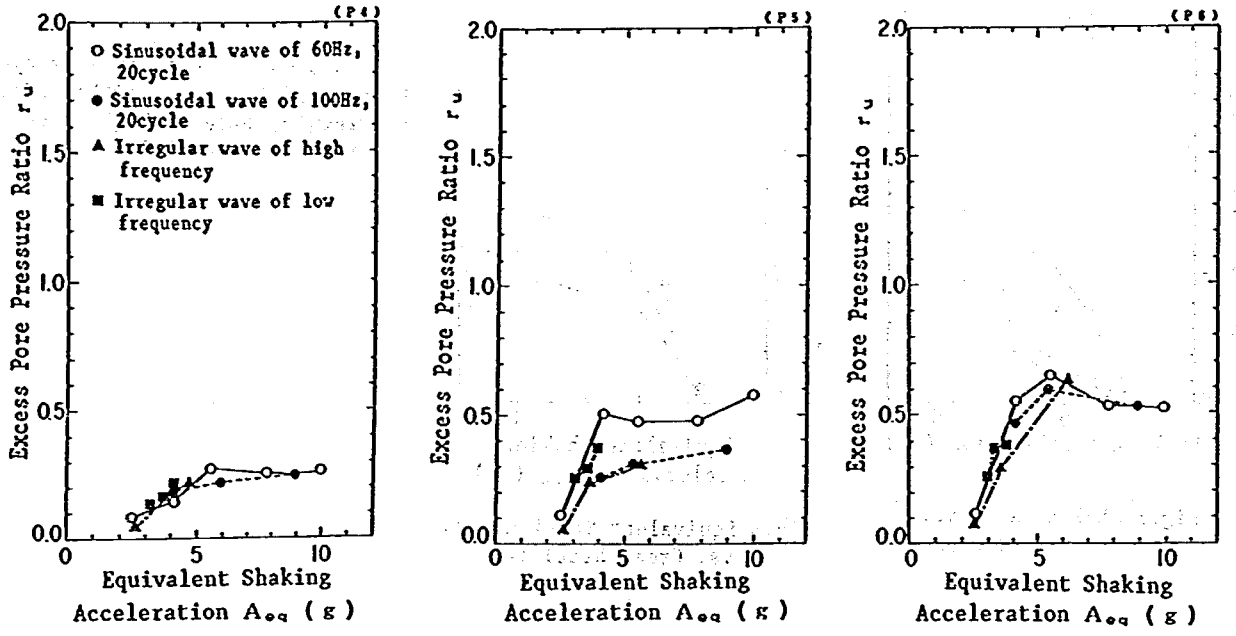
Fig.8 Equivalent Shaking Acc. vs. Crest Settlement





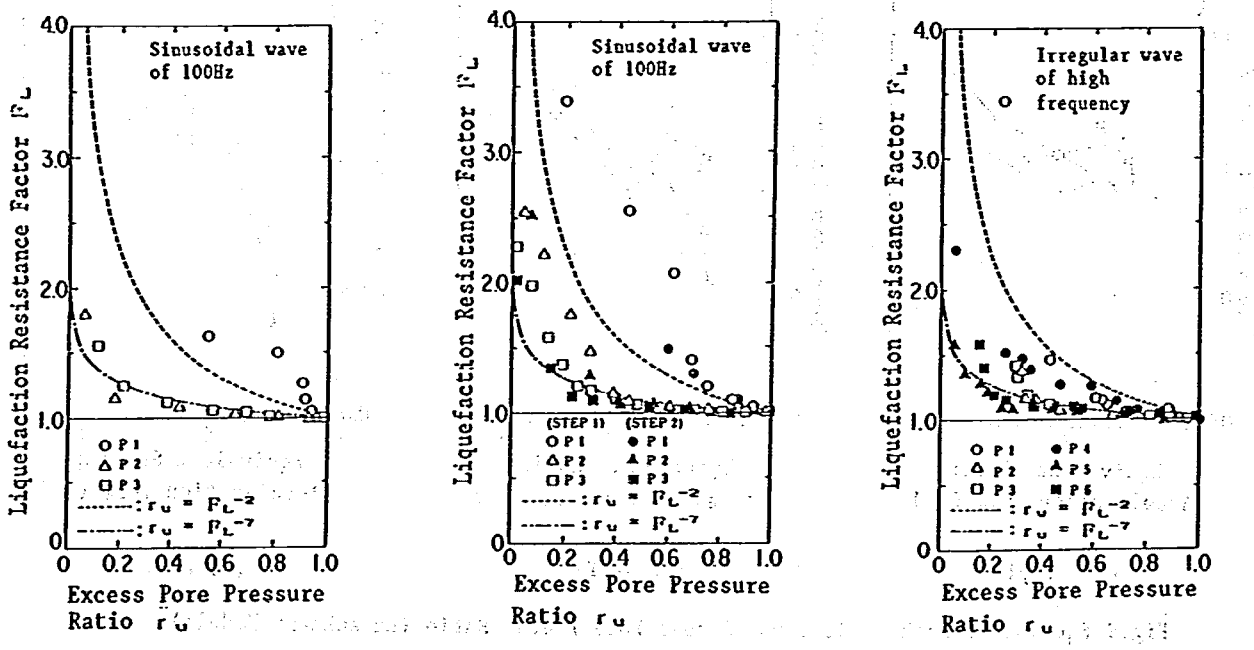
(1) at P1 (2) at P2 (3) at P3

Fig.9 Equivalent Shaking Acc. vs. Excess Pore Press. Ratio (Embankment Models)



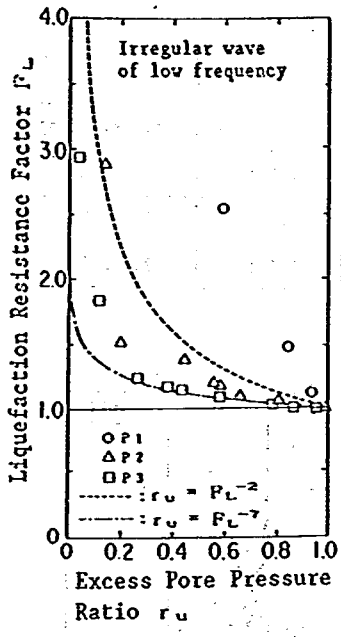
(4) at P4 (5) at P5 (6) at P6

Fig.9 Equivalent Shaking Acc. vs. Excess Pore Press. Ratio (Embankment Models) (continued)

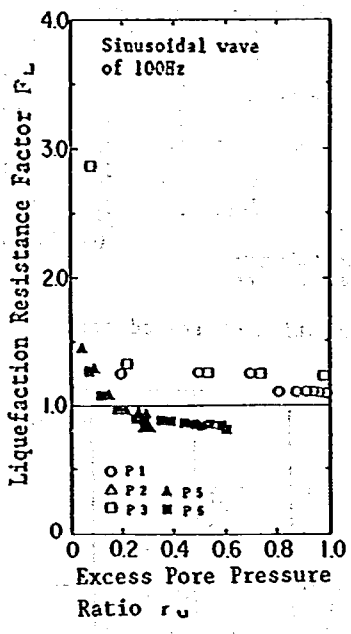


(1) Case 2-2-1, Step1 (2) Case 2-2-2, Step1 and Step2 (3) Case 2-2-4, Step1

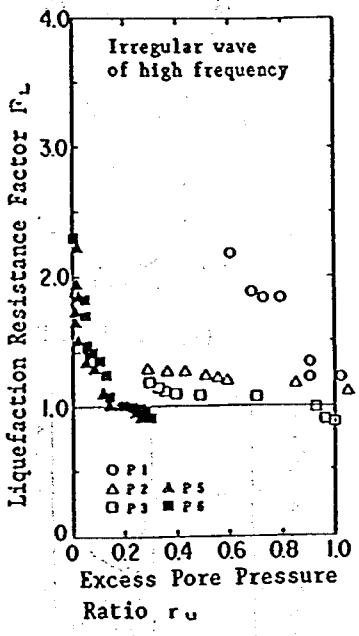
Fig.10 Liq. Resistance Factor vs. Excess Pore Press. Ratio (Horizontal Layer Models)



(4) Case 2-2-6, Step1

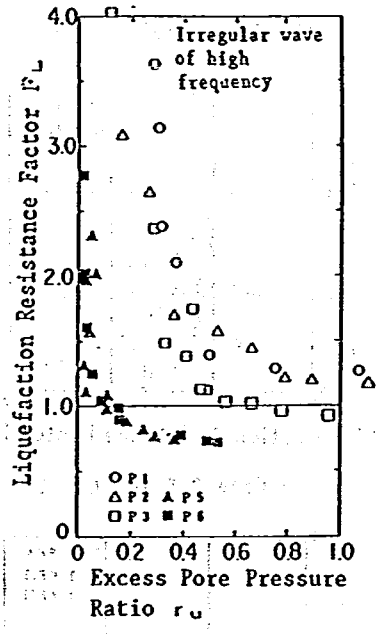


(1) Case 2-1-2, Step1

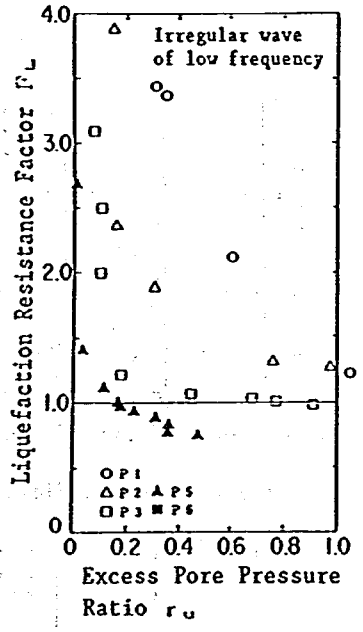


(2) Case 2-1-4, Step1

Fig.10 Liq. Resistance Factor vs. Excess Pore Press. Ratio (Horizontal Layer Models) (continued)

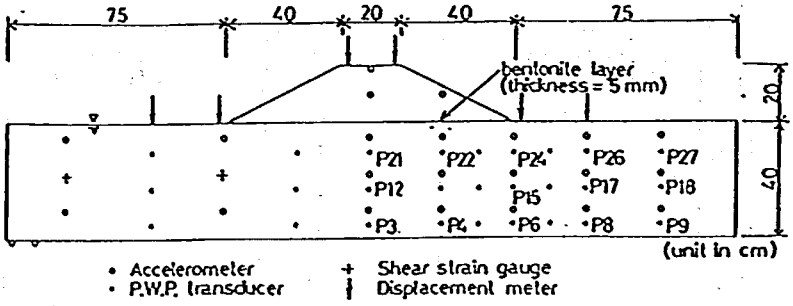


(3) Case 2-1-4, Step2



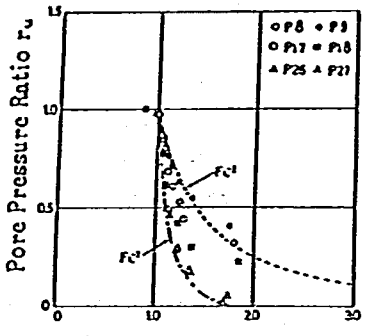
(4) Case 2-1-9, Step1

Fig.11 Liq. Resistance Factor vs. Excess Pore Press. Ratio (Embankment Models)

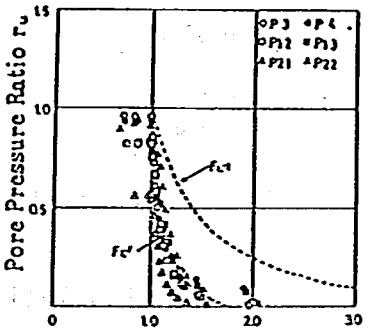


Case	Sinusoidal wave of 20cycle	
	Frequency	Acceleration
1	2.5Hz	190 gal
2	10.0Hz	315 gal
3	5.0Hz	208 gal

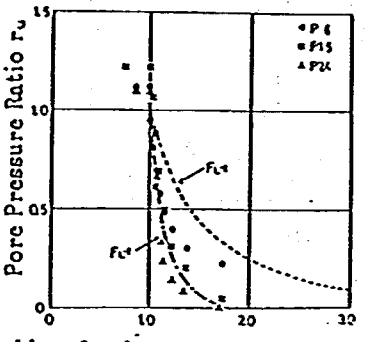
(1) Analized Model and Location of Transducers



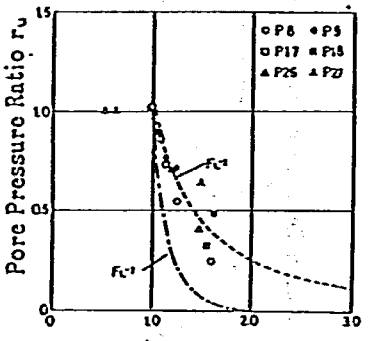
(2) Case 1 Free Field



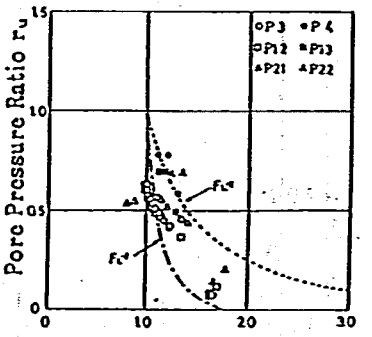
(3) Case 1 Below Crest



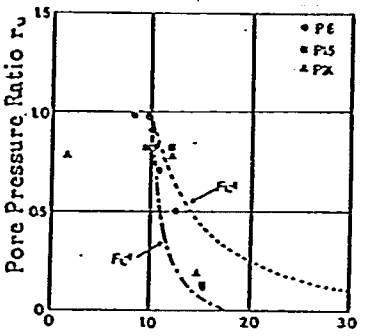
(4) Case 1 Below Slope Toe



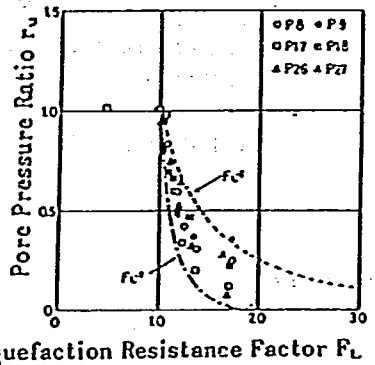
(5) Case 2 Free Field



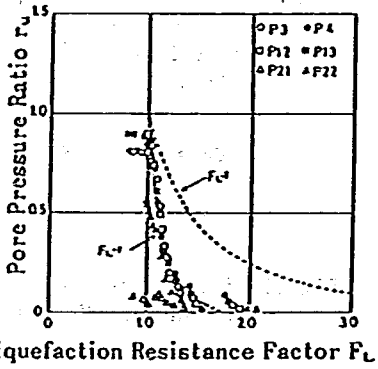
(6) Case 2 Below Crest



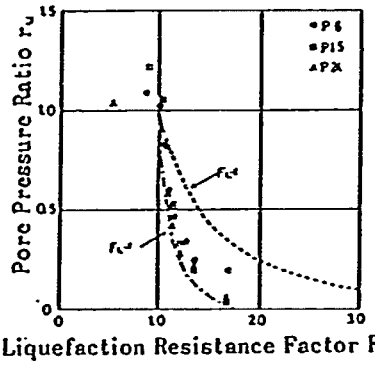
(7) Case 2 Below Slope Toe



(8) Case 3 Free Field



(9) Case 3 Below Crest



(10) Case 3 Below Slope Toe

Fig.12 Liq. Resistance Factor vs. Excess Pore Press. Ratio (Shaking Table Test[6])

Earthquake-Induced Landslides in the Santa Cruz Mountains

by

M. E. Hynes* and M. Hudson**

ABSTRACT

The magnitude 7.1 Loma Prieta earthquake which occurred at 5:04 p.m on October 17, 1989, ruptured a 25-mile long segment of the San Andreas fault in the Santa Cruz Mountains, California, and caused numerous landslides in the epicentral vicinity. A detailed study of the ground failure in this area is documented by Keefer et al. 1991. This paper summarizes slope stability and deformation analyses of damaged slopes in the Upper Schultheis Road and Villa Del Monte areas that were performed to assess the potential for future movement caused by earthquakes, intensive rainfall, or a combination of the two. The Loma Prieta earthquake and resulting slope movements provided a full scale field test of the strength of the materials in the slope. The stability and deformation analyses were used to back-calculate the mobilized strength of the in situ materials during the earthquake from the observed displacements and estimated ground motions, sliding surface locations, and groundwater conditions. This stability model was then adjusted to simulate intense rainfall conditions with higher groundwater levels, and used to assess static (no earthquake) slope stability for these same sliding surfaces. The resistance to earthquake-induced sliding deformations on postulated surfaces by future seismic events was determined for low groundwater and high groundwater conditions to simulate the relatively dry conditions that existed during the Loma Prieta event and the relatively wet conditions that would result from intense rainfall. It was concluded that (1) the slopes would become marginally stable when the water levels rise to within about 10 ft of the ground surface, and (2) if the Loma Prieta earthquake had occurred during a period of elevated groundwater level, then the maximum landslide displacements could have been three times as large or more; i.e., on the order of at least 25 ft rather than 8 ft.

1. INTRODUCTION

The 1989 Loma Prieta earthquake which occurred at 5:04 p.m on October 17, 1989, ruptured a 25-mile long segment of the San Andreas fault in the Santa Cruz Mountains, California, and caused numerous landslides in the epicentral vicinity. A detailed study of the ground failure in this area is documented by Keefer et al. 1991. This paper summarizes the engineering analyses of two landslide areas in the Santa Cruz Mountains, the Upper Schultheis Road landslide and the Villa Del Monte landslide complex, both of which are located in the epicentral region, about 1000 ft from the trace of the San Andreas fault. Location maps are shown in Figures 1 and 2, and mapped observations of surface cracking are shown in Figures 3 and 4. The geologic materials in the slide areas consist of colluvium and regolith on upturned and folded interbedded shale, sandstone and siltstone that have been complexly sheared and are variably weathered.

The Upper Schultheis Road landslide and the Villa Del Monte landslide complex were modeled in slope stability and deformation analyses first to backcalculate static and dynamic strengths of the slope materials. These backcalculated strengths were then used in the slope stability and deformation analyses to estimate the stability of the slopes under higher groundwater conditions and the displacements that could occur in future earthquakes. In this case history, backcalculated strengths are preferred to laboratory test results because they represent the strength of in situ materials averaged over the entire failure surface, which passes through several types of materials, at full-scale stress conditions. Laboratory tests were performed on selected samples, but these test results represent only point estimates of strength of individual materials in this complex geologic setting.

The slope stability and deformation analyses were calibrated using observed past performance of the slopes. The past performance of the slopes against which the models were calibrated are:

Condition (a): The slopes are generally stable under deep groundwater conditions and static loading as observed during periods of drought and low rainfall before and after the 1989 Loma Prieta earthquake.

Condition (b): Deep basal sliding surfaces of large areal extent are stable to marginally stable under elevated groundwater conditions and static loading, since areally extensive sliding of this type has not been observed during past periods of high rainfall.

Condition (c): Shallow sliding surfaces of limited areal extent are marginally stable to unstable under elevated groundwater conditions and static loading, as observed from localized slope movements experienced in the slopes during past periods of high rainfall.

Condition (d): The severe earthquake ground shaking caused by the 1989 Loma Prieta earthquake triggered instability of deep basal sliding surfaces of large areal extent and resulted in downslope displacements as large as approximately 8 ft. This occurred during a period of drought when the groundwater levels were relatively deep.

The models were developed to reflect condition (a), and then slope stability for conditions (b) and (c) were computed. Thus, it was concluded that the models provided reasonable estimates of averaged static strengths of the slope materials.

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The slope stability and deformation models were then adjusted to represent the dynamic loading case in condition (d). The computed dynamic strengths from this case were compared to the estimated static strengths, and it was found that the dynamic strengths reflected typical levels of strength degradation associated with dynamic loading and displacements on the order of several feet (Idriss 1985, Seed 1987). Thus it appears that the analytical models provided realistic estimates of both static and dynamic strengths of the slope materials.

The backcalculated static and dynamic strengths were then used to develop charts and procedures from which slope displacements caused by future earthquakes and groundwater conditions could be estimated. For example, the analyses indicated that had the groundwater levels been significantly higher when the 1989 Loma prieta earthquake occurred, then the maximum displacements could have been three times as large or more; i.e., on the order of at least 25 ft rather than 8 ft.

This paper describes the slope stability and displacement models that were developed, and presents the resulting charts and procedures for use in analysis of slope stability for a range of earthquake and groundwater conditions.

2. STABILITY AND DEFORMATION APPROACH

In the Newmark sliding-block deformation analyses adopted for this study, the part of a slope displaced by earthquake shaking was idealized as a rigid block sliding on an inclined plane, as originally proposed by Newmark (1965). Since 1965, a number of researchers (Goodman and Seed 1966, Ambraseys and Sarma 1967, Sarma 1975, Franklin and Chang 1977, Hynes 1979, Makdisi and Seed 1979, Sarma 1979, Wilson and Keefer 1983, Hynes and Franklin 1984, Idriss 1985, Wilson and Keefer 1985) have developed Newmark's conceptual model into a widely used, practical procedure for estimating permanent, earthquake-induced deformation of slopes.

A conventional, limit-equilibrium slope stability analysis with slight modifications was used to determine the shearing resistance along a failure surface. Because the base motions may be amplified upon being propagated upward through a slope, consideration of the dynamic response of the slope was incorporated to account for amplified accelerations in the slope. The shearing resistance determined in the limit-equilibrium slope stability analysis was then used to determine the relative displacement that will accumulate along a slide surface for a representative group of earthquake ground-motion histories. Each of these steps is described in greater detail below.

3. LIMIT-EQUILIBRIUM SLOPE STABILITY ANALYSES

3.1 Method used

A two-dimensional, force-equilibrium solution method using Lowe and Karafiath's side force assumption was used for the slope stability calculations (Lowe and Karafiath 1960). The programs used were UTEXAS2 (Edris and Wright 1987) and UTEXAS3 (Wright 1990). In this method, a pseudo-static horizontal acceleration, k_h, expressed as a fraction of gravity, can be applied to the center of gravity of the sliding mass. The value of k_h that causes the factor of safety against sliding of the slope to equal unity is termed the yield acceleration, k_y. A parametric study was performed to determine values of k_y and static (k_h = 0) safety factors against sliding, FS_S, for a range of shear strength values and groundwater conditions. A unit weight of 134.5 pcf was used as an average estimate for all materials in the slope based on laboratory test results.

3.2 Geometry of Failure Masses

An analysis of the field evidence used to estimate the geometry of the failure masses and locations of the shear surfaces is described by Keefer et al. (1991). Several sliding surface locations were assumed in the slope stability analyses ranging from deep surfaces (greater than approximately 100 ft) to fairly shallow surfaces (approximately 50 ft or less). This range provided (1) insight about the controlling mechanism of failure acting in the slope during sliding caused by the 1989 Loma Prieta earthquake, (2) an indication of the sensitivity of the results to possible slip surface locations, and (3) information about potential slope stability during future rainfall and earthquake events.

A single cross section, denoted SR, was selected through the Schultheis Road landslide area, as shown in Figures 5 (plan) and 6 (section). Two cross sections, denoted DM and SB, were chosen through the Villa Del Monte landslide complex, as shown in Figures 7 (plan) and 8-9 (sections). Estimated deep-seated basal surface locations are shown in Figures 6, 8 and 9 for sections SR, DM and SB, respectively. Additional assumed shear surfaces are shown in Figures 10-12 for sections SR, DM and SB, respectively.

3.3 Groundwater Levels

A range of water-level conditions were assumed in the analyses to simulate the range from the 1989-1990 drought conditions to high groundwater levels, such as may be caused by intense or long-duration rainfall. Monitoring of groundwater levels is discussed in greater detail by Keefer et al. (1991). A key step in the stability analyses is to estimate the pore pressures that existed in the slope just prior to the earthquake; it is estimated that the piezometric levels measured during the spring of 1990 were indicative of those that existed just prior to the earthquake. Since no significant rainfall occurred in the area for several months prior to the earthquake, and the winter and spring months of 1990 had a similar rainfall environment, it is estimated that transient effects related to the earthquake had passed and the groundwater regime had again reached a steady-state condition similar to that before the earthquake. Estimated groundwater levels along the three analyzed sections are shown in Figures 6, 8 and 9. Due to the complex stratigraphy, it was expected that the in situ groundwater pore pressure fields are also complex. However, for the purposes of analysis, hydrostatic pore pressure fields were used as an approximation of the in situ conditions.

In the Upper Schultheis Road landslide, Figure 6, a hydrostatic groundwater level at a depth of 50 ft was used to represent pre-earthquake pore pressure conditions. Analyses were

conducted for hydrostatic groundwater levels ranging from 0 to 50 ft below the ground surface to represent the effects of low to high rainfall.

In the Villa Del Monte area, Figures 8-9, the pre-earthquake groundwater level was assumed to be at a depth of 100 ft, and analyses were conducted for hydrostatic groundwater levels ranging from 0 to 200 ft below the ground surface for sensitivity analyses and to represent groundwater conditions for low to high rainfall.

3.4 Shear Strength

Friction angles represented the shear strength on the stability computations. Friction angles ranging from 10½ to 40½ were used in the parametric studies.

3.5 Parametric Results from Slope Stability Calculations

The sliding masses bounded by deep-seated basal shear surfaces shown in Figures 6, 8 and 9 were analyzed in a detailed parametric study for a wide range of shear strength values and groundwater levels. The resulting relations determined between k_y, mobilized friction angle and groundwater level are shown in Figures 13-18. Minimum values of friction angle required for static stability for the range of groundwater levels correspond to k_y = 0 in these plots and range from 11½ to 24½. These figures were used to estimate the in situ strength mobilized during earthquake-induced sliding and critical levels of groundwater increase, as described later in this paper.

4. ESTIMATED GROUND MOTION AND SLOPE AMPLIFICATION

The next step in the Newmark analysis was to estimate the ground motions (accelerations) that were generated at the base and crest of the mountain by the Loma Prieta event, particularly the peak horizontal ground accelerations. No strong-motion instruments were located in the immediate area. Two California strong-motion instrument stations were close to the epicenter of the earthquake--Corralitos, to the south, at an epicentral distance of approximately 10 km, and Lexington Dam, to the north, at an epicentral distance of approximately 18 km (Shakal and others 1989). For the Loma Prieta event, the peak horizontal ground accelerations at Corralitos were 0.48 and 0.63 g; at Lexington Dam (left abutment), the peak horizontal ground accelerations were 0.41 and 0.44 g.

The Upper Schultheis Road landslide and Villa Del Monte landslide complex are located approximately 10 km north of the earthquake epicenter. The Krinitzsky, Chang and Nuttli (1988) attenuation chart shown in Figure 19 is based on a large body of California data, and indicates a peak horizontal ground acceleration of about 0.63 g would be estimated for a magnitude 7 earthquake occurring at a distance of 10 km; about 0.40 g would be estimated at a distance of 18 km. Since the recorded peak acceleration (maximum of two horizontal records) at Corralitos located at the same distance as the slide areas and the predicted value from a large body of California data both result in a peak acceleration of 0.63 g, it was concluded that the observed values at Corralitos of 0.48 and 0.63 g were realistic estimates of the peak motions at a rock outcrop in the study area.

The possible amplification of the base motions as they propagated upward through the slope to the ridge crest was estimated from consideration of typical values from analytical studies and field observations of natural slopes and embankments, and from a simplified shear-beam numerical model described by Makdisi and Seed (1979).

Typically, analytical studies and field observations indicate that embankments amplify base (toe-of-slope) accelerations, a, by factors ranging from approximately 1 to 4 to obtain peak crest accelerations, u. In special cases, amplification factors greater than 4 have been observed in the field (based on data from Hynes and Franklin 1984, and the Corps of Engineers Strong-Motion network). At high levels of acceleration, embankment amplification factors tend to approach unity, as shown in Figure 20 (from Harder 1991). Simplified shear-beam models tend to yield fairly high amplification factors; values of 4 to 5 were computed in this case. A range of amplification values was used in the sensitivity analysis shown in Table 1 to estimate the pseudo-static acceleration, k_{max}, most appropriate to represent the earthquake motions in the deformation calculations.

The yield acceleration, k_y, from the slope stability analyses is compared to the mass averaged acceleration, k_{max}, induced by the earthquake in the sliding mass. Both shear-beam and finite-element numerical analyses have been used in earlier studies (Sarma 1979, Makdisi and Seed 1979, Hynes and Franklin 1984) to develop generalized charts to compute representative values of k_{max} for a range of sliding mass shapes, slope stiffnesses, and geometries (which affect amplification), and incoming earthquake ground motions.

The value of k_{max} should range from about 0.75 to 1.5 times the base motion on the basis of numerous dynamic finite element analyses of slopes (Idriss 1968, Idriss 1991). The chart for estimating k_{max} from embankment and slope crest peak accelerations developed by Makdisi and Seed (1979) is shown in Figure 21. This chart indicates that deep-seated slides typically have ratios of k_{max} to crest acceleration of about 0.20 to 0.45, depending on the amount of amplification of the accelerations in the mountain. Based on the sensitivity analysis shown in Table 1, values of k_{max} of 0.5 to 0.6 seem to best represent the dynamic load for the 1989 Loma Prieta event. These values were used in the deformation calculations described next.

5. DEFORMATION CALCULATIONS, ESTIMATED FIELD STRENGTHS, AND CALCULATION OF STATIC STABILITY

Charts of the relative displacement, u, that can accumulate between a sliding block and an inclined plane for a range of ratios of k_y/k_{max} have been developed by Franklin and Chang (1977), Seed and Makdisi (1979), and Hynes and Franklin (1984), based on many earthquake accelerograms. The Seed and Makdisi (1979) curves were selected for this study. Generalized displacement curves from Seed and Makdisi (1979) for earthquake magnitudes of 6.5 and 7.5 were averaged to estimate a curve for magnitude 7 earthquakes. This estimated curve used to backcalculate mobilized strength is shown in Figure 22.

The deformations observed in the field were analyzed to estimate the displacement of the center of gravity (CG) of the sliding mass for the various failure surfaces. Both the upper bound and average displacements were used, determined as a sum of observed displacements along each cross section analyzed. The displacements are listed in Table 2 (from Keefer et al. 1991). The total displacement is approximately the cumulative value of downslope displacements observed along the ground surface along and near the limits of a section. Because the sliding mass is compressed during movement, it was estimated that the CG displacement might best be associated with about half of the total displacement (Keefer et al. 1991). Thus, half of the total displacement was defined as the average displacement and represented the best estimate for the backcalculation of strength.

The estimated CG displacements were analyzed with the sliding block model to determine the mobilized shear strength in the field and to estimate the amount of groundwater rise that would lead to static slope instability. The computations are shown in Tables 3 and 4 for upper and lower deep-seated basal shear surfaces identified in Figures 6, 8 and 9. Backcalculated dynamic friction angles for the additional assumed surfaces shown in Figures 10-12 are listed in Tables 5-6.

The computed mobilized friction angles for deep basal sliding surfaces ranged from approximately 16 to 19 for the Upper Schultheis Road landslide and from about 18 to 23 for the Villa Del Monte landslide complex. These backcalculated strengths are from dynamic loading conditions and are typically lower than the strength actually available for static loading conditions. The calculations shown in the last three columns of Table 4 were performed to obtain conservative estimates of the stability of the slopes and the critical rises in groundwater levels for triggering slope instability under static conditions. These conservative results indicated that a groundwater rise of 15 to 30 ft, from a depth of 50 ft to 20 to 35 ft below the ground surface in the Upper Schultheis Road landslide would result in renewed landslide movement without earthquake shaking. At Villa Del Monte, the range of computed critical groundwater levels was broader; the results suggested that some analyzed sliding masses would become statically unstable with a groundwater level increase of as little as 30 ft, while others would remain stable with rises of more than 100 ft (which would require artesian conditions). Computed static factors of safety using dynamic strengths, with estimated pre-earthquake groundwater levels, were in the range of 1.07 to 1.16 for the Upper Schultheis Road landslide and 1.15 to 1.62 for the Villa Del Monte landslide complex. These results are summarized in Table 4. These safety factors for the low 1989-1990 groundwater levels are all quite low, but greater than unity, even when the low dynamic strengths are used in the computations.

The strengths and static stability conditions discussed above are backcalculated from a dynamic loading event. Usually, there is some degradation of strength caused by dynamic loading of geologic materials, especially if they are below the water table. To estimate the amount of strength degradation, the additional assumed sliding masses shown in Figures 10-12 were analyzed. The surfaces were chosen based upon estimated critical locations of slope geometry. Two cases were analyzed to determine (1) the friction angle that would have been mobilized during the Loma Prieta event, and (2) the friction angle required to have a static factor of safety against sliding of unity if the water table is near the surface of the slope, at a depth of approximately 10 ft. These results are listed in Table 5 for shallow (approximately 50 ft or less) surfaces and Table 6 for deep (approximately 100 ft or more) surfaces.

Localized movements have been observed in these slopes after periods of heavy rains. The strengths of the materials in the slope were probably at least as high as the values calculated for static conditions with the water table near the surface and the factor of safety against sliding equal to one. If this were not the case, large failures would have been expected to be seen as the water table rose at times in the past. Instead, localized slope movements have been observed during wet weather, indicating a factor of safety near unity. The strength values in Tables 5 and 6 indicate that the friction angles for the static high-water case are larger than those necessary to produce the observed displacements during the 1989 Loma Prieta event. Therefore, in order to satisfy both conditions, the strength of the slope materials must have degraded due to the shaking of the earthquake.

Degradation of strength is most likely to occur in saturated conditions. The materials along shallow surfaces were not saturated at the time of the earthquake. All of the deep failure surfaces had some of their lengths at or below the water table. Therefore, it is most likely that materials along deep surfaces experienced strength degradation. It was therefore concluded that deep-seated sliding surfaces were more likely to have been responsible for the landslide activity in the Upper Schultheis Road and Villa Del Monte areas, with a reduction in strength occurring in the materials along the deep surfaces due to earthquake shaking.

Another approach was used to estimate the strength of the materials in the slope under static loading conditions. It is known that the slopes were stable before the 1989 Loma Prieta earthquake, with the groundwater table at a depth of about 50 ft in the Upper Schultheis Road area and about 100 ft in the Villa Del Monte area. A factor of safety against sliding of 1.5 was used as a reasonable estimate for the static factor of safety of the slopes for these groundwater conditions. A series of slope stability calculations was performed using these additional slip surfaces to determine the strengths required to have a static factor of safety of 1.5. These strengths were then used to compute the static factor of safety with the water table near the surface. The results of these calculations are shown in Table 7. As can be seen from this table, the friction angles thus generated seem reasonable in that they provide a low factor of safety against sliding after the water table has risen significantly. This correlates well with the observations of localized movements during periods of high water table.

In summary, the strengths backcalculated from the Loma Prieta 1989 event probably

underestimate the available strength of the materials in the slope for the static case of a rise in groundwater levels because of strength degradation of saturated materials due to dynamic loading. The static strengths are likely to be greater than the backcalculated dynamic values and approximately equal to or less than the static values listed in Tables 5-7. Also, if groundwater levels rise, slope movements may involve both shallow and deep surfaces.

6. SLIDING DURING FUTURE EARTHQUAKES WITH HIGHER GROUNDWATER LEVELS

The Newmark sliding block calculation procedure demonstrated in Table 3 can be used in conjunction with the yield accelerations in Figures 13-18 to estimate the potential for further sliding-block displacement in future earthquakes with higher groundwater levels. For example, displacement computations that simulate occurrence of the Loma Prieta-type event with higher groundwater levels indicate that displacements on a deep seated shear surface along cross section SB, in the Villa Del Monte landslide complex, might have been two to three times greater than they actually were if the groundwater level had been at a depth of 50 ft rather than 100 ft during the earthquake. This is determined by observing in the last column in Table 3 that the average mobilized friction angle for the slip surface denoted SB-U was approximately 21°. One then enters Figure 17 at a friction angle of approximately 21° to find the value of k_y corresponding to a water table depth of 50 ft, by intersecting this contour. The corresponding value of k_y is approximately 0.03. Then, one can compute the ratio k_y/k_{max} using a value of k_{max} equal to 0.5 for a nearby magnitude 7 event. The resulting ratio is 0.03/0.5 = 0.06. One then enters the displacement chart in Figure 22 at a ratio value of 0.06. For a magnitude 7 event, this yields a displacement of about 43 inches (110 cm). This displacement value is twice as large as the estimated average observed value of 22 inches (55 cm) during the Loma Prieta event when the water table was at a depth of approximately 100 ft.

If an earthquake occurs when the water table are higher in the slopes, more materials will be saturated and likely to suffer some degradation of strength due to earthquake shaking; consequently, for these conditions, slope instability may not be limited to areally extensive, deep-seated basal shear surfaces, but may also include shallower surfaces and deep surfaces with more limited lateral extents.

The occurrence of an earthquake when the water table is near the surface of the slopes could result in large displacements if the epicenter is close to the landslide areas. The meaning of "close" varies with the magnitude of the earthquake event and the depth to the water table at the time of the event. For the relatively deep water tables that existed during the Loma Prieta event, a magnitude 7 epicenter would need to be as close as approximately 10 miles (16 km) to cause calculated slide displacement on Villa Del Monte slopes of more than a foot; for the Upper Schultheis Road area, the magnitude 7 epicenter could be as far away as about 25 miles (40 km) and still cause this magnitude of displacement. It should be noted that the southern end of the Hayward-Calaveras fault is approximately 25 miles away from the area involved in this study.

7. SUMMARY AND CONCLUSIONS

Slope stability and deformation analyses of the Upper Schultheis Road landslide and Villa Del Monte landslide complex were performed to backcalculate the strengths mobilized in the slopes during the 1989 Loma Prieta earthquake, and to use this and other field performance information to assess the potential for future movement caused by earthquakes, rainfall, or a combination of the two. Both shallow and deep block sliding were analyzed using a limit-equilibrium pseudo-static slope stability procedure (Lowe and Karafiath 1960) and a Newmark sliding-block deformation analysis (Newmark1965). These analyses indicated the following:

(a) It is likely that the saturated slope materials suffered some degradation of shear strength due to earthquake shaking. Strengths backcalculated from observations of the Loma Prieta earthquake can be used to assess future earthquake stability, but are likely to underestimate the available strength for static loading conditions.

(b) The analyses confirm that deep-seated basal shear surfaces were the likely controlling failure mechanism for these landslide areas in the Loma Prieta event, since these deeper materials were saturated and subject to earthquake-induced strength degradation. It is unlikely that the controlling slope failure mechanism for these landslides was shallow slip surface movement since these materials were well above the water table and would not be likely to have suffered earthquake-induced strength degradation.

(c) If there is significant rise in groundwater levels, slope failure mechanisms may include shallower and deeper slip surfaces of limited areal extent. Large slope failures have not been observed in this area during past heavy rainfall events, but rather have involved small movements on sliding masses of limited extent. This past field performance of the slopes and the analyses indicate that the factor of safety against sliding for such surfaces is close to unity when the water table is near the ground surface.

(d) If there is a significant rise in the water table and another earthquake shakes the slopes, the slope failure mechanisms may include deep basal shear surfaces of large areal extent as well as shallow and deep surfaces of limited areal extent. This is because the shallower slope materials would then be saturated and subject to earthquake-induced strength degradation.

(e) If the Loma Prieta event had occurred when the water table was higher, maximum slope displacements might have been on the order of 25 ft rather than 8 ft. This level of displacement was indicated for deep basal shear surfaces as well as the shallow and deep surfaces of limited areal extent for a nearby magnitude 7 seismic event.

Consequently, the analyses indicated that deep-seated basal shear slide masses could become marginally stable when the groundwater rises to within about 20 ft of the ground surface at the Upper Schultheis Road landslide

area and to within about 30 ft of the ground surface at the Villa Del Monte landslide complex. If the groundwater rises to within about 10 ft of the ground surface, the slopes could become marginally stable against deep-seated, large scale, basal shear sliding; the computed safety factors for this case ranged from about 1 to 1.2. The analyses also indicated that shallow and deep surfaces of more limited areal extent may possibly become unstable to marginally stable if the groundwater rises to within 10 ft of the ground surface; the computed safety factors in this case ranged from about 0.8 to 1.1. However, if an earthquake occurs nearby when the water table is high, there may be extensive damage.

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Table 1

Sensitivity Analysis of Dynamic Load, K_{eax}

Acceleration, g				
Base a	Crest ū	Ratio ū/a	Dynamic Load Ratio K_{eax} / ū	Dynamic Load K_{eax}, g
0.4	2.4	6	0.20	0.48
0.4	2.0	5	0.25	0.50
0.4	1.6	4	0.30	0.48
0.4	1.2	3	0.35	0.442
0.4	0.8	2	0.40	0.32
0.48	1.44	3	0.35	0.50
0.5	2.5	5	0.25	0.63
0.5	2.0	4	0.30	0.60
0.5	1.5	3	0.35	0.53
0.5	1.0	2	0.40	0.40
0.6	2.4	4	0.30	0.72
0.6	1.8	3	0.35	0.63
0.6	1.2	2	0.40	0.48
0.63	1.26	2	0.40	0.50
0.7	2.1	3	0.35	0.74
0.7	1.4	2	0.40	0.56

Based on sensitivity analysis above,
K_{eax} = 0.5 to 0.6 g selected for displacement computations

Table 2

Summary of Slope Displacements
Provided by the T.A.G.

Section	Displacement	
	Total in. cm	Average in. cm
Villa del Monte		
DM	28 72	14 36
SB	44 111	22 36
Schultheis Road		
SR	96 244	48 122

Table 3

Displacement Computations for Villa del Monte Landslide Complex and Upper Schultheis Road Landslide

Section (U-upper, L-lover surfaces)*	Dynamic Load K_{max} g	Total Displacement u in.	u cm	Ratio K_y / K_{max}	Yield Acceleration K_y g	Mobilized Friction ϕ'_{mob} degrees	Average Displacement u in.	u cm	Ratio K_y / K_{max}	Yield Acceleration K_y g	Mobilized Friction ϕ'_{mob} degrees
Villa del Monte											
DM-U	0.5	28	72	0.11	0.055	17.6	14	36	0.21	0.105	20.7
DM-U	0.6	28	72	0.11	0.066	18.3	14	36	0.21	0.126	22
SB-U	0.5	44	111	0.06	0.03	17.8	22	55.5	0.15	0.075	21
SB-U	0.6	44	111	0.06	0.036	18.3	22	55.5	0.15	0.09	22
SB-L	0.5	44	111	0.06	0.03	19.3	22	55.5	0.15	0.075	22.5
SB-L	0.6	44	111	0.06	0.036	19.8	22	55.5	0.15	0.09	23.5
Schultheis Road											
SR-U	0.5	96	244	0.03	0.015	17.6	48	122	0.05	0.025	18.3
SR-U	0.6	96	244	0.03	0.015	17.8	48	122	0.05	0.03	18.6
SR-L	0.5	96	244	0.03	0.015	18.6	48	122	0.05	0.025	19.4
SR-L	0.6	96	244	0.03	0.018	18.9	48	122	0.05	0.03	19.8

* Surfaces identified in Figures 6, 8 and 9

Table 4

Summary of Mobilized Dynamic Friction, Static Stability, and Critical Ground-Water Depths

Section*	Pre-Quake Ground-Water Conditions: Estimated Ground-Water Depth at Time of Quake ft	Dynamic Load K_{max} g	Displacement u in.	u cm	Mobilized Friction ϕ'_{mob} degree	Friction Required for Static Stability $\phi'_{req.}$ degrees	Factor of Safety Against Sliding** FS_s	Ground-Water Depth That Would Trigger Static Instability** ft	Critical Change in Ground-Water Depth** ft
DM-U	100	0.5	28	72	17.6	14	1.27	45	55
DM-U	100	0.6	28	72	18.3	14	1.33	30	70
DM-U	100	0.5	14	36	20.7	14	1.52	10	90
DM-U	100	0.6	14	36	22	14	1.62	>0	>100
SB-U	100	0.5	44	111	17.8	15.5	1.15	70	30
SB-U	100	0.6	44	111	18.3	15.5	1.19	60	40
SB-U	100	0.5	22	55.5	21	15.5	1.38	15	85
SB-U	100	0.6	22	55.5	22	15.5	1.46	5	95
SB-L	100	0.5	44	111	19.3	16.9	1.15	55	45
SB-L	100	0.6	44	111	19.8	16.9	1.19	45	55
SB-L	100	0.5	22	55.5	22.5	16.9	1.36	>0	>100
SB-L	100	0.6	22	55.5	23.5	16.9	1.43	>0	>100
SR-U	50	0.5	96	244	17.6	16.5	1.07	35	15
SR-U	50	0.6	96	244	17.8	16.5	1.08	35	15
SR-U	50	0.5	48	122	18.3	16.5	1.12	30	20
SR-U	50	0.6	48	122	18.6	16.5	1.14	30	20
SR-L	50	0.5	96	244	18.6	17.2	1.09	35	15
SR-L	50	0.6	96	244	18.9	17.2	1.11	30	20
SR-L	50	0.5	48	122	19.4	17.2	1.14	25	25
SR-L	50	0.6	48	122	19.8	17.2	1.16	20	30

* Surfaces identified in Figures 6, 8 and 9

** Based on dynamic strengths

Table 5

Calculated Friction Angles for Shallow Failure Surfaces Shown in Figures 10, 11, 12

Failure Surface Designation	Friction Angle Back-Calculated From Loma Prieta Earthquake (degrees)	Friction Angle Required for Static F.S = 1 With Water Table Near Surface (degrees)
SR-2	19	33
DM-7	33	>40
DM-8	20	22
SB-3	22	26

Table 6

Calculated Friction Angles for Deep Failure Surfaces Shown in Figures 10, 11, 12

Failure Surface Designation	Friction Angle Back-Calculated From Loma Prieta Earthquake (degrees)	Friction Angle Required for Static F.S. = 1 With Water Table Near Surface (degrees)
SR-1	25	32
SR-U	18	22
SR-L	16	22
DM-1	20	29
DM-3	21	27
DM-U	20	23
SB-1	22	29
SB-2	19	22
SB-U	18	22

Table 7

Estimated Static Friction Angles for Deep Surfaces Shown in Figures 10, 11, 12

Failure surface designation	Friction angle required for static F.S. = 1.5 with a deep water table (degrees)	Factor of safety water table at ground Surface
SR-1	35	1.09
SR-U	26	1.18
SR-L	24	1.10
DM-1	24	0.81
DM-3	26	0.94
DM-U	23	1.01
SB-1	28	0.98
SB-2	25	1.16
SB-U	23	1.03

Table 8

Computed Slope Displacements with No Strength Degradation
(Surfaces Identified in Figures 10, 11, 12)

Failure Surface Designation	Yield Acceleration Based on Friction Angles Listed in Table 7	K_{max} (g's)	K_y/K_{max}	Calculated Displacement in.	cm
SR-1	0.150	0.5	0.300	24	9
		0.6	0.250	32	13
SR-U	0.100	0.5	0.201	44	17
		0.6	0.167	51	20
SR-L	0.100	0.5	0.199	44	17
		0.6	0.166	51	20
DM-1	0.136	0.5	0.272	38	11
		0.6	0.227	38	15
DM-3	0.147	0.5	0.294	24	9
		0.6	0.245	32	13
DM-U	0.104	0.5	0.209	44	17
		0.6	0.174	51	20
SB-1	0.145	0.5	0.290	24	9
		0.6	0.242	32	13
SB-2	0.138	0.5	0.276	28	11
		0.6	0.230	25	10
SB-U	0.108	0.5	0.217	38	15
		0.6	0.181	48	19

Table 9

Estimated Displacements for Postulated Scenarios
(Surfaces Identified in Figures 10, 11, 12)

	Scenario 1:				Scenario 2:			
Failure Surface Designation	Water at 100 ft in.	cm	Water at 50 ft in.	cm	Water at 50 ft in	cm	Water at 25 ft in.	cm
SR-1			76	194			65	165
SR-L			120	306			215	545
SR-U			120	306			194	494
DM-1	80	204			165	419		
DM-3	76	194			144	365		
DM-U	109	278			241	612		
SB-1	76	194			165	419		
SB-2	80	204			157	398		
SB-U	109	278			215	545		

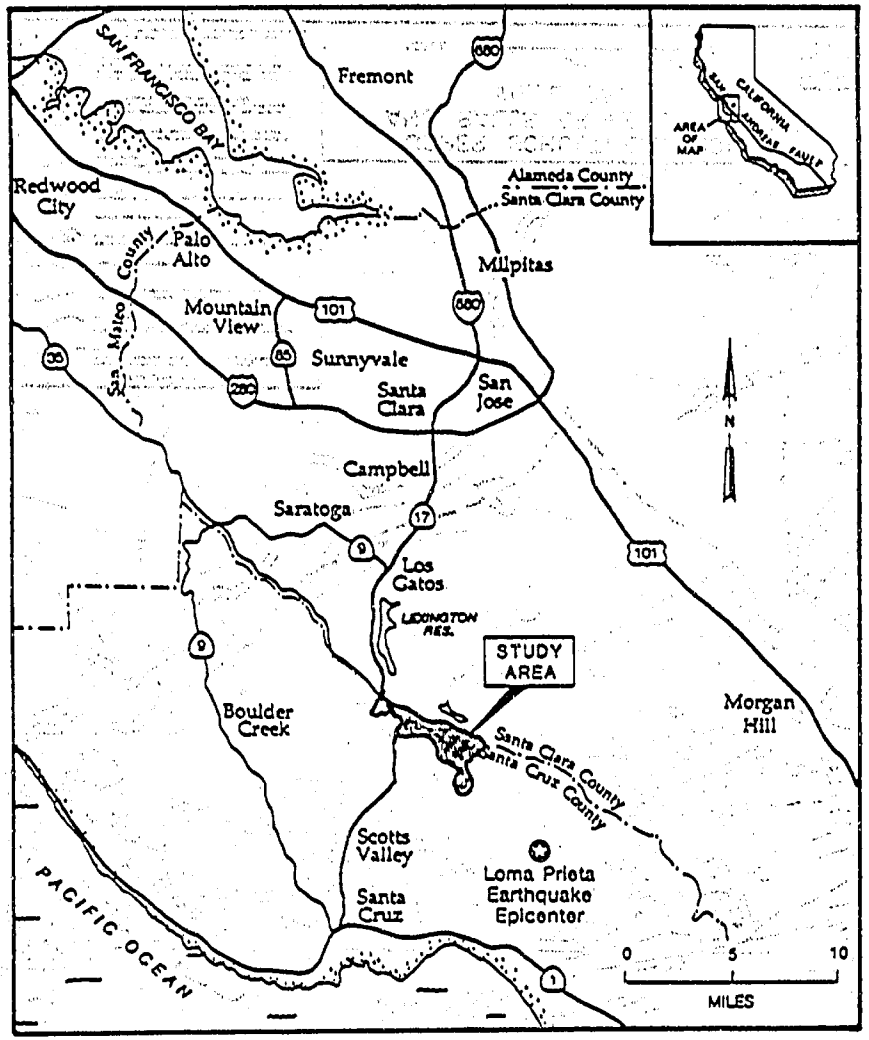


Figure 1. Location of study area
(after Keefer et al. 1991)

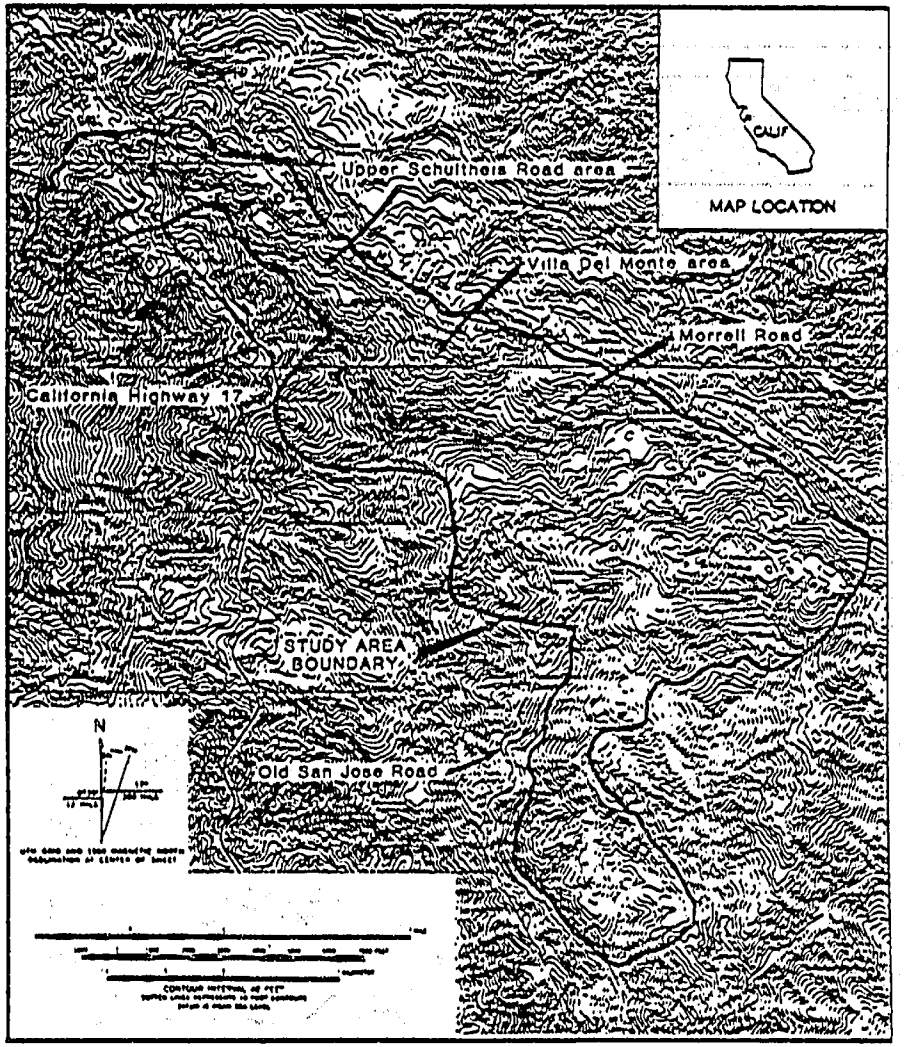


Figure 2. Contour map of study area
(after Keefer et al. 1991)

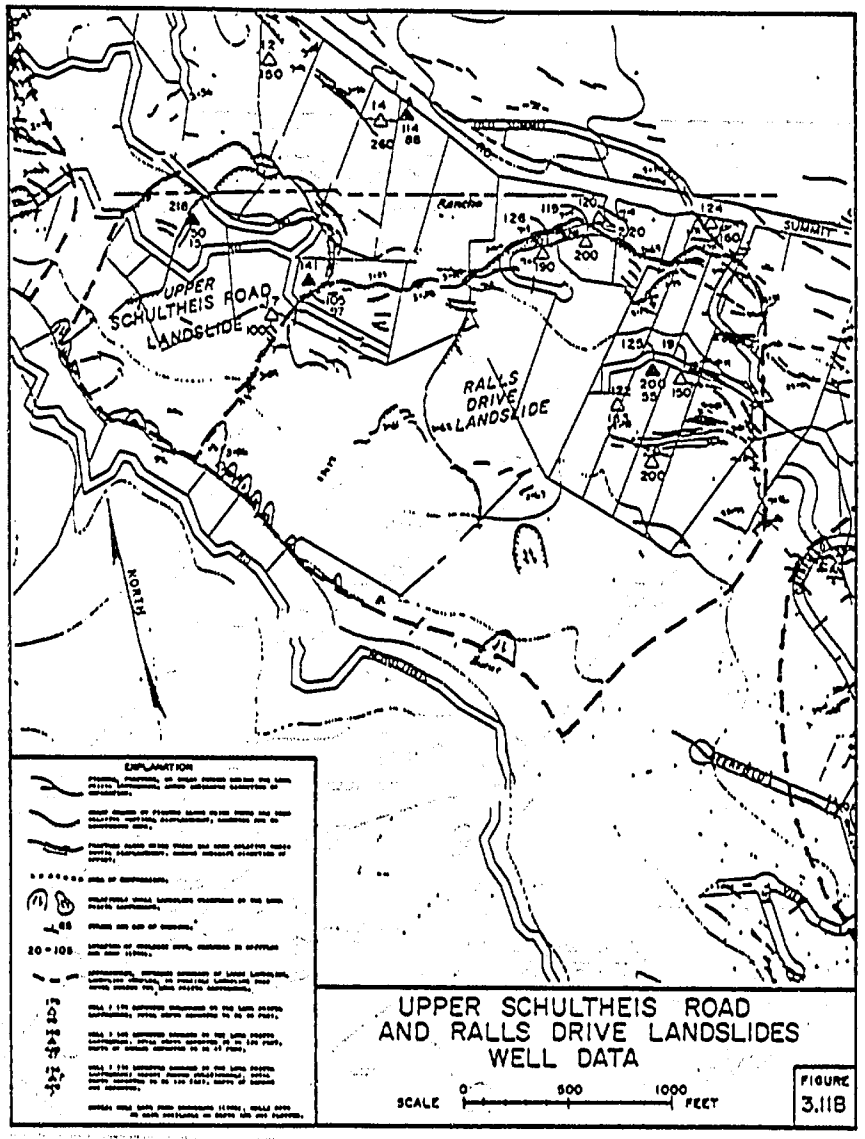


Figure 3. Mapped surface cracks in Upper Schultheis Road area (after Keefer et al. 1991)

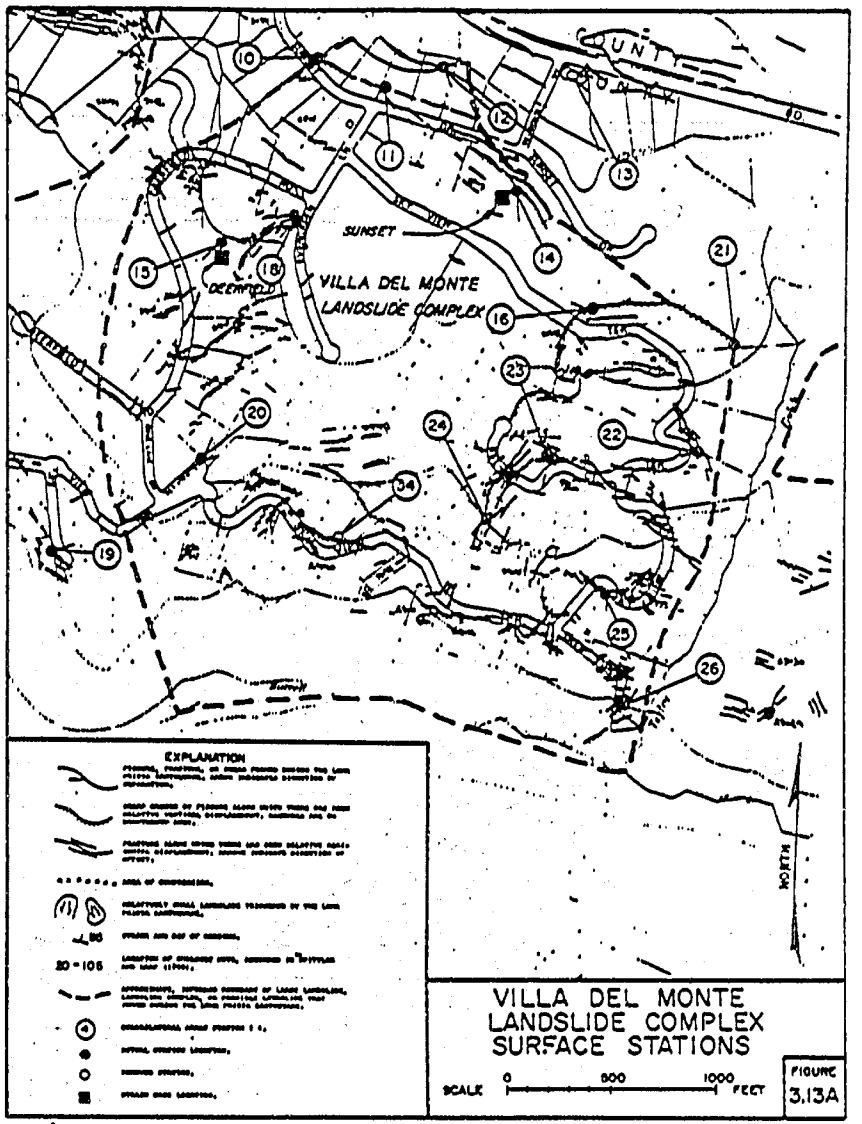


Figure 4. Mapped surface cracks in Villa Del Monte area (after Keefer et al. 1991)

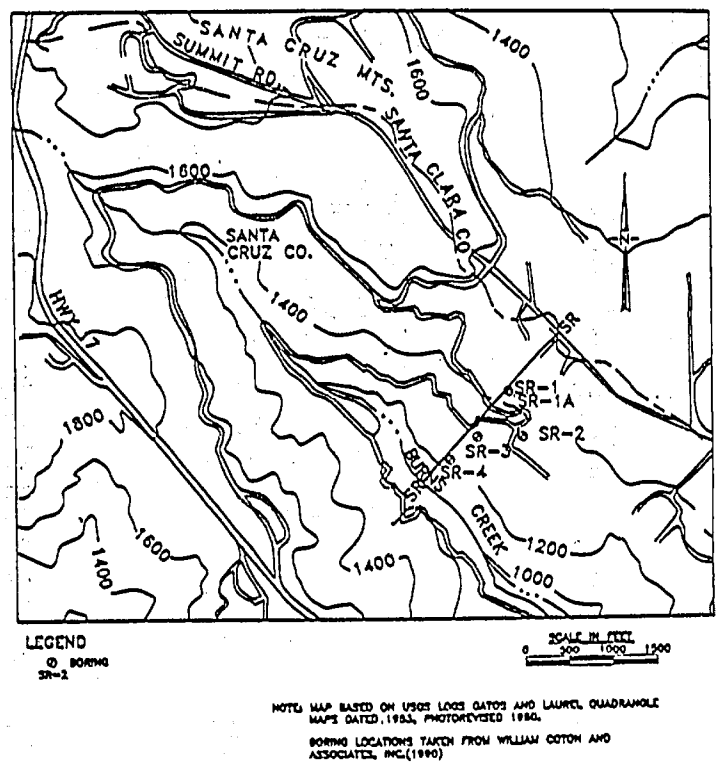


Figure 5. Locations of borings and section SR in Upper Schultheis Road area

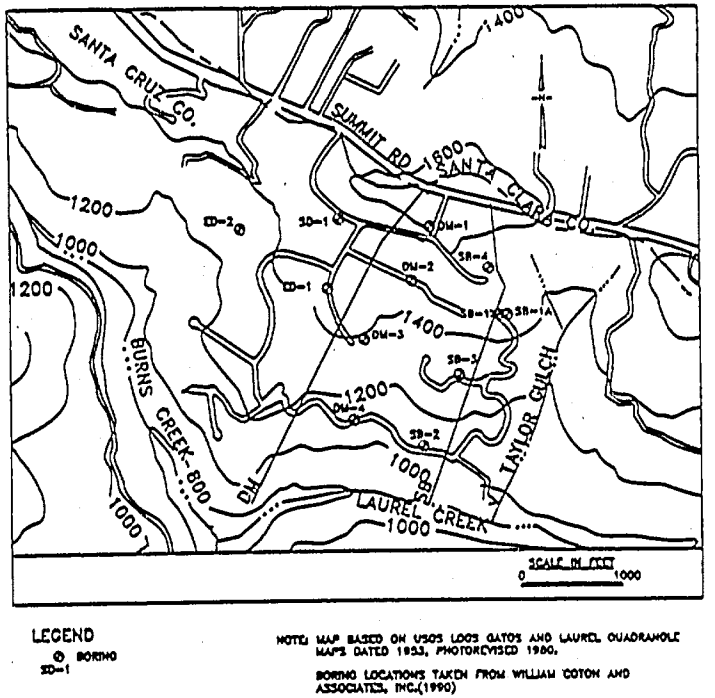


Figure 7. Locations of borings and sections DM and SB in Villa Del Monte area

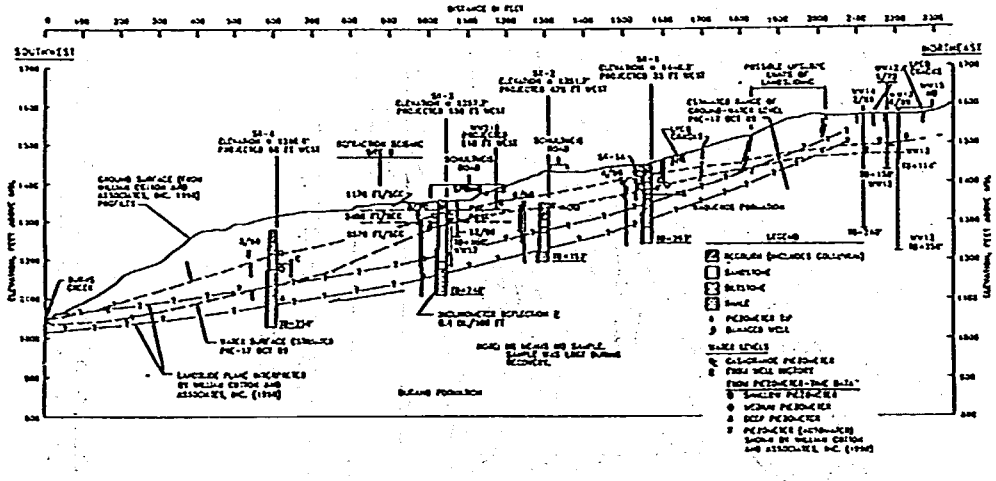


FIGURE 6 Estimated Upper and Lower Locations of Basal Shear Surfaces for Section SR, Upper Schultheis Road Landslide, from William Cotton and Associates, Inc. (1990)

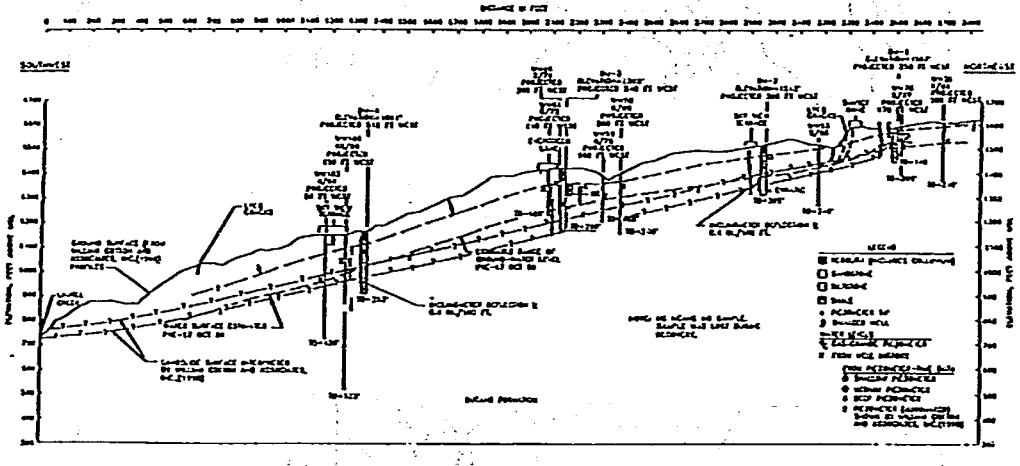


FIGURE 8 Estimated Upper and Lower Locations of Basal Shear Surfaces for Section DH, Villa Del Monte Landslide Complex, from William Cotton and Associates, Inc. (1990)

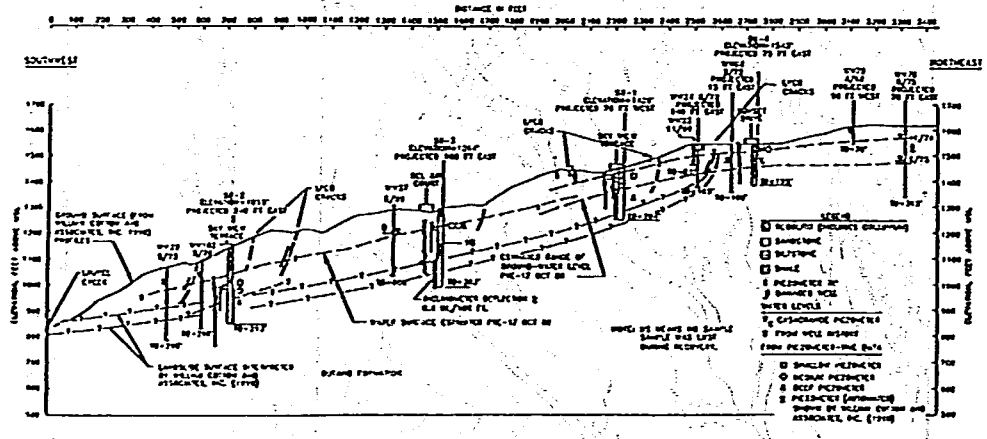


FIGURE 9 Estimated Upper and Lower Locations of Basal Shear Surfaces for Section SB, Villa Del Monte Landslide Complex, from William Cotton and Associates, Inc. (1990)

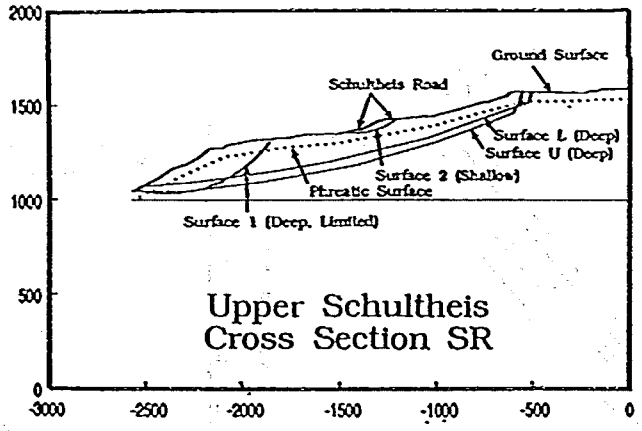


FIGURE 10 Assumed Locations of Shallow and Deep Sliding Surfaces for Section SR, Upper Schultheis Road Landslide

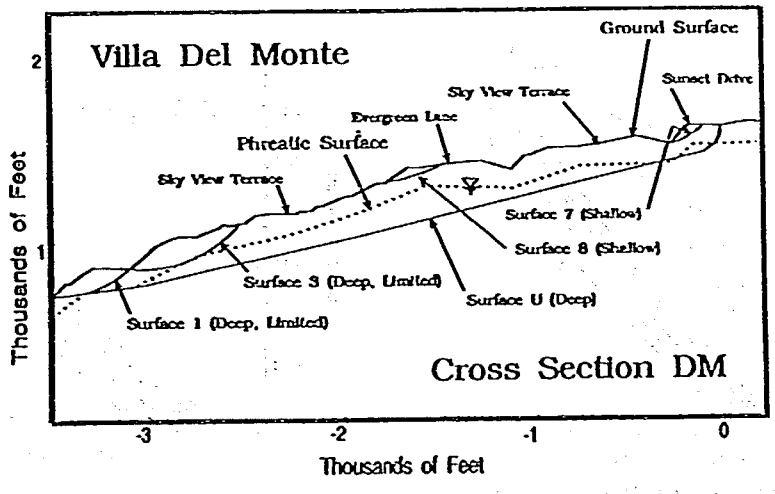


FIGURE 11 Assumed Locations of Shallow and Deep Sliding Surfaces for Section DM, Villa Del Monte Landslide Complex

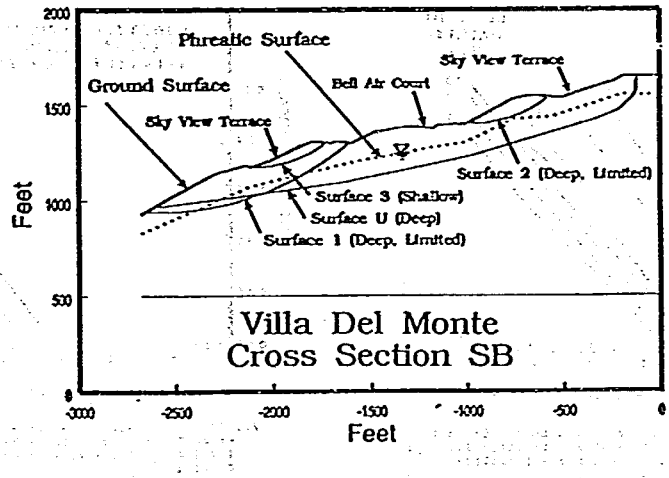


FIGURE 12 Assumed Locations of Shallow and Deep Sliding Surfaces for Section SB, Villa Del Monte Landslide Complex

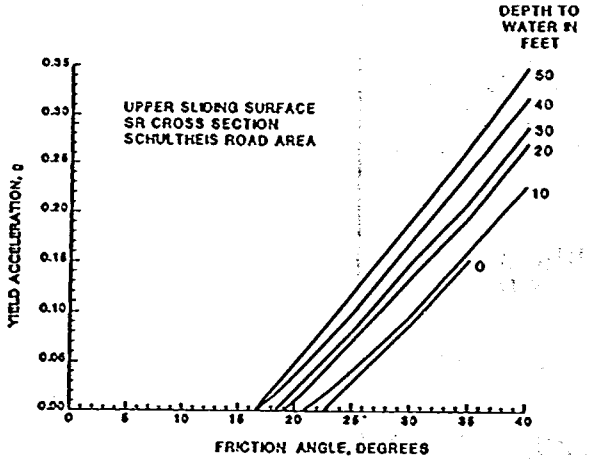


FIGURE 13 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Upper Basal Shear Surface, Section SR, Upper Schultheis Road Landslide

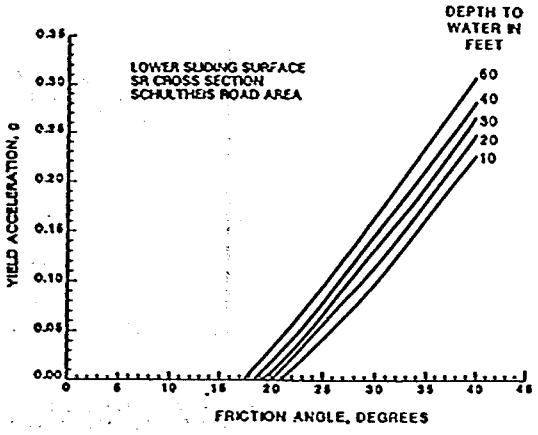


FIGURE 14 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Lower Basal Shear Surface, Section SR, Upper Schulthels Road Landslide

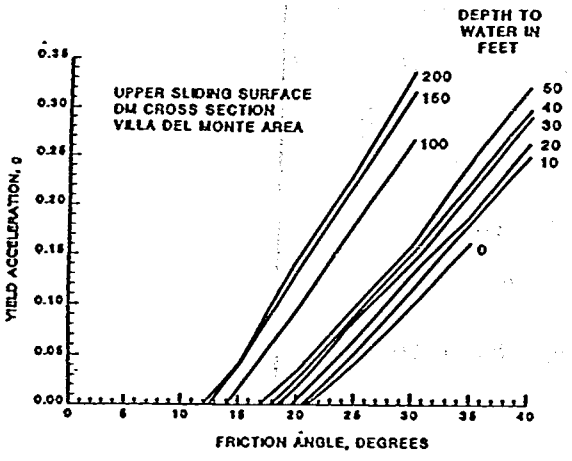


FIGURE 15 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Upper Basal Shear Surface, Section DM, Villa Del Monte Landslide Complex

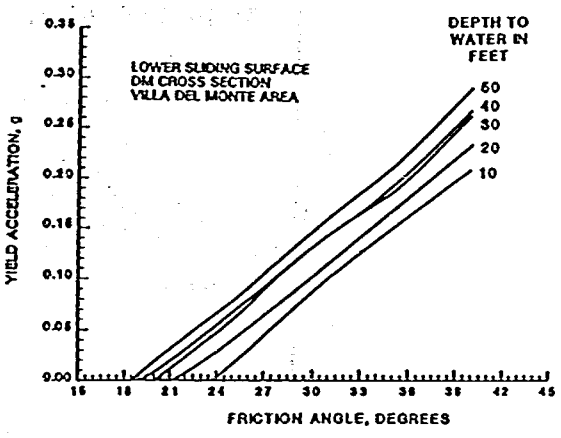


FIGURE 16 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Lower Basal Shear Surface, Section DM, Villa Del Monte Landslide Complex

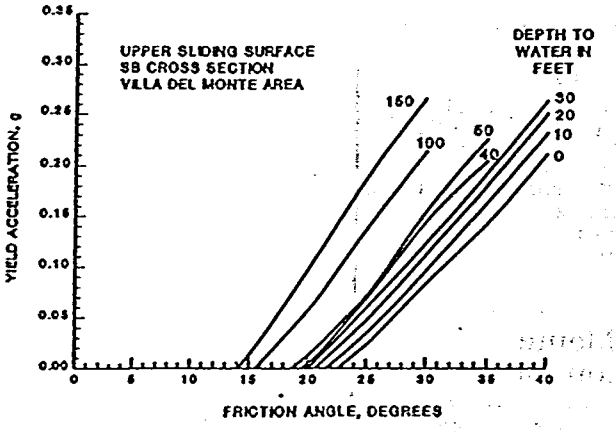


FIGURE 17 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Upper Basal Shear Surface, Section SB, Villa Del Monte Landslide Complex

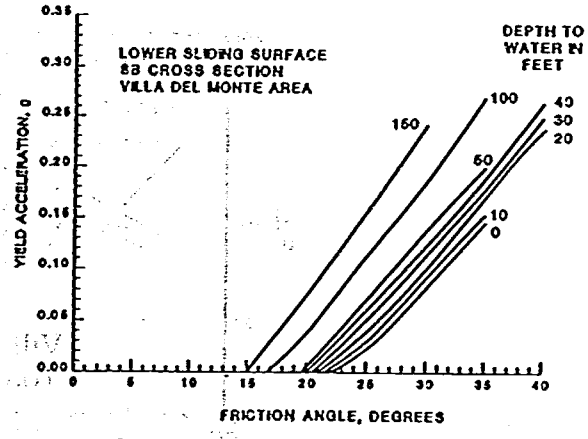


FIGURE 18 Relation Between Mobilized Friction Angle, Depth to Water Table, and Yield Value of Horizontal Acceleration for Lower Basal Shear Surface, Section SB, Villa Del Monte Landslide Complex

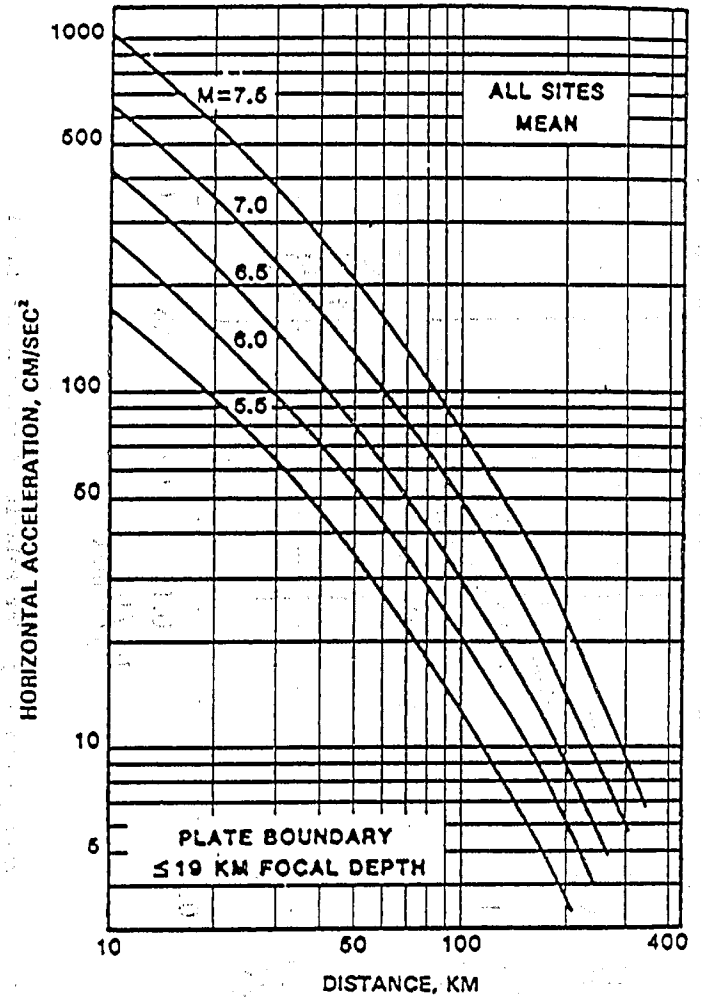


FIGURE 19 Attenuation of Peak Ground Acceleration with Distance. Average Curves for All Site Types for Plate Boundary Seismic Events with Focal Depths Less than 19 km, from Krinitzsky, Chang, and Nuttli (1988)

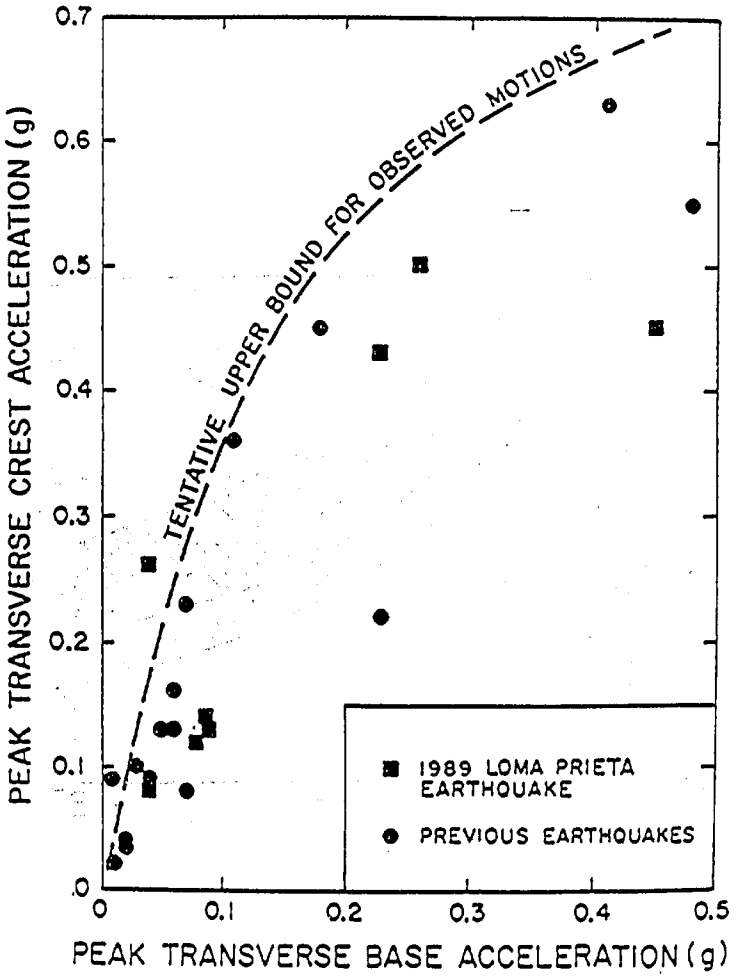


FIGURE 20 Comparison of Peak Base and Crest Transverse Accelerations Measured at Earth Dams (from Harder 1991)

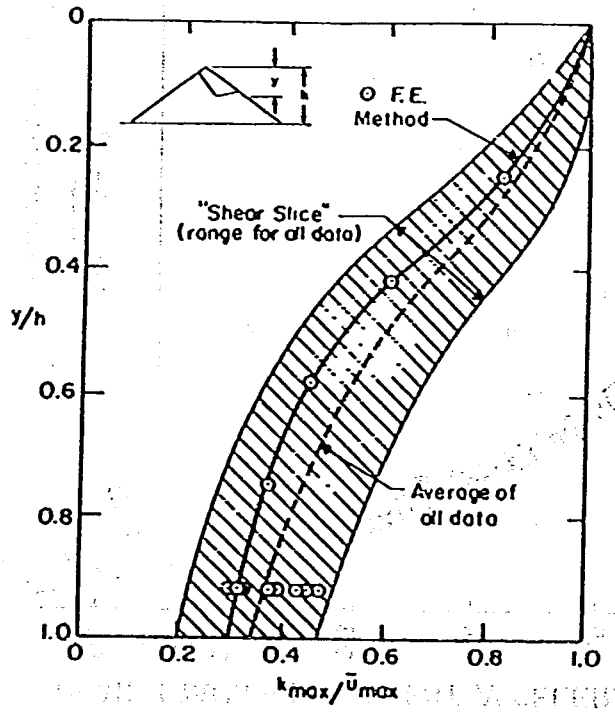


FIGURE 21 Relation between Acceleration Ratio k_{max} / $\bar{u}_{max}$ (averaged dynamic load to peak crest acceleration) and Relative Height of Sliding Surface (Y/h)

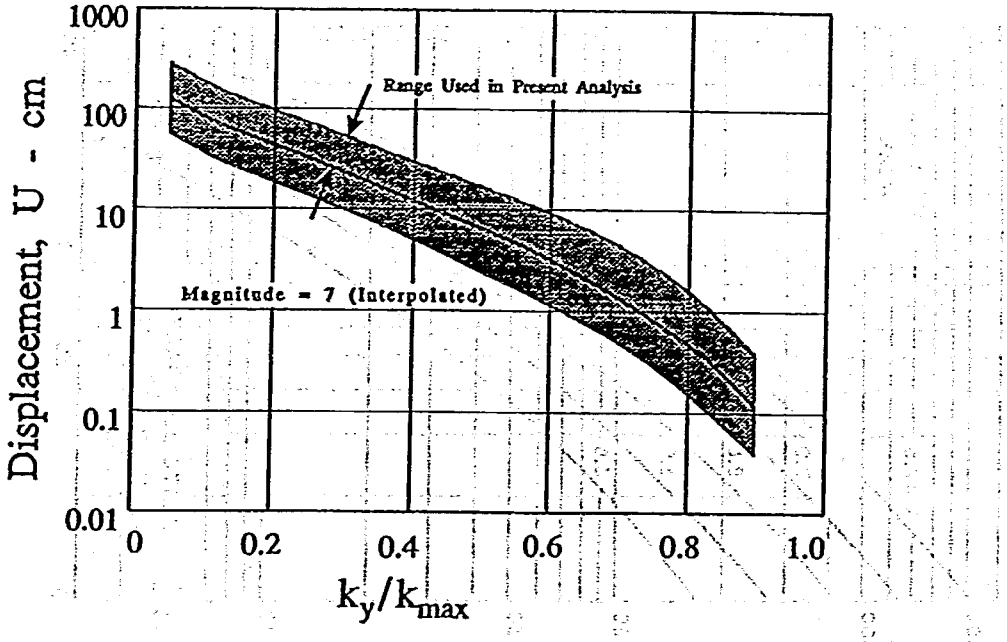


FIGURE 22 Average and Range for Relation Between Accumulated Newmark-Sliding-Block Displacement, U, and Acceleration Ratio, k_y/k_{max}, for Interpolated Magnitude 7 Earthquakes after Makdisi and Seed (1979)

液状化による地盤流動の予測及び杭に及ぼす影響に関する研究

常田　賢一[1]、松本　秀臣[2]、東畑　郁生[3]、佐々木　康[4]

要約／1．まえがき
液状化による地盤流動は、既往の地震で数多く発生している。液状化による地盤流動を考慮した構造物の耐震設計法の確立のためには、地盤流動に関係する基本要因、永久変位の予測方法および下部工に作用する外力を研究することが必要である。
本報告では、液状化による地盤の永久水平変位を予測するために、エネルギー最小原理に基づく解析手法が提案されている。また、液状化により流動する地盤中の杭に作用する外力を明らかにするために、実験的な研究が実施されている。

キーワード：液状化、地盤流動、永久変位、予測、最小エネルギー原理、パラメトリック解析、模型実験、外力、杭

2．大型振動台実験

2．1　実験方法

まず、液状化による地盤流動に対する斜面の長さの影響を検討するために、4種類の地盤模型の振動台実験を実施した。地盤模型の条件を図－1に示す。実験土層は、長さ8m，幅1m，高さ1mである。基本的に、地盤模型は3層（表層非液状化層：150mm、液状化層：350mmおよび非液状化層）で構成されており、地盤のモデル－9、10、11および12での模型長さは、それぞれ2m，4m，8mおよび8mである。モデル－12はモデル－11に対して表層の非液状化層が無い場合である。液状化地盤のの材料は浅間山砂（平均粒径0．25mm）であり、表層非液状化層は礫（平均粒径7mm）である。地下水位は液状化層上面に一致させるようにしており、地表面勾配は5％とした。なお、振動台の加振は周波数2Hzの正弦波で、20秒間行った。

2．2実験結果

図－2は、振動台の入力加速度と地盤模型表面の中央における水平方向の累積変位の関係を示す。地盤流動は地盤表面の長さの増加に伴って、小さい入力加速度値で上昇し始めることが分かる。

3．水平地盤流動の予測

3．1最小エネルギーの原理

図－3は、液状化による地盤の永久水平変位を予測する解析手法の提案のための地盤モデルを示す。本研究で提案されたエネルギー最小化手法の詳細な原理はここでは省略する。

3．2最小エネルギー原理の実例への適用

提案された解析手法は、振動実験における地盤モデル－10に適用されたが、地表面での最大水平変位の分布について実験結果と解析結果を比較したのが図－4である。解析による最大水平変位は16cmであるが、実験結果では20～25cmであるので、解析値は実測値よりやや大きいことが分かる。

また、提案された解析手法は、既往地震における原位置での事例研究にも適用された。図－5は、1983年の日本海中部地震で被害を受けた能代市での計測値と地表面での水平変位の計算値を比較した例の一つである。やや、過大な予測となっているが、観測値との類似性は良好といえる。
提案した解析手法は、それぞれの地盤条件に応じて原位置に適用できるが、工学的な視点からはもう少し簡易かつ実用的な方法が必要である。

3．3最小エネルギー原理を用いたパラメトリック解析による簡易法

本研究では、最大水平変位を予測するための簡易手法を提案するために、提案した解析手法を用いたパラメトリック解析を実施した。図－6に示すように、既往の実験的な研究の結果に基づいて、主要な4つの指標（地盤流動の範囲長、液状化層の厚さ、表層の非液状化層の厚さおよび勾配）を考慮した。
本研究では、最大水平変位を表わすために式（7）を仮定した。

$$D = a \times L^{b} \times H^{c} \times T^{d} \times \theta^{b} \quad \cdots\cdots\cdots\cdots (7)$$

結果は式（8）式の通りである。既往地震において液状化による水平方向地盤流動が発生した地点に、式（8）を適用した。
図－7および図－8は、原位置での計測された最大水平変位と地盤分類別に応じた計算値との関係を示す。式（8）により計算された変位は、実測での大きな変位領域を除いて、実測値と相対的に良い対応が見られる。

4．杭に及ぼす地盤流動の影響

4．1 流動地盤中の杭に関する振動台実験

杭模型に対する地盤流動の影響を調べるために、2つの模型地盤（模型－11、模型－12）に対して振動台実験が行われた。図－9に示すように、塩化ビニール製のパイプが土槽の底に固定された。
図－10は、杭模型のB2からB5測点での曲げモーメントの時刻歴を示す。杭模型の曲げモーメントは、部分的に液状化した地盤が流動を始める経過時間約5秒において増加を始める。B2からB4測点はオーバースケールしているが、地盤が完全に液状化した後は、曲げモーメントが減少し始めている。

4．2 液状化地盤中を牽引する杭に関する実験

4．2．1 実験方法

液状化地盤での地盤流動により荷重を受ける杭に関する実験が、図－12に示す簡易実験装置を用いて行われた。液状化土層は、土層の側壁へのハンマーによる打撃

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により加振され、液状化している。液状化の発生の直後に、ウィンチにより移動板に固定された杭模型を液状化層内で強制的に移動させ、牽引荷重を計測した。

3種類の杭の移動速度（4．5mm／秒、2．6mm／秒、1．5mm／秒）が設定され、杭の数およびその配置を図－14のように変化させた。なお、杭の移動速度4．5mm／秒、2．6mm／秒および1．5mm／秒毎に、牽引台車と支持枠との摩擦力が計測されているが、それぞれ0．9kg／cm^2、0．8kg／cm^{2}および0．8kg／cm^{2}であった。

4．2．2 実験結果

図－16は、全荷重、過剰間隙水圧および杭模型の変位の関係を示す。杭模型は段階Ⅰで増加するが、全荷重は小さいまま、ほとんど一定である。一方、段階Ⅱでは、変位の増加に伴って、全荷重は増加する。

図－18は、ほとんど完全に液状化した場合および液状化していない場合で、全荷重と格子配列の9本の杭の移動速度との関係を示す。液状化が部分的に発生している場合、杭の移動速度の増加に伴って、全荷重は増加している。

図－19は、全荷重、杭模型先端での過剰間隙水圧、杭模型の移動速度および杭の配列の関係を示す。過剰間隙水圧の増加に伴って、全荷重は減少することが分かる。

4．まとめ

本報告の結果をまとめると以下の通りである。

（1）液状化した傾斜地盤は、斜面の長さおよび表層の非液状化層の厚さの減少に伴い、水平方向に流動しやすくなる。

（2）地盤流動による最大水平地盤変位を予測するために、エネルギー最小原理に基づく解析手法が提案された。

（3）パラメトリック解析により、地盤流動による最大水平地盤変位の簡易予測手法が提案された。

（4）液状化により流動する地盤中の杭に作用する最大荷重は、完全液状化状態では発生しないで、部分的な液状化状態で発生する。

（5）液状化に伴う地盤流動により杭に作用する全荷重は、過剰間隙水圧比の増加により減少し、地盤と杭の相対速度の増加により増加する。

（6）杭に作用する全荷重、言い換えると、地盤流動に対する杭の抵抗力は、地盤流動に直交する方向の杭の数の増加に伴って増加する。

地盤の液状化による地盤流動の影響を考慮した耐震設計法の確立のために、今後一層の研究を実施することが必要である。

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Table 1. Condition of ground models

Ground Model No.	Gradient of Ground Surface (%)	Length of Ground Model /Slope (m)	Thickness of Liquefiable Layer (cm).	Thickness of Uppermost Non-Liqufied Layer(cm)
9	5	2.0	35	15
10	5	4.0	35	15
11	5	8.0	35	15
12	5	8.0	35	0

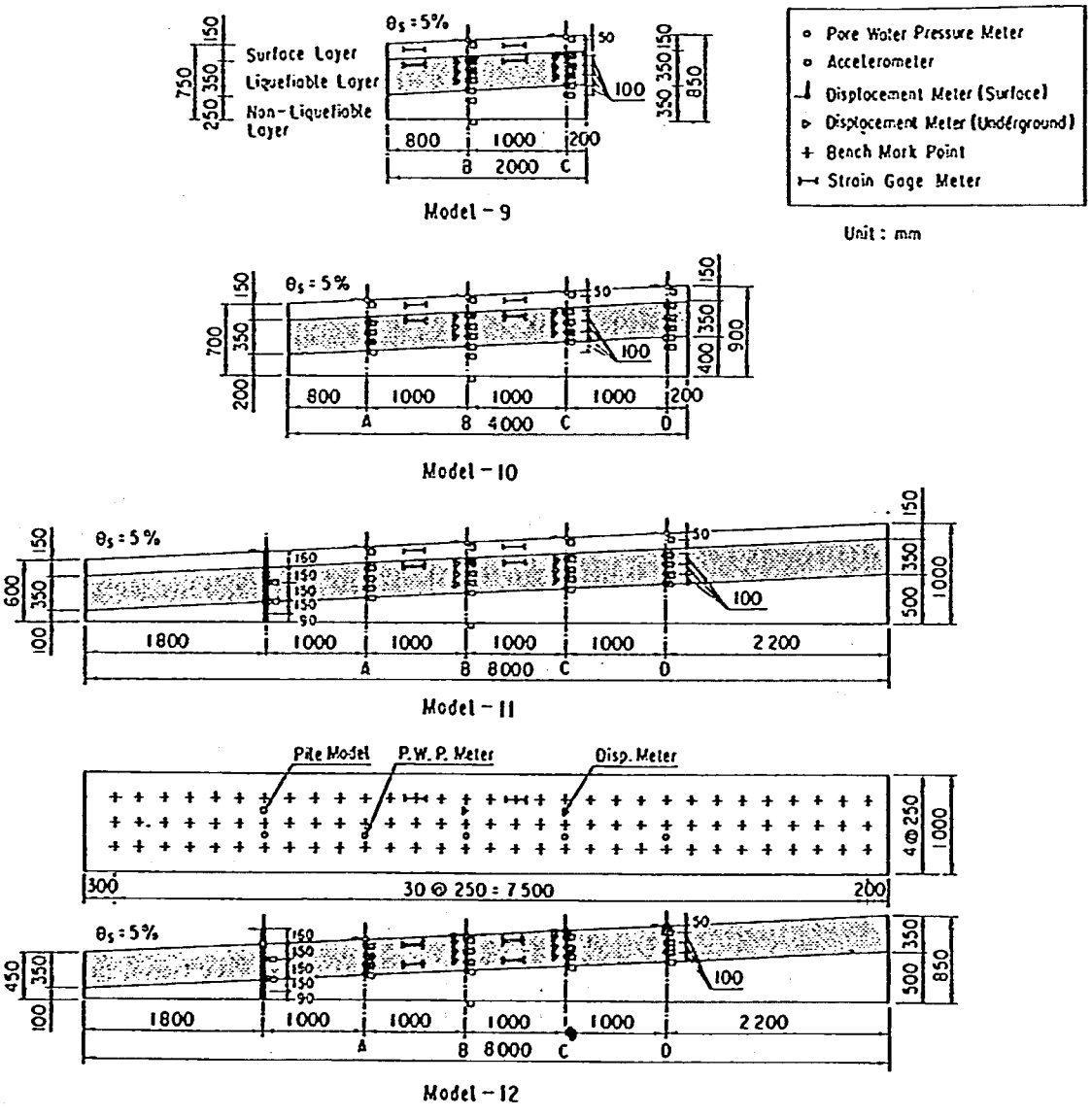


Figure 1. Ground models and measurements for large scale shaking table tests

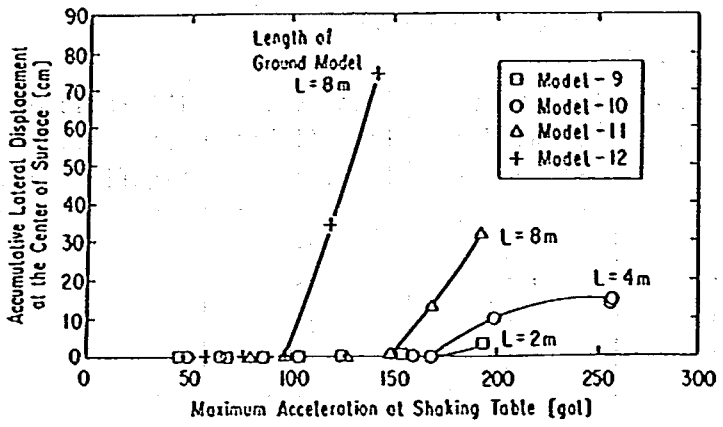


Figure 2. Relation between lateral ground displacement at center of ground surface, input acceleration and length of ground surface

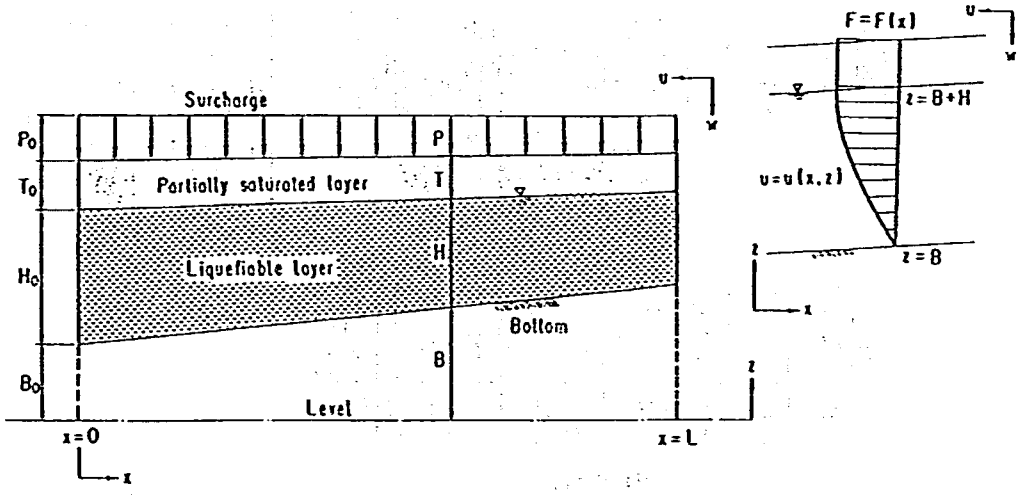


Figure 3. Ground model applied for minimum potential energy method

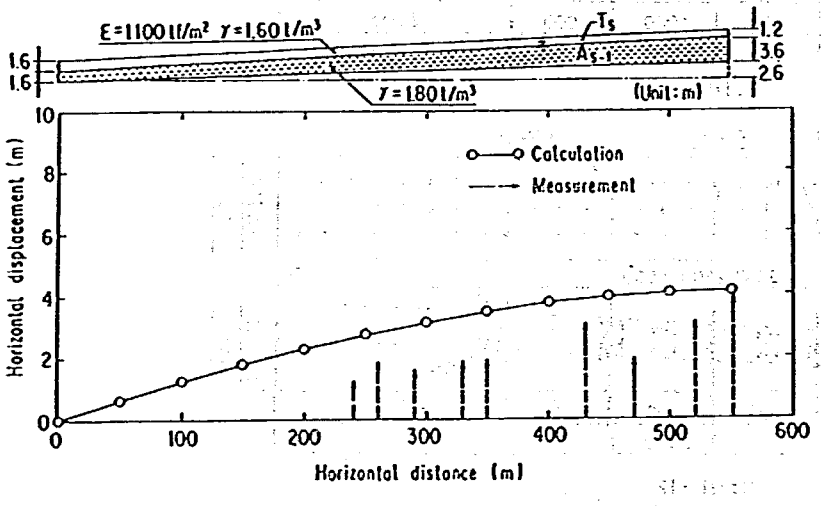


Figure 5. Case history study at Maeyama Area in Noshiro City in the 1983 Nihon-Kai Cyubu Earthquake, Japan

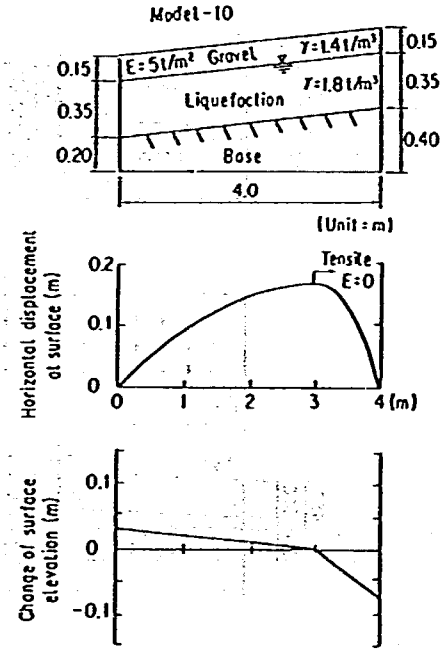


Figure 4. Case study applied for shaking table test: Model-10

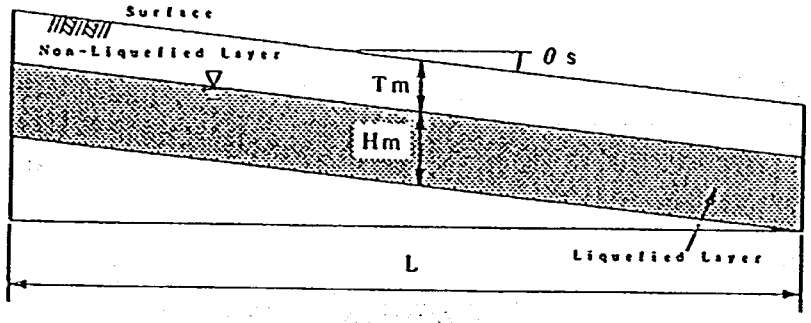


Figure 6. Ground model for parametric study with use of minimum potential energy method

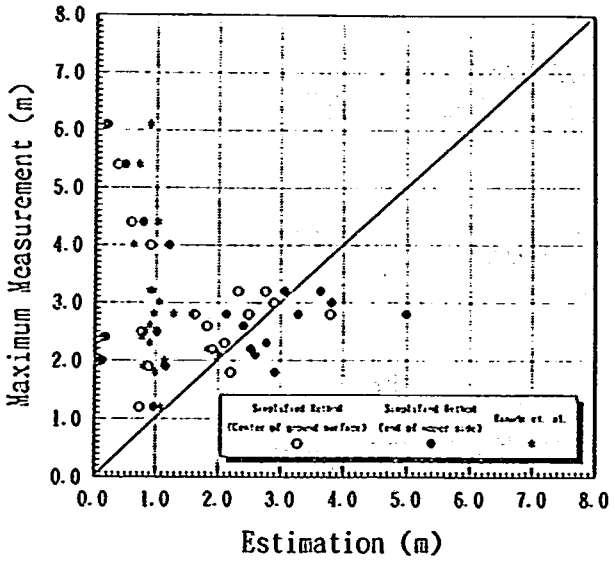


Figure 7. Relation between measured displacement in situ and calculated one (Types Ⅰ, Ⅱ, Ⅲ)

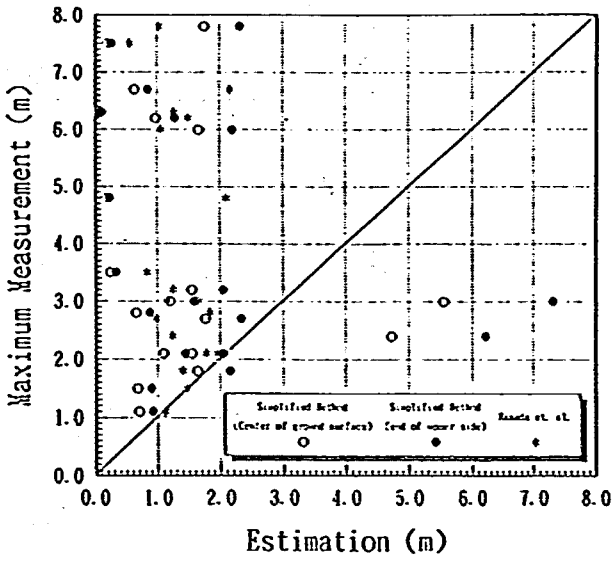


Figure 8. Relation between measured displacement in situ and calculated one (Type Ⅳ)

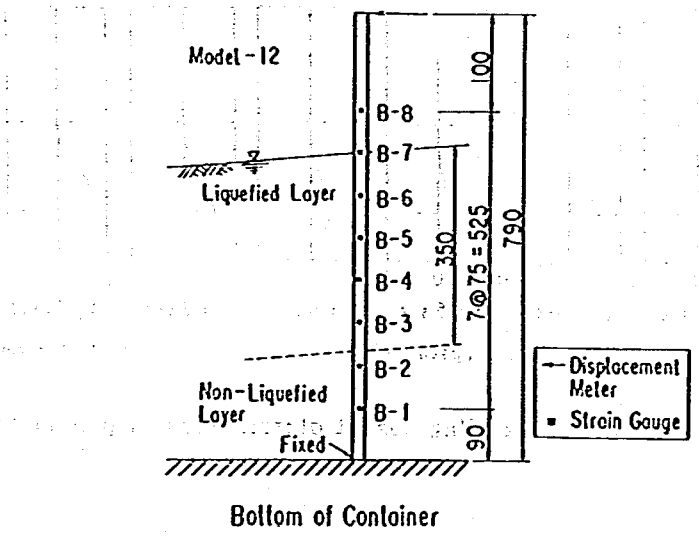


Figure 9. Condition of pile model and measurements

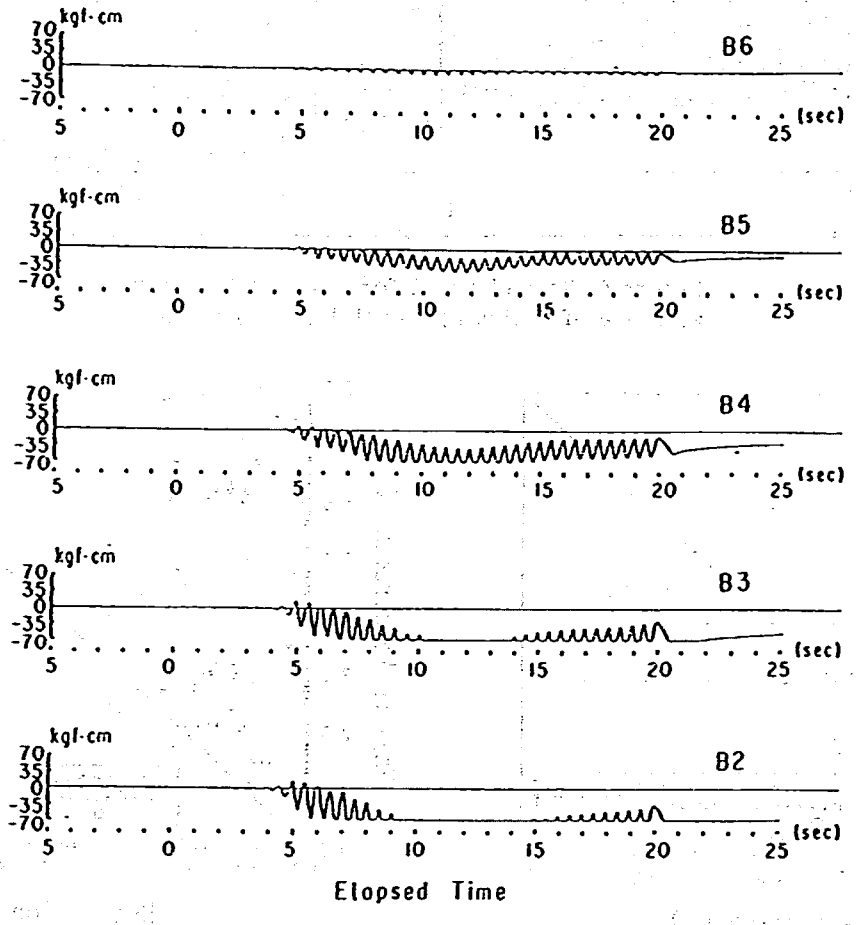


Figure 10. Time histories of bending moment of pile model

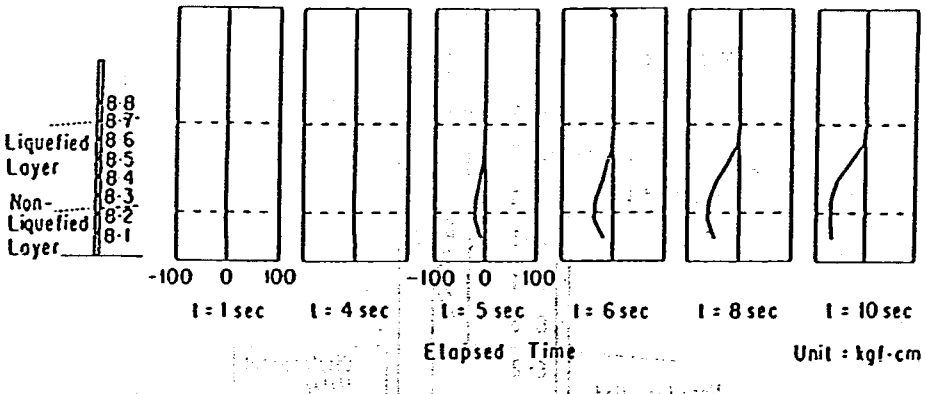


Figure 11. Time history of bending moment distribution of pile model

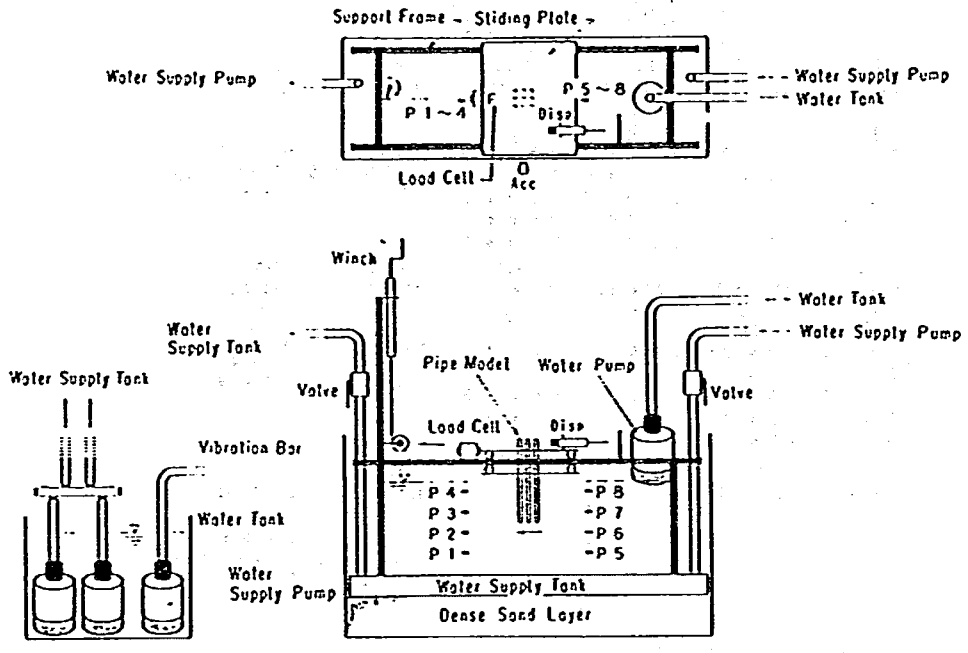


Figure 12. Test apparatus to drive piles in liquefied soils

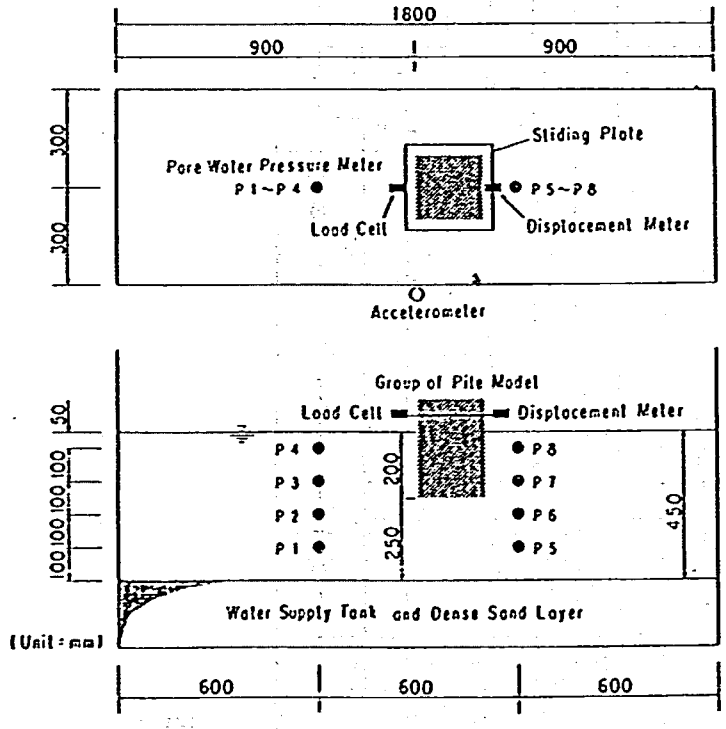


Figure 13. Ground model and measurements

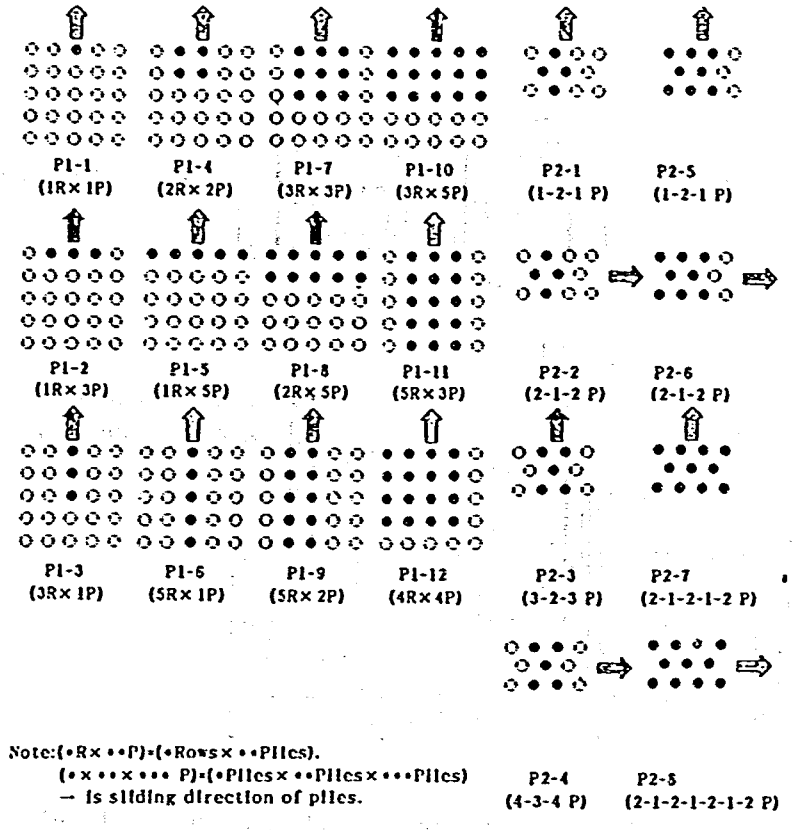


Figure 14. Condition of pile arrangement and sliding direction of piles

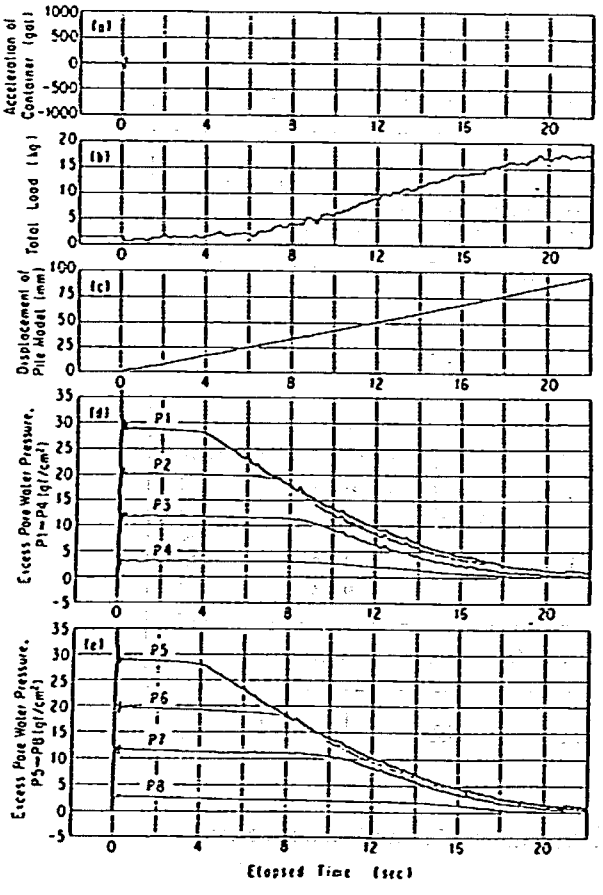
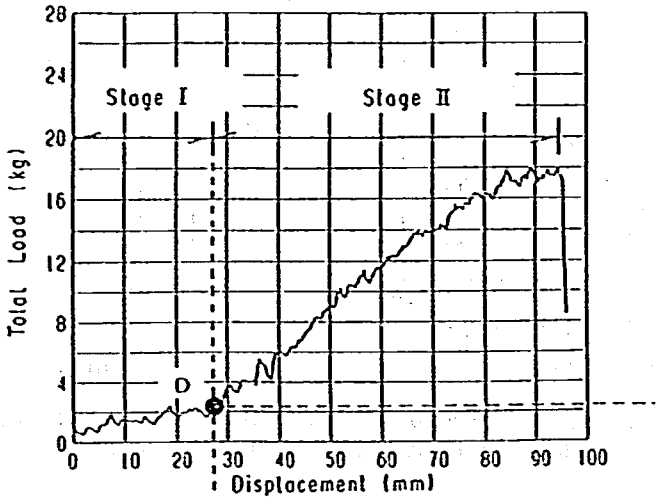
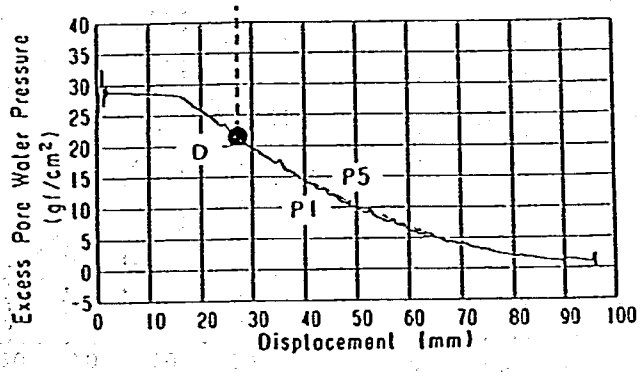


Figure 15.
Example of time histories of dynamic behavior of ground model and pile model



(a) Load~Displacement



(b) Excess Pore Water Pressure~Displacement

Figure 16. Example of relation between total load, excess pore water pressure and displacement of pile model

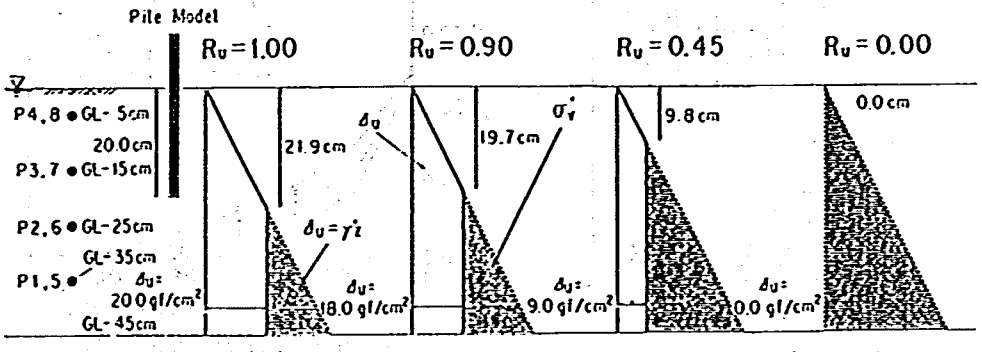
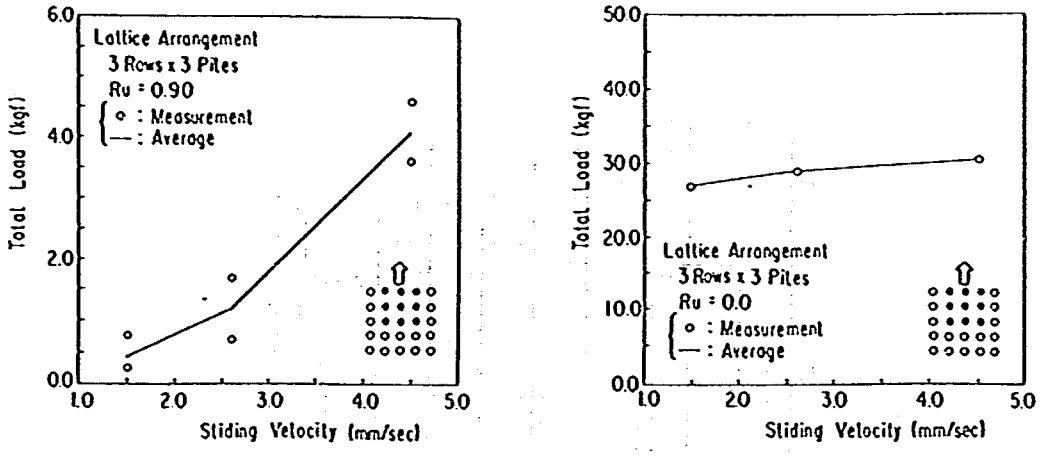
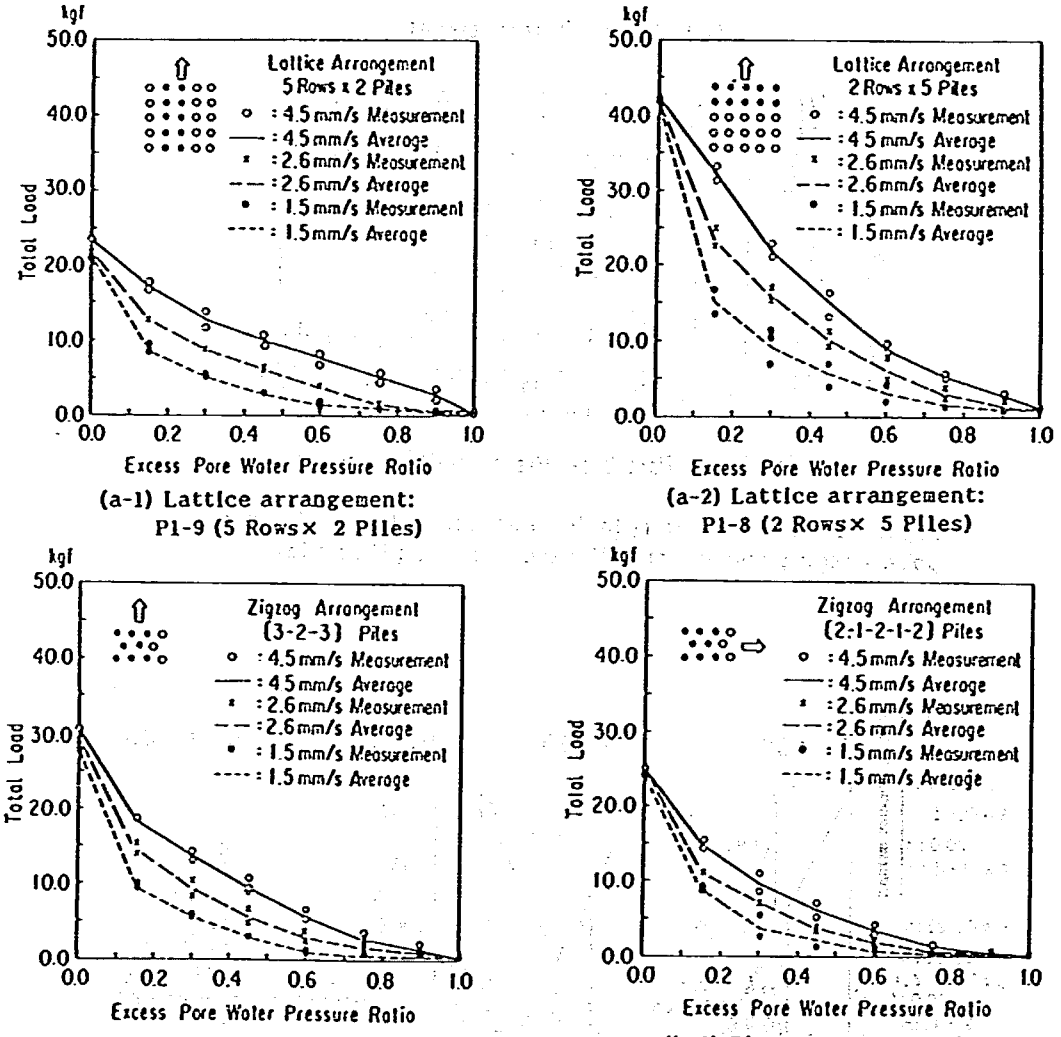


Figure 17. Identification of excess pore water pressure ratio at tip of pile model



(a) Ru = 0.90 (b) Ru = 0.00

Figure 18. Relation between total load acting on piles and sliding velocity of piles in the case of P1-7



(a-1) Lattice arrangement: P1-9 (5 Rows× 2 Piles)

(a-2) Lattice arrangement: P1-8 (2 Rows× 5 Piles)

(b-1) Zigzag arrangement: P2-6 (3-2-3 Piles)

(b-2) Zigzag arrangement: P2-5 (2-1-2-1-2 Piles)

Figure 19. Relation between pore water pressure at tip of pile and total load acting on pile

水平載荷試験による杭基礎の保有水平耐力

藤原稔[*1]、岡原美知夫[*2]、中谷昌一[*3]、木村嘉富[*4]、高木繁[*5]

要　約

現在、構造物の設計基準は従来の許容応力度設計法から限界状態設計法に移行しつつある。限界状態設計法とは、構造物が所要の機能を確保し得ることを検証するための状態として限界状態を定義し、設計耐用期間に構造物に作用する荷重などの外的作用に対して所要の安全性を有して限界状態に至らないことを照査する設計法である。ここで、基礎の安定計算における限界状態としては、基礎の挙動が可逆的な限界である弾性限界、および、変形が進行し転倒などに至る終局限界などが考えられる。杭基礎に限界状態設計法を導入するためにはこれら設計において照査すべき限界状態を明確にする必要があり、また、その状態を表す解析モデルの確立が不可欠である。

このため、多数の杭の水平載荷試験結果を解析し、水平方向の弾性限界を整理するとともに、この弾性限界内での杭基礎の安定計算に用いる水平方向地盤反力係数の推定式を提案した。さらに、弾性限界を超えた大変形時における杭基礎の挙動を明らかにするために、地盤反力度，群杭効果，杭体の曲げ剛性の低下などについて、水平載荷試験結果を解析することにより検討している。

キーワード：杭基礎，水平載荷試験，保有水平耐力，弾性限界，水平方向地盤反力係数，地盤反力度

1．はじめに

現在、諸外国の構造物設計規準は従来の許容応力度設計法（Working Stress Design Method）から限界状態設計法（Limit States Design Method）または荷重抵抗係数設計法（Load Resistance Factor Design Method）に移行しつつある[1)2)3)]。我国においても「コンクリート標準示方書」（土木学会，1986）の改訂を契機として、各機関において新しい設計法の採用が検討されている。限界状態設計法とは、構造物が所要の機能を確保し得ることを検証するためのための状態として限界状態を定義し、設計耐用期間に構造物に作用する荷重などの外的作用に対して所要の安全性を有して限界状態に至らないことを照査する設計法である。この設計法においては、構造物が所有すべき機能が限界状態として表されているため、設計目的が明確になる。また、構造物の安全性についても、設計における各種の不確定要因を信頼性理論を用いて評価し、荷重係数や抵抗係数など安全係数として導入できることから、より合理的な設計が可能となるといわれている。

このような限界状態設計法を杭基礎の設計に導入するためには、設計で照査すべき限界状態を定めるとともに、その状態を精度よく表すことのできる安定計算モデルを確立する必要がある。ここで、杭などの弾性体基礎の水平方向挙動の解析については、杭周面の地盤抵抗（地盤反力度ｐ）の取扱いによって、①極限地盤反力法、②弾性地盤反力法および③複合地盤反力法に分けられる。

(1) 極限地盤反力法：地盤の極限状態における地盤反力度の分布形状を仮定して、杭に作用する外力との釣合だけから水平抵抗を推定しようとするものである。与えられる地盤反力度が変位と無関係であり、また、杭の曲げ変形を考慮していないため、極限抵抗時の変位を推定できない。

(2) 弾性地盤反力法：地盤反力度ｐを深さｘと杭のたわみ量ｙの関数ｐ＝ｐ(x,y)として表す方法で、ｐ(x,y)の取扱いによって、線形，非線形およびｐ－ｙ曲線法に区分される。線形地盤反力法に用いられる地盤バネとしては、杭の載荷試験データから推定されたいわゆる逆算値を用いるのが一般的である。非線形弾性地盤反力法は、地盤抵抗の非線形性を考慮するために、地盤反力度と変位の関係を非線形的に取り扱う方法である。ｐ－ｙ曲線法は、試験などにより地盤反力度－変位量関係を推定し、数値解析的に解く方法である。

これらの方法では、地盤を弾性体と仮定しているので極限状態の地盤反力は求められず、地盤から決まる極限支持力は求められない。

(3) 複合地盤反力法：変形の大きい地表面近くの地盤は塑性領域にあるとし、その下の変形の小さい地盤

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は弾性領域にあるとして解く方法である。塑性領域の地盤抵抗としては(1)の考え方を用い、弾性領域の地盤抵抗としては(2)の考え方を用いるものである。地盤のモデル化については検討課題として残されているが、実際の基礎の挙動を最も精密に計算することができる方法である。

上記のように、杭基礎の水平方向の解析法には各種方法があるが、杭基礎の設計における安定計算にどの手法を用いるべきかは、その設計で想定している変位量の範囲と密接な関係にある。

現在、道路橋の下部構造に関する技術基準として用いている「道路橋示方書Ⅳ下部構造編」[4]（日本道路協会，1990）（以下、「道示」という。）の設計思想は、現行の設計荷重レベルに対しては基礎は支持力に対して十分安全であるとともに、"基礎には有害な残留変位を生じさせない"というものである。すなわち、杭基礎では杭を線形なばね定数に置換して解析を行い、得られる変位および杭頭反力が基礎が弾性的な挙動をする範囲内に抑えられるよう設計がなされている。

一方、限界状態設計法においては、設計で照査すべき限界状態を今後を明らかにしていく必要があるが、前述の"有害な残留変位を生じさせないようにする"に加え、"巨大地震など設計耐用期間に極めて稀に作用する荷重に対し、落橋など構造物に壊滅的な損傷が生じないように設計する"という考え方も提案されている（図－1.1 参照）[5]。このため、設計に用いる安定計算モデルとしては、基礎が弾性的な挙動をする範囲を超えた大変形時にまで適用できる、あるいは基礎の保有水平耐力を算定できるモデルが望ましく、この場合には地盤抵抗あるいは杭体の曲げ剛性の非線形性の影響も無視できなくなると考えられる。

本稿はこれらの状況を踏まえ、杭基礎の水平方向挙動および保有水平耐力を明らかにするために水平載荷試験結果を解析したものである。単杭についての多数の水平載荷試験結果を統計的に解析し、設計上弾性的な挙動をするとみなしうる限界の変位量を定量的に把握するとともに、この変位量内での設計に用いる水平方向地盤反力係数の推定式の提案を行った。さらに、この変位量を超えた領域での杭基礎の挙動を明らかにするために、単杭および群杭の水平載荷試験結果の解析を行い、地盤抵抗や杭の剛性の非線形性や群杭による影響について検討した。

2．使用した杭の載荷試験データ

本解析に用いるデータは、昭和63年度に土木研究所が各地方建設局、都道府県および道路4公団等の関係機関から収集した実杭の原位置における水平載荷試験データである。収集した全載荷試験データは415件[5]にのぼるが、載荷試験の内容およびデータの充実度からデータ選別を行った。ここで採用したデータ数は表－2.1 に示すとおりである。なお、検討の内容に応じては、さらに同表に示すデータから適宜有効なデータを採用している。

大変形時の杭基礎の挙動に関する検討については、上記データの内大変形時までの載荷試験が行われているデータを基に単杭の解析を行い、群杭の影響については土木研究所の土槽における模型杭の実験、および、首都高速道路公団により原位置で行われた杭基礎の実験結果を用いている。

3．杭の水平方向弾性限界

3.1 水平方向弾性限界の定義

杭基礎に水平荷重が作用したときの杭頭における荷重－変位曲線は強い非線形性を示す。その一例として、直径318㎜，長さ1.5mの鋼管杭を9本（3x3本）用いた杭基礎の水平載荷試験結果[7]を図－3.1 に示す。この図のように、荷重－変位量の関係は載荷の初期段階から非線形となっており、また、荷重除荷時に残留変位が残っている。これは、土自身の変形特性の非線形性とともに、変位の増加により地盤の塑性化領域が地表から深さ方向に順次進行したためと考えられる。このため、たとえ荷重を低いレベルにとどめたとしても厳密な意味で弾性限界を定義するのは困難であるといえる。

図－3.2 は、杭の水平方向載荷試験における各載荷サイクルごとの杭頭残留変位量と、除荷前の杭径に対する杭頭変位量の比率（杭頭ひずみSy/D）との関係の1例を示したものである。これから、ある変位レベルを境に残留変位量が急増することが読みとれる。荷重作用時の杭頭の変位量をこの残留変位量急増点Sry以内に抑えておけば、基礎の過大な残留変位を防止することができる。すなわち、地震などの短期荷重が作用した場合に基礎の変位をこの変位レベル内に抑えるよう設計すれば、荷重除荷後に大きな残留変位が残らないため補修を行なわずに使用可能となる。したがって、工学的意味での弾性的挙動、すなわち設計上無視しうる残留変位量となる変位の限界を杭の水平方向弾性限界と定義し、その定量的評価を行なう。

評価は、2．で紹介した多数の水平載荷試験データに基づいて行うこととし、上記の残留変位急増点Sryの他、図－3.3 に定義するワイブル関数分布曲線上の

降伏荷重に相当する点Sy について調査を行った。

ここで、場所打ち杭の水平載荷試験データと他の杭、たとえば鋼管杭のそれと比較すると、荷重－変位曲線の形状に特異性が見られた。その理由として次の２点が考えられる。

① 場所打ち杭は比較的小さい荷重レベルで杭体に曲げひび割れが発生し、それに伴い杭体の断面剛性が変化して杭体の曲げ剛性の非線形性の影響がでてくる。

② 場所打ち杭は施工時に周面地盤を緩めるので、載荷により周面地盤がもとの自然地盤の応力状態に達するまでは地盤の緩みの影響がでてくる。

このため解析にあたり場所打ち杭に対するワイブル関数分布曲線の適合性を高めるため、曲線式の変位指数mを１に固定しないで変化させている（他の杭についてはm＝１で十分に近似できる）。

残留変位急増点Sryおよびワイブル関数分布曲線上の降伏点Sy について、載荷試験のデータを用いて比較を行う。図－3.4(a)に鋼管杭の場合について、同図(b) に場所打ち杭の場合について両者の比較結果を示す。この結果、両者は比較的良い相関を示し、残留変位量が急増する点とワイブル曲線上の降伏点とはほぼ対応すると考えて差し支えないと思われる。

3.2 降伏変位量と地盤の変形係数

上記の水平方向弾性限界は、杭径，杭種のほか、杭の曲げ剛性や地盤の変形係数Eo とも関係していると考えられる。荷重－変位曲線をワイブル関数分布曲線で近似し、推定された降伏変位量Sy と地盤の変形係数Eo との関係を示したのが図－3.5 である。また、降伏ひずみ（Sy/D）と地盤の変形係数の関係を示したものが図－3.6 である。ただし、地盤の変形係数Eo は標準貫入試験のＮ値からEo＝７Ｎとして推定したものである。

これから次のことがいえる。

① 全体的にSy およびSy/DとEo との相関をみた場合、両者ともばらつきが大きい。

② 全体的な傾向として、Sy およびSy/Dは地盤の変形係数が大きくなるにつれ、小さくなる。しかしながら、Sy/Dについてはその傾向はあまり顕著ではない。

③ Sy およびSy/Dの値は鋼管杭，ＰＣ杭，ＰＨＣ杭では大きく、場所打ち杭では小さい。この傾向は特にSy/Dにおいて顕著である。

3.3 降伏変位量の統計的性質

上記のようにSy およびSy/Dと地盤の変形係数との相関があまり強くないため、地盤の変形係数とは無関係にSy およびSy/Dの統計的性質について分析する。

Sy およびSy/Dの密度分布を、図－3.7，図－3.8 に示す。両者とも対数正規分布に比較的よく適合しているといえる。図－3.9 に各種の杭のSy とSy/Dの密度分布を対数正規分布で近似させたものを示す。この対数正規分布を用いて、各種の杭に対するSy およびSy/Dの統計的性質を示したのが表－3.1 および表－3.2である。

これらより次のことがいえる。

① Sy およびSy/Dとも変動係数は100%を超えておりばらつきはきわめて大きい。

② Sy の平均値は、鋼管杭で26㎜、ＰＣ，ＰＨＣ杭で14㎜である。一方、Sy/Dの平均値は、鋼管杭で4.3%、ＰＣ，ＰＨＣ杭で3.7%である。

③ 場所打ち杭については、Sy ，Sy/Dとも小さく他の杭との違いがかなり大きいといえる。例えば、Sy/Dの平均値は１%である。これは場所打ち杭の水平抵抗が小さいというよりも、3.1 で述べたように杭体のひび割れ発生などが降伏点の判定に影響を与えているためと考えられる。

3.4 水平方向の弾性限界

荷重－変位量曲線における降伏荷重を超えるような大きな水平荷重を杭が受けると、杭の変形が急増し杭基礎の安定性が損なわれることは前に示したとおりである。このため、設計において照査すべき状態の一つとして杭基礎の変形をいわゆる弾性領域にとどめておく限界の状態がある。しかしながら、降伏変位量のばらつきは大きいので水平方向の弾性限界を設定する場合、どの程度このばらつきを考慮して合理的に決めるかが問題となる。

道示（1990）では、ここで解析した結果を基に杭の許容変位量として杭径の１%（≧15㎜）と決めたが、杭の特性に応じて個々に弾性限界を設定する方がより合理的であるかもしれない。例えば、鋼管杭は変形性能が大きいので杭径の２～３%程度、一方、場所打ち杭はその逆であるので杭径の0.7~1.0%程度とすることも考えられる。

4 弾性限界内での杭基礎の安定計算法

4.1 安定解析モデル

3.1 で定義したように、荷重作用時の基礎の変位が

弾性限界以内であれば、基礎の変位量も比較的小さく、また残留変位量も小さいレベルにある。したがって、この範囲においては杭を線形なバネに置換して安定計算を行っても大きな誤差は生じないと考えられる。このため、道示における杭基礎の安定計算は、フーチングを剛体とし、フーチングの変位（鉛直・水平・回転変位）を考慮した弾性解析法によって行うこととしている。

ここで、杭は押込み，引抜き，曲げともに線形弾性的であると仮定して、図－4.1 の解析モデルのように各杭を杭頭における等価な支点バネとしてモデル化している。すなわち、杭基礎の安定計算は、杭群を表す複数の支点バネで支えられた剛体（フーチング）の安定計算に置き換えられている。支点バネとしては、杭軸方向バネ定数Kv および以下に定義される杭軸直角方向バネ定数K_1～K_4から構成される。

K_1, K_3： 杭頭部に回転を生じないようにして、杭頭部を杭軸直角方向に単位量だけ変位させるとき、杭頭部に作用させるべき軸直角方向力(tf/m)および曲げモーメント(tf･m/m)

K_2, K_4： 杭頭部に移動を生じないようにして、杭頭部を単位量だけ回転させるとき、杭頭部に作用させるべき軸直角方向力(tf/rad)および曲げモーメント(tf･m/rad)

これらの杭軸直角方向バネ定数は、水平方向地盤反力係数k_H を用いた弾性床上の梁の理論に基づき算出される荷重と変位量との関係から、表－4.1 のように求められる。したがって、弾性限界内における杭基礎の安定計算においては、水平方向地盤反力係数の評価が重要な項目となる。以下、水平方向地盤反力係数の推定法について、水平載荷試験結果を基に検討を行う。

4.2 逆算による水平方向地盤反力係数

水平載荷試験より求めた水平方向地盤反力係数を逆算k_Hというが、次のように求めることができる。

載荷試験における杭頭変位量Sは、弾性床上の梁の式から、次のようになる。

$$S = \frac{P}{2EI\beta^3} \qquad (4.1)$$

ここに、

P ：杭頭載荷荷重（kgf）

S ：杭頭水平変位量（cm）

EI：杭体の曲げ剛性（kgf･cm²）

β ：杭の特性値（cm⁻¹）

$\beta = \sqrt[4]{k_H D / 4EI}$

D ：杭径（cm）

すなわち、式(4.1) に基準変位量So における載荷試験で得られたPを代入してβを求め、そのβを用いてk_Hを求めることができる。

4.3 推定による水平方向地盤反力係数

旧道示（1980）における杭の水平方向地盤反力係数の推定式は以下のとおりであった。

$$k = k_o y^{-1/2} \qquad (4.2)$$

$$k_o = \alpha E_o D^{-3/4} \qquad (4.3)$$

ここに、

k ：弾性体基礎の水平方向地盤反力係数（kgf／cm³）

k_o：設計地盤面での変位量を1cmとした時の水平方向地盤反力係数（kgf／cm³）

E_o：表－4.2 に示す方法で求めた地盤の変形係数（kgf／cm²）

α ：杭基礎の場合のE_oの求め方に対応する表－4.2に示す係数

D ：杭径（cm）

y ：設計地盤面での設計上着目する変位量（cm）

式(4.2)，(4.3)からもわかるように、水平方向地盤反力係数は地盤強度，変位量，杭径のみの関数で表されており、杭の深さ方向の変形モードの影響を無視している。この結果、杭長に比べて杭径の大きい杭基礎などでは小さめのk値が推定され、実状にあわない面があった。さらに、剛体基礎など他の基礎形式のk値との整合性に欠けていた。そこで、これらの点を解決するため、弾性体基礎の水平方向地盤反力係数の推定法として以下のとおり提案した。

土質試験等により地盤の変形係数が得られると水平方向地盤反力係数は次のように計算される。

$$k_H = k_{H0} \left(\frac{B_H}{30} \right)^{-3/4} \qquad (4.4)$$

ここに、

k_H ：水平方向地盤反力係数（kgf/cm³）

k_{H0}：直径30cmの剛体円盤による平板載荷試験の値に相当する水平方向地盤反力係数（kgf/cm³）で、各種土質試験・調査により求めた変形係数から推定する場合は次式により求める。

$$k_{H0} = \frac{1}{30} \alpha E_o \qquad (4.5)$$

B_H ：荷重作用方向に直交する基礎の換算載荷幅（cm）。一般に、弾性体基礎の水平抵抗に関与する地盤としては、設計地盤面から$1/\beta$までを考慮する（$=\sqrt{D/\beta}$）

E_o ：表－4.3 に示す方法で測定または推定した、

設計の対象とする位置での地盤の変形係数（kgf/cm²）。

α　：地盤反力係数の推定に用いる係数で、表－4.3に示す。

β　：基礎の特性値（cm⁻¹）

ここで提案している弾性体基礎の水平方向地盤反力係数の算定式は、従来の剛体基礎の算定式と基本的な式の構成は同じであるが、杭体部分の弾性変形の影響を考慮したものとしている。すなわち基礎上部に作用する水平力に対して、杭体の弾性変形が大きいほど、水平抵抗に寄与する地盤反力は上層部地盤に集中してくるからである。そこで、実験的，経験的に判断して、水平抵抗に寄与する地盤の抵抗範囲を$D \times 1/\beta$（D：基礎幅，$1/\beta$：水平抵抗に関与する深さ方向の地盤の長さ）とし、基礎の換算載荷幅を$\sqrt{D/\beta}$とした。この考え方を用いると上記で指摘したk値算定に関する矛盾や基礎形式間の不整合がかなり整理されたことになる。

4.4 推定によるk値と載荷試験による逆算k値との比較

以下、杭の載荷試験データに基づいて式(4.4)の推定精度について検討する。図－4.1は旧道示（1980）に基づき式(4.3)により推定したk_Hと基準変位量10mmの逆算k_Hを比較したものであり、図－4.3は基準変位量を杭径の1％とした逆算k_Hと、式(4.4)から求めたk_Hを比較したものである。また、図－4.4，4.5は、それぞれ逆算k_Hと推定k_Hの比をとり、その分布を対数正規分布として処理したものである。

用いたデータの90％以上が杭径が1,000mm以下（約半分が杭径が600mm以下）であることもあり、旧道示式によるk値に比べて提案式によるk値の精度があまり向上していない。これは、提案式が剛体基礎のk値および大口径基礎のk値に対する整合性を高めた結果であるが、k値の推定式にはこの程度のばらつきがあることも認識しておくことが肝要である。

5．大変形時の杭基礎の挙動

5.1 地盤抵抗の非線形性の影響

杭基礎の変形が小さく弾性限界内にあるときには、4.1で述べたように、地盤反力特性は線形弾性であると仮定し、地盤の抵抗特性を水平方向地盤反力係数k_Hによりモデル化している。この仮定は変位が小さい領域での設計においては妥当と考えられるが、大変形時には誤差が大きくなるため、地盤反力度の非線形性を考慮する必要がある。

地盤の各深度における地盤反力度pと変位量yとの関係としては、DNV[8]などでは図－5.1に示すp－y曲線を用いている。基礎の挙動を精度良く表すには地盤反力特性を忠実にモデル化するのが望ましいが、設計にあたっての地盤調査精度や計算の簡便さを考慮すると、一般の橋梁にこの方法を適用することは必ずしも妥当とはいえない。したがって、ここでは地盤反力特性を図－5.2に示すようなバイリニアとしてモデル化することとし、水平載荷試験結果によりこの解析手法の有効性を検討した。

解析に用いるデータとして、2．で収集した載荷試験の内、杭径の30％以上変形しているデータを抽出した。地盤抵抗の定数として次式により定めた水平方向地盤反力係数k_{HS}と最大地盤反力度p_{u1}をパラメータとし、荷重－変位量曲線および杭の曲げモーメント分布の誤差が最小となる組み合わせを選定した。

$$k_{HS} = \alpha_k \times (\text{道示による } k_H) \qquad (5.1)$$

$$p_{u1} = \alpha_p \times (\text{Coulombの受働土圧強度}) \qquad (5.2)$$

解析範囲としては、地表面での水平変位が杭径の10％までの試験結果を対象とし、上記パラメータを変化させて解析結果との誤差を求めた。その一例として、砂質地盤中に打設された直径61cm，杭長21mの鋼管杭の試験における誤差のコンター図を図－5.3に示す。この事例においては、地盤反力度の上限値として受働土圧強度の3.5倍，地盤反力係数として道示による値の1.5倍程度で誤差が最小となっているのが認められる。ここで、道示による水平方向地盤反力係数k_Hは4．で述べたように杭頭での変位が杭径の1％の状態で定められているのに対し、弾塑性解析に用いる水平方向地盤反力係数はそれより小さい変位レベルでの特性を表しているため、α_kは1以上となっている。また、α_pについては、地盤の抵抗領域が三次元的に広がっており、塑性化した領域が杭幅の3倍程度となっていたと考えられる。このときの荷重－変位量曲線を図－5.4、に杭の曲げモーメント分布を図－5.5に示す。両図とも比較的良く一致しており、単純化したモデルにおいても定数を適切に設定することにより、杭の挙動を表すことが可能であることが認められた。

上記の結果は他の試験杭においても同様に認められたが、地盤抵抗のパラメータα_kおよびα_pの値については表－5.1に示すように各試験により異なっている。このように、本解析法の有効性は認められたが、設計に用いる定数の評価法については今後の検討課題であり、解析データの蓄積が必要である。

5.2 群杭による影響

橋梁基礎に用いられている杭基礎は一本の単杭で用いられることは少なく、一般的には複数の杭をフーチングで結合して用いられる。このような組杭に水平力が作用し、杭が大きく変形した時、地盤を介して各杭が相互に干渉し合うため、各杭の地盤反力度は杭位置によって異なることが知られている。しかしながら、その干渉の度合いについては、杭種、杭間隔、土質などによる差異が大きく、未だ明らかにされていない。道示においては、杭の最小中心間隔を原則として杭径の2.5 倍以上とし、この場合には単杭についての水平方向地盤反力係数を群杭を構成する全杭について同じ値を用いている。近年、これらの問題について、様々な機関で杭の水平抵抗機構を把握するための実験が行われており、ここでは土木研究所、および首都高速道路公団で行われた鋼管杭による水平載荷実験を基に、群杭による影響について検討を行った。

土木研究所での実験[9]は図－5.6 に示すような単杭、直列杭，並列杭，および組杭（模型鋼管杭φ101.6㎜）について、土槽内に作成した実験地盤で載荷したものである（図－5.7参照）。また、首都高速道路公団[7]では、図－5.8 に示すように実大鋼管杭（φ318.5㎜）で原位置に９本組杭を作成し、水平載荷試験を行っている。

図－5.9 は土木研究所の実験における平均載荷荷重（杭１本当たりの平均荷重）～フーチング変位量曲線を示す。図より、変位量が大きい範囲では荷重－変位量曲線はいずれも単杭の曲線の下方にあり、これは杭相互の干渉による影響と考えられる。その影響度合いは９本組杭が一番大きく、直列杭、並列杭の順で小さくなり、単杭に近くなる。これは、群杭においては、同一フーチング内の杭の位置によって水平力に対する水平反力の分担が異なるからである。

図－5.10は９本組杭において中央の杭に対する前・後杭の分担する水平反力の分担比を示したものである。両方の実験についても、前杭が大きな荷重を分担し、中央杭、後杭となるのに従って分担荷重が小さくなっていることがわかる。この傾向は荷重が大きくなるにつれて顕著になっている。

このような実験から、群杭における各杭の反力分担の差異、および大変形時の挙動について、設計手法の確立を目指して解析によるシミュレーションを試みた。シミュレーションでは大変形時に問題となる地盤のｐ－ｙ特性における非線形性を、5.1 で検証したバイリニア型で扱うこととした。また、各杭の荷重分担の差異を考慮するため、各杭位置に対して、別々のα_p，α_kをパラメータとした地盤抵抗特性（p_u，k_H）を試行錯誤的に設定し、

①荷重～変位曲線　　②杭体変位分布

③曲げモーメント分布　④地盤反力度分布

の４項目について実験値と解析値が最もよく適合するようなα_p、α_kの値を選定した。ここで、シミュレーションする変位レベルは杭体が降伏するまでとし、ここでは杭径の50%程度の変位まで適合させるようにした。

図－5.11は９本組杭のフーチング部の荷重－変位曲線について実験値に適合するα_p，α_kを選定したもので、適切な値を設定することによりかなりの精度でシミュレーションできることがわかる。９本組杭において図－5.11で最も適合性がよかったα_p、α_kの組み合わせにおける、曲げモーメント分布、および地盤反力度分布を図－5.12，5.13示す。これらの値も実験値と計算値は比較的よく合っているといえる。このほか、杭体変位についても同程度の適合性を得ることができた。

このような適合性を得ることができるα_p、α_kを他のケースについてもシミュレーションによって選定し、その結果を示したのが表－5.2 である。直列杭について前杭は単杭と同程度の α_pとなり、後方の杭は徐々に低下している。並列杭は単杭に比べてα_p、α_kともに、並列方向の本数が多くなるほど低下することがわかる。直列杭と並列杭の組合せとなる９本組杭では、α_p，α_kともに直列杭，並列杭に比べてさらに低下していることがわかる。これらの現象は、換算載荷幅の増大による地盤反力係数の低減や、各杭の抵抗範囲が互いに重なることによる受働抵抗の減少などによるものと考えられる。

5.3 杭体の曲げ剛性の非線形性による影響

5.1および5.2の解析において対象とした載荷試験はいずれも鋼管杭であり、また、解析した変位量の範囲は杭体が降伏するまでであったため、杭体自身は弾性体として扱っている。これに対し、場所打ち杭や、鋼管杭においても降伏点を超えた領域に対しては、杭体の曲げ剛性の非線形性の影響も顕著になってくる。

一般にコンクリート杭に曲げモーメントが作用した場合の曲げモーメントMと曲率φとの関係は、コンクリートのひびわれ開始時（C），引張り鉄筋の降伏開始時（Y）および断面の終局時（U）が認められる。載荷試験の解析においてこの影響を考慮する場合、軸方向力Nと曲げモーメントMを受ける円形鉄筋コンクリート断面に対して求めたこれらの点を結ぶトリリニ

ア曲線でモデル化する方法がある（図－5.14）。

その一例として、場所打ち杭の載荷試験に関する解析例[10]を紹介する。

図－5.15は場所打ち杭の載荷試験において計測された荷重－変位曲線およびそのシミュレーション結果を示したものである。ここで、地盤抵抗特性は、バイリニア型としている。杭体を線形とした解析では、変形量が小さい領域では一致しているが、変形量が大きくなると解離してくる。これに対し、杭体の曲げ剛性の非線形性を考慮することにより、大変形時までシミュレーションできている。

上記のように、杭基礎の大変形時の挙動は、地盤抵抗や杭体剛性の非線形性の影響および群杭効果などを考慮することによりシミュレーションできることが認められた。なお、一般の杭基礎の設計計算に用いる解析モデルについては、設計で考慮すべき変位レベルとも合わせて検討したい。

6．おわりに

杭基礎の設計に限界状態設計法を導入するために、杭の挙動を把握しその解析モデルを確立するために水平載荷試験の解析を行った。本研究結果をまとめると次のように要約される。

1）杭基礎の弾性限界を可逆性が保証される限界と考え、残留変位が急増しない点を設計における弾性限界とする。この弾性限界は荷重－変位量曲線の降伏点と概ね一致している。

2）上記の弾性限界は変動係数が100 %を越えておりばらつきはきわめて大きいが、全杭平均で杭径の3.5 %，鋼管杭で4 %，場所打ち杭で1 %程度（いづれも対数正規分布）である。ただし、場所打ち杭については杭体のひび割れ発生が降伏点の判定に影響を及ぼしている可能性がある。

3）弾性限界内での安定計算に用いる水平地盤反力係数の算定式を提案した。ここで、杭の深さ方向の変形モードの影響を考慮し、基礎の換算載荷幅を$\sqrt{D/\beta}$と提案した。また、旧道示は基準変位量10㎜時の水平方向地盤反力係数であったが、2）に伴い水平地盤反力係数も杭径の1 %時で算定することとした。これらによる水平方向地盤反力係数の推定精度は旧道示と同程度であることが確認された。

4）大変形時の杭基礎の挙動については、地盤抵抗の非線形性，群杭による影響および杭体の曲げ剛性の非線形性などを考慮することによりシミュレートできることが確認された。

本研究の実施に当たっては、各機関から杭の水平載荷試験結果を提供していただいた。なお、本研究の一部は鋼管杭協会との共同研究により行っているものであり、ここに記して謝意を表す。

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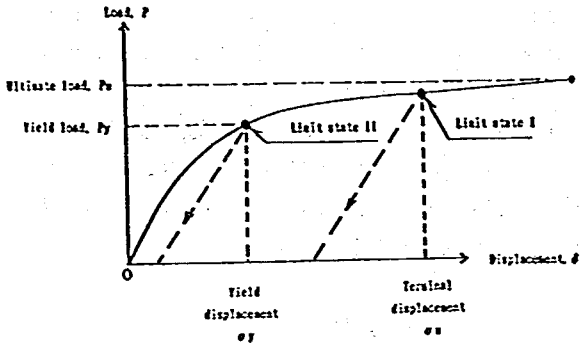


Figure 1.1. Limit states for stability calculations of foundations

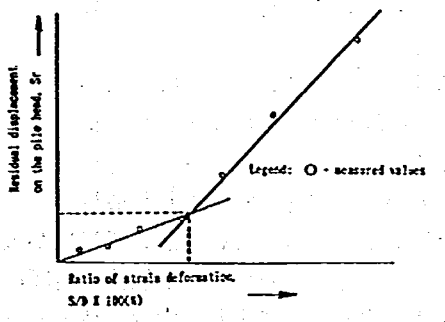


Figure 3.2. A case in which the point where the residual displacement of the ground increases suddenly is defined as the elastic limit

Table 2.1 Data obtained from horizontal loading tests

Type of Pile	Number of date
Steel pipe pile	120
Cast in pile	29
PC pile	31
PHC pile	24
Total	204

(note) Steel pipe piles, PC piles and PHC piles are placed by the driven-pile method.

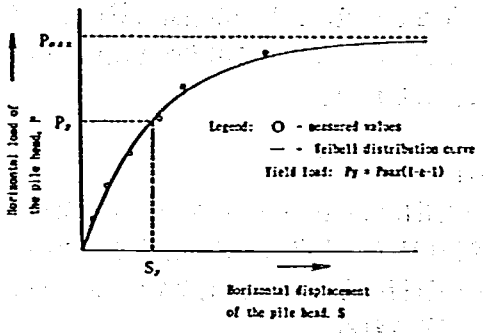


Figure 3.3. A case in which the yield load on the Weibull curve is defined as the elastic limit

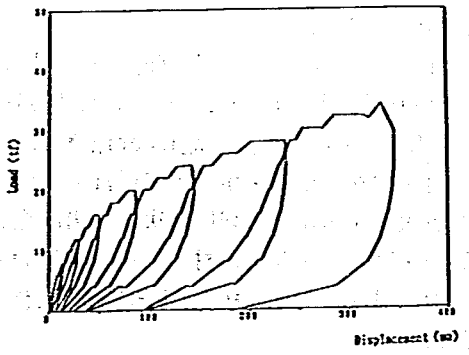
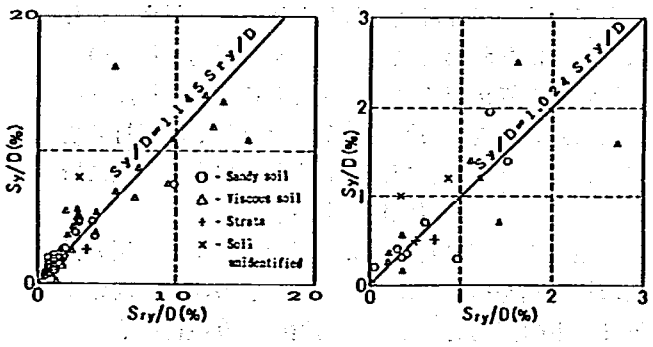


Figure 3.1. Load - head displacement (horizontal loading tests)



Sy: Yield displacement defined by the Weibull distribution curve. d=(mm)
Sry: Yield displacement obtained from the point where the residual displacement increases suddenly (mm)
D: Pile diameter (m)

Figure 3.4. Comparison of yield strain deformation obtained from formula of the Weibull distribution curve and the point where the residual displacement increases suddenly

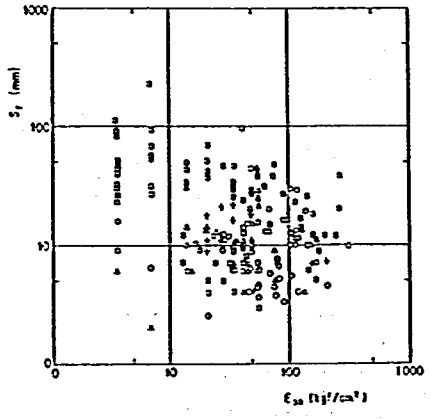


Figure 3.5. Correlation between Sy and E50

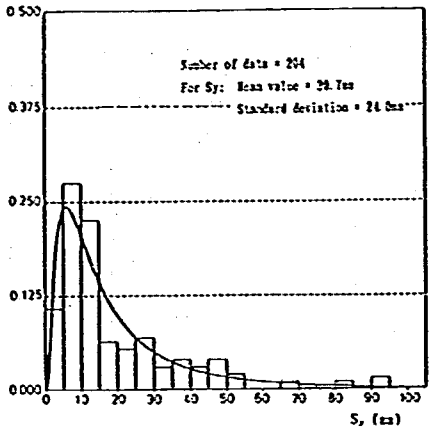


Figure 3.7. Probability density distribution of Sy

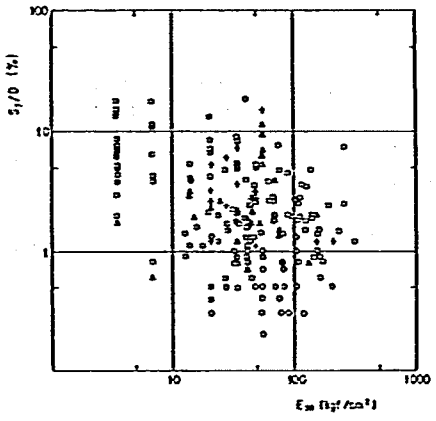


Figure 3.6. Correlation between Sy/D and E50

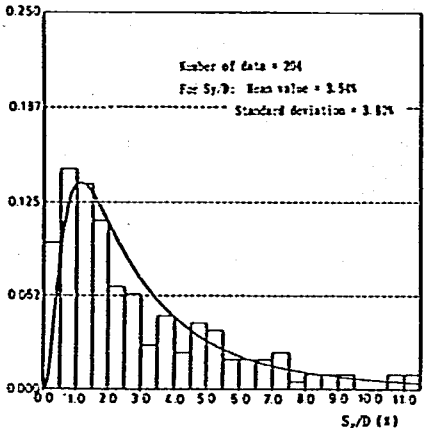


Figure 3.8. Probability density distribution of Sy/D

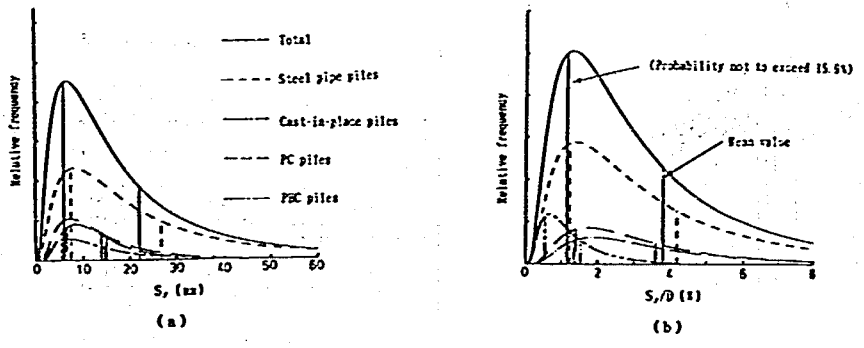


Figure 3.9. Probability density distribution of Sy and Sy/D, expressed by logarithmic normal distribution

Table 3.1 Statistical value of S_y

		Steel pipe pile	Cast in place pile	PC pile	PHC pile	Total
Number of sample		128	29	31	24	224
Extent	(cm)	2.4~224.8	2.5~18.5	3.8~45.9	4.8~14.8	2.8~224.8
Mean value	(cm)	25.8	10.5	14.8	13.6	28.2
Standard deviation	(cm)	28.8	10.1	11.5	8.5	24.8
Coefficient of variation (%)		111	96	78	62	116
S_y (cm)	Non-exceed probability 5 (%)	4.8	2.8	3.8	4.6	3.8
	10 (%)	5.8	2.7	4.8	5.6	4.2
	15.8 (%)	2.2	3.4	5.8	6.8	5.4
	30.8 (%)	11.8	5.8	8.2	8.7	8.4

Table 3.2 Statistical value of S_y /D

		Steel pipe pile	Cast in place pile	PC pile	PHC pile	Total
Number of sample		128	29	31	24	284
Extent	(%)	0.3~22.4	0.2~4.8	0.2~15.3	0.1~14.9	0.2~22.4
Mean value (cm)	(%)	4.1	0.8	14.8	3.8	3.5
Standard deviation	(%)	4.8	0.8	11.5	2.8	3.8
Coefficient of variation(%)		105	98	78	76	109
S_y / D (%)	Non-exceed probability 5 (%)	0.7	0.3	3.8	1.8	0.6
	10 (%)	0.8	0.2	4.8	1.3	0.8
	15.8 (%)	1.2	0.3	5.8	1.5	1.8
	30.8 (%)	1.8	0.5	8.2	2.1	1.5

Table 4.1. Spring constant in the vertical direction to the axis of piles ($\beta l \geq 1$)

	Rigidly connected pile heads		Hinge-connected pile heads	
	$h = 0$	$h > 0$	$h = 0$	$h > 0$
K_1	$\frac{12EI\beta^3}{(1+\beta h)^3+2}$	$4EI\beta^3$	$\frac{3EI\beta^3}{(1+\beta h)^3+0.5}$	$2EI\beta^3$
K_2, K_3	$K_1 \cdot \frac{h}{2}$	$2EI\beta^2$	0	0
K_4	$\frac{4EI\beta}{1+\beta h}\frac{(1+\beta h)^3+0.5}{(1+\beta h)^3+2}$	$2EI\beta$	0	0

where

β : Characteristic value of piles $\beta = \sqrt[4]{\frac{kD}{4EI}}$ (cm^{-1})

λ : $h = \frac{\lambda}{\beta}$ (m)

k : Coefficient of horizontal ground reaction (tf/m^3)

D : Pile diameter (m)

EI : Flexural rigidity of piles (tf/m^2)

h : Length in the axial direction above the design ground of piles (m)

Table 4.2 Relationships between E_0 and α, of elastic foundation

Coefficient of variation of the ground E_0 (kgf/cm^2)	α
Coefficient of variation of the ground measured inside boring pores	0.5
Coefficient of variation of gained by unconfined or triaxial compression test	0.3
Coefficient of variation of estimated from $E_0=28N$ of N value of standard penetration test	0.2

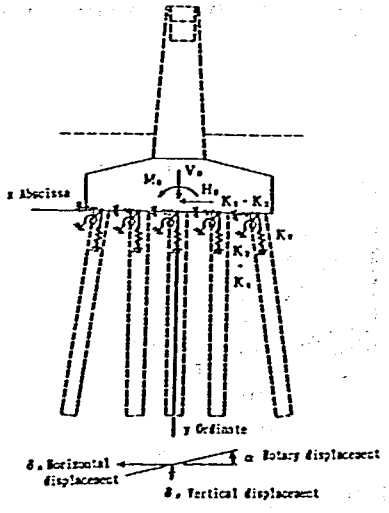


Figure 4.1. Analytical model in the stability calculation of pile foundations

Table 4.3. E_0 and α

Modulus of deformation E_0 (kg/cm^2)	α	
	Ordinary state	During earthquake
Modulus of deformation obtained from the repetition curve of 30cm diameter plate loading test	1	2
Modulus of deformation measured in a bored hole	4	8
Modulus of deformation obtained from the unconfined compression test or triaxial compression test of specimens	4	8
Modulus of deformation estimated by $E_0 = 28N$ from N value of standard penetration test	1	2

(note) In case of a windstorm the values of ordinary occasions should be used.

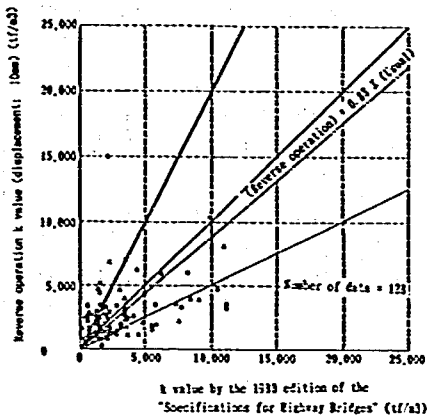


Figure 4.2. Reverse operation k value(displacement:10mm) and estimated k value in the 1980 edition of the "Specifications"

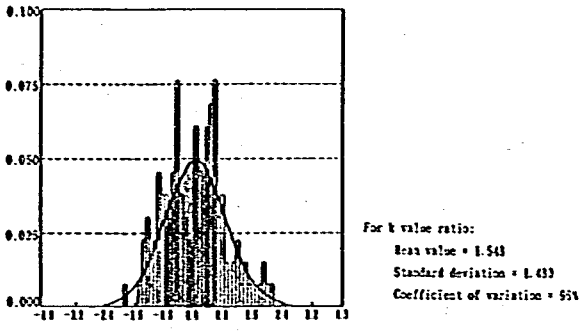


Figure 4.5. Distribution of Log (k value ratio : reverse operation k value (displacement:1% of pile diameter) / k value in proposed formula)

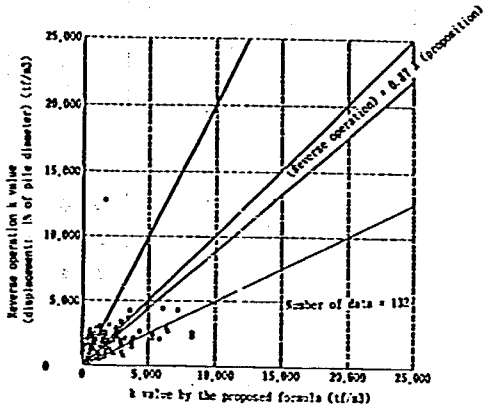


Figure 4.3. Reverse operation k value (displacement: 1% of pile diameter) and estimated k value by the proposed formula

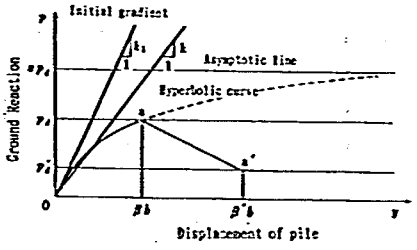


Figure 5.1. Ground resistance characteristics in DNY

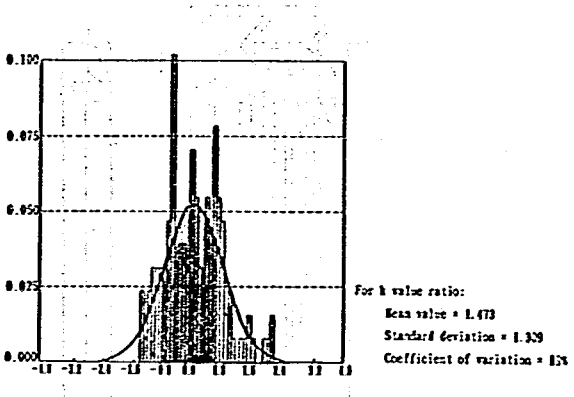


Figure 4.4. Distribution of Log (k value ratio : reverse operation k value (displacement:10mm)) / k value in the 1980 edition of "Specifications"

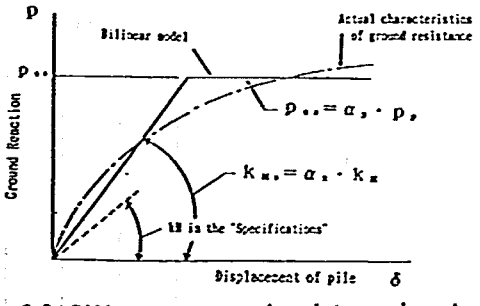


Figure 5.2. Bilinear-type ground resistance characteristics

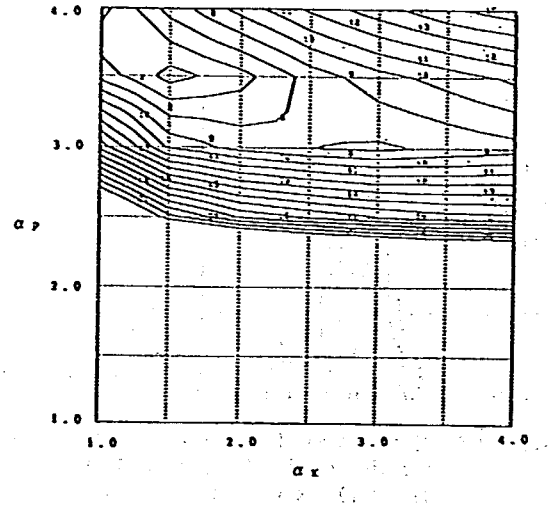


Figure 5.3. Contour map of errors (S3)

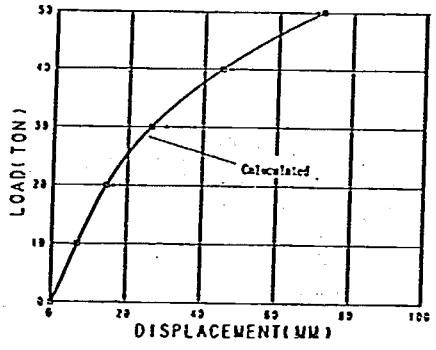


Figure 5.4. Load-displacement Curve (S3)

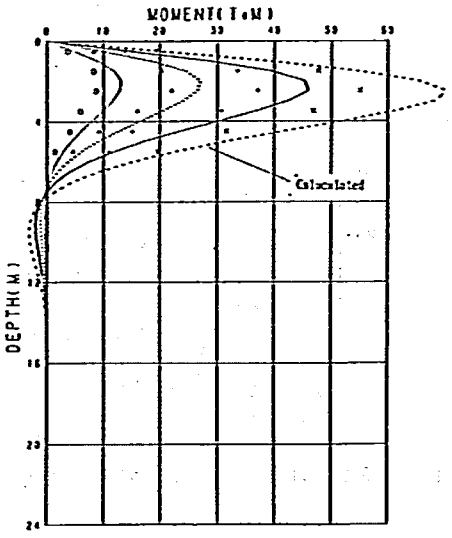


Figure 5.5. Distribution of bending moment of pile (S3)

Table 5.1. Most adequate parameters

Test	Pile D x t(mm)	Pile L(m)	αp	αk	Ground
S 1	φ 191x 5.3	4.5	2.0	1.5	Sand
S 2	φ 319x 6.9	15.0	6.0	6.0	Sand
S 3	φ 610x 9.5	21.0	3.5	1.5	Sand
S 4	φ 610x12.7	36.0	4.0	1.5	Sand
S 5	φ 813x15.0	17.0	4.5	1.5	Sand
C 1	φ 191x 5.3	5.0	0.5	1.0	Soft Clay
C 2	φ 813x14.0	30.7	2.5	1.5	Stiff Clay
C 3	φ1000x27.2	20.0	1.0	2.5	Stiff Clay
C 4	φ1500x13.0	33.1	1.0	1.5	Soft Clay
C 5	φ2000x22.0	51.5	2.0	6.0	Soft Clay

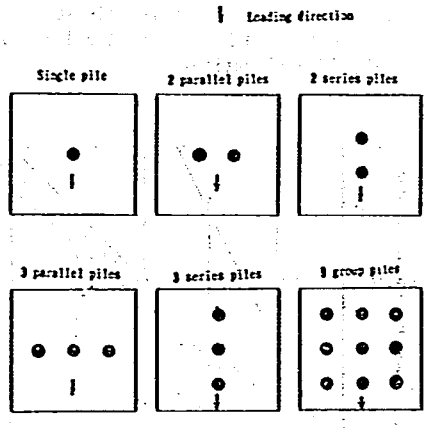


Figure 5.6.

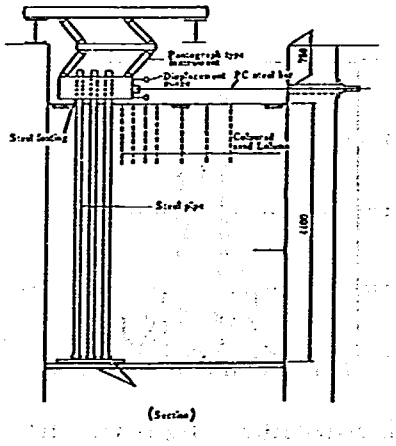


Figure 5.7. Test method for group piles (PTRI)

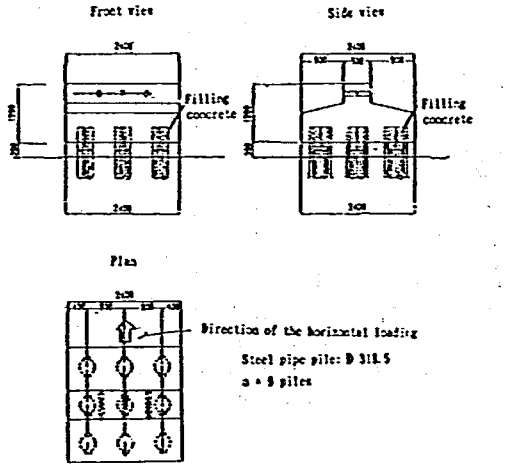


Figure 5.8. Horizontal loading tests of piles (MEPC)

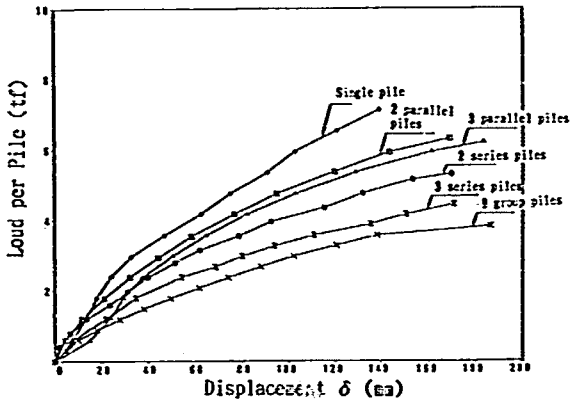


Figure 5.9. Load-displacement curve

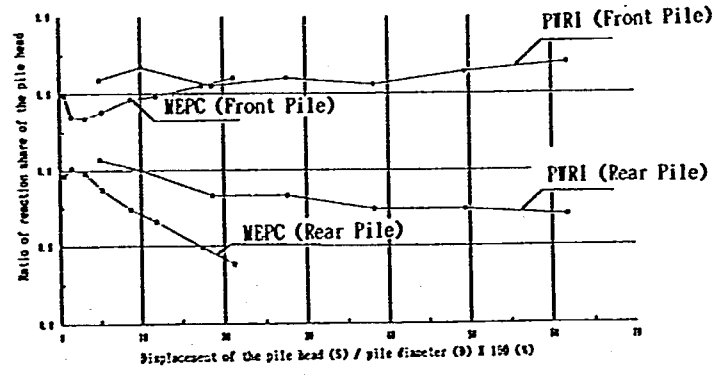


Figure 5.10. Ratio of load share to the central pile (9 piles used)

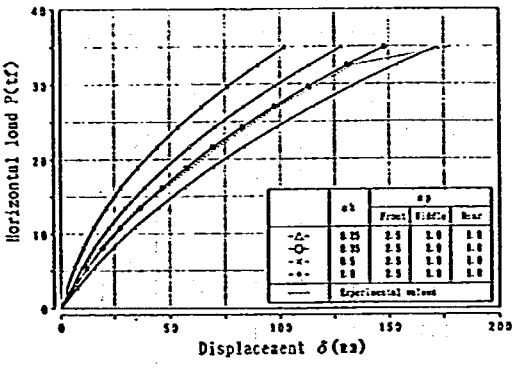


Figure 5.11. Load-displacement curve of the 9 piles

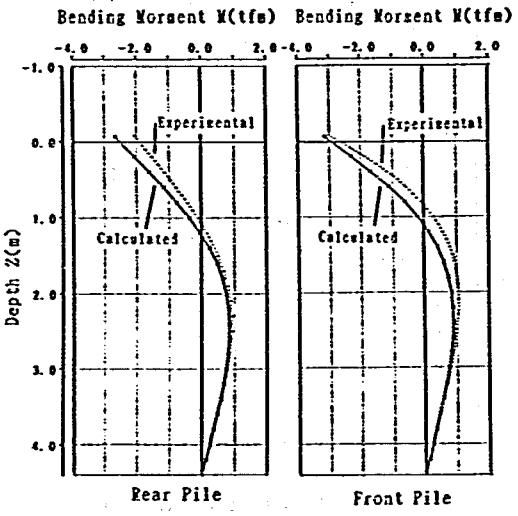


Figure 5.12. Distribution of bending moment of the 9 piles

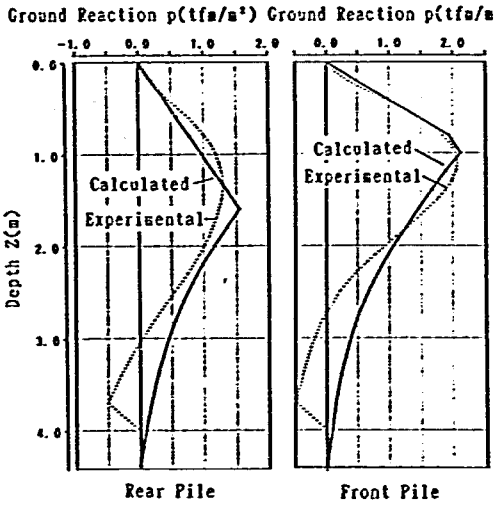


Figure 5.13. Distribution of ground reaction of the 9 piles

Table 5.2. Comparison between experimental values and analytical values

CASE	α_s	α_p		
		Front piles	Middle piles	Rear piles
Single pile	1.5	(3.0) 3.0	-	-
2 series piles	1.5	(3.0) 3.0	(0.8) 1.0	-
3 series piles	1.0	(3.0) 3.0	(1.3) 1.0	(0.8) 0.5
2 parallel piles	1.0	(2.5) 2.5	-	-
3 parallel piles	0.75	(Center 2.8) (Side 3.2) 2.5	-	-
9 group piles	0.35	(Center 2.5) (Side 2.4) 2.5	(Center 1.6) (Side 1.3) 1.0	(Center 1.3) (Side 1.2) 1.0

Figures in parenthesis are experimental values

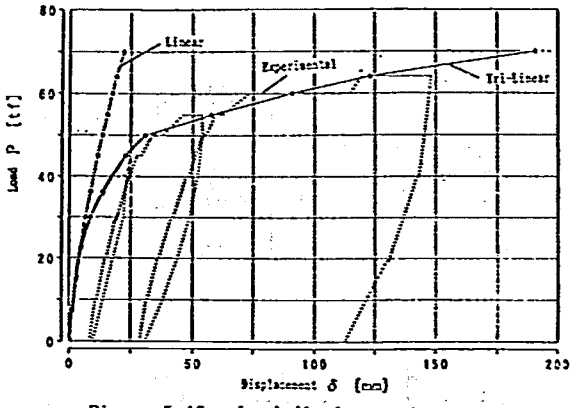


Figure 5.15. Load-displacement curve of Cast-in-place Pile

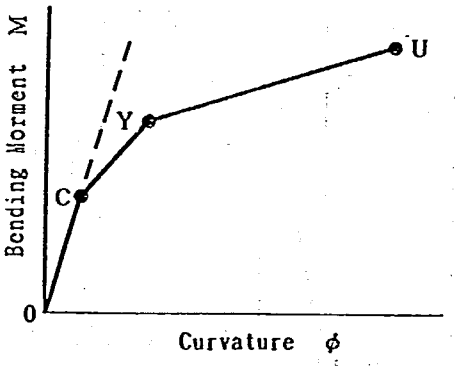


Figure 5.14. M-ϕ of Cast-in-place Pile

Seismic Effects on Port and Harbour Structures

by

Hajime Tsuchida[1] and Setsuo Noda[2]

ABSTRACT

In Japan, many port facilities were damaged by the earthquakes. Efforts has been being continued for designing and constructing facilities of larger seismic resistance with less construction cost. Design standards for the earthquake resistant design were proposed and used widely. After earthquakes of large scale, performance of port facilities in a zone of sever shaking were investigated in details. Results of those investigations were reflected to revision of the design standards.

This report presents Japanese experience on earthquake damages, current practice in seismic design and countermeasures against earthquake disaster in connection with port facilities.

KEYWORDS: Earthquake damage, earthquake resistance design, liquefaction of soil, Port and Harbour structures,.

1. DAMAGE EXPERIENCE

1.1 Earthquakes which caused seismic damage

Many of the port facilities in Japan were damaged by earthquakes. The Kanto earthquake in 1923 is well known as one of the earthquakes which caused serious damage to many kinds of structures. Port facilities in Yokohama and Yokosuka were damaged by that earthquake, too. However, none of seismographs recorded peak value of the ground motions in the areas where damage to port facilities took place.

In 1960's, a number of deployed accelerographs increased considerably and since then, more reliable information on ground motions became available. Major earthquakes which caused damage to port facilities since that period are given in Table 1.

The peak acceleration shown in the table were all recorded in port area except Niigata. However, the instrument which recorded the Niigata earthquake was located not much distant from the Niigata Port. In the 1973 Nemuro-Hanto-oki earthquake, the most serious damage took place in Hanasaki Port. Unfortunately, no accelerograph was installed at the port. Although the peak ground acceleration in Kushiro Port is given in the Table 1, the damage in Kushiro Port by this earthquake was not serious.

In the 1987 Chiba-ken-Toho-oki earthquake, there was no damage to port facilities; but liquefaction was observed at many locations in artificially filled land.

Field reconnaissances in detail on seismic performance of port facilities (damaged and undamaged) were carried out after the major earthquakes and reported (Bureau for Ports and Harbours, Ministry of Transport, et al., 1964, 1965, 1968, and 1973; Tsuchida, et al., 1979; Tsuchida, et al., 1985; Iai, et al., 1988).

1.2 Damage examples

1) Liquefaction of ground

During the Niigata earthquake in 1964, seismic damage to large variety of structures took place due to liquefaction of sandy grounds. By this earthquake people started to recognize the importance of liquefaction and very intensive research activities were undertaken by many researchers. Liquefactions were observed in almost all major earthquakes which hit coastal areas in Japan.

2) Gravity Type Quaywall

By the 1983 Nippon-kai-Chubu earthquake, a quaywall using concrete caissons in Akita Port was seriously damaged. Fig. 1 shows the typical cross section and displacements of the quaywall (broken line : after earthquake, solid line : original). The damage was sliding of the caissons towards the sea side more than 1 meter and settlement of backfill was about 1.5 meter in maximum. It was evident that liquefaction had taken place behind the quaywall.

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3) Sheet pile bulkhead
Fig. 2 also shows a cross section of the steel sheet pile bulkhead (Ohama No. 2 Wharf) in Akita Port. The sheet piles were broken due to large pressure caused by the liquefied backfill. Adjacent to this bulkhead, there located another sheet pile bulkhead (Ohama No. 1 Wharf) of very similar construction. There was no liquefaction at this site and the structure survived without any damage.

4) Trestle type pier
Since 1960's many trestle type piers with steel pipe piles were constructed. Some of them were subjected to strong ground motions. However, none of them was damaged by the earthquakes. Fig. 3 is a cross section of the trestle type pier in Muroran Port. This pier experienced ground motions of peak acceleration 209 Gal, during the 1968 Tokachi-oki earthquake. The structure was designed for the seismic coefficient 0.10.

5) Cellular bulkhead
The cellular bulkhead in Muroran Port was subjected to the ground motions of 209 Gal, but no damage was seen. Fig. 4 is showing a cross section of the bulkhead.

6) Breakwater
No damage due to the earthquakes listed in Table 1 was seen on the breakwaters. During the Kanto earthquake in 1923, a breakwater in Yokosuka Port settled more than 5 meters. Fig. 5 is a cross section of the breakwater.

7) Warehouse
Many warehouses were damaged by earthquakes in past. The warehouse in Gaiko District of Akita Port was seriously damaged by the 1983 Nippon-kai-Chubu earthquake. The floor slab settled significantly and the foundation piles were broken.

2. OBSERVATION OF STRONG-MOTION EARTHQUAKES

The authors and their colleagues have observed the strong-motion accelerations during earthquakes for 29 years in Japan. As of December 1990, 4802 strong motion earthquake records have been obtained by the observation network of 81 strong-motion accelerographs deployed throughout Japanese port areas (Kurata and Iai, 1991). Fig. 7 shows one of records recovered from this network during the major earthquake. The records were the ground accelerations at Hachinohe Port during the 1968 Tokachi-oki earthquake which caused serious damage to many kinds of engineered structures.

To study seismic wave propagations and strains of ground due to seismic waves, the observation of ground motions at individual points have been found to be insufficient. An array observations have been considered essential. Many research groups started observation of this type. PHRI deployed seismometers three dimensionally in the field of the Tokyo International Airport (Haneda). Fig. 8 is showing the deployment of seismometers of the PHRI array in Haneda.

3. CURRENT PRACTICE IN EARTHQUAKE RESISTANT DESIGN

3.1 Procedure

Structures in ports and harbours in Japan are designed in accordance with the Technical Standard of Port and Harbour Facilities and its Commentary (Japan Port and Harbour Association, 1989). Chronologically, the procedure for earthquake resistant design was compiled as early as in 1950, revised, supplemented or legislated in 1959, 1967, 1971 1975 and 1979. The most recent revision in 1989 aims at assuring, in practice, the greater safety against liquefaction; i.e. the lesson learned from the case history in 1983 Nippon-kai-Chubu Earthquake of magnitude 7.7.

According to the Technical Standard, the steps to be taken for earthquake resistant design are as follows:

(a) take into account the effects of seismic inertia force upon the safety and stability of structures or the effects of seismic displacement upon lifeline facilities and
(b) assess the liquefaction potential of soil due to earthquakes and, if necessary, take measures against liquefaction.

Current procedures taken for these steps will be briefly reviewed in the following sections.

3.2 Stability against seismic inertia force

Seismic inertia force considerably affects the seismic stability of port facilities such as quaywalls, piers and breakwaters. Those effects are, in the routine design practice, treated by a method well known as seismic coefficient method; the seismic inertia force is idealized into a static horizontal load of which magnitude is specified by the seismic coefficient.

The seismic coefficient is determined by taking

1) regional seismicity,
2) condition of foundation soil and
3) importance of structures

into consideration. The regional seismicity in Japan is studied by Kawasumi (1951) with case history data of 343 earthquakes since the year of 599 A.D.. The results obtained by Kawasumi are critically reviewed and idealized, for use in design procedure, into

seismic zoning with three-regions, of which regional seismic coefficients are specified as 0.15, 0.10 and 0.05.

Because the results given by Kawasumi are of the alluvial ground, a factor should be multiplied to the regional seismic coefficients for taking variety in soil conditions into account. The factors are given for three kinds of subsoil conditions, specified as 0.8, 1.0 and 1.2.

In addition to the subsoil condition, importance of structures should be considered for determining seismic coefficient. The factors to be multiplied to the seismic coefficients are given for four levels of importance, specified as 1.5, 1.2, 1.0 and 0.5.

Seismic inertia force thus specified will, when loaded, produce increase in the stresses acting in material of structures. Such stresses will be compared with the allowable limits, which are specified as 1.5 times high as those specified for the static stability analysis.

Thus, basically all kinds of structures are designed by seismic coefficient method for the routine practice. Among the various port structures, piers with piles (i.e. trestle type piers) show characteristic dynamic response during earthquakes. For the piers of this type, the validity of the design will be checked by the analysis in which dynamic response of the pier has been taken into consideration (Yamamoto et al, 1970).

4. VALIDATION WITH CASE HISTORY AND STRONG MOTION DATA

4.1 Effect of vertical component on stability

Every fives years or so often the structures designed by the simple procedure presented in the previous section have been exposed to great earthquakes in Japan. With the strong-motion observation network deployed by the Port and Harbour Research Institute since 1963, validity of the simple procedure has been critically evaluated by the research groups of the same research institute.

During the earthquake, structures receive vertical seismic inertia force as well as horizontal one. It is the practice in design, however, that the seismic inertia force is represented by nothing but a horizontal load. Uwabe et al.(1976), using vertical components of 574 strong-motion accelerograms, concluded that ratios of vertical vs. horizontal components in the peak values are less than 1/2 as shown in Fig. 8 and the vertical component has indeed little effect on the stability of gravity type quaywalls.

4.2 Relation between max. acceleration and seismic coefficient

During the earthquake, the peak value of the seismic inertia force is applied instantaneously. The seismic load considered in the design procedure is, however, a continuously applied load. If duration of intense seismic loading is limited to a very short period, nothing but a insignificant deformation will be induced. Besides, if slight deformation is allowed for dense saturated sand (i.e. non-liquefiable sand), dilatant behavior under undrained condition will instantaneously increase the strength of soil. Thus, the instantaneously applied peak value of the seismic inertia force can be higher than the seismic load to be considered in the seismic coefficient method. Noda and Uwabe (1975), using case history data of 129 gravity type quaywalls during 12 earthquakes, studied the relation between the maximum ground acceleration recorded by SMAC-B2 type accelerograph and the seismic coefficient for gravity type quaywall. The results are obtained as shown in Fig. 9 with the relation to be used for the conversion from maximum acceleration to seismic coefficient as follows.

$$k_h = \alpha/g \qquad (\alpha \leq 0.2g) \qquad (1)$$
$$k_h = 1/3(\alpha/g)^{1/3} \qquad (\alpha > 0.2g)$$

in which

k_h: seismic coefficient
α : maximum acceleration (Gal)
g : acceleration of gravity (Gal)

Kitajima and Uwabe (1979) conducted similar study for anchored steel sheet pile quaywalls and, using 110 case history data, concluded that the relation for the gravity type quaywall is also applicable for the steel sheet pile quaywalls.

As shown in Fig. 10, the relation in Eq.(1) contains conservative errors. If an average relation is sought for representing the general correlation between these quantities, it will be given as follows.

$$kh \cong 0.5\ \alpha/g \qquad (2)$$

Obviously the relation by Eq.(2) will not be used in practice for determining the seismic coefficient from maximum acceleration. This relation will, however, be discussed again in the following section in the context of maximum acceleration to be used for assessing liquefaction potential.

4.3 Stability against liquefaction

Soil liquefaction, if occurs, considerably affects the stability of structures. A design criteria to assess liquefaction potential for a sandy ground in coastal areas was originally proposed based on field experiences and laboratory tests (Tsuchida, 1970). Liquefaction potential, in the current practice, is assessed by the following procedure which is a revision of the original one (Iai et al, 1989).

1) Classify the soils by comparing the grain size accumulation curves with the ranges shown in Fig.10;

Soils within the range (A) : very easily liquefiable soil.
Soils within the range (Bf) or (Bc) : easily liquefiable soil but less liquefiable than the soils within the range (A).

Consider that no liquefaction will take place for the soils which do not belong to either the ranges (A), (Bf) or (Bc).

2) Obtain the equivalent N-values from SPT N-values with corrections for effective confining pressures to 0.66 kgf/cm2 and with corrections for fines content. Conduct the earthquake response analysis of ground by equivalent linear method and obtain the equivalent acceleration from the maximum shear stress ratio (the ratio of maximum shear stress over vertical effective stress) by multiplying the factor of 0.7g as a conversion factor.

3) Use the design charts shown in Fig. 11 in accordance with the ranges for grain size accumulation curves. Assess the liquefaction potential in accordance with the regions to which the equivalent N-value and the equivalent acceleration belongs;

Region I : Consider that liquefaction will take place.
Region II : Conduct a further study such as laboratory test of soils if desirable. Otherwise, consider that liquefaction will take place.
Region III : Conduct a further study such as laboratory tests of soils if high reliability is required in the stability of the soil deposit. Otherwise, consider that liquefaction will not take place.
Region IV : Consider that liquefaction will not take place.

Present criterion has evolved from the studies since 1964 Niigata Earthquake (Tsuchida and Hayashi, 1971). Though the evolution process is quite independent from the studies by Tokimatsu and Yoshimi (1983) and Seed et al.(1985), all of these criteria show remarkably good agreement as shown in Fig. 12 (Iai et al, 1989).

In assessing liquefaction potential by the present procedure, maximum accelerations should be specified for the input motions to be given at the base rock for earthquake response analysis. Based on the recent study on seismic zoning (Kitazawa et al, 1984), maximum accelerations as an rock outcrop motions (i.e. double amplitudes of incident waves) are specified for five regions as 300, 250, 200, 150 and 100 Gals.

It is important to assure that the level of safety against liquefaction should be well balanced with the level of safety against seismic inertia force. As partly suggested in the previous section, Eq.(2) shows such balance are indeed assured in the present design procedure.

Based on the assessment of liquefaction potential, adequate measures will, if necessary, be taken against liquefaction. Relevant studies can be referred to on designing the area of ground compaction (Iai et al, 1987) and on designing spacing of gravel drains (Iai et al, 1988).

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The authors express sincere appreciation to Dr. Takamasa Inatomi, Chief of Structural Dynamics Laboratory; Dr. Tatsuo Uwabe, Chief of Earthquake Disaster Prevention Laboratory, Dr. Susumu Iai, Chief of Geotechnical Earthquake Engineering Laboratory, and Mr. Yasuo Matsunaga, Member of Geotechnical Earthquake Engineering Laboratory of Port and Harbour Research Institute for their considerable support to prepare this report.

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Table 1 Recent earthquakes which caused damage to port facilities

Date	Name	Mag.	Highest recorded peak acc. in Gal (Location)
1964/06/16	Niigata earthquake	7.5	159 (Niigata)
1968/05/16	1968 Tokachi-oki	7.8	233 (Hachinohe)
1973/06/17	1973 Nemuro-Hannto-oki	7.4	164 (Kushiro)
1978/06/12	1978 Miyagi-ken-oki	7.4	280 (Shiogama)
1982/03/21	1982 Urakawa-oki	7.1	253 (Hiroo)
1983/05/25	1983 Nihonkai-Chubu	7.7	205 (Akita)
1984/08/07	1984 Hyuuganada	7.1	268 (Hososhima)
1987/12/17	1987 Chiba-ken-Toho-oki	6.7	171 (Chiba)

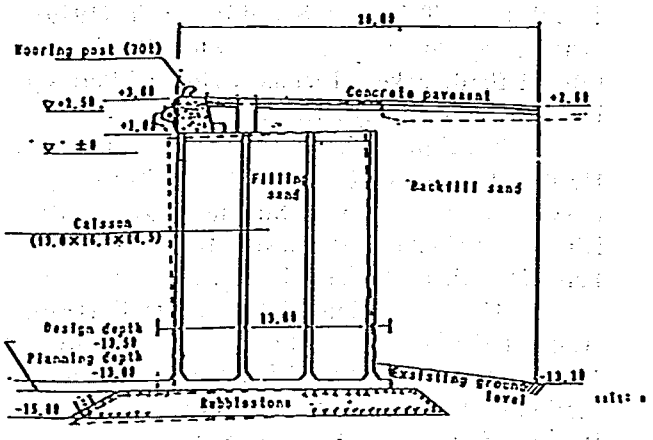


Fig. 1 Cross section of gravity type quaywall (Gaiko Wharf) in Akita Port

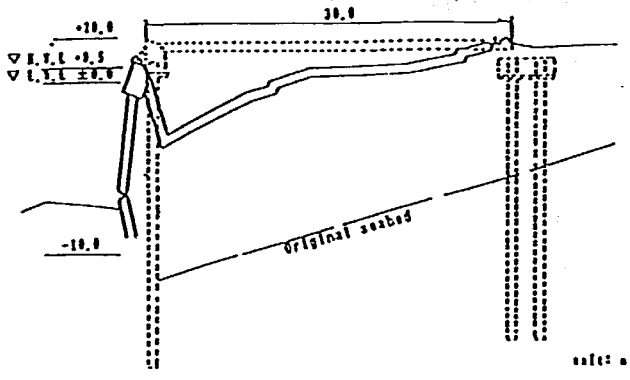


Fig. 2 Cross section of damage to steel sheet pile bulkhead (Ohama No. 2) in Akita Port

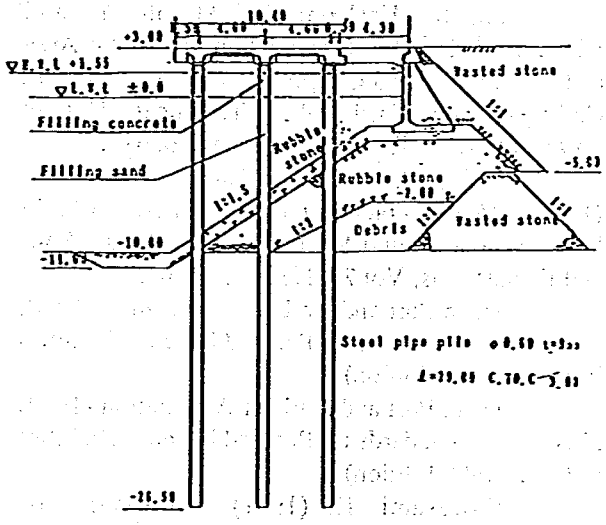


Fig. 3 Trestle type pier (West No. 2 Pier, East Side) in Muroran Port

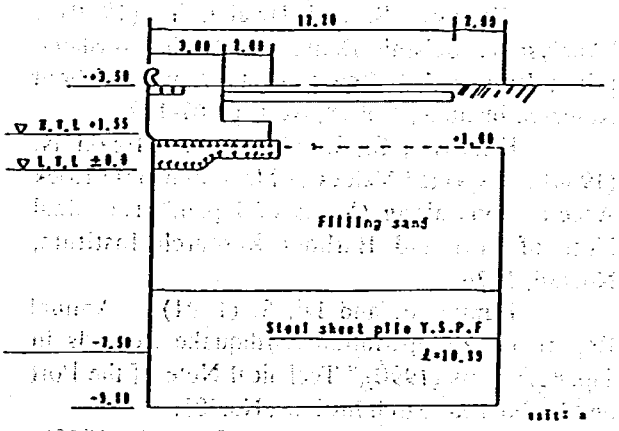


Fig. 4 Cellular bulkhead (Nittsu Wharf) in Muroran Port

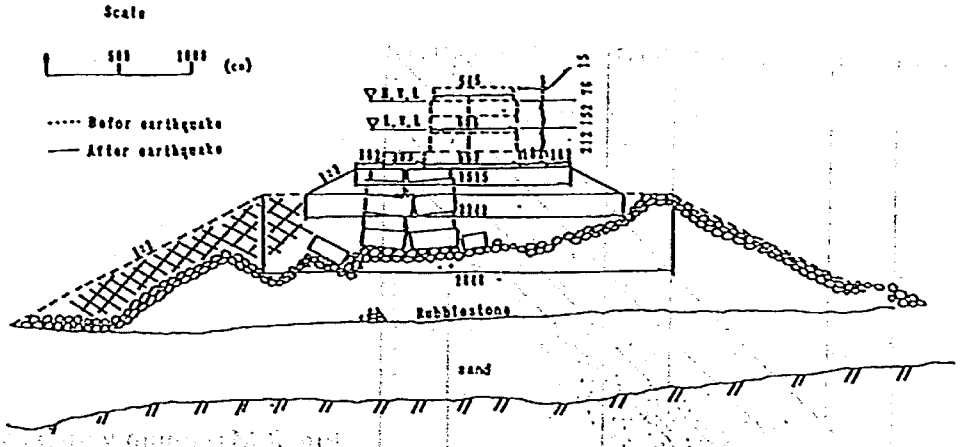


Fig. 5 Breakwater in Yokosuka damaged by Kanto earthquake (1923)

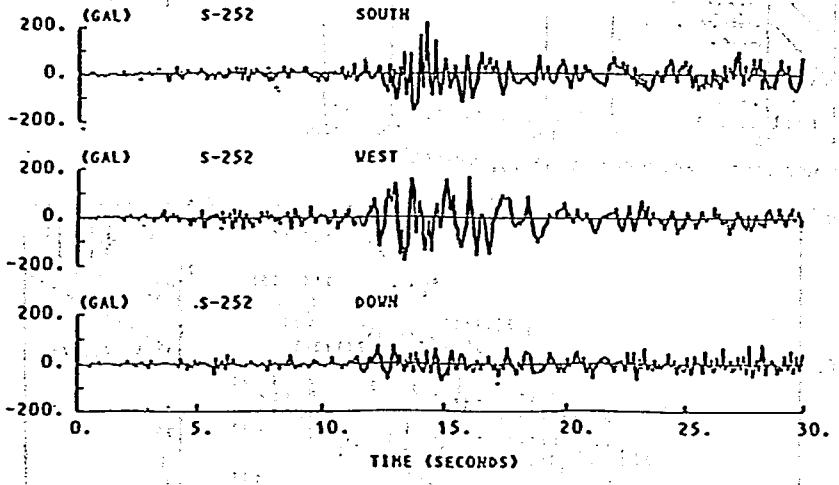


Fig. 6 Accelerations of 1968 Tokachi-oki earthquake at Hachinohe Port

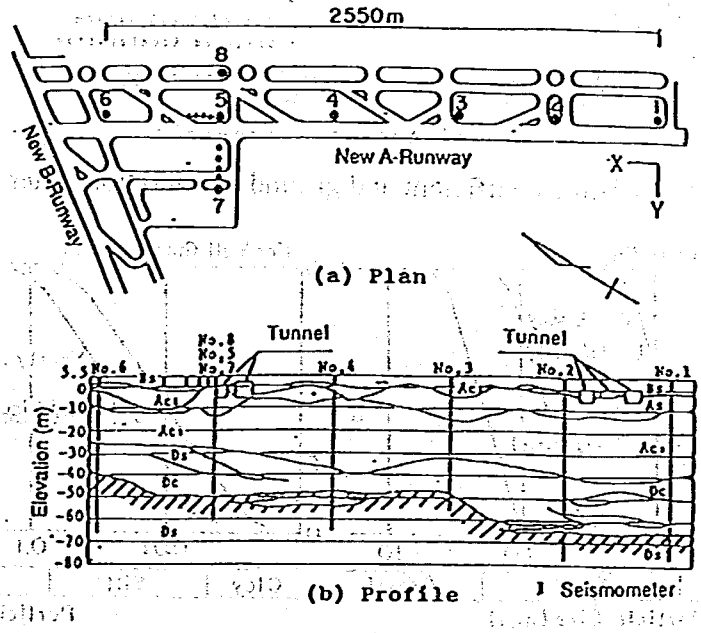


Fig. 7 Deployment of seismometers of array observation system of PHRI(Haneda Airport)

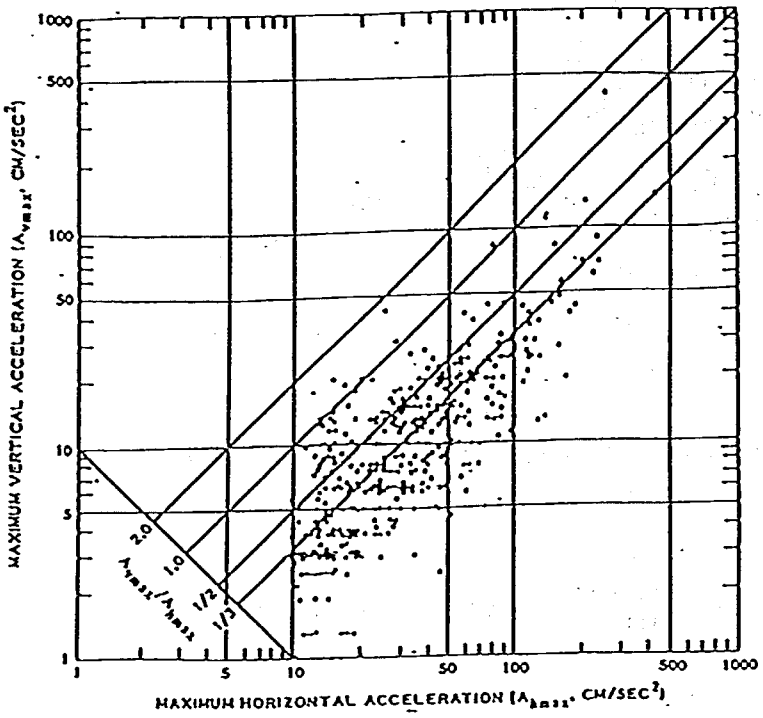


Fig. 8 Maximum vertical vs. horizontal acceleration in Japan (after Uwabe et al, 1976)

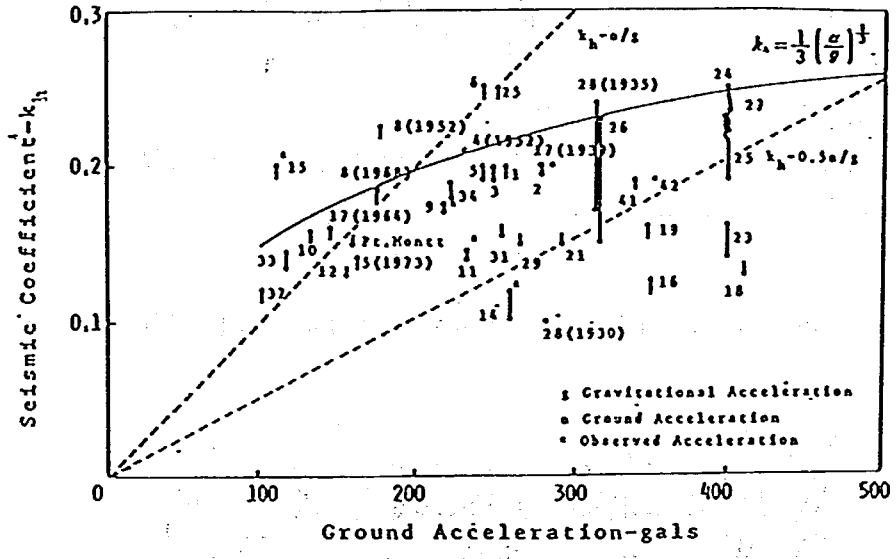


Fig. 9 Relation between seismic coefficient and ground acceleration (after Noda et al, 1975)

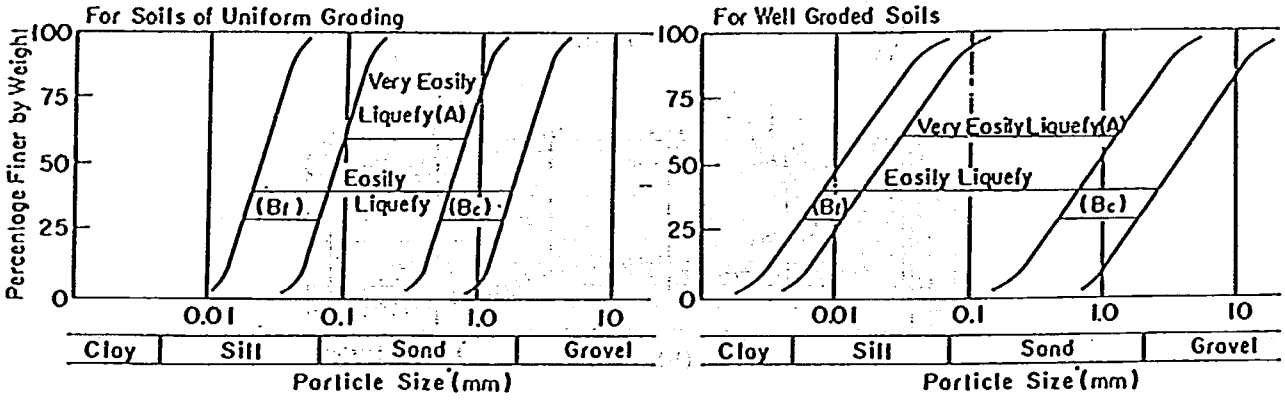
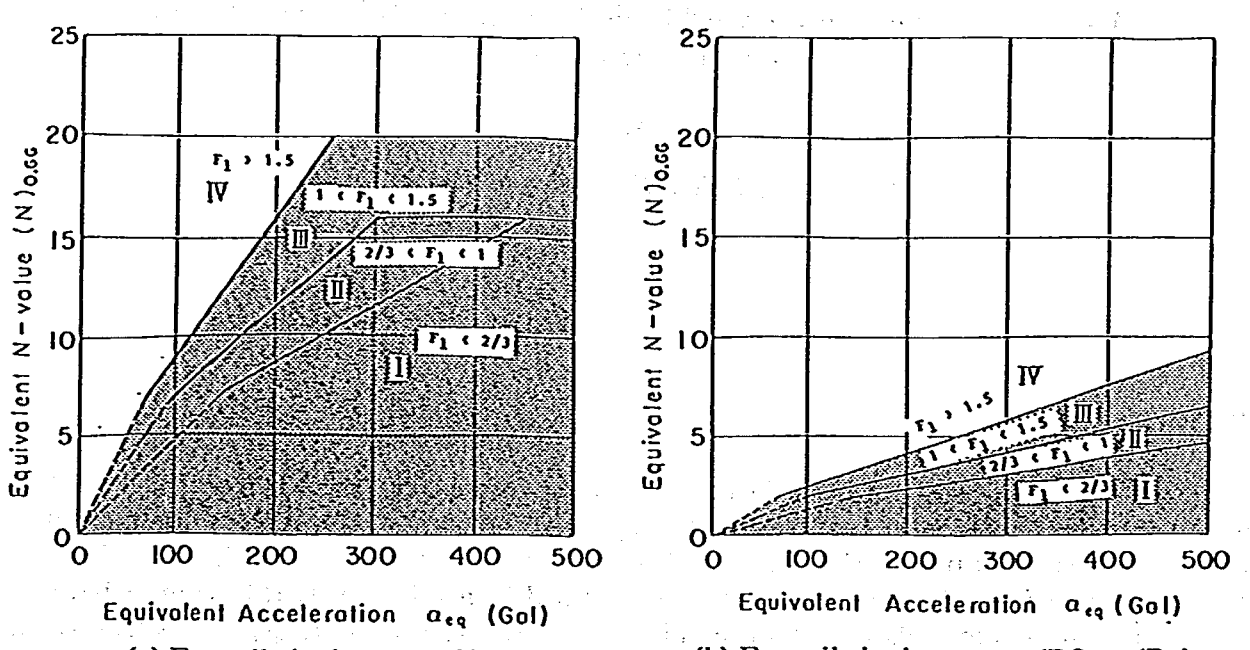


Fig. 10 Ranges of grain size accumulation curves for liquefiable soils (after Iai et al, 1989)



(a) For soils in the range (A)

(b) For soils in the ranges (Bf) or (Bc)

Fig. 11 Design chart for assessing liquefaction potential (after Iai et al., 1989)

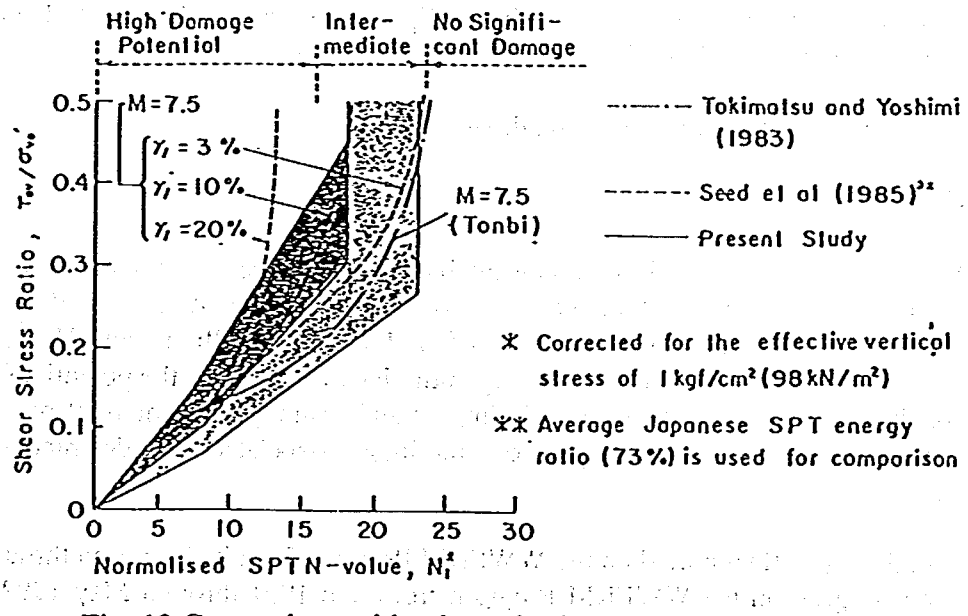


Fig. 12 Comparison with other criteria (after Iai et al, 1989)

A Progress Report on the Worldwide Earthquake Risk Management (WWERM) Program

by

S. T. Algermissen[1], Walter W. Hays[1], and Paul R. Krumpe[2]

ABSTRACT

Considerable progress has been made in the Worldwide Earthquake Risk Management (WWERM) Program since its initiation in late 1989 as a cooperative program of the Agency for International Development (AID), Office of U.S. Foreign Disaster Assistance (OFDA), and the U.S. Geological Survey. Probabilistic peak acceleration and peak Modified Mercalli intensity (MMI) maps have been prepared for Chile and for Sulawesi province in Indonesia. Earthquake risk (loss) studies for dwellings in Gorontalo, North Sulawesi, have been completed and risk studies for dwellings in selected areas of central Chile are underway. A special study of the effect of site response on earthquake ground motion estimation in central Chile has also been completed and indicates that site response may modify the ground shaking by as much as plus or minus two units of MMI. A program for the development of national probabilistic ground motion maps for the Philippines is now underway and pilot studies of earthquake ground motion and risk are being planned for Morocco.

KEYWORDS; Earthquake; hazard; international; risk

1. INTRODUCTION

The WWERM Program was developed as an outgrowth of an Executive Briefing on "Strategic Planning to Reduce Economic Impacts of Earthquake Hazards throughout the World" held at the National Academy of Sciences, Washington, D.C., on March 8 and 9, 1988 (Williams and Hays, 1988). This Executive Briefing established the need for a more quantitative evaluation of the potential for earthquake disasters and for the assessment of the economic impact and casualty loss potential of such disasters. In addition, the need to assess the long-range economic losses associated with destructive earthquakes was also outlined.

Algermissen and others (1991) reported on the WWERM Program from its inception through mid-1991. Here we report on progress in the WWERM Program from mid-1991 through May 1992.

2. OBJECTIVES OF THE PROGRAM

The initial objectives of the program were to develop an action plan and conduct a pilot program in each of three geographic regions to demonstrate the utility, importance, feasibility, and need for uniform

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[1]U.S. Geological Survey

[2]Department of Energy

global earthquake hazard mapping and risk assessment/management as a contribution to the International Decade for Natural Disaster Reduction (IDNDR).

The initial three geographic areas for pilot studies were Morocco, Perú/Chile, and Indonesia. Since the initiation of the project in 1990, the scope of the work has been greatly expanded through the funding of additional program in Indonesia and the Philippines by the AID Mission in each country. The supplemental funding from the AID Office in Indonesia has permitted the following additional work: (1) a comprehensive assessment of earthquake ground motion in North Sulawesi Province; and (2) the calculation of estimates of expected future losses to masonry dwellings in Gorontalo, North Sulawesi. Funding from the AID Mission in the Philippines has allowed us to undertake a comprehensive program of earthquake ground-motion assessment for the entire country together with pilot studies of future expected earthquake losses. As in Indonesia, OFDA has provided supplemental funding that has made it possible to purchase and transfer a MacIntosh computer and associated equipment to the Philippine Institute of Volcanology and Seismology (PHIVOLCS) for training and technology transfer for seismic hazard mapping and risk assessment.

As work progresses, we are continuing to expand the technical horizons of WWERM with the hope that the project will eventually become global in scope.

3. PROGRESS

3.1 Chile

Because of funding and technical results available from an earlier OFDA-USGS Project "Seismic Zoning and Implementation of Improved Earthquake Resistant Design in Chile" which in many ways complemented the WWERM Program, it has been possible to develop a considerably broader program in Chile with results and products having widespread application. These products include: (1) probabilistic acceleration and seismic intensity maps of the entire country for an exposure time (period of time of interest) of 50 years and 10 percent chance that the mapped values will be exceeded; (2) comprehensive evaluation of the "site effect" in central Chile, that is, the contribution of the surface geology to the modification of ground shaking during earthquakes; and (3) collection of data for loss assessment.

A probabilistic ground acceleration map for a time period of interest of 50 years is shown in Figure 1. A similar map of seismic intensity for central Chile is shown in Figure 2. These maps are prepared for "average" site conditions in Chile. The primary use of the acceleration maps is to provide the ground motion basis for the earthquake resistant design of structures either for a specific building or as part of the seismic provisions in a building code. The primary use of the intensity maps is to present the expected levels of damage (in descriptive form) for disaster preparedness, mitigation, and land use planning. Figure 3 shows the magnitude of the variation in ground shaking (in units of intensity) associated solely with variations in surface geology. Figure 4 shows how these variations in shaking can be incorporated into probabilistic seismic intensity maps to provide a more realistic estimate of the maximum ground shaking levels that may occur in an area in a specific time period of interest. The level of refinement of estimated ground motion shown in Figure 4 is generally not available, even for most areas of the United States, and represents a state-of-the-art product. The importance of surface geology in controlling ground shaking during earthquakes was first appreciated in Chile after the M_s=7.9, central Chile earthquake of 1985. This is, however, the first study to describe this effect in a

quantitative manner. We anticipate that the intensity maps in this study will be of considerable use in disaster preparedness, mitigation, and land use planning in central Chile.

During October 1991, USGS personnel participated in an international seminar and a separate workshop, both related to the WWERM Project. The first meeting was organized by UNDRO, MAPFRE (a Spanish reinsurance company, ONEMI (the national emergency office in Chile), the Universidad de Chile, the Caja Reaseguradora de Chile (a Chilean insurance company), and the USGS. This "Seminario Internacional" held in Santiago from October 21-24, 1991, was entitled "Disastres Sismicos Urbanos, Codigo de Construccion Sismoresistente y Seguros" (Urban Seismic Disaster, Earthquake Resistant Construction Codes and Insurance).

A workshop entitled "Applicacion de Mapas Probabilisticos de Movimientos Fuertes para la Reduccion del Riesgo Seismico en Chile" (Application of Probabilistic Maps of Strong Motion to the Reduction of Seismic Risk in Chile) was organized by the USGS and the Universidad de Chile during the week following the international seminar. This highly technical workshop was designed to make the results of the two OFDA-USGS projects in Chile available to scientists, engineers, and disaster management specialists in Chile. In particular, scientists and engineers from the major Chilean universities and from major consulting firms involved in the development and revision of the earthquake resistant design provisions of the Chilean National Building Code, together with disaster preparedness and mitigation specialists from ONEMI (the national emergency office in Chile), were invited to attend and participate in the workshop. The papers presented at the USGS-Universidad de Chile workshop are currently being edited. These papers will be published in a special issue of "Revista Geofisica", a professional journal published by the Pan American Institute of Geography and History with wide circulation in Latin America.

3.2 Indonesia

The earthquake hazard and risk study of North Sulawesi, Indonesia, was completed in August 1991, in advance of a workshop held in Bandung in September 1991. Products of the Pilot Study include: (1) synthesis of the geologic structures related to earthquake occurrence in the area; (2) expected peak horizontal ground acceleration on both hard rock and soft soil that have a 90 percent probability of not being exceeded in 50 years; (3) development of inventory and earthquake vulnerability relationships for dwellings; and (4) estimation of both future average annual loss and catastrophe potential for the dwelling stock in Gorontalo, North Sulawesi. Papers on all aspects of the Pilot Study were presented at the workshop in Bandung. A technical paper "Probabilistic Seismic Hazard Estimates in North Sulawesi Province, Indonesia" describing the results of the Pilot Study will be presented at the SECOND-ASIA CONFERENCE on Engineering for Mitigating Natural Hazards Damage that is scheduled for Yogyakarta, June 22-26, 1992, (Thenhaus and others, 1992).

3.3 The Philippines

The WWERM Program will begin in the Philippines in June 1992. The intention is to: (1) prepare national probabilistic hazard maps for all of the country; (2) to undertake pilot studies of future earthquake losses in selected areas of the country; and (3) to conduct training and technology transfer in earthquake hazard and risk assessment in conjunction with the Philippine Institute of Volcanology and Seismology (PHIVOLCS).

3.4 Morocco

The Morocco phase of WWERM has been much delayed, first because of the Persian Gulf War, and second, because of the accelerated program of hazard and risk assessment that was undertaken in Indonesia. The work in Morocco will include a demonstration project of earthquake hazard (ground motion) and risk (loss to dwellings) in selected areas. The work will be done cooperatively with the Center for Technical and Scientific Research (CTSR) in Rabat, Morocco. Work in Morocco will begin in August 1992.

4. SUMMARY

Major studies of earthquake ground motion estimation have been completed in Chile and investigations of risk to dwellings in Central Chile are nearing completion. Similar studies have been completed in Sulawesi, Indonesia. Very successful workshops highlighting the technical results have been held in Chile and Indonesia. A major study of earthquake ground motion and pilot studies of seismic risk to dwellings will begin in the Philippines in June 1992 followed by pilot studies of earthquake ground motion and risk in Morocco.

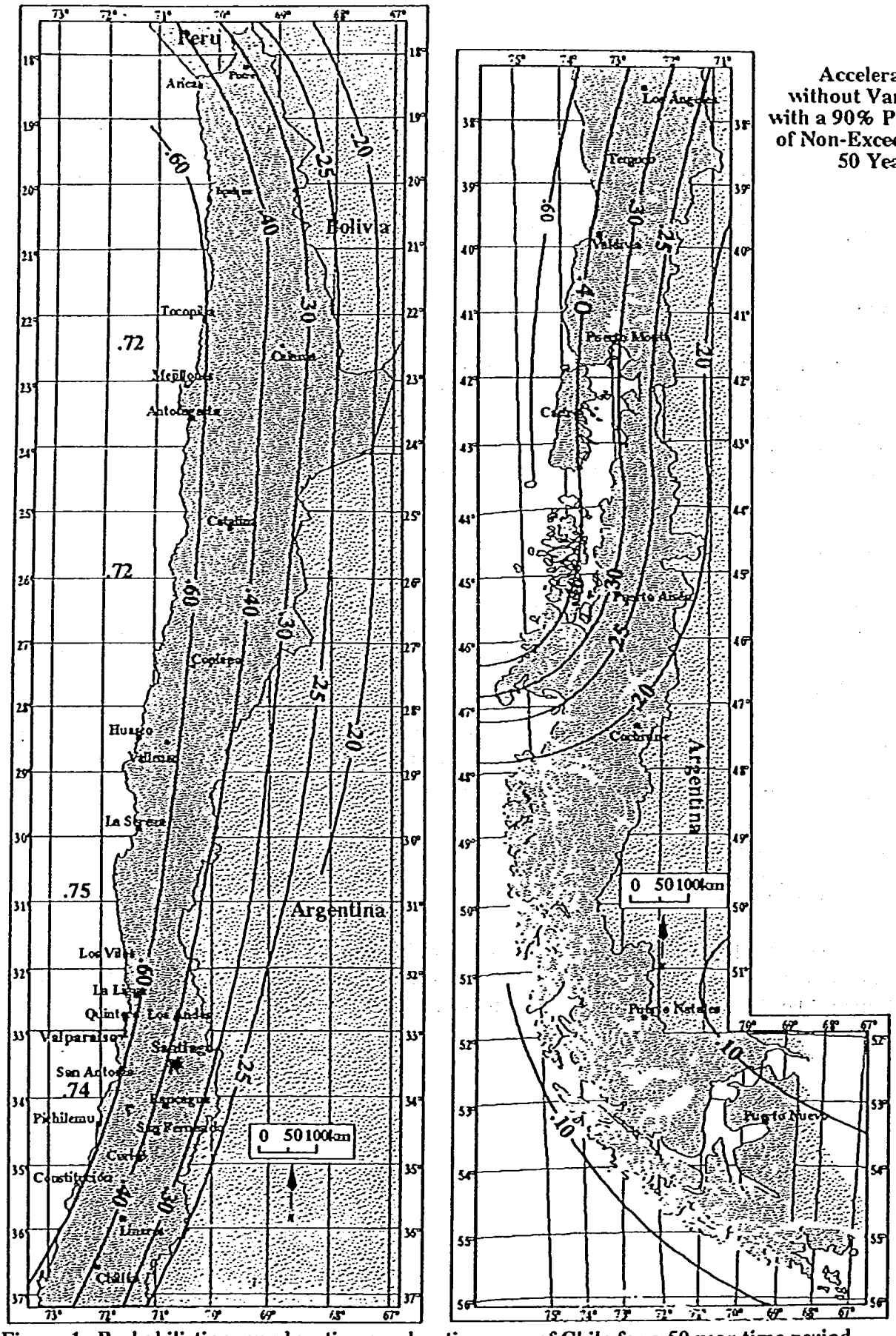
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Acceleration without Variability with a 90% Probability of Non-Exceedance in 50 Years

Figure 1. Probabilistic ground motion acceleration map of Chile for a 50 year time period of interest. The ground motions mapped have only an estimated 10% chance of being exceeded in 50 years. Maps of this type are widely used as design maps in the earthquake resistant design provisions of building codes.

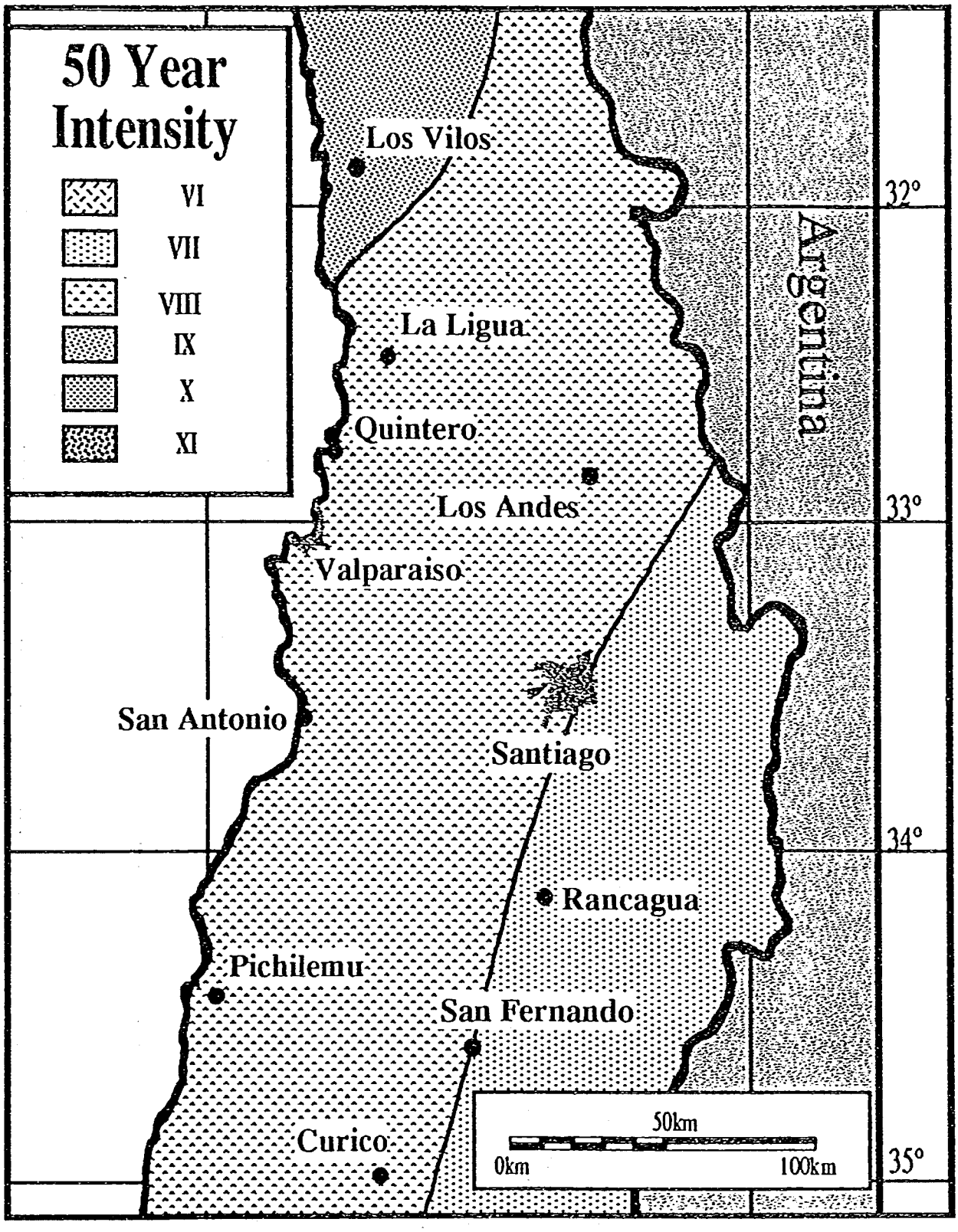


Figure 2. Probabilistic ground intensity map of Chile for a 50 year time period of interest. The ground motions mapped have only an estimated 10% chance of being exceeded in 50 years. Maps of this type are often combined with site specific shaking maps (such as Figure 3) to produce probabilistic ground shaking maps that include the site specific response associated with the near surface materials beneath a site.

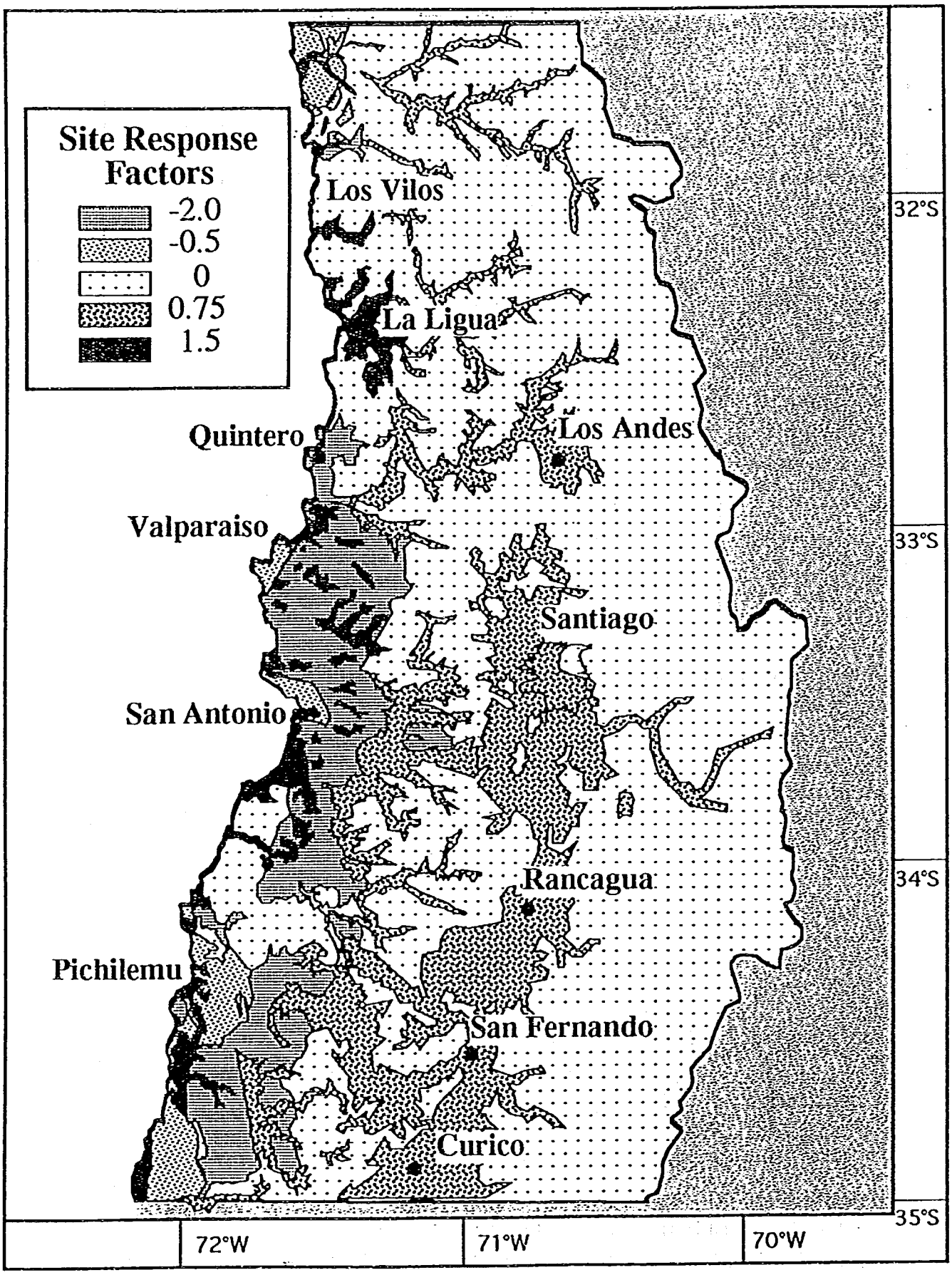


Figure 3. Site specific response factors associated with the near surface materials in central Chile. This map is combined with the regional ground motions shown in Figure 2 to produce the map in Figure 4.

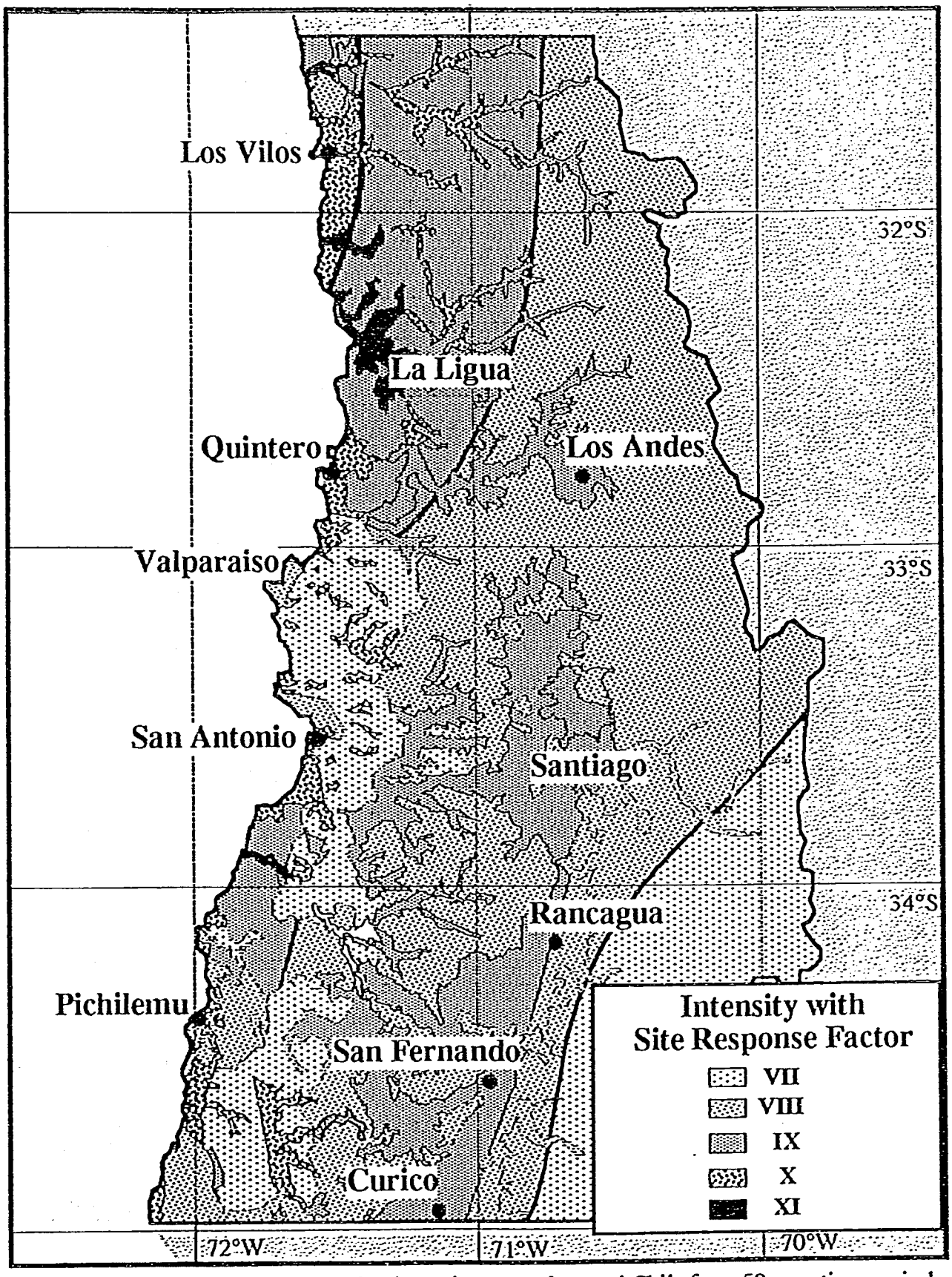


Figure 4. Probabilistic ground motion intensity map of central Chile for a 50 year time period of interest. This map combines the mapped values of Figure 2 with those of Figure 3 to produce a probabilistic ground motion map that includes the effect of near surface materials. These maps are valuable in disaster preparedness, mitigation, land use planning and earthquake risk (loss) studies because they represent an estimate of the maximum damage (and consequently loss) that is expected to occur in the period of time considered in the mapping.

Implementation of Executive Order 12699, "Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction "

by

Richard N. Wright*

ABSTRACT

The Interagency Committee on Seismic Safety in Construction (ICSSC) assists federal agencies involved in construction to develop and incorporate earthquake hazards reduction measures in their ongoing programs. ICSSC proposed an executive order for seismic safety in construction that became the basis for Executive Order 12699, "Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction," dated January 5, 1990. ICSSC is working with federal agencies, state and local governments and private sector organizations to implement the Executive Order.

KEYWORDS: building codes; building standards; earthquake hazard reduction; existing buildings; retrofitting buildings; seismic safety

1. THE EARTHQUAKE PROBLEM

Losses of property approaching $10 billion, losses of 62 lives and severe disruption of human activities from the October 17, 1989, Loma Prieta, California, earthquake reminded the United States of the severe threats posed by great earthquakes. In perspective, losses were modest because the earthquake was far from the San Francisco Bay area where the major damages occurred. Unless enhanced efforts are made to reduce earthquake hazards, a large earthquake close to a major metropolitan area can kill tens of thousands of people, cause tens of billions of dollars of direct property losses, and, through consequent losses, severely disrupt economic activity for the whole nation.

It is important to note that earthquake hazards are not only a California problem. Forty-six of our fifty states are vulnerable to strong earthquakes, Figure 1. In the eastern half of the country, the area that will be affected by an earthquake could be 10 times as large as for a similar earthquake in California. Records from historic earthquakes have illustrated this pattern, Figure 2, which is believed to be due to differences in rock and overlaying soil. One or more earthquakes, equal to or larger than that that struck California in 1989, are expected to affect U.S. metropolitan areas in the next ten years.

However, while earthquakes are an inevitable hazard, they are not inevitable disasters. Properly designed and constructed buildings and lifelines (public works and utilities structures) can resist economically the effects of strong earthquake ground shaking.

2. POLICY REQUIREMENTS

The National Earthquake Hazard Reduction Program (NEHRP) was established by the President in 1978 in accord with the Earthquake Hazards Reduction Act of 1977 (P.L. 95-124) to reduce loss of life, injuries, and economic costs resulting from the occurrence of earthquakes in the United

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States. The Interagency Committee on Seismic Safety in Construction was established in 1978 to assist the federal agencies involved in constructions to develop and incorporate earthquake hazard reduction measures in their programs. Twenty-nine agencies participate in ICSSC.

On January 5, 1990, the President issued Executive Order 12699, "Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction." The Executive Order is based on a proposal by the Interagency Committee on Seismic Safety in Construction.

The Executive Order responds to the policy requirements created by the Earthquake Hazards Reduction Act of 1977. The Act requires "the development and promulgation of specifications, building standards, design criteria, and construction practices to achieve appropriate seismic resistance." It calls for "the examination of alternative provisions and requirements for reducing earthquake hazards through federal and federally financed construction, loans, loan guarantees and licenses." The Act requires attention not only to federal facilities, but also to federally assisted or regulated construction. Each agency is responsible for its own actions. Neither the Federal Emergency Management Agency (FEMA) nor ICSSC are given authorities over the agencies' programs.

3. PROVISIONS OF THE EXECUTIVE ORDER

The requirements for the earthquake safety of federal buildings have these purposes: to reduce risks to the lives of occupants; to reduce secondary risks of failures; to improve the capability for important federal buildings to be functional during or after the emergency; and to reduce losses of public buildings. Any new federal buildings entering the detailed design stage after January 5, 1990, shall be designed and constructed in accord with appropriate seismic standards.

For federally leased, assisted or regulated buildings, the purpose of the Executive Order is to reduce risk to lives of occupants of buildings leased for federal uses or purchased or constructed with federal assistance; to reduce risk to the lives of persons potentially affected by earthquake failures of federal assisted or regulated buildings, and to protect public investments. For space constructed and leased for federal occupancy, the Executive Order applies to agreements executed since January 5, 1990.

Federal domestic assistance programs affect most new residential construction because states, municipalities and developers want to be eligible for VA or FHA mortgages. The Executive Order requires agencies, within 3 years, to plan and initiate measures to assure appropriate consideration of seismic safety in domestic assistance programs.

The concurrent requirements are extremely important to the private sector and to the federal agencies. Agencies and ICSSC are not expected to develop federal standards for new building construction. Administration policy in OMB Circular A-119 for the use of nationally recognized standards to the extent possible in federal programs is reflected in the Executive Order. Agencies would use nationally recognized private sector standards unless none are available that meet special agency requirements. The local building code may be used if the agency determines it provides adequate seismic safety. ICSSC is working collectively to provide guidance to the agencies so that each agency doesn't itself have to carry out an assessment of the more than 15,000 local building codes in the country.

Each agency is responsible for issuing its regulations or procedures, planning for implementation through its own budget process and regularly reviewing its regulations and procedures. FEMA is to request from

each agency annually the status of its work and to report on execution of the order annually to Congress.

4. IMPLEMENTATION OF THE EXECUTIVE ORDER

ICSSC is now working on implementation of the Executive Order. Guidelines [1] have been developed for implementation of the Executive Order. ICSSC also is working with the private sector in implementation of the Executive Order. The National Earthquake Hazards Reduction Program has developed, in cooperation with the Building Seismic Safety Council and the Applied Technology Council, "Recommended Provisions for Development of Seismic Regulations for New Buildings." These are being used in the national standards and model codes. The American Society of Civil Engineers is considering the NEHRP Recommended Provisions in revision of its widely used national Standard A7, "Minimum Design Loads for Buildings and Other Structures."

A study with the Council of American Building Officials (CABO) [2] compared model code provisions to NEHRP Recommended Provisions. Based on its review of the CABO assessment, the ICSSC recommended as substantially equivalent to the NEHRP Recommended Provisions:

- 1991 ICBO Uniform Building Code
- 1992 Supplement to the BOCA National Building Code
- 1992 Amendments to the SBCC Standard Building Code

State or local ordinances adopting and enforcing these model codes are deemed adequate for implementing the Executive Order.

These nationally recognized private sector standards will become important resources for new federal buildings. ICSSC also is working with the private sector in preparing guidance documents for the federal agencies to use to assess the adequacy of the local building codes in dealing with the seismic safety of federally assisted or regulated facilities. A study with the National Conference of States on Building Codes and Standards [3] assessed the status in the United States of the adoption and enforcement of adequate seismic provisions.

Considering the strong efforts of the private sector and state and local government to implement seismic design and construction standards for new buildings, and the federal agencies' implementation of the Executive Order, most U.S. new building construction should be appropriately seismically resistant by the mid-1990s.

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Seismic Hazard of the United States

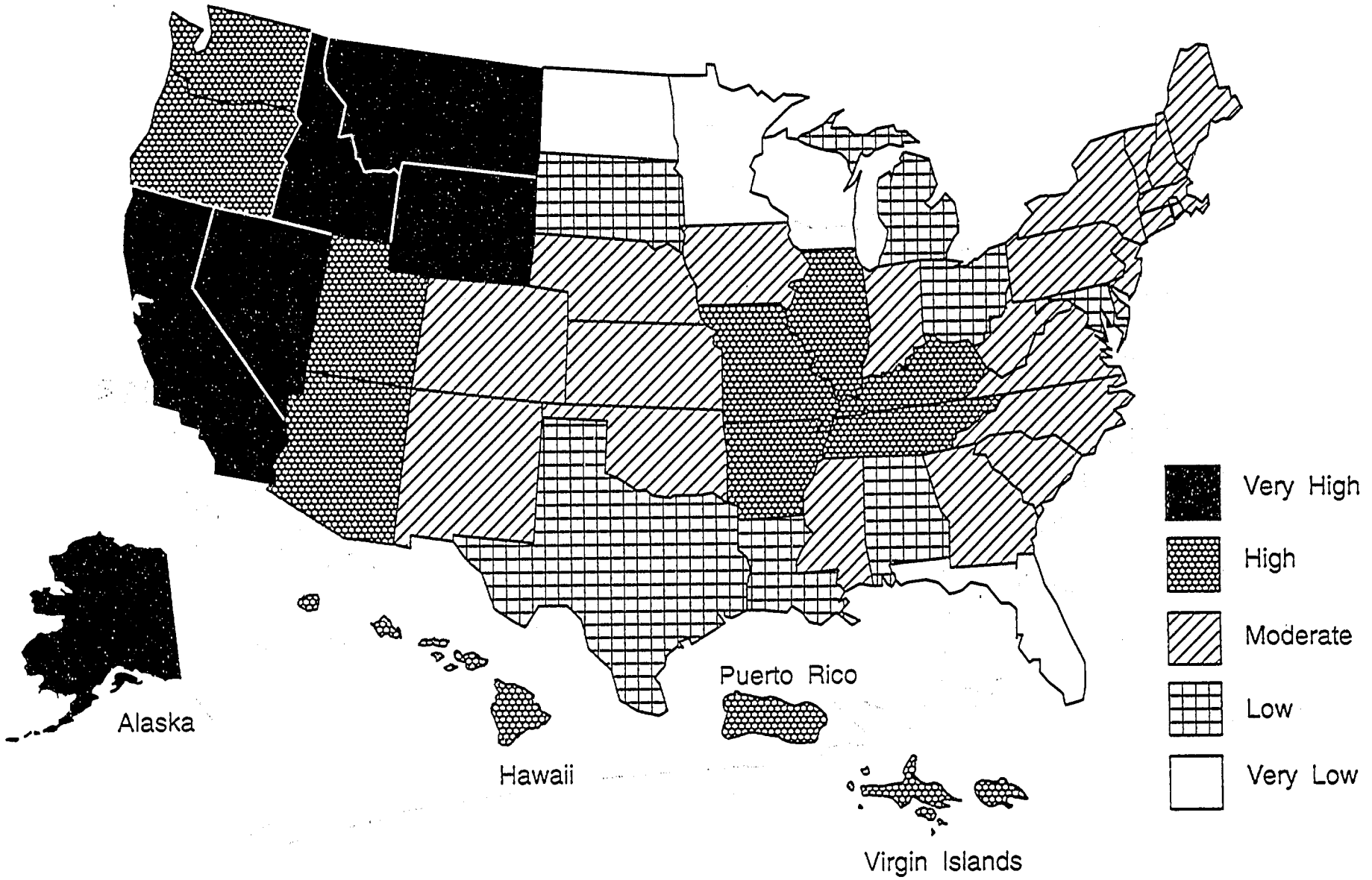


Figure 1

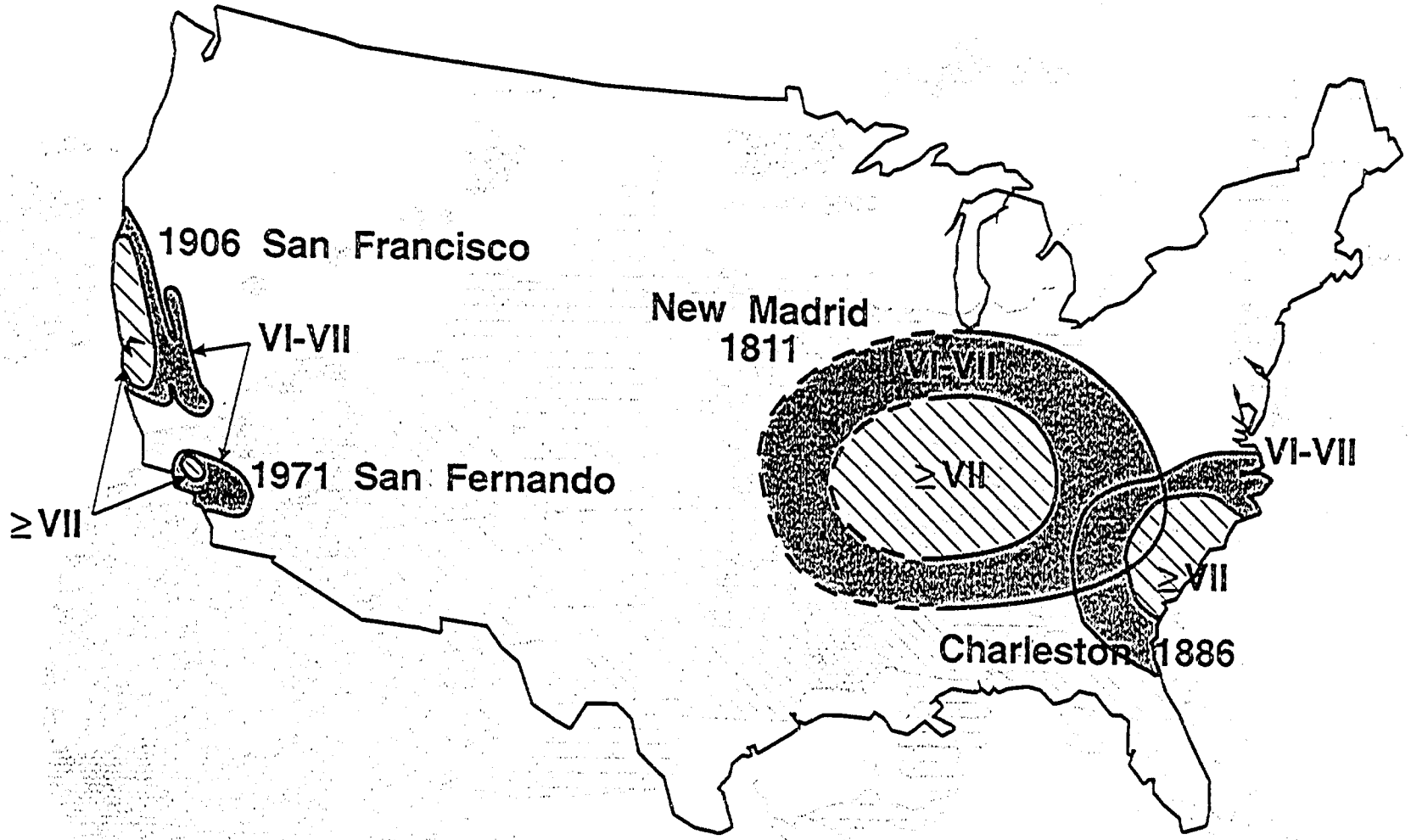

1906 San Francisco
VI-VII
≥ VII
1971 San Fernando
New Madrid
1811
VI-VII
≥ VII
VI-VII
≥ VII
Charleston 1886

Figure 2

U.S. Army Corps of Engineers Earthquake Engineering Research Program

by

W. D. Roper*, M. E. Hynes**, and R. F. Davidson*

ABSTRACT

To ensure the rapid recovery of Corps infrastructure and provide effective, rapid response to earthquake-induced flooding, landslides, and other life threatening emergencies, the Corps has recently organized a focused research and development program, called the Quake Program, to address the primary sources of seismic damage, namely ground shaking and ground failure, and their impact on Corps facilities. Quake Program efforts are prioritized according to the needs of our field engineers who are responsible for the design, construction, operation and maintenance of the Corps' flood control and navigation facilities. The objectives of the program are: (1) to develop practical, more effective seismic design procedures where they are now missing, (2) to reduce the uncertainty in our seismic safety assessments and avoid costly defensive design, (3) to reduce the risk of catastrophic failure of Corps infrastructure, primarily dams, and (4) to save money on new construction and remediation efforts. The Quake studies incorporate the research results of universities and other government agencies in this country and abroad to develop seismic design tools tailored to Corps needs. This paper describes some of the problems we have encountered in our projects which motivated the establishment of the Quake program at this time, the scope of the studies planned for the next few years, and the coordination of these activities with others involved in earthquake-related research.

KEYWORDS: buildings; dams; earthquakes; embankments; foundations; ground motions; liquefaction; locks; outlet works; seismology; walls; lifelines.

1. INTRODUCTION

As part of our Civil Works mission, the Corps of Engineers designs, constructs, operates and maintains approximately 400 flood control dams and reservoirs, and about 150 navigation locks and reservoirs. We have built thousands of miles of levees, and do most of the dredging needed to keep our nation's ports open. In addition, the Corps has been assigned certain emergency responsibilities under the U. S. Federal Response Plan that would be activated after a major earthquake. It is in connection with our Civil Works mission that the Corps is currently organizing an earthquake engineering research program, called the Quake Program.

This paper describes a number of problems we have encountered in our large construction and seismic safety projects which motivated the development of this new research program, the organization and scope of the studies planned for the next few years to address the perceived gaps in our knowledge, and the coordination of these studies with the activities of universities and other government agencies in this country and abroad.

2. THREATENED FACILITIES AND RESEARCH NEEDS

A sudden great earthquake poses the danger of catastrophic failures in major works of the Corps of Engineers including dams, levees, river control structures, navigation channels, harbors, airfields, and hazardous and toxic waste sites, as well as essential military facilities. At least 39 U.S. states are subject to strong earthquakes and more than 20 million people are at risk. In the central United States, where we have little modern experience with earthquakes, at least 20 locks and dams, 51 pumping plants, 19 reservoirs, and 23,000 miles of levees could be impacted by a major seismic event in the New Madrid area.

In spite of major advances over the last 25 years, serious gaps in our knowledge base exist in the areas of earthquake hazard predictions; site characterization for seismically sensitive parameters; constitutive behavior, failure mechanisms and material properties of rock, soils, concrete and composite (reinforced or remediated) materials under seismic loads, and the stress and deformation response of sites and facilities to seismic loading. Cost-efficient remediation and defensive design techniques are needed in addition to careful calibration of fast advancing numerical methods to actual field performance. Part of such calibration includes rapid deployment to sites of earthquake damage to collect and monitor transient data. In addition, some of these same monitoring techniques can be developed for use in immediate urban search and rescue efforts in damaged areas.

It is our experience that every time the Corps is faced with a serious earthquake engineering safety study it becomes a research project because the state of the art in earthquake engineering falls far short of our needs. Thus, earthquake safety studies tend to take a long time since we must perform the research necessary to develop design solutions. These research studies require considerable experimentation in the field, in the laboratory, and on the computer. Each step in this solution process involves a great deal of uncertainty. Once a logical line of reasoning to solve the problem has been developed the results may be tentative because new knowledge provided when earthquakes occur or with a major state-of-the-art advance may necessitate revisions.

This uncertainty combined with disastrous consequences in the event of a failure of one of our critical structures requires diligent

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attention to continuously improve our seismic design capabilities and to confirm that our existing designs conform to the highest standards of safety. At this time, we design our structures defensively, to account for the shortcomings in the state of the art to analyze and design efficiently for earthquake problems. Our research is aimed at improving our ability to design effectively, to reduce uncertainty in our end results and reduce the cost of our construction.

3. PROBLEMS ENCOUNTERED IN OUR PROJECTS

The motivating force in all Corps Civil Works research is the need expressed by our field engineering offices for better design procedures and tools. Engineering problems have been encountered in a number of large Corps projects that have highlighted the need for research in earthquake engineering. These problems involve our inability to accurately predict earthquake ground motion levels, our incomplete understanding of materials in situ, material behavior and mechanisms of failure during and after earthquake loading, the need for validation and simplification of our analytical approaches, and the need for more efficient, economical methods of remediation. Three projects are described next to illustrate the knowledge gaps we have been faced with in our efforts to achieve efficient, well engineered designs for seismic loading of these key structures. Two projects are existing dams which have been found to need remediation to withstand their design earthquakes. The other project is a new navigation structure in the design stage.

3.1 Folsom Dam and Reservoir Project

The Folsom Dam and Reservoir Project is located on the American River about 20 miles northeast of the city of Sacramento, California (see location map in Figure 1). The reservoir is retained by about 4.5 miles of earth and rockfill embankment dams and a 1400-ft long concrete gravity section which includes the spillway. At maximum pool, the reservoir retains one million acre-ft of water. The design earthquake for this project is a magnitude 6.5 event occurring 15 km away. Catastrophic failure of this dam is estimated to result in 6,000 deaths, put downtown Sacramento under six ft of water, and cause six billion dollars' worth of property damages. Clearly this is a critical project.

The embankment shells are rolled-fill constructed from gravelly dredge tailings. A portion of one auxiliary embankment dam, Mormon Island Auxiliary Dam, has upstream and downstream shells founded on these loose, gravelly dredge tailings. Figure 2 shows a plan and Figure 3 shows cross sections of this dam. This is the only section of the Folsom Project found to be unsafe. We are now in the process of designing and constructing a remedial measure at this location.

At the time this safety evaluation was started in 1978, there were no established procedures to measure the properties of these gravelly materials, in situ or in the laboratory, and to assess their earthquake behavior. Consequently, we had to perform the research necessary and incorporate the work of others to develop testing and analysis procedures adapted to gravelly soils for which there is little or no earthquake experience -- large-scale in situ density tests, geophysical tests in steel-cased holes, interpretation of geophysical tests in loose gravels and rockfill, laboratory compaction and density tests for gravels, interpretation of Becker Hammer penetration tests and equivalent SPT values, cyclic loading behavior in large-scale laboratory tests and corrections for membrane compliance for extremely loose and moderately dense gravels, and estimation of residual strength for gravels).

For the concrete gravity section, there were also problems, such as realistic ways of determining the adequacy of the complex, three-dimensional connection between the embankment wing dams and the partially embedded concrete gravity dam (see typical section in Figure 4) subjected to seismic loads, procedures for assigning values of reservoir bottom absorption for hydrodynamic response analyses of the concrete dam, and methods of accurately determining the potential for sliding deformation in the dam or in fracture planes in the rock foundation.

Through almost ten years of testing, analysis, and conservative interpretation, we were able to acquire enough information to conclude that only the 800-ft long section of Mormon Island Auxiliary Dam with embankment shells on loose gravels required remedial action. We are now faced with the poor understanding the profession has today concerning the actual behavior of liquefying soils and the design, construction, verification and seismic behavior of remediated (composite) zones. A possible remediation plan under consideration involves a system of drains (shown in Figure 5) to control pore pressures.

We now find that we do not really understand the effective stress states and stress-strain behavior of soils as they are deforming while excess pore pressures increase during seismic loading, and after liquefaction as they develop residual strength. We find we do not fully understand the seismic behavior of improved soils with zones of higher density from dynamic compaction, or with stone columns with higher permeability, or reinforced with piles, and the pore-pressure, stiffness and strength interaction of these relatively loose and dense zones under dynamic loads.

We cannot accurately predict the change in density to be expected with distance from a remedial action. We cannot easily verify that we have indeed achieved the desired results, since the remedial action typically radically increases horizontal stresses which are difficult to measure and for which none of our seismic safety evaluation procedures have been developed and calibrated. For example, SPT blowcounts corrected to one tsf is a widely adopted practice, however, the correction factor C_N was developed for horizontal stress levels typically found in natural deposits, not the high values that result in remediated zones.

This wide range of uncertainty in the design and effectiveness of remedial actions causes us once again to adopt conservative interpretations of our analytical and test results. In this project, our efficiency as engineers is hampered because the design tools were not readily available but had to be developed, and the end results still have considerable uncertainty requiring costly defensive design.

3.2 Sardis Dam and Reservoir Project

Sardis Dam is located in northwestern Mississippi, 70 miles from the southernmost extension of the New Madrid fault zone in the central United States (see location map in Figure 1, project plan in Figure 6, and typical sections in Figure 7). Sardis is a hydraulic fill dam, and study of its seismic safety began right after the near failure of the hydraulic fill Lower San Fernando Dam, Los Angeles, California, in the 1971 San Fernando earthquake. More than once in the process of our seismic safety evaluation we were about to conclude that the dam was safe, only to have new developments in the state of the art of earthquake engineering force us to reconsider.

One of the first issues was the level of shaking to be expected at the site -- early estimates were that it was too small to be of concern. In 1980, Dr. Ellis Krinitzsky of the Waterways Experiment Station at Vicksburg, Mississippi, conducted a survey of 18 individuals and organizations, all of whom were considered to be experts at determining ground motions. All were given the same "hypothetical" sites, one of which was, for practical purposes Sardis Dam. The peak ground accelerations that were recommended ranged from 0.03g to 0.5g -- from imperceptible to very strong. We eventually reached the conclusion that the expected ground motions were sufficiently large to require us to assess the liquefaction potential of the dam and foundation.

After extensive investigations of the sandy portions of the dam and foundation, we had concluded that the dam was safe, when Dr. H. B. Seed (Seed et al. 1983) published data from China indicating that some clays were susceptible to liquefaction -- up to that time clays had generally been assumed to be non-liquefiable. Weak, sensitive clays meeting the Chinese criteria were in fact found in the foundation of Sardis Dam, and in 1988 an upstream berm was completed in the area believed to be most likely to fail. It was recognized at that time that more work was needed to evaluate the safety of the rest of the dam.

The site characterization difficulties for this site and several other complexly stratified alluvial dam sites in the central United States challenged our in situ investigation techniques and consequently we improved our ability to perform and interpret cone penetrometer tests. We pursued laboratory investigations to better understand the behavior of these fine-grained soils under cyclic loading and to better define the criteria indicated by the data from China. These studies helped define additional portions of the dam and foundation that required remedial action. We devoted more than a decade to development and validation of a sophisticated effective stress nonlinear large strain finite element technique to estimate the deformed shape that would result from an earthquake and to develop design parameters for the remediated zones. The deformed shape of the embankment after earthquake shaking using this technique is shown in Figure 8.

In January 1992, nearly 20 years after the San Fernando earthquake that started this reevaluation, the conclusion of the Sardis studies is that additional remedial work is needed at a cost of several million dollars. Design of the proposed solution -- driving several thousand large concrete piles (see Figure 9) -- will be a challenge to the state of the art. Because of the current lack of design tools, we are again forced into a certain amount of concurrent research and design, even though that is not usually a very efficient way to do either.

3.3 Olmsted Locks and Dam Project

Another challenge we now face is to design and construct a new double lock and dam on the Ohio River at Olmsted (see location map in Figure 1). A portion of the dam lies down flat on the river bed so that barge traffic only needs to use the lock in periods of low water. This river carries some of the heaviest barge traffic in the world. Loss of use of this important shipping route would have a serious impact on the economy of our nation as evidenced by the economic damage caused by the severe drought which made shipping impossible on portions of the Mississippi River in 1991. While we believe that the potential for loss of life caused by earthquake-induced failure of this structure is small, the economic losses could be substantial, and failure could shut down navigation on the Ohio River for an extended period.

This important navigation link is located on the northernmost extension of the New Madrid fault zone. Since we have little modern experience with significant earthquakes in the central United States, the earthquake ground motion levels appropriate for design remain controversial, as they were for Sardis Dam. Since the seismic design level this project is largely an economic decision, the many issues that enter into a detailed risk analysis (such as ground motion levels, uncertainty in engineering design aspects, and quantification of consequences) remain controversial because of differences in opinion as to how to define them, and because the construction costs associated with these aspects are substantial.

The design of this massive, complex, rigid structure remains a challenge. The realistic analysis of stiff, rigid, embedded, submerged walls subjected to dynamic loads is a distinct challenge to our dynamic soil-water-structure interaction analysis capability. We have much to learn yet before we have accurate, efficient, practical tools for dynamic design of its pile foundations. Our methods for determining hydrodynamic effects need improvement and validation. To add further geotechnical interest, there is an old landslide on the landside of the locks. The challenge here is to estimate the material strengths in this old landslide and estimate the extent of deformations in an earthquake. Like Sardis and Folsom Dams, the Olmsted Project involves concurrent research and design.

On projects such as Olmsted, we would like to be able to make intelligent decisions concerning the tradeoffs between the cost of providing additional seismic resistance and the expected damage to be avoided -- direct and indirect, tangible and intangible. At the present time, we are not able to define these aspects with sufficient accuracy to be useful. Improvements will require, for example, better estimates of expected ground motions, including recurrence rates, and better methods

of predicting structural damage for a given level of shaking. It has also become clear as we progress in this project and others that the tools for estimating vertical ground motion, empirically and analytically, have not been developed and validated.

The Folsom Dam and Sardis Dam seismic safety evaluations and the Olmsted Locks and Dam design phase are three examples of our field experiences in the shortcomings of the state of the art in earthquake engineering. These and other projects like them lead us to conclude that once we have done the best we can to evaluate the seismic safety of our critical facilities within the current limitation of the state of the art, the remaining uncertainties are large, uncomfortably large, considering the costs and risks involved. These issues have thus motivated us to organize the Quake Program. Our planning process and the current scope of studies in the program are described in more detail in the next section.

4. FOCUS AND SCOPE OF QUAKE PROGRAM

4.1 Program Focus and Planning Process

Since the motivation for Corps-funded research related to Civil Works is primarily driven by needs in the field, it is the nature of Corps Civil Works research programs that they are highly problem-oriented, and generally classify as applied research. These programs are also directly focused on the Corps Civil Works facilities. Thus, the niche that these programs fill, the Quake Program in particular, is a link between the more basic research pursued at universities and the rather unique Corps field engineering needs for practical design of critical flood control and navigational facilities.

The Corps is actually responsible for the enormous capital investment that the United States has made in our system of dams, locks and related facilities. As public servants, it is our mission to protect public safety and to spend public dollars wisely. This serious responsibility sharpens our perspective of research needs for the Corps. Consequently, our selection of research topics is driven by creative ideas that help us solve a real field problem, rather than creative ideas that would be simply intellectually interesting to study. The process by which we formulate our study topics and prioritize them is a combination of (1) needs identified by senior engineers in Corps field offices, (2) knowledge gaps identified by research elements within the Corps, and (3) Corps policy guidance provided by program monitors from Corps Headquarters.

We shape our program based on suggestions provided by a Field Review Group composed of the senior field engineers, who review and prioritize all proposed topics. The Field Review Group provides guidance for the direction of various studies to better meet field needs, and suggests new topics for consideration.

The Corps research elements manage and execute the program, suggest topics to the Field review group for their consideration, provide guidance on technical feasibility, and distinguish between problems actually requiring research and technology transfer (if the needed tools are actually available but have not yet made their way into practice). Technical Monitors from Corps Headquarters maintain an oversight role throughout the process to provide technical and policy guidance for the program. The proposed program incorporating the field comments and priorities is approved, guided and reviewed by Corps senior policy officials to assure coherent, sensible, productive, cooperative, mission-oriented expenditure of our limited research funds.

4.2 Structure and Scope of Proposed Studies

The program structure follows the solution procedure for an earthquake engineering problem: 1) What are the ground motions? 2) How will they be delivered to the site/facility? 3) Analyze the facility for stress and deformation using appropriate boundary conditions, properties, material behavior and numerical and physical models, 4) Estimate actual performance, and 5) Revise design or implement remedial action if needed.

The program has three primary focus areas (1) earthquake ground motions, (2) geotechnical earthquake engineering, (3) structural earthquake engineering, and one secondary focus area in emergency response. Other Corps civil works related disciplines, such as hydraulics, electrical and mechanical, have indicated that they had no pressing earthquake engineering problems requiring research, and thus are not included in the program at this time.

As recommended by our Field Review Group, the initial efforts are directed toward those areas where greatest loss of life and property would result from an earthquake in conjunction with those structures most likely to fail from earthquakes. That area is unquestionably reservoir dams. Within the arena of reservoir dams, the Field Review Group recommended prioritization of research efforts dealing with liquefaction, followed by nonlinear behavior of concrete dams. Thirdly, they recommended that the next area of emphasis should be navigation locks and dams and other hydraulic structures. In addition, the Field Review Group recommended that additional strong motion instruments be placed in representative structures at sites where earthquake events are likely to occur, to provide additional data for validation of modeling and analysis techniques.

Evaluating the adequate seismic performance of our facilities usually consists of estimating the deformations that we expect and deciding whether the facilities can safely withstand these deformations. There are presently serious limitations in our ability to estimate deformations, and improvements will require (1) better estimates of expected ground motions, (2) improved analytic tools for calculating deformations including better constitutive models, (3) improved site and laboratory investigation methods that provide the information needed to use those analytic tools, and (4) validation of items 2 and 3 with full scale field data, particularly from earthquakes and possibly centrifuge testing.

4.2.1 Earthquake Ground Motions

The greatest uncertainty for all of our projects, be they earth dams, concrete dams, locks, levees, harbors, etc., is by far the level of earthquake ground motion that we should design for. Determining the incoming

ground motions and other possible seismic hazards is common to all design efforts. It is in this step that we define the "loads" we will design for. We find that with each new earthquake we learn a great deal. In this topic area we have planned several work units to improve our procedures for selecting design ground motions for engineering analyses. This effort includes: 1) reconnaissance of earthquake stricken locations world-wide to collect full-scale performance data, 2) continued development of ground motion and response spectra design charts and data bases, 3) development of empirical and analytical procedures to estimate vertical motions in time and frequency domains, 4) investigation of the effects of fault mechanics on vertical motions, 5) two- and three-dimensional boundary effects on ground motions, traveling waves and spacial variation of motions, and 6) instrumentation of a site and structure to capture detailed full scale response data for calibration of analytical procedures.

The continued effort to improve our ability to determine design ground motions is essential since this aspect of earthquake problems typically is a prime controlling factor in the overall cost of a project. Overestimation of ground motions leads to costly overdesign, and underestimation may lead to loss of life and costly, time consuming remediation.

4.2.2 Geotechnical Earthquake Engineering

This portion of the Quake Program is focused on liquefaction and its effects on our facilities. There are four main topic areas addressed:

* Site characterization and material behavior, to include development of innovative methods such as geophysical techniques for in situ identification of potentially liquefiable soils; rapid, less expensive assessment of dynamic material properties in situ; and investigation of the behavior of soils as they approach liquefaction, liquefy, and after liquefaction.

* Seismically-induced deformations to include a series of work units to develop improved methods for estimating deformations, to include the effects of three-dimensional boundary conditions and geometry, and to examine the potential behavior under strong aftershocks.

* Remedial measures and ground improvement to include a series of studies on the fundamental behavior of remediated soils which are now composites; development of procedures for realistically analyzing these composite materials; investigation of the effects of lateral stresses and migration of excess pore pressures into improved ground.

* Dynamic soil-structure-water interaction problems to include design of deep and shallow foundations, walls, tunnels and shafts, and embedded and submerged structures, in cooperation with the structural counterparts.

The best type of data for improving our design procedures is full scale performance data. The above topics are planned to be integrated with data from reconnaissance of earthquake damaged sites and quantitative information from an instrumented structure mentioned in the previous section.

4.2.3 Structural Earthquake Engineering

The structural earthquake engineering studies focus on concrete dams and the outlet works essential for controlled release of the reservoir. In the course of working on these types of projects it has become clear that the output from finite element analyses is not easily translated to the kind of information needed by structural designers; so, another aspect of the proposed studies is to develop such basic tools. The areas for research planned at this time are:

* Seismic structural response of concrete dams to include sliding structural stability, nonlinear material behavior and failure mechanisms, nonlinear behavior of roller-compacted concrete, dynamic uplift pressures, seismically-induced displacements, design earthquake level selection, nonlinear behavior of arch dams, field and model verification tests, reconnaissance of earthquake stricken sites to collect full scale performance data, and instrumentation of a site to collect detailed, quantitative full scale performance data for calibration of analytical procedures.

* Seismic response of outlet works to include overturning stability, nonlinear material behavior, equipment-structure interaction, bridge-tower interaction, and tower-abutment interaction.

* Development of basic engineering tools for extraction of design force information from response spectra analyses, and pre- and post-processors for finite element output.

* Additional soil-structure-water interaction studies for piles, walls, concrete channels, tunnels and shafts, in cooperation with the geotechnical counterparts.

4.2.4 Emergency Response

Another important mission of the Corps is emergency response in the aftermath of a natural disaster. We have certain responsibilities under the Federal Response Plan that would be activated after a major earthquake, such as assisting with removing debris, and restoring transportation routes and utilities. We are also often asked for assistance by others. For example, after the Loma Prieta earthquake, the Corps had more than three hundred engineers in the field supporting the U. S. Federal Emergency Management Agency.

Our plans for this portion of the Quake program are still in the formative stage. At this time we envision (1) periodic review of progress in the general field of earthquake engineering to select information that would impact our emergency response mission, (2) coordination of Quake research and emergency response research, and (3) action to assist transfer of technology to Corps emergency response offices.

5. COORDINATION OF QUAKE PROGRAM WITH OTHERS

For many years, the Corps has exercised well established, existing forums for keeping abreast of current research in earthquake engineering, primarily through national and international, governmental and private professional activities. We typically work with leaders in the field in our earthquake engineering safety evaluation problems, and

perform research in our Corps Laboratories. We learn much from participation, communication and coordination by our headquarters, field and research laboratory staff with:

a. Activities of national and international professional societies such as American Society of Civil Engineers, Earthquake Engineering Research Institute, Seismological Society of America, U. S. Committee on Large Dams, and the International Committee on Large Dams.

b. Activities of national organizations such as the National Research Council Committee on Earthquake Engineering and Transportation Research Board, and the Electric Power Research Institute.

c. Activities of federal agencies, such as the four primary agencies involved in the National Hazards Earthquake Reduction Program, namely the Federal Emergency Management Agency, the U. S. Geological Survey, the National Science Foundation (NSF) which includes the National Center for Earthquake Engineering Research, and the National Institute for Standards and Technology, as well as the many contributing agencies.

d. Activities of university organizations such as the consortium of California Universities for Research in Earthquake Engineering, and the Earthquake Engineering Research Center at the University of California, Berkeley.

e. Activities of international government groups such as the Joint U.S.-Japan Panel on Wind and Seismic Effects.

f. Activities of private companies such as Woodward Clyde and Associates, Dames and Moore and Associates, H. J. Deggenkolb and Associates, and others involved in state-of-the-art earthquake engineering projects.

Through our Field Review Group meetings, close communication with federal agencies, and work with other experts, we develop the perspective needed to plan our research program in earthquake engineering to meet our Civil Works mission.

6. CONCLUSIONS

In the development of a new federally funded research program, it is essential to carefully identify research needs and expected products, coordinate and communicate with other federal agencies, plan and execute the proposed program, and transfer results to the federal field engineers.

We proposed the Quake Program in response to an outcry from our district and division offices who repeatedly encounter the shortcomings in the state-of-the-art in earthquake engineering in their seismic safety evaluations of existing and design of new Civil Works projects, primarily flood control and navigation facilities. The focus of our proposed Quake Program is applied research -- to take fundamental research contributions such as scientific agency and university research, and develop practical design tools tailored to Corps needs for use in Corps offices. At this time, the focus of the program is on the seismic safety of reservoir dams -- the improvements needed in ground motion selection and geotechnical and structural engineering to reduce uncertainties and develop more efficient, effective procedures for assessing dam safety under seismic loading and designing remedial actions if required.

7. REFERENCES

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Seed, H. B., Idriss, I. M., Arango, I. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 3, pp. 458-482.

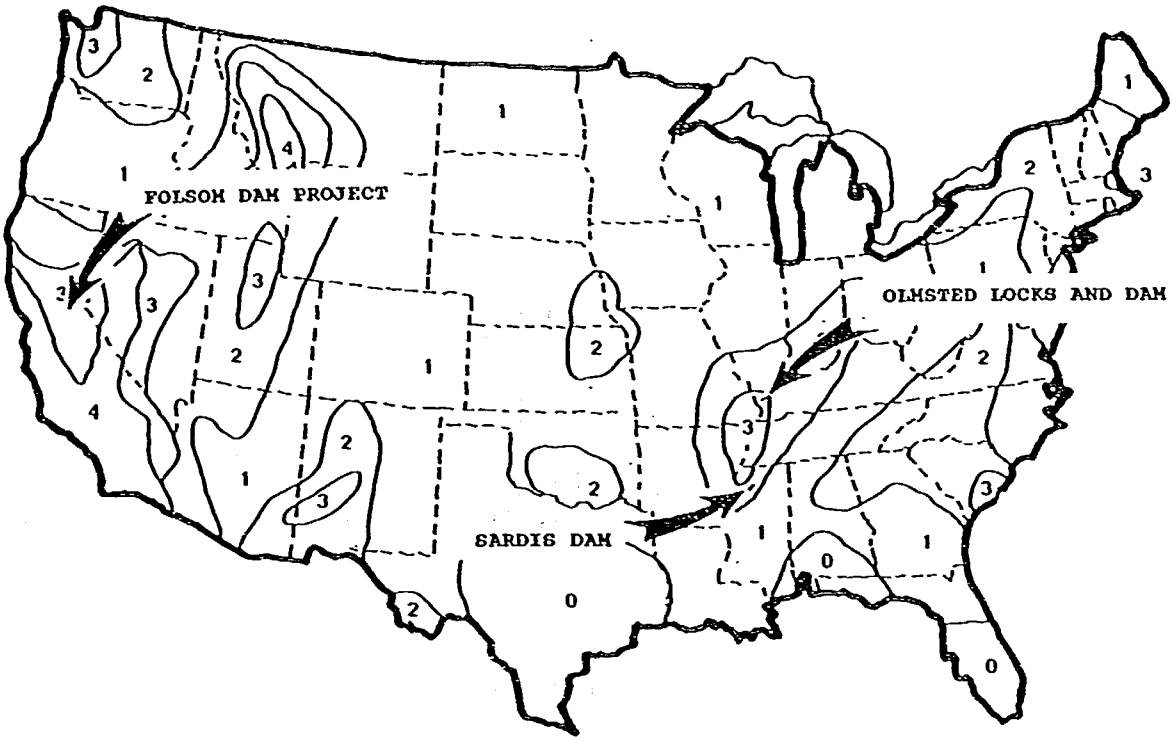


Figure 1. Locations of three Corps projects

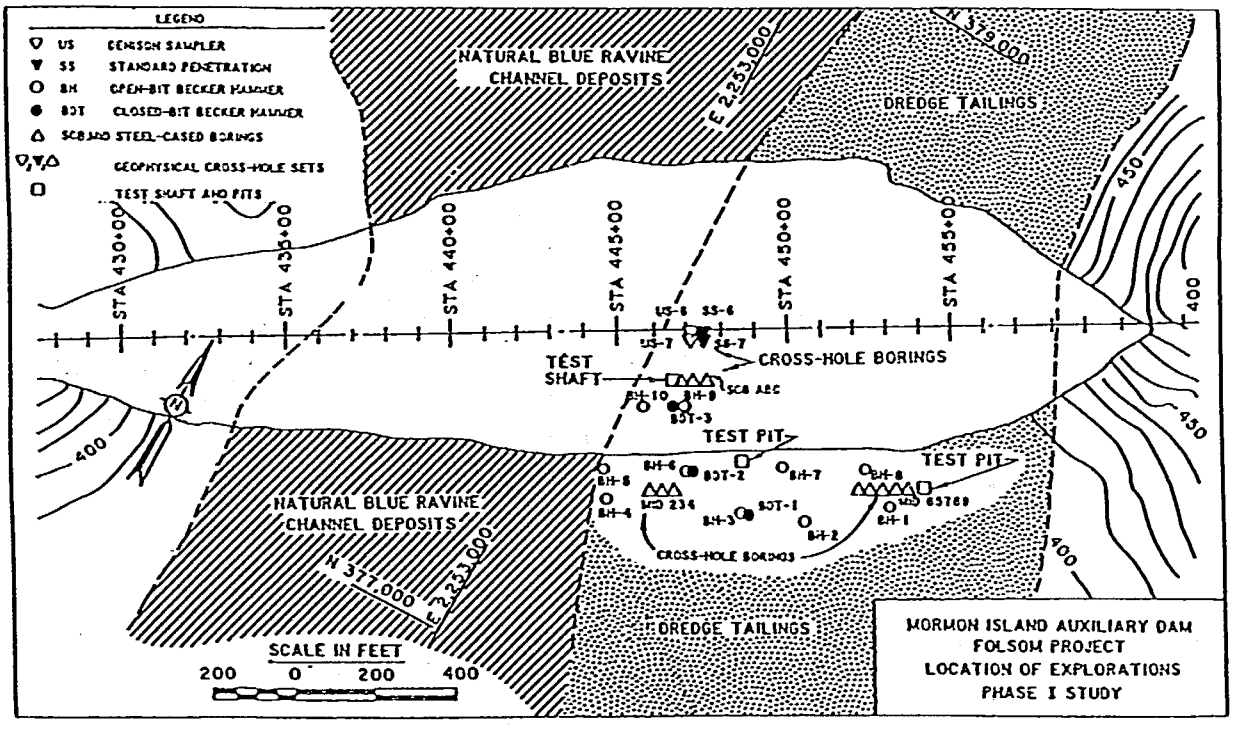


Figure 2. Plan of Mormon Island Auxiliary Dam, Folsom Project, showing locations of field investigations

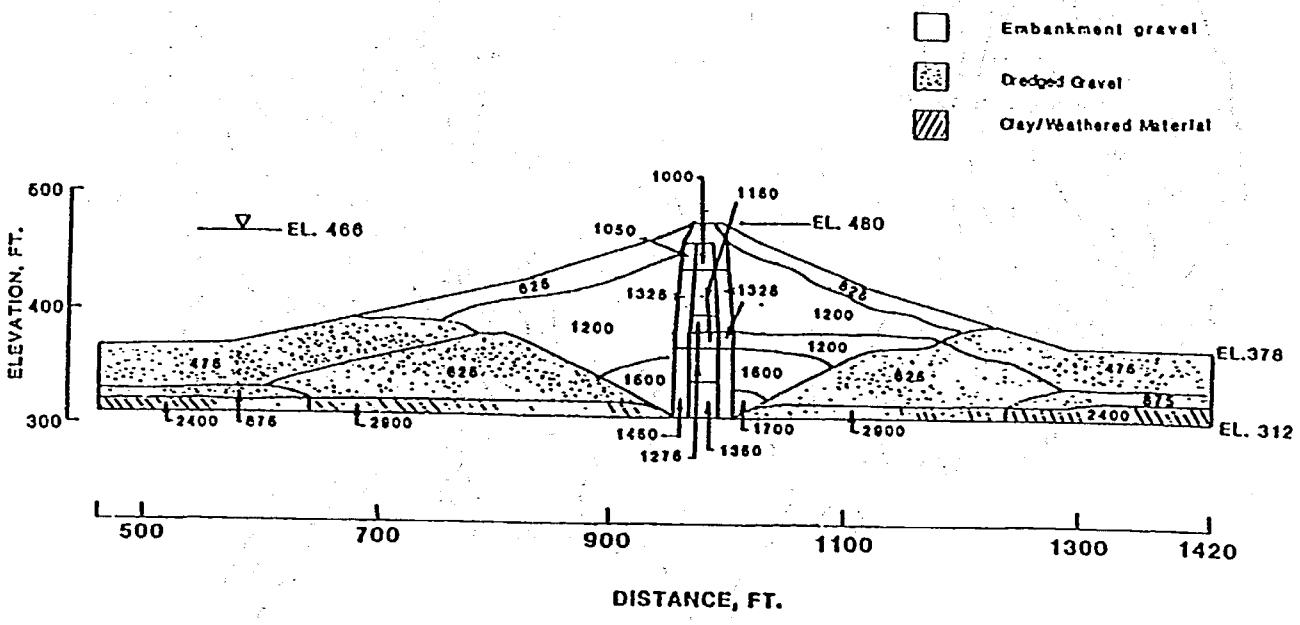


Figure 3. Idealized section of Mormon Island Auxiliary Dam, Folsom Project, showing estimated shear wave velocities in fps

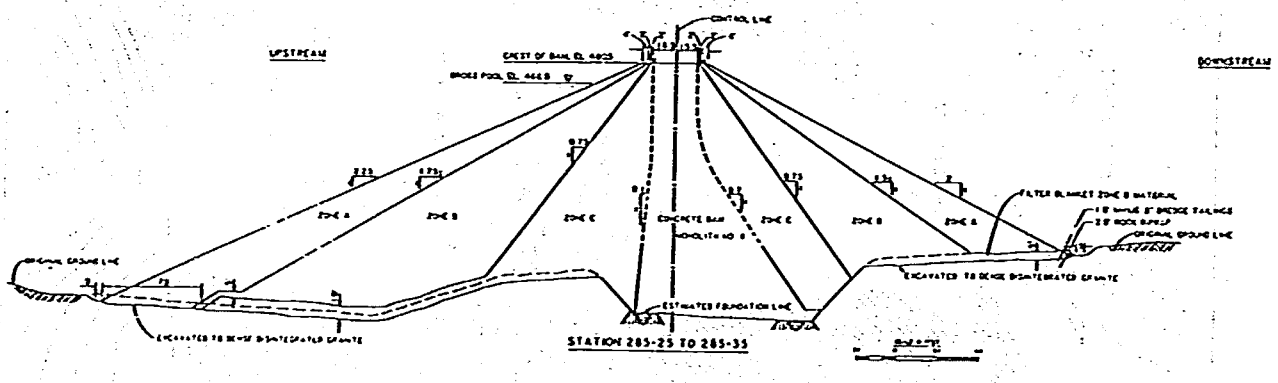


Figure 4. Typical section of Wing Dam, Folsom Project, at interface with Concrete Gravity Dam

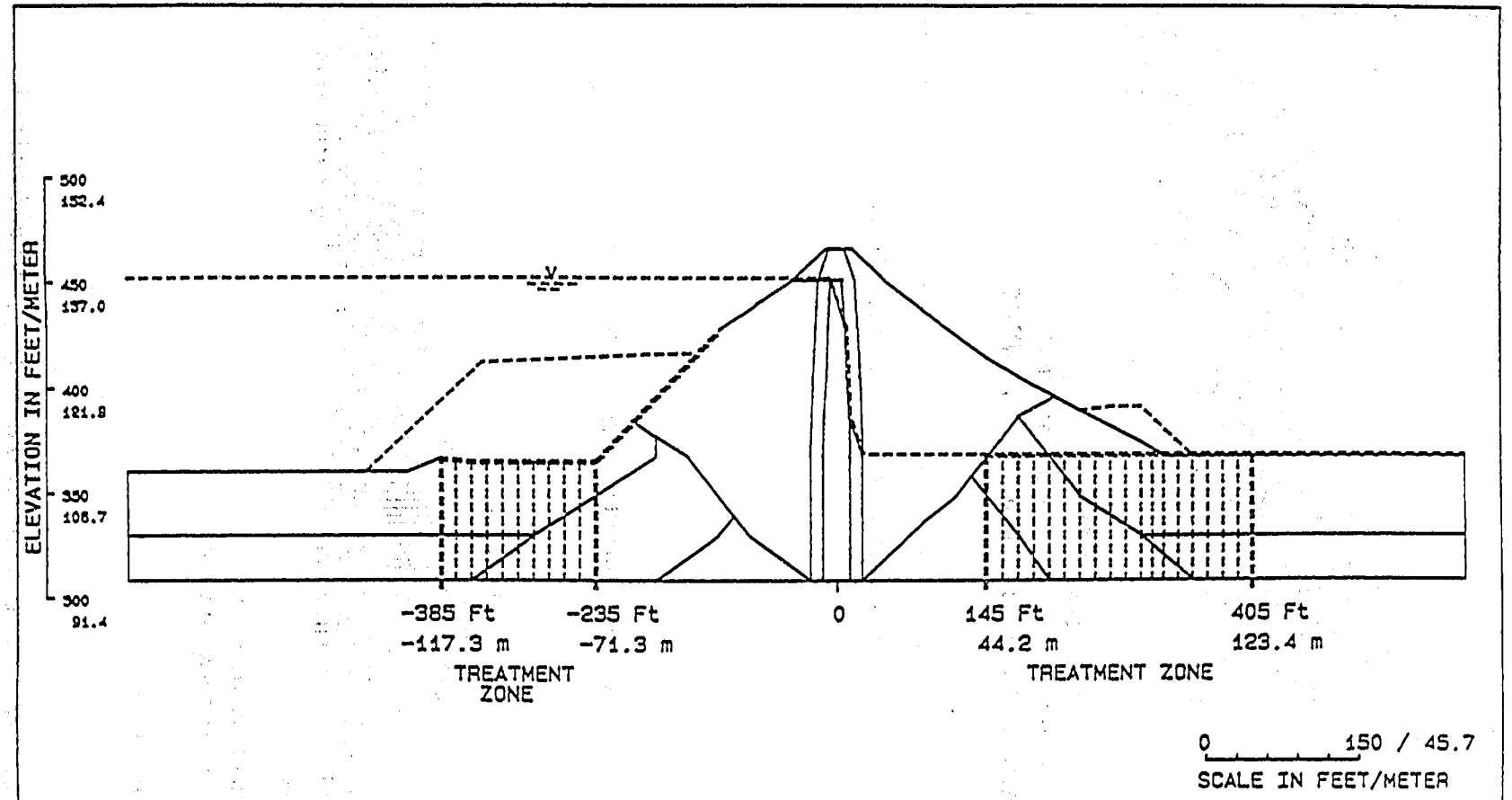


Figure 5. Conceptual plan of pore pressure control alternative for remediation at Mormon Island Auxiliary Dam (after Ledbetter et al. 1991)

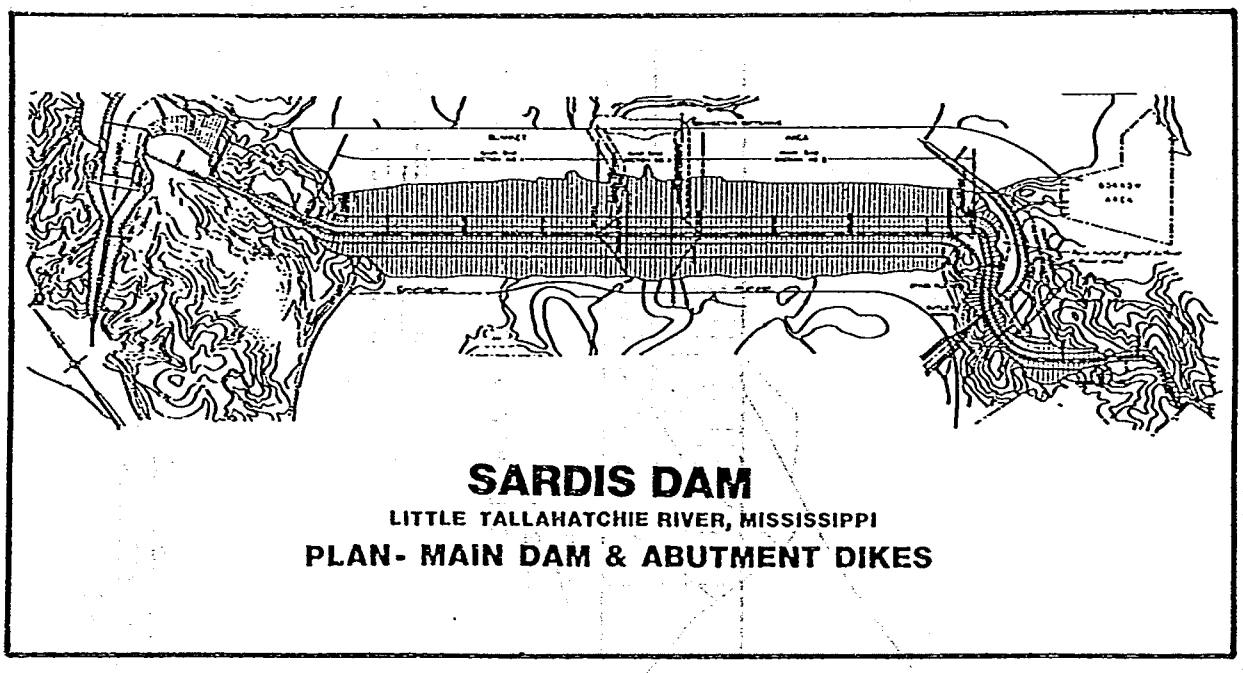


Figure 6. Plan of Sardis Dam, Mississippi

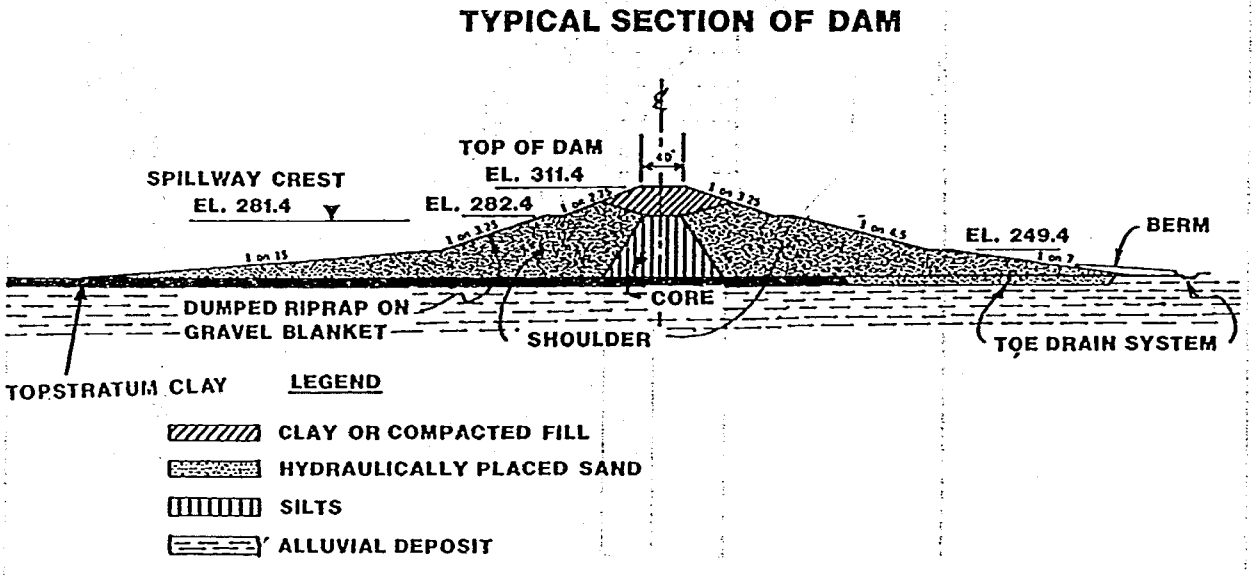


Figure 7. Idealized section of Sardis Dam, Mississippi

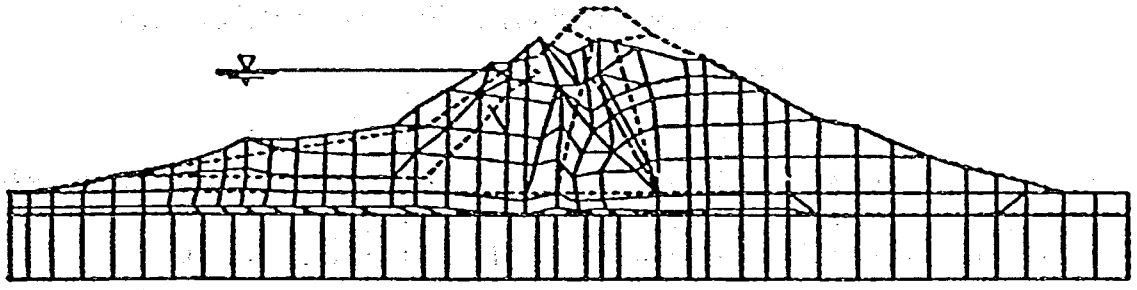


Figure 8. Initial and post-liquefaction estimated shapes for Sardis Dam (without remediation, after Finn et al. 1991)

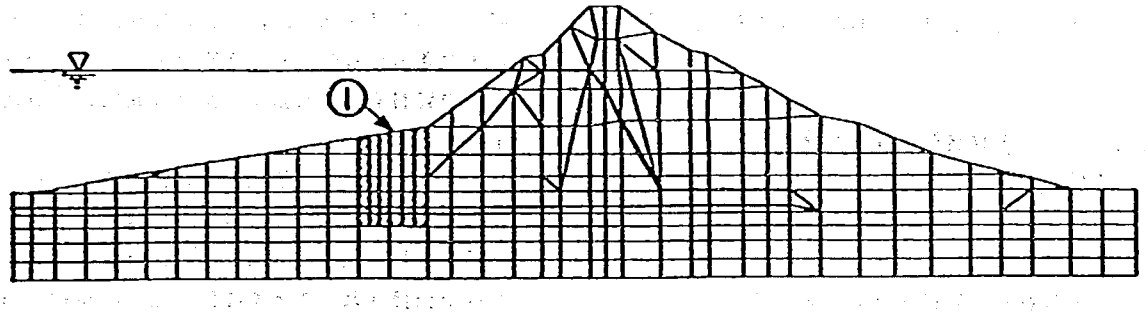


Figure 9. Conceptual plan for remediation of Sardis Dam by strengthening and stiffening upstream foundation materials (after Finn et al. 1991)

鉄筋コンクリート造建築物の超軽量・超高層化技術の開発

構造性能に関する研究（１９９１）

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【要約】

本論文は、昭和63年度から５ヶ年計画で実施されいる建設省総合技術開発プロジェクト「鉄筋コンクリート造建築物の超軽量・超高層化技術の開発」(略称New RC)の平成３年度に行われた構造性能に係る主要な成果の概要を記したものである。

【キーワード】

高強度コンクリート、高強度鉄筋、超高層建築物、構造性能

１．序

鉄筋コンクリート構造は、優れた経済性、耐久性、維持管理性等により、従来から中規模建築物に構造として多用されてきた。しかしながら、現在の材料強度では、現在極めて社会的要求の高い超高層化・長大スパン化に対応するにはおのずと限界がある。本総プロは、300 ～1,200kgf/cm^{2}の圧縮強度を有するコンクリート、4,000～12,000kgf/cm^{2}の引張降伏強度を有する鉄筋を開発し、かつそれらを用いた新たな鉄筋コンクリート造建築物の開発を目的としたものである。（図１）

２．高強度鉄筋および高強度鉄筋コンクリート要素に関する研究

2.1 研究課題

平成３年度に行った研究課題を以下に記す。

2.1.1 高強度・高品質鉄筋の開発

a. 高強度・高品質鉄筋の製造技術の検討、試作及び試作品の基本性能
b. 高強度・高品質鉄筋のガス圧接性
c. 高強度・高品質鉄筋の溶接性
d. 高強度・高品質鉄筋の曲げ・曲げ戻し
e. 高強度・高品質鉄筋の歪み時効硬化
f. 高強度鉄筋の継手単位の性能
g. 継手を有するはり部材の性能
h. 高強度・高品質鉄筋の品質規格の骨子

2.1.2 拘束コンクリートと高強度コンクリートの付着特性

a. 拘束コンクリートの力学特性に関する資料のとりまとめ
b. 塑性ヒンジの主筋の座屈防止効果の定量化
c. 鉄筋コンクリート構造物への有限要素法適用のガイドラインの検討
d. 機械式定着・機械式付着の性能
e. 付着性能試験方法の標準化
f. 付着作用に依存する曲げ部材の付着性能評価

2.2 研究成果

得られた主要な成果を項目毎に以下に記す。

2.2.1 高強度・高品質鉄筋の開発

a. 高強度・高品質鉄筋の製造技術の検討、試作及び試作品の基本性能

明瞭な降伏棚を有し、主筋に使用される SD685と明瞭な降伏棚がないが主筋に使用されるSD980 及びせん断補強筋に使用する SD785とSD1275の高強度鉄筋の試作を行い、品質規格作成に必要な資料として、(1)化学成分、(2)降伏点のばらつきの品質管理の可能性、(3)破断伸び、(4)降伏比、(5)降伏棚の長さ，(6)問題なく曲げ加工できる曲げ内のり半径等の資料が収集できた。これらの資料を元に高強度・高品質鉄筋の品質規格の骨子を策定した。

b. 高強度・高品質鉄筋のガス圧接性

SD685のD25を用いたガス圧接供試体を製作し、引張試験と曲げ試験を行ったところ、母材破断で規格引張強度が発揮されることを確認した。圧接部の 180度曲げはできなかった。

c. 高強度・高品質鉄筋の溶接性

引張り強さ100kgf/cm^{2}の溶接用ワイヤーを開発し、

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[*3]建設省建築研究所第三研究部振動研究室室長
[*4]建設省建築研究所第四研究部実大構造物研究室主任研究員
[*5]建設省建築研究所国際地震工学部第二耐震研究室主任研究員

*6建設省建築研究所第三研究部構造研究室研究員

SD685 のD25とD32の溶接試験供試体を製作し、引張試験と曲げ試験を行ったところ、熱影響部での破断が起きて母材の規格引張強度には達しなかった。高強度せん断補強筋の SD785の場合には、フラッシュバット、アップセットバットのいずれの方法でも母材と同等の強度が確保できた。

d. 高強度・高品質鉄筋の曲げ・曲げ戻し

SD685のD25及びSD980のD25の試作鉄筋について、鉄筋工事中の曲げ・曲げ戻しがどの程度許容されるかを明らかにする必要があるため、曲げ内のり半径 3.6d、4.8d 及び 8.0dとして、0° →90° →0° の曲げ曲げ戻し試験を行った。SD685では、4.8dで 90° 曲げが可能であった。SD980では、ねじ筋を除いて4.8dで 90° 曲げが可能であった。曲げ曲げ戻しでは、8.0dでほぼ可能となった。

e. 高強度・高品質鉄筋の歪み時効硬化

鉄筋の製造方法によっては、加工歪みを受けると時間経過にしたがって硬化し折損するものがある。この有無を調査するため、SD685、SD980の高強度鉄筋に破断伸びの1/2 の歪みを付与した後100° C 1時間の時効促進を行い引張試験を行った。成分調整アズロール系SD685のD25では、歪み時効硬化がないことを確認した。

f. 高強度鉄筋の継手単位の性能

SD685のD25のねじ筋鉄筋のグラウト注入カップラーによる継手供試体を製作し、平成３年１月の建設省住宅局建築指導課長通達「特殊な鉄筋継手の取扱いについて」の別添1の1に示された継手の判定基準を準用して適用したところ SA 級継手としての性能を満足することが確かめられた。

g. 継手を有するはり部材の性能

降伏比が90％の高強度鉄筋を用い、危険断面に機械式継手カップラーを有する 1/2スケールの鉄筋コンクリート片持ちばり試験体 3体を製作し、漸増振幅の静的正負繰返し加力を行い、歪集中により引き起こされる主筋の破断の危険性を検討した。その結果、降伏比の高い鉄筋を主筋に用いるとモーメント勾配のある部材の曲げ危険断面での主筋の引張り歪の集中が大きくなり、大変形時に主筋破断を起こすことがあることが確かめられた。したがって、ヒンジゾーンの主筋に使われる鉄筋には降伏比の制限が行われることが望ましいものと考えられた。（図2.1）

h. 高強度・高品質鉄筋の品質規格の骨子

今年度までに試作された鉄筋について得られた鉄筋の製造方法、化学成分、力学特性などのデータをもとに、高強度・高品質鉄筋の規格案の骨子を作成した。

2.2.2 拘束コンクリートと高強度コンクリートの付着特性

a. 拘束コンクリートの学力特性に関する資料のとりまとめ

拘束コンクリート応力・歪み関係に関する既往の諸提案式に関する比較研究を行い、 (1)簡便性、(2)実験結果との適合性、 (3)高強度材料への拡張性等を考慮して適切なモデルを選択する。モデルを表現する式の中の変数は、既往の実験資料に基づいて決定する。以上のようにして求めた応力度歪度関係モデルが高強度材料を用いた本プロジェクトの実験結果に適用できるかを検討し、拘束コンクリートの応力歪関係のモデルを提案した。（図2.2）

b. 塑性ヒンジの主筋の座屈防止効果の定量化

主筋と帯筋に高強度鉄筋を使用した柱試験体24体の中心軸圧縮試験を行って、主筋の座屈性状及び補強筋の座屈防止効果を検討した。上下の加力部分では鉄筋は鉄骨のスタブに溶接し、主筋のみに軸力が作用しコンクリートが軸力を負担しないように、スタブと試験区間の境にスタイロフォームのスリットを設けた。試験体の変数は、主筋座屈を拘束する帯筋の間隔（4d, 6d, 8d, d:主筋の公称径）と主筋径とした。実験の結果、(1) 主筋の座屈は塑性座屈となり最大圧縮応力度は主筋の圧縮降伏強度によって決まり帯筋量の影響は受けなかった。 (2)最大応力度時の変位では帯筋間隔の影響が見られ、帯筋間隔が小さいほど変形能力が向上した。（図2.3）

c. 鉄筋コンクリート構造物への有限要素法適用のガイドラインの検討

非線形有限要素法解析を鉄筋コンクリート部材に適用し、高強度部材のせん断・付着性能が普通高強度材料の場合とどう異なるのか、また、設計にどのように貢献出来るかを検討する。研究は平成３年と４年の２年計画とし今年度は、次の作業を行った。

①高強度材料の非線形構成則のモデル化とプラットフォームを含む複数プログラムへのインプリメンテーション。②プラットフォームを含む複数プログラムによる、普通強度材料、高強度材料の部材の解析とプログラムの信頼性の検討。（図2.4）

d. 機械式定着・機械式付着の性能

外柱はり接合部のはり主筋の90度折り曲げ定着が使えない場合の代替の定着方法として、プレート又はナットによる機械定着の利用が考えられる。そこで、機械定着強度を検討するため、外柱はり接合部の応力状態を再現する加力方法を用いて、56体の試験体の機械

式定着の引き抜き強度の実験を行い、①定着長さ、②支圧面積等の変数の効果を検討した。（図2.5）

e. 付着性能試験方法の標準化

簡易な試験体で割裂付着破壊を起こすことのできる付着試験法を適用して、３種類の異形鉄筋（横節、波節、螺旋溝付）計９体の付着試験をコンクリート圧縮強度 1000kgf/cm^{2}の条件で行い節形状の付着性能に及ぼす影響を検討した。波節鉄筋は、横節鉄筋と比較して約90％の付着強度となり、螺旋溝付鉄筋では約65％の付着強度を発揮した。

f. 付着作用に依存する曲げ部材の付着性能評価

高強度鉄筋・高強度コンクリートを使用した鉄筋コンクリート造はりおよび柱部材の途中の付着性能を検討するため、コンクリート強度、付着長さ、補強筋の量と配置を変数とし、サイドスプリット型の付着割裂破壊を起こすように設計された計26体の実験を行い、従来の付着実験結果と併せて統計的に処理し、付着割裂強度式を導いた。

3．構造性能に関する研究

3.1 平成３年度に行った研究課題

平成３年度に行った研究課題を以下に示す。

3.1.1 柱、はりの曲げ性能に関する研究

a. 柱の曲げ降伏後の変形性能および付着割裂破壊に関する研究
b. 柱の圧縮曲げ降伏後の変形性能に関する研究
c. はりの曲げ降伏後の付着割裂破壊に関する研究
d. はりの最小鉄筋比の規定に関する実験的研究
e. 柱、はりの曲げ性能に関するミクロモデルによる研究

3.1.2 柱、はりのせん断性能に関する研究

a. 柱のせん断性能および付着割裂破壊に関する研究
b. ２方向曲げせん断応力下での柱のせん断性能に関する研究
c. はりのせん断性能に関する研究
d. 有孔ばりのせん断実験
e. ミクロモデルによる実験補完

3.1.3 柱・はりの接合部に関する研究

a. 内柱・はりおよび外柱・はり接合部の定着付着性能に関する研究
b. 外柱・はり接合部のせん断性能に関する研究
c. ２方向加力を受ける直交ばり付柱・はり接合部のせん断性能に関する研究
d. １階柱と基礎のコンクリート強度に関する研究
e. ミクロモデルによる実験補完

3.1.4 耐震壁に関する研究

a. 耐震壁のせん断強度に関する研究
b. 壁板せん断圧縮破壊型耐震壁の曲げ性能に関する研究
c. ２方向変形を受ける耐震壁の変形性能に関する研究
d. 解析モデルの作成およびパラメトリックスタディ

3.2 研究成果

得られた主要な成果を項目毎に以下に記す。

3.2.1 柱、はりの曲げ性能に関する研究

a. シアスパン比が 1.5 および 2.0 の柱では、軸力が高くなるほど変形性能が劣る（図3.1）。またシアスパン比 1.5の柱では、部材中央に縦ひび割れが生じる（図3.2）。
b. 高軸力を受ける柱において、主筋がすべり始める部材角と付着指標との関係が提案された。
c. ２段配筋されたはりでは、２段目の主筋の付着劣化が先行して生じた。また、曲げ降伏後の変形性能は、１段配筋のはりの方がすぐれている。
d. 長期荷重によるはりのひび割れについては、ひび割れ幅が 0.2mm～ 0.3mmであれば、引張側主筋位置での引張応力度は 3000kgf/cm^2～ 4000kgf/cm^{2}であった。また既往のひび割れ評価式で安全側に評価できる。

3.2.2 柱、はりのせん断性能に関する研究

a. 柱のせん断耐力は、建築学会の「鉄筋コンクリート造建物の終局強度型耐震設計指針」［1］（以下、「ＲＣ終局指針」）のＡ法ではやや低く、Ｂ法はやや高く評価される。
b. せん断破壊型のはりでは、そのせん断耐力をＲＣ終局指針［1］のＡ法によって精度よく推定できる。
c. 開孔を持つはりにおいては、開口補強筋の定着を十分に確保する必要がある（図3.3）。

3.2.3 柱・はりの接合部に関する研究

a. 内柱・はり接合部においては、高強度コンクリートを用いた場合、せん断耐力余裕度が同じ普通コンクリートを用いた場合より、コンクリートの有効係数νの低下が激しく、接合部破壊が促進される。
b. 外柱・はりの接合部では、２段配筋の折曲げ定着の各段の定着耐力は、１段配筋の定着耐力より大幅に低下する。また、90度折曲げ定着鉄筋の内法半径を鉄筋径の２～３倍としても実用が可能である。

c. 1階柱にFc600、基礎にFc200のコンクリートを用いて、基礎ばりの十字形、T字形、L字形の交点で柱の中心圧縮を行うと、最大荷重は、十字形の場合に最大で、T字形ではその約8割、L字形では約6割になる。

3.2.4 耐震壁に関する研究

a. 今回行われた壁板せん断圧縮破壊型の耐震壁の実験では、曲げ強度は既往の略算式で、せん断強度はRC終局指針式で評価できる。

b. 2方向に載荷された耐震壁の供試体（図3.4,3.5）では、面外加力の影響で面内方向で耐力が低下する。最終的には、側柱及び側柱近くの壁の圧縮ひずみが進展して壁板が圧壊した（図3.6）。

c. せん断破壊先行型の耐震壁においては、高強度コンクリートの平板実験より得られた構成則モデルを用いて解析を行うと、実験結果に近づいた。

3.3 成果のとりまとめ

以下の項目毎に、これまでの成果のとりまとめを行いつつある。

a. 曲げ降伏型の柱の設計のための資料整理
b. 曲げ降伏型のはりの設計のための資料整理
c. 柱のせん断強度式の精度
d. はりのせん断強度式の精度
e. 柱・はり接合部のせん断強度の評価式、破壊モードの判定
f. 柱・はり接合部のせん断変形及び鉄筋の抜け出し量、定着耐力

4. 設計分科会

設計分科会ではNew RC設計ガイドライン、クライテリアの作成と設計例の作成を最終目標として活動を行なってきている。New RC設計ガイドラインの作成進捗状況と関連する研究課題を以下に述べる。

4.1 適用範囲

200mまでのフレーム及びフレームに連層壁が組込まれた偏心の少ない整形な構造の建物を対象と考えている。200mの根拠としては、60階建ての住宅ならびに40階建て事務所建築物がその高さになることによっている。またこの高さ範囲内では設計用（許容応力度設計）標準1階せん断力係数を 0.2とすれば風圧力が設計を支配することはほぼない。この適用範囲内で

60階集合住宅：　純フレーム、及びフレーム＋壁
40階事務所建築物：ダブルチューブ、及びチューブ＋コアー
25階事務所建築物：純フレーム
15階事務所建築物：純フレーム、及びフレーム＋壁

の試設計とその検討を行ってきた。

4.2 使用材料

試設計を始めた時点では使用材料はゾーン分けに縛られず、スリムな高層建築物の実現を目指した。

60階 集合住宅では、Fc=990、σy=SD685、40階事務所建築物では、Fc=900、σy=SD785、15階 事務所建築物では、Fc=800、σy=SD785 の使用を仮定した。その後実用的なゾーンIの材料（Fc=600、σy=SD685）の使用を仮定した検討も行った。

コンクリートと鉄筋の強度の組合わせ制限は構造規定で行なう予定である。

普通強度材料の使用、混用は許容する。バランスは構造規定で考える。

4.3 設計の基本方針

4.3.1 長期設計

許容応力度設計を行う。許容応力度として、コンクリートの圧縮＝1/3Fc、鉄筋の引張＝2,000～2,200kgf/cm²、圧縮＝2/3σyを考えることにした。ひび割れ幅の制限を設ける。

コンクリートに対する鉄筋のヤング係数比、クリープ係数の検討が必要である。

4.3.2 地震力に対する設計

中小地震動(25kine)に対しては各部材には降伏を生じさせない。

大地震動(50kine)に対しては設計限界変形内(1/80以内)に変形をとどめる。

この為に、

(1) 設計用標準せん断力係数：標準1階せん断力係数(C_0=)0.2に対して、部材の降伏の有無の確認、C_0=0.25以上を設計限界変形時に確保することにしている。これらを静的設計により行い、動的解析でそれを確認する。

動的解析では考慮できていない要因（2方向、強度、変形能のバラツキ、捩れ等）に対する強度の余裕率を確保する。

C_0を動的解析により定められるようにすることも検討中である。その場合に、C_0を0.25よりも小さくすることが可能の決断が必要である。

(2) 2方向荷重考慮設計：梁の設計は1方向の地震荷重に対して行う。但し、多次元性を有する地震動に対

して、柱、壁、接合部の設計は

1) X,Y それぞれの方向での設計応力に（直交方向設計応力の0.5倍）を加えて行う。

2) 任意１方向の地震荷重に対して行う。すべての方向に対して設定以上の安全率を確保出来るようにする。斜方向の解析、設計にかえてX, Y両主軸の応力より任意方向の応力の推定を可能とする。推定方法は検討中である。簡便法としては曲げ、せん断応力は √2、軸力は２倍とする案もある。

(3) 軸力の評価：高層建築物では軸力制限が１番の問題となろう。２方向はもちろん鉛直動の影響も考えた軸力の評価方法を提案する。

(4) 耐力壁関係：耐力壁が入った場合の梁、床の立体効果、耐力壁の高層建築物における高次モードを制御する効果を設計に取入れる。

(5) 解析手法の方法：動的解析による確認は静的設計の後に行うことになっているが、設計限界変形、ならびにそれに対応する設計強度を事前の動的解析により行うこととなる。その為には簡便かつ精度の高いモデル化の方法、解析手法を提案する。

動的解析は平面フレームモデルとする。質点系及び簡易モデルでの解析を認めるためにの根拠ともなる。

静的解析は部材レベルのフレーム増分解析法で行う。部材の降伏点剛性の評価は構造性能分科会線材WGより提案される。外力分布は予備動的解析より決定する。２方向の解析(立体解析)を原則とする。

(6) 保証変形の検証：設計で想定した以上の地震力に対する安全策および設計限界変形の算定に対する信頼性の問題を解消するため、および、P-Δ効果による剛性、強度の低減ならびに分布性状の変化を担保するために保証変形までの変形能の確保を行なう。

(7) 偶心に対する対応：整形な建物を適用範囲とし原則的には偶心は生じないが、部材の塑性化、部材強度のバラツキに起用する偶心に対する設計を行なう。

4.3.3 風圧力に対する設計

現案ではC_0=0.2相当の地震力と風圧力を比較し、風圧力が地震力より小さければ風圧力に対する設計を省略可としている。設計C_0を動的検討より決める場合には風圧力が設計を支配することが十分有り得る。但し本分科会ではそれに対する特別な検討は為されていない。

4.3.4 その他の応力に対する設計

温度応力、乾燥収縮、不同沈下、事故(爆発、衝突)に対する設計も行なう。但し本分科会ではそれに対する特別な検討は為されていない。

4.4 断面算定

4.4.1 曲げ強度

材料(コンクリート、鉄筋)の s-s関係より部材断面の性能を求める。ファイバーモデル、ＦＥＭ解析を行なって求める。これらのs-s関係の提案が必要。
略算式による算定も可とする。

4.4.2 その他の強度

せん断強度 付着強度、柱-梁接合部の強度、耐力壁の強度,は各担当WGより提案される。

4.4.3 算定式と部材強度、変形能のばらつき

強度算定式の精度、ばらつきと安全率（余裕率）の取り方を定める。

4.4.4 各部材の変形能力

部材の破壊形式としては曲げ破壊を想定する。この破壊形式も最終的には圧壊、付着、せん断破壊にモード変化することにより限界変形が算定される。

この値に基づき建築物の保証変形、設計限界変形、強度が定まる。設計分科会提案の設計限界変形(1/80以内)値との関係の調整が必要。

SD490 までは、圧縮筋降伏時の釣合鉄筋比で表わせる。高強度の鉄筋を使用する際には圧縮鉄筋が弾性状態にあることが考えられそれを考慮した終局釣合鉄筋比が必要となる。

4.5 地震動

耐震設計検討用地震動 (New RC波) と、既往波数種を用いて動的解析を行い耐震性の確認を行う。地震動荷重WGより建設地盤(工学基盤)で想定される、大きさならびに特性をもつNew RC波の作成手法が提案される。同時に地表若しくは建築物底面位置へのNew RC波の変換手法も提案される。

既往波の大きさは原則として50kineに基準化する。

4.6 基礎構造

大きな鉛直力並びに転倒モーメントによる付加軸力、水平力を受ける基礎構造の合理的設計手法の提案が為される。

4.7 構造規定

構造規定となり得る項目を挙げた。この項目から設計できるものは除き、設計できない項目に対して規定を定める。

4.7.1 梁、柱

(1)断面

・形状1)　　：矩形柱2)、幅とせいの比の制限3)

柱の最小径とその主要支点間距離4)
矩形はり、T型はり5)
1)実験及び解析の対象範囲による制限
2)円柱の扱い、L型・T型柱及び箱柱等の特殊な形状の柱の扱い
3)柱と壁柱の区別
4)長柱としての座屈の検討の必要性
5)I型梁等の特殊な形状の梁の扱い
・内法長さ1)2)：部材せいと内法長さの比、シアースパン比
1)実験の対象範囲及び解析モデルの精度による制限
2)部材の変形性能確保の為の下限値
(2)主筋
・種類1)　：鉄筋とコンクリートの組合わせ2)3)
1)使用可能な鉄筋の種類の記載
2)実験の対象範囲による制限
3)種類の異なる鉄筋の使用の適否(同一断面、同一部材、同一層等)
・径1)　：降伏点強度と主筋の径
1)使用可能な鉄筋の種類の記載
・かぶり厚1)2)
1)耐火性、耐久性、構造耐力上の必要寸法の記載
2)施工上の最小寸法
・間隔1)2)：主筋相互のあき
1)構造耐力上の必要寸法の記載
2)施工上の最小寸法
・継手　：継手の種類1)、継手の性能と使用箇所2)継手のかぶり厚3)5)、継手相互の位置とあき4)5)
1)使用可能な継手の種類の記載(機械式継手のみ)
2)継手の性能と継手を設ける位置の制限(ヒンジ計画部と非ヒンジ部)
3)耐火性、耐久性、構造耐力上の必要寸法の記載
4)構造耐力上の必要寸法の記載
5)施工上の最小寸法
・引張鉄筋比　：最小値1)、上限値(梁の終局釣り合い鉄筋比等)2)
1)長期荷重に対する最小鉄筋比の制限
2)靭性確保の為の必要条件
・梁の複筋比　：最小値1)2)
1)長期荷重によるクリープたわみの防止
2)靭性確保の為の必要条件
・柱の主筋比　：最小値1)
1)RC柱としての基本的な条件(不測の応力変動への対処等)
・配筋形式1)2)：カットオフ筋、X型配筋
1)実験の対象範囲による制限
2)設計式の前提条件
・カットオフ筋の定着長さ1)：梁2段筋の定着長さの算定式
1)実験の対象範囲による制限
(3)横補強筋
・種類1)　：鉄筋とコンクリートの組合わせ2)3)
1)使用可能な鉄筋の種類の記載
2)実験の対象範囲による制限
3)種類の異なる鉄筋の使用の適否(同一断面、同一部材等、または、せん断補強筋、付着割裂防止、拘束等目的に応じた鉄筋の使用)
・径1)　：降伏点強度と横補強筋の径、最小径2)
1)使用可能な鉄筋の径の記載
2)太径主筋拘束の為の必要条件
・かぶり厚1)2)
1)耐火性、耐久性、構造耐力上の必要寸法の記載
2)施工上の最小寸法
・間隔1)2)3)：径と最大間隔(ヒンジ計画部と非ヒンジ部)
1)主筋の座屈防止及び拘束の為の必要条件
2)実験の対象範囲による制限
3)施工上の最小寸法
・形状1)2)：スパイラル溶接閉鎖型、フック付(フックの折曲げ角度、余長)
1)横補強筋による拘束の為の必要条件
2)実験の対象範囲による制限
・補強筋比：最小値1)2)3)(ヒンジ計画部と非ヒンジ部、高軸力柱、耐力壁側柱)
最大値2)3)4)
1)RC柱としての基本的な条件(不測の応力変動への対処等)
2)実験の対象範囲による制限
3)設計式の適用範囲による制限
4)施工上の上限
(4)梁貫通孔
・貫通径1)2)：最大値(梁せいと径、貫通位置と径)、位置
1)実験の対象範囲による制限
2)設計式の適用範囲による制限

4.7.2 耐力壁

項目は略(ほぼ4.7.1)と同じ。

5．その他の研究

5.1 超高層フラットスラブ建築物の開発

豊かな内部空間を有する超高層フラットスラブ構造の構造設計技術を開発する目的で、

(1)以下の2例について検討を進めている（表5.1）。

(i) コア壁型超高層フラットスラブ建築物（図5.1）

(ii) 大壁型超高層フラットスラブ建築物（図5.2）

(2)文献調査を以下の2点に注目して行った。

(i) フラットスラブ構造実験と計算値の比較

(ii)耐力壁の変形と強度性状

(3)フラットスラブ一柱、フラットスラブ一壁の接合部の実験を行なう。

(i) 内柱シリーズ 4体 (ii)側柱シリーズ 8体

(iii)柱シリーズ 4体 (iv)壁柱シリーズ 7体

を計画中である。

5.2 超高強度ＲＣ造大規模建築物のフィジビリティスタディ

超高強度材料の材料特性を活かした大規模建築物のフィジビリティスタディを行っている。具体的には、以下に示すような構造方法および高さの異なる6例のネガストラクチャーの検討用構造モデルを設定している。今後、それらの構造性能クライテリアの作成、構造設計手法の検討、略式設計を行う。また、施行性、経済性および行政上の問題点についても検討を行う。この他、委員会では上記検討用構造モデル以外の未来型New RC建築物の可能性について検討する予定である。

(1)高さ200mのオープンフレーム …図5.3

(2)高さ300mのオープンフレーム（ストレートタイプ） …図5.4

(3)高さ300mのオープンフレーム（テーパー付き） …図5.5

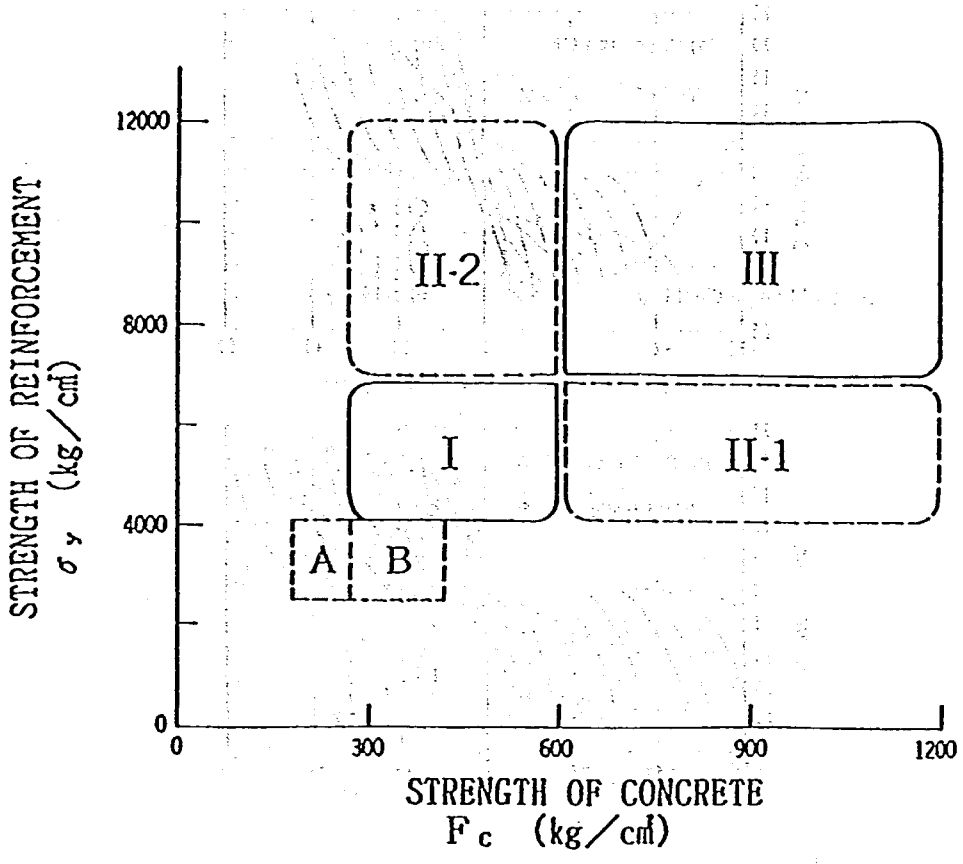
(4)高さ200mのブレース付きフレーム（K形ブレースタイプ） …図5.6

(5)高さ200mのブレース付きフレーム（片ブレースタイプ） …図5.7

(6)高さ300mのブレース付きフレーム …図5.8

参考文献

[1] 日本建築学会：鉄筋コンクリート造建築の終局強度型設計指針、1988年5月



ZONE : TYPE OF RC BUILDINGS & MATERIALS
A: LOW-RISE BUILDINGS IN COMMON
B: HIGH-RISE BUILDINGS CONSTRUCTED IN LAST DECADE
I : HIGH-STRENGTH CONCRETE & REINFORCEMENT
II-1: ULTRA HIGH-STRENGTH CONCRETE & HIGH-STRENGTH REINFORCEMENT
II-2: HIGH-STRENGTH CONCRETE & ULTRA HIGH-STRENGTH REINFORCEMENT
III: ULTRA HIGH-STRENGTH CONCRETE & REINFORCEMENT

Fig.1 Strength of materials and fields of research and development

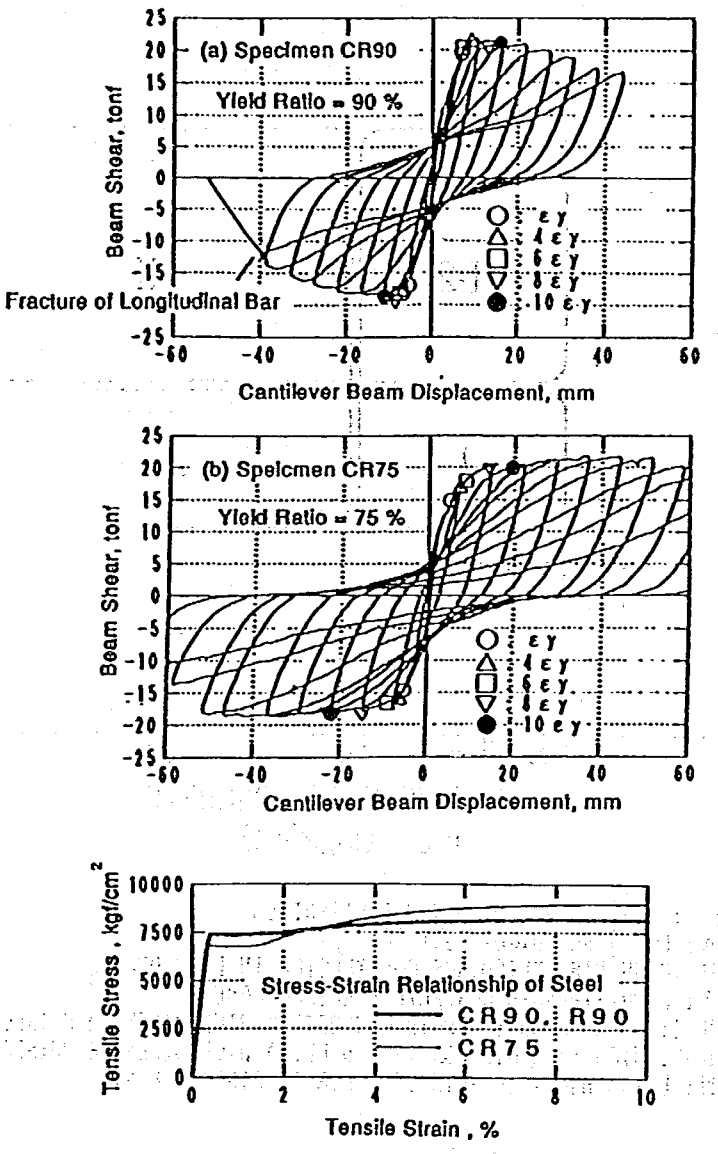


Fig. 2.1 Test of Cantilever Beams using High-yield Ratio Longitudinal Reinforcing Bars

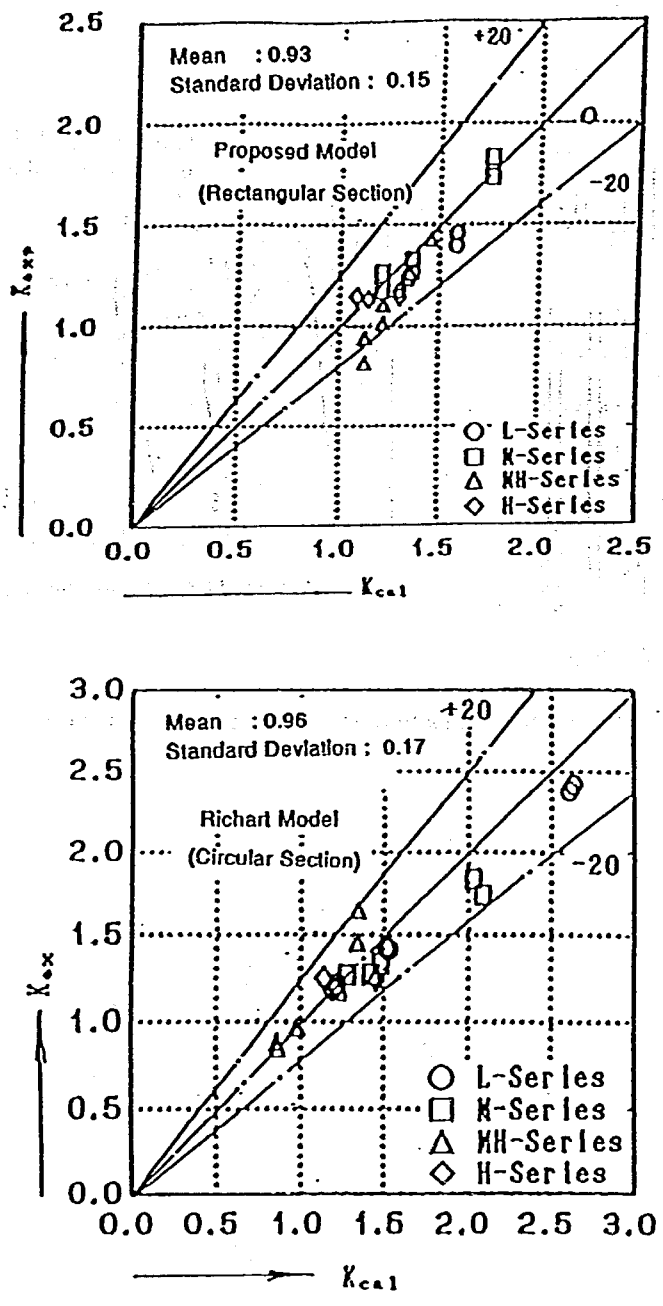
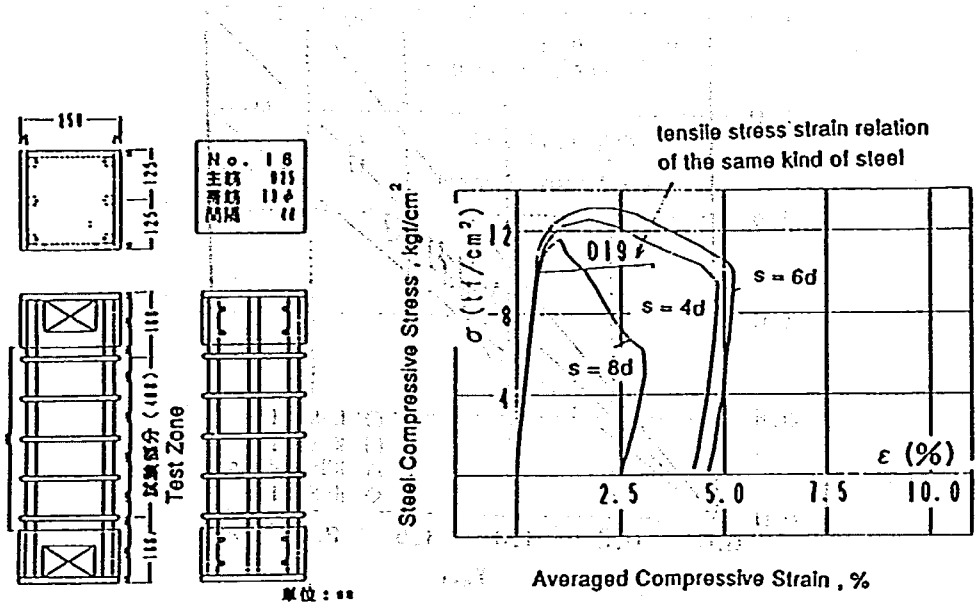


Fig. 2.2 Model Predicting Ultimate Strength Increase of Confined Concrete



Geometry of Typical Specimen

Fig. 2.3 Effect of Hoops Spacing for Preventing from Local Buckling of Longitudinal Bar

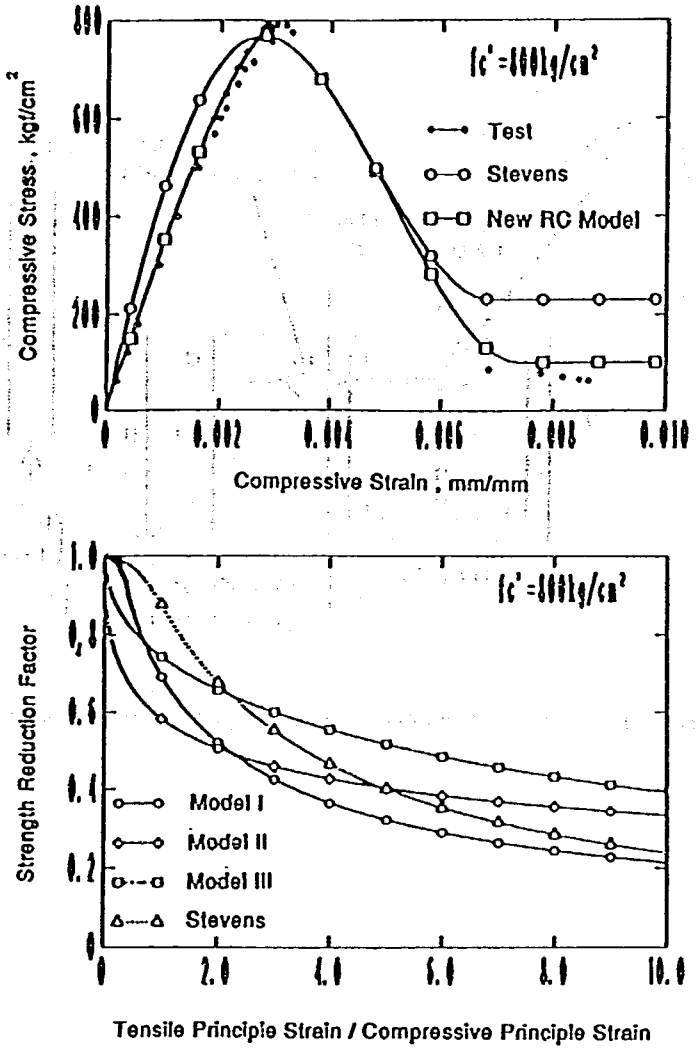


Fig. 2.4 Example of Constitutive Model for Concrete for Non-Linear FEM Analysis

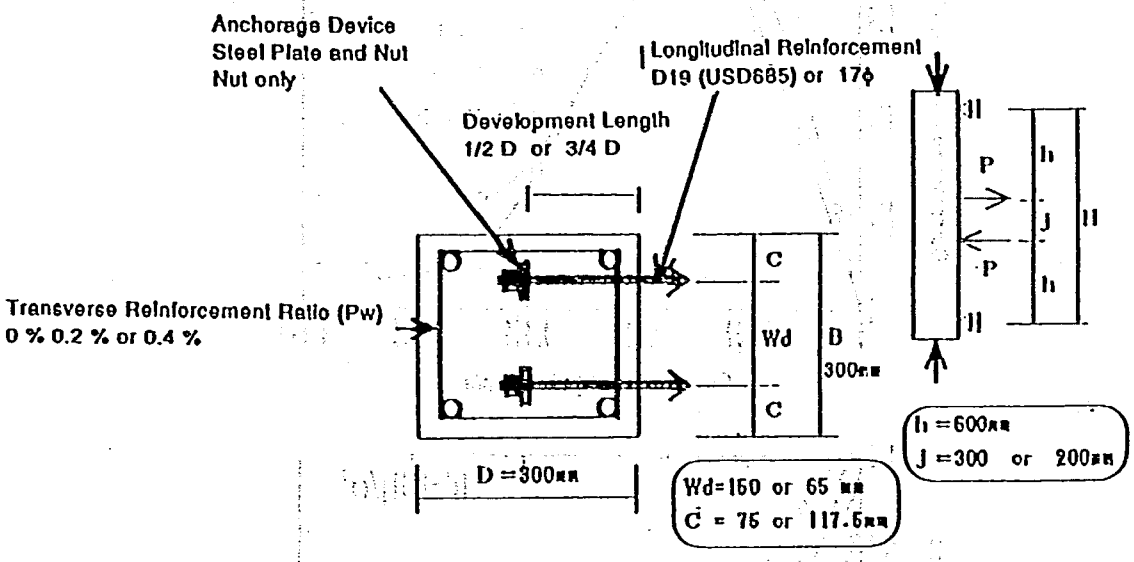


Fig. 2.5 Tensile Strength Test of Plate Anchorage Developed in Exterior Beam Column Joint

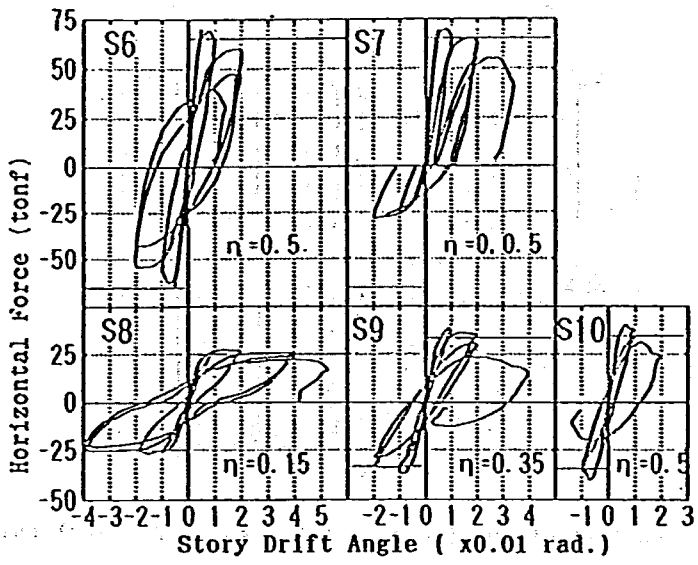


Fig 3.1 Horizontal Force and Story Drift Angle Relationship

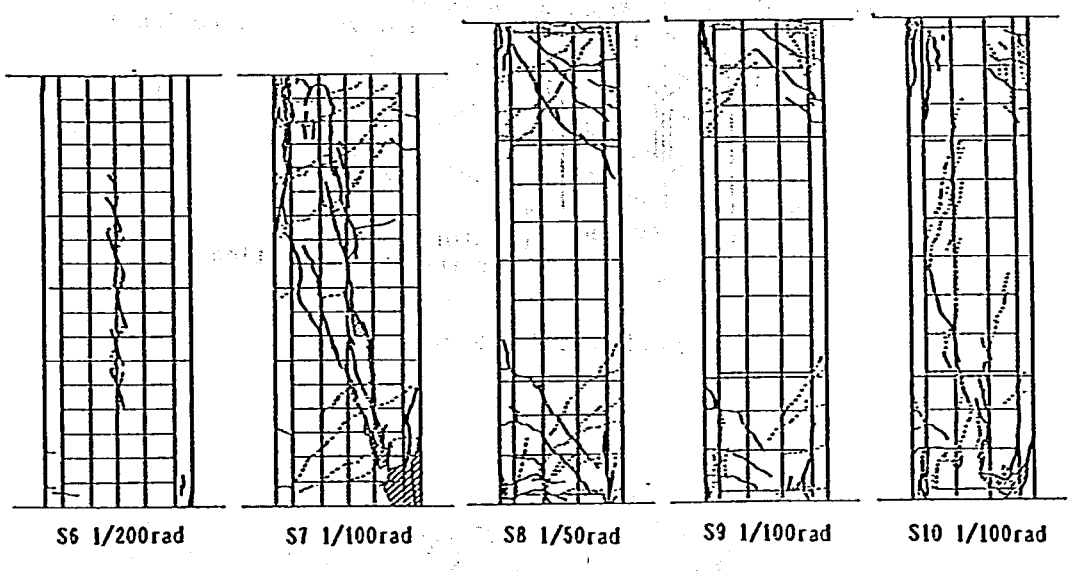


Fig 3.2 Crack pattern of Specimens (at Maximum Horizontal Force)

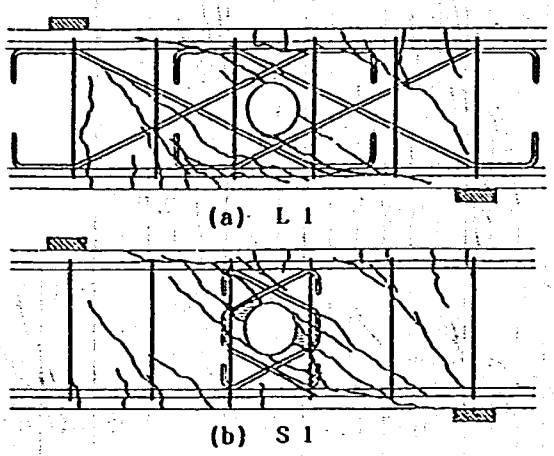


Fig 3.3 Crack pattern of Specimens (at Maximum Horizontal Force)

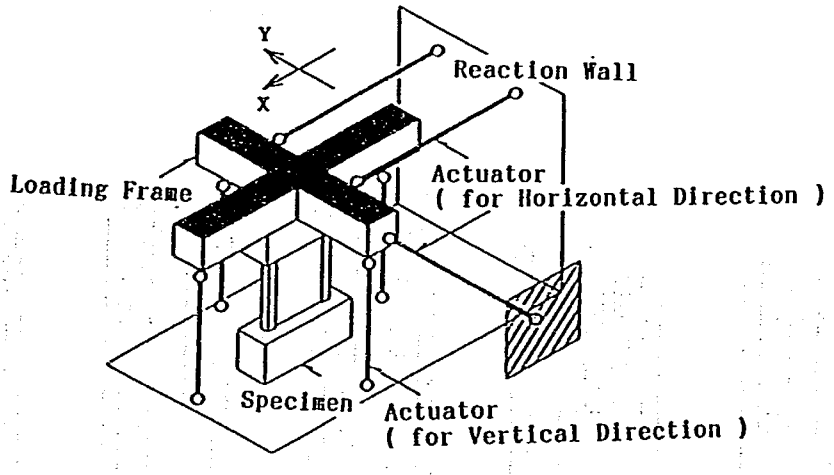


Fig 3.4 Loading System

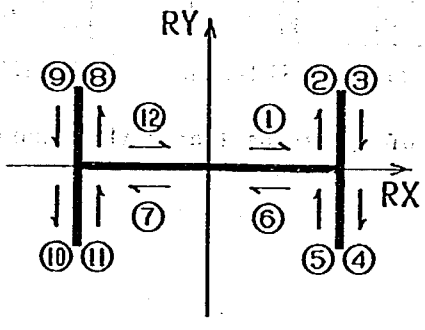
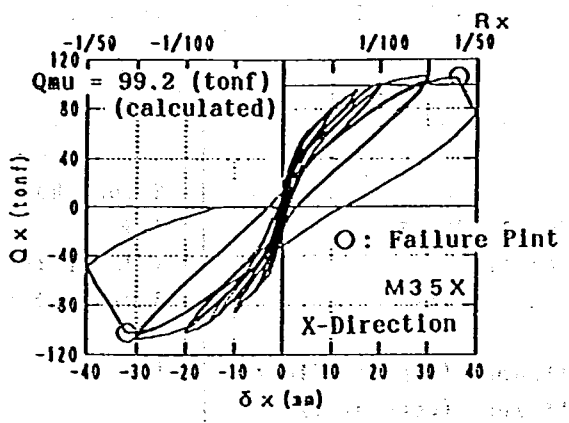
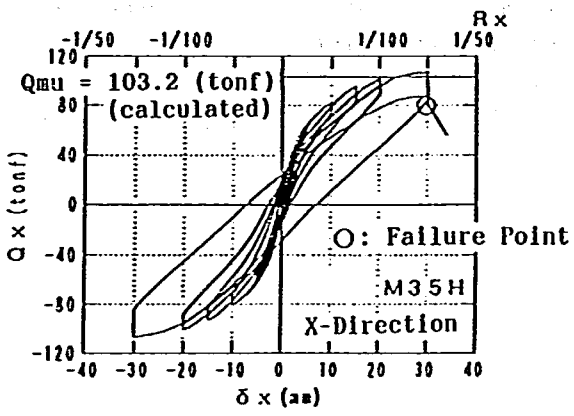


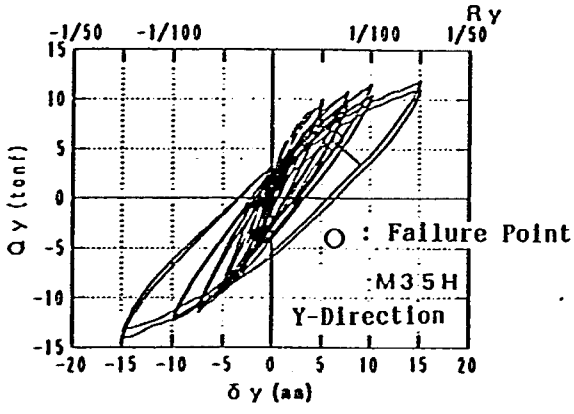
Fig 3.5 Loading Path



(a) One Directional Loading (X-Direction)



(b) Bi-Directional Loading (X-Direction)

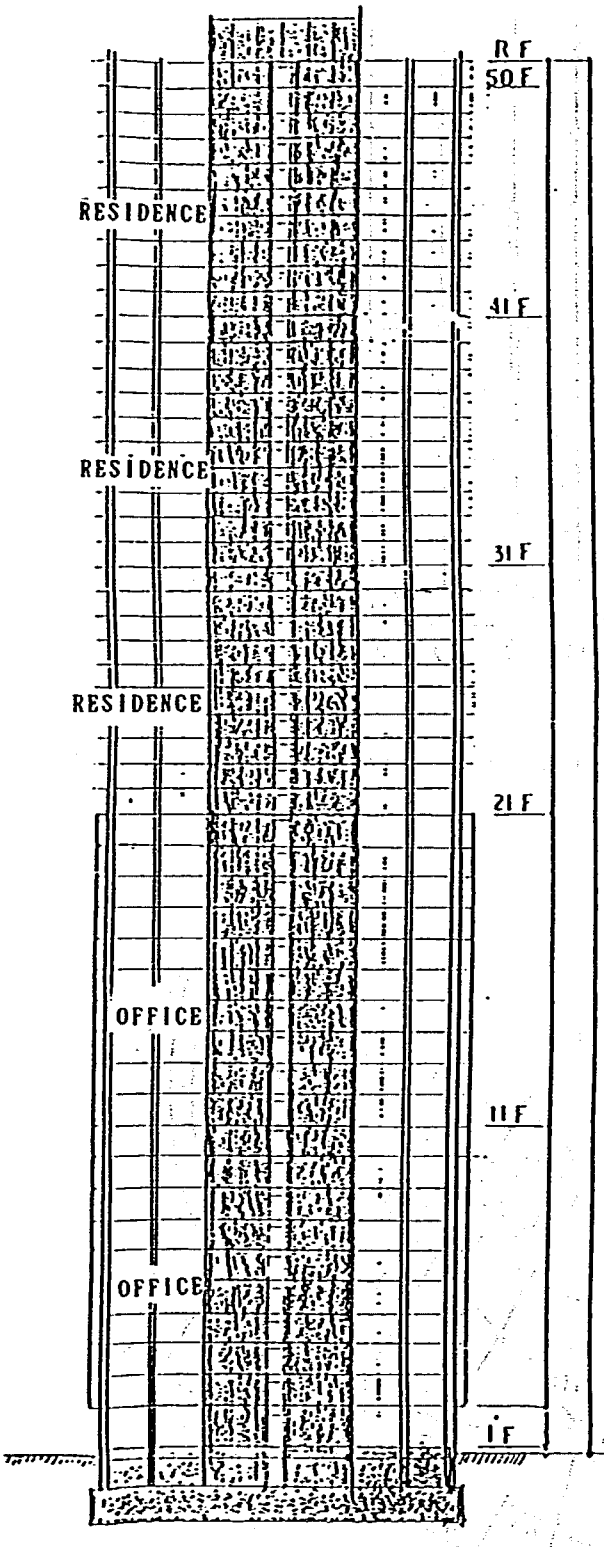


(c) Bi-Directional Loading (Y-Direction)

Fig 3.6 Horizontal Force and Displacement Relationship

Table 5.1 outline of structure

	(1) Core wall type	(2) Wall type
Purpose	Apartment	Resort Hotel
Stories	50F、B－1、P－2	40F
Height	GL＋151.5m	GL＋123m
Story Height	3.0m　1F：4.5m	3.0m　1F：6.0m
Area	41.8×41.8m　(typical floor) 16.2×16.2m　(core arer)	1,090㎡
Concrete	Fc＝98(For slab Fc=56？)MPa	Fc＝98MPa
Steel	σy=980MPa σy=295MPa(for slab)	σy=980MPa
Section	Wall thickness 70cm Boundary beam　70×90cm Column 90×90cm(interior) 80×80cm(exterior) Slab　20or25cm	Wall thickness 40cm Boundary beam　50×300cm Slab　20cm



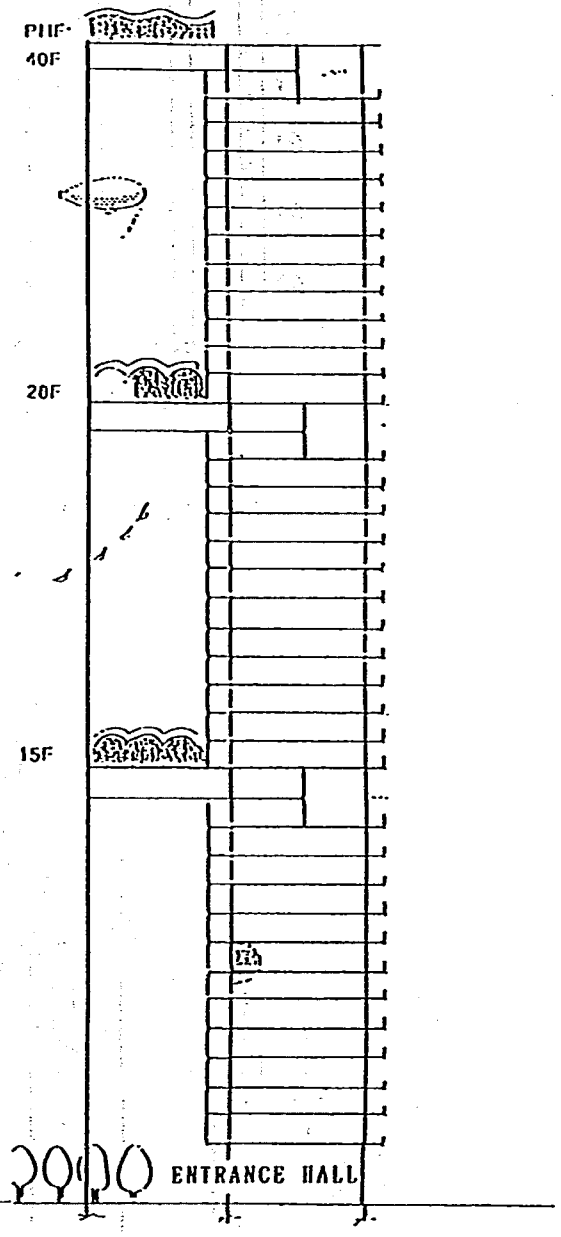


Fig. 5. 1 (b) Section of core wall type

Fig. 5. 2 (b) Section of wall type

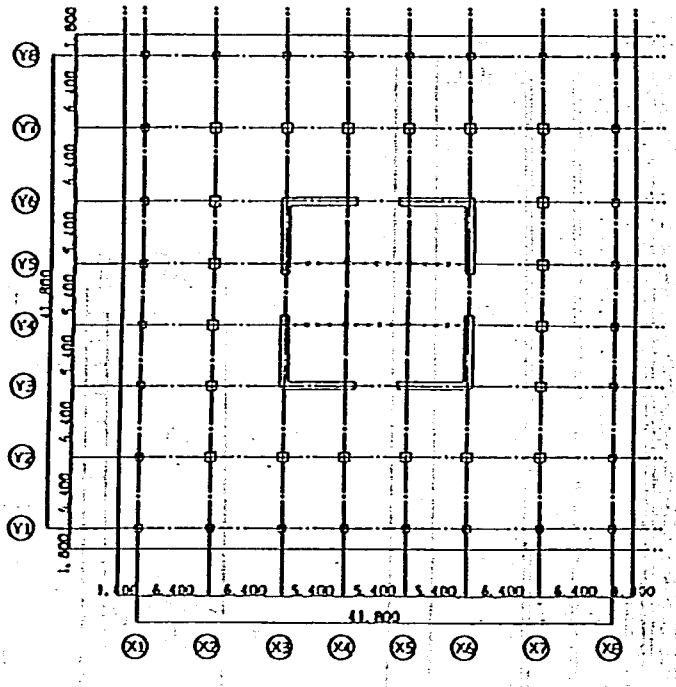


Fig. 5. 1 (a) Plan of core wall type

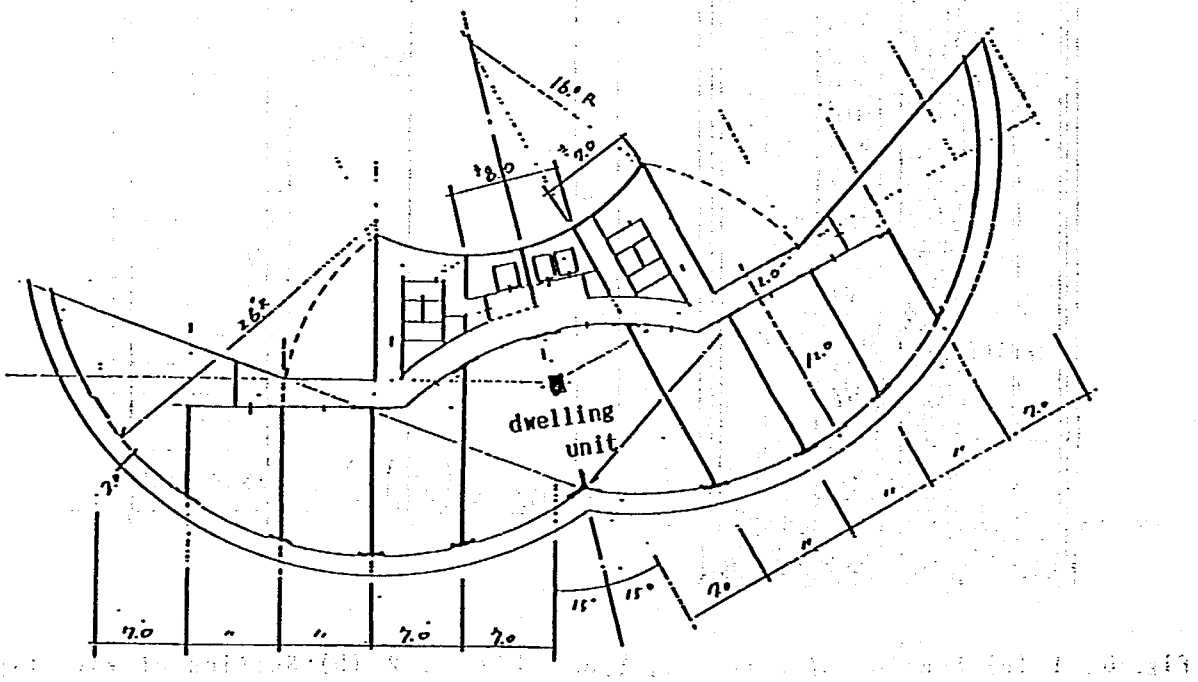


Fig. 5. 2 (a) Plan of wall type

S:1/2000

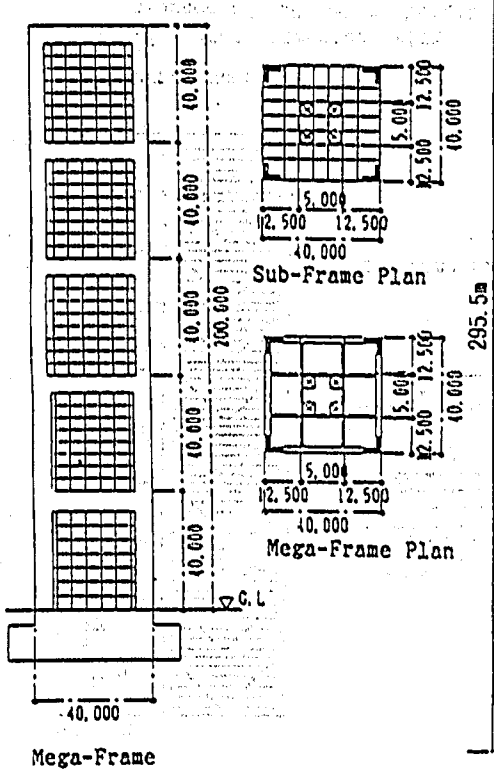


Fig. 5.3. Frame structure of 200 meters height

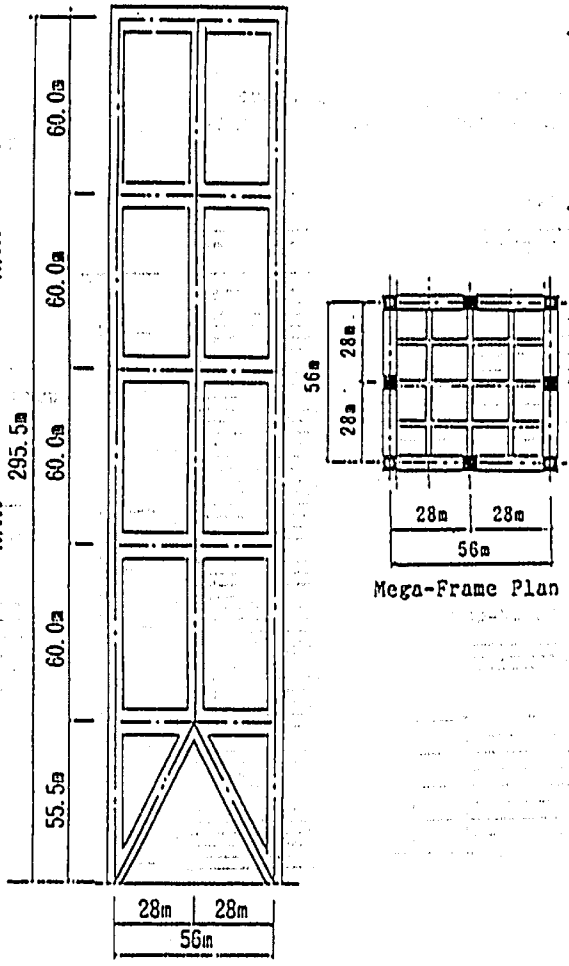
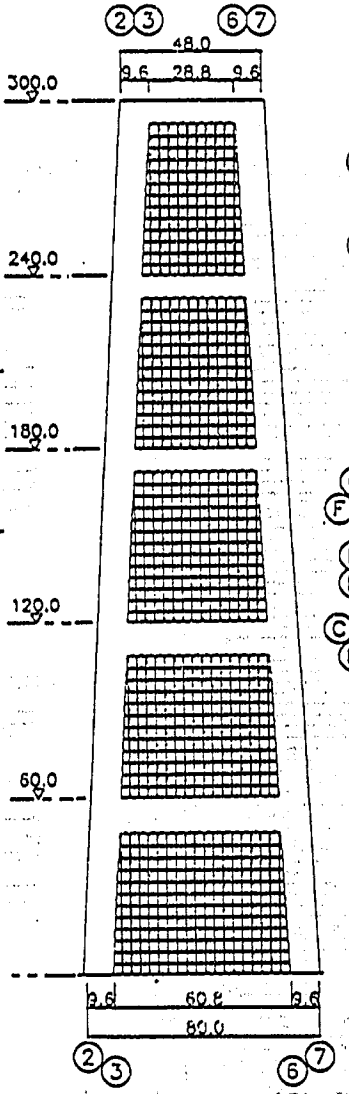


Fig. 5.4. Frame structure of 300 meters height



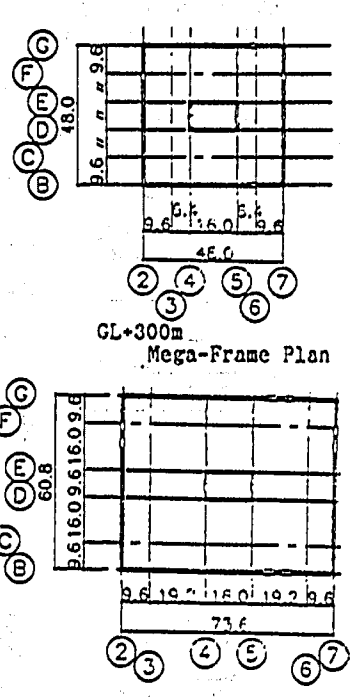
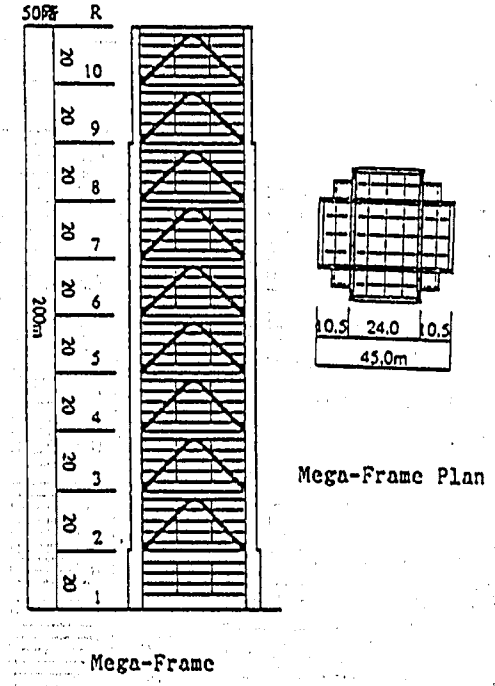


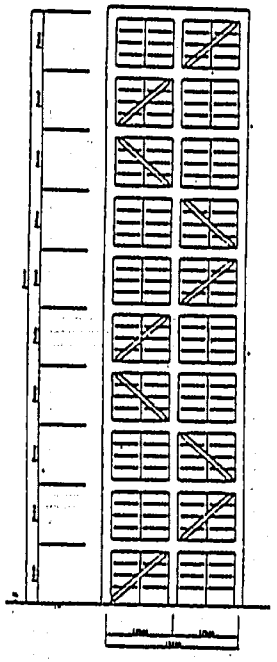
Fig. 5.5. Frame structure of 300 meters height



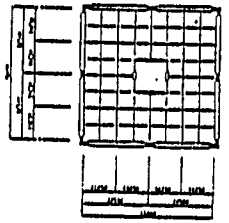
Mega-Frame

Mega-Frame Plan

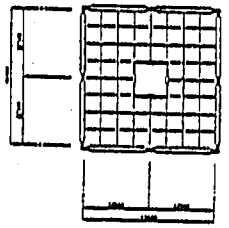
Fig. 5.6. Frame structure with K-brace of 200 meters height



Mega-Frame

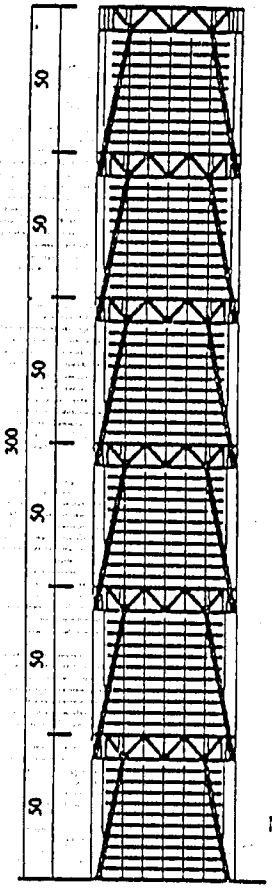


Sub-Frame Plan

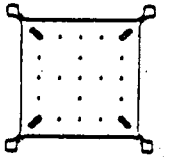


Mega-Frame Plan

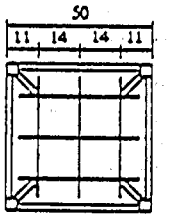
Fig. 5.7. Frame structure with one way brace of 200 meters height



Mega-Frame



Sub-Frame Plan



Mega-Frame Plan

Fig. 5.8. Frame structure with brace of 300 meters heigh

Dynamic Testing of Reinforced Concrete Frames

by

Pamalee Brady[1] and Steven C. Sweeny[2]

ABSTRACT

This paper discusses ongoing research to study the behavior of concrete frame structural systems typical of U.S. military construction, and ways of improving their resistance to seismic loading. One test program focuses on reinforced concrete frames with masonry infill. Frame stiffness and its interaction with the infill is investigated. In-plane and out-of-plane behavior is being studied, including tests on plain infill and infill repaired both before and after damage to the wall has occurred. The second test program focuses on evaluating the capacity of lightly reinforced concrete frames and the feasibility of retrofitting them with viscoelsatic dampers to improve their seismic performance. Both beam/slab/column and slab/column details typical of existing three story concrete frame barracks on the west coast are being tested with and without the dampers to study building response.

KEYWORDS: concrete frames, dynamic tesing, non-ductile, repair.

1. INTRODUCTION

Most permanent buildings on Army installations have been constructed, in accordance with seismic criteria applicable during their design, to provide some protection from seismic hazards. However, concern for the seismic safety of these buildings arises from experiences in recent earthquakes, where structures such as hospitals, communications centers, emergency facilities, and other essential and high-loss-potential facilities have been damaged by ground shaking or other seismic hazards. Since the early 1970s the Army has maintained seismic design criteria that are current with national codes and standards. After the 1971 San Fernando Earthquake the Army realized there may be hazards associated with their existing building inventory. Research was initiated to develop methods for evaluating these existing structures. In 1988 technical manual, TM 5-809-10-2, "Seismic Upgrading of Existing Buildings," was published to standardize the procedure for determining building vulnerability analytically.

The large inventory of built structures means that reducing seismic hazards in existing buildings is a complicated and costly issue that must be carefully addressed. The Army alone currently operates and maintains about 164 installations and major activities in the United States. More than 127,000 buildings are located on these installations. A phased plan involving inventory screening, hazard indexing,

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preliminary seismic safety evaluations, detailed seismic safety evaluations and development of seismic hazard mitigation plans has been initiated by each of the military services to reduce military seismic hazards. These activities have been accelerated in response to the National Earthquake Hazard Reduction Program Reauthorization Act of 1990.

The U.S. Army Construction Engineering Research Laboratory (USACERL) has performed preliminary seismic evaluations of buildings at several Army and Air Force installations in Seismic Zones 4 and 3. The objective of these evaluations is to determine whether a building provides an acceptable degree of safety, or whether a more extensive detailed investigation is required. At three Air Force bases in Seismic Zone 4, 56 buildings were identified as significant seismic hazards requiring detailed analysis and possible upgrading. Projected cost to bring these buildings up to current seismic standards is $118 M.

Among the vulnerable buildings, we identified several standard barrack designs used by both the Army and Air Force during extensive construction in the 1950s, 1960s and 1970s. Approximately 80 percent of the vulnerable facilities evaluated were of two common structural systems. These systems are 1) concrete frames without infill and 2) concrete frames with full infill, infill with openings, or partial height infill walls. The systems have performed poorly in earthquakes and the American Concrete Institute code requirements changed significantly in 1971 to address the identified problems. Details of the reinforced concrete frames constructed prior to these code changes are the primary reasons for poor behavior of the structural systems and are defined below:

1. Lap splices in columns occur at the base of the columns which for lateral loading is the region of maximum moment. These splices were also not well confined by tie or spiral reinforcing.
2. Positive beam moment reinforcing was discontinuous at the column.
3. Little or no reinforcement was provided in the beam-column joints
4. Construction joints are located above and below the beam column joint.

Additionally, infill walls alter the behavior of the concrete frames. Infills are much stiffer than the surrounding frame and tend to share or resist the lateral loads in the system. Research has shown that partial height walls produce a short column effect in the frames and therefore larger shears in the columns.

Because of the prevalence of these standard designs, the cost benefit of a study to accurately assess the capacity of the existing systems with the goal of developing upgrading solutions to improve their behavior is extremely high. Only recently has research addressed assessing the strength and ductility of existing buildings and dynamic tests have not been performed on these structural system types.

USACERL has undertaken two related research programs to investigate the seismic hazard associated with buildings having these structural systems. The programs evaluate the seismic resistance of the buildings' critical elements to determine the extent of any deficiencies so that practical strengthening concepts can be developed and their effectiveness assessed. One program is directed toward conceptually modelling a prototype structure of concrete frames with masonry infill walls and the other uses scale models of concrete frame prototype structures.

The prototype structures upon which the research is based are three story barrack buildings. There are four prevalent building configurations with similar structural systems; in plan they represent H, rolling pin, hammer head, and T shapes (Figure 1). A typical

building of the H configuration consists of three separate structures each with overall dimensions of 3566.2 cm by 1178.7 cm (117 ft-0 in. by 38 ft-8 in), (Figure 2). Each structure is six bays in the longitudinal direction and two bays in the transverse direction. Bay width is 586.7 cm (19 ft - 3 in.) and story height is 304.8 cm (10 ft). Exterior transverse frames contain full height infill walls. Interior longitudinal frames have no infill and exterior longitudinal frames have partial height infill walls. The balance of this paper discusses the dynamic testing programs to evaluate these reinforced concrete frame systems.

2. Concrete Frames with Masonry Infill

2.1. Objective.

The objective of this project is to conceptually model a transverse frame with masonry infill to study dynamic behavior, failure mechanisms, and the effects of repair and rehabilitation. Two small scale concrete frame models are designed to be used in a multi-year research effort. A strong frame, with larger beam and columns and ductile detailing, and a weak frame with smaller elements and non-ductile detailing will be tested. The test program for each frame is being conducted in three stages. The first stage is determination of the behavior of the bare frame, including the effects of initial cracking and column post-stressing on the behavior of the model. The second phase is determination of the effect of the infill on frame behavior and failure mechanisms of the infill both in-plane and out-of-plane. The final phase is determining the effect of repair and retrofit of the infill on system behavior.

2.2 Test Specimen.

The strong frame is one-half scale of a frame designed in accordance with Chapter 21 of ACI 318-89 (Figure 3). The columns are 15.2 x 15.2 cm (6 in x 6 in), and 88.9 cm (35 in) from base to the center of the beam-column joint. Longitudinal reinforcement consisted of 6 - #3 grade 60 deformed bars. Stirrups are formed from 6 gauge basic wire. A 1.6 cm (5/8 in) brass tube runs down the center of the column to accommodate the post-stressing cable. The beams are 12.7 cm wide by 15.2 cm deep (5 x 6 in), 137.2 cm (54 in) center to center of the beam-column joints. Each beam has 3 - #3 grade 60 deformed bars top and bottom and 6 gauge basic wire stirrups. In addition, the beam has an unsymmetric top flange 5.1 cm (2 in) thick extending 24.1 cm (9.5 in) from the edge of the beam. Both beams and columns extend 15.2 cm (6 in) beyond the joint to provide for continuity of the reinforcement. Concrete is a 2.7 kN/cm^2 (4000 psi) mix with a maximum aggregate size of .95 cm (.375 in). The weak frame has the same clear dimensions for infill as the strong frame with smaller beam and column dimensions and non-ductile reinforcing. The columns will be 12.7 x 12.7 cm (5 x 5 in) with 4 - #3 bars and 6 gauge wire stirrups. Final design of this frame has not been completed.

To facilitate dynamic testing on the shaketable two identical frames in parallel are tested (Figure 4). The two frames are coupled with a reinforced concrete slab. The slab also provides for attachment of 25.4 kN (5.7 kips) additional mass during the in plane tests. Diagonal tension cross bracing is provided at the ends between the two frames to prevent torsion during in plane and to provide all lateral support out of plane. Columns are post-stressed during testing to simulate dead load stresses.

Instrumentation consists of an array of strain gauges, displacement transducers, and accelerometers. One frame is heavily instrumented and the second frame lightly instrumented to verify similar response between the two frames. Strain gauges are mounted on longitudinal bars at the base of the columns and in the beam near the beam-column joints. Two gauges, at 2.5 and 10.2

cm (1 and 4 in) from the base or joint, are mounted at each location. At each strain gauge pair displacement transducers monitor cracking and curvature. Global displacement transducers are attached from a fixed reference frame to the model at the base, mid-height of the column, mid-depth of the beam, and at mid-width of the slab. Displacement transducers within the shake table actuators are used to measure table displacements. Accelerations are measured at the base, mid-height of each column, mid-span of the beam, center of the slab, the top of the mass plates and at the shake table. Accelerations will also be measured at various locations on the infill when tested.

2.3 Test Procedure.

Dynamic testing was conducted on the USACERL Biaxial Shock Test Machine (BSTM). Three tests are used to measure the behavior of the model under dynamic loading; a) a random frequency sine wave, b) free vibration, and c) a time scaled El Centro record. The frame is initially subjected to a low level random sine wave motion to determine the natural frequency and to approximate the dynamic amplification of the base motions. The random input is increased until a significant response is measured. This input level is used for all subsequent random vibration tests. Next, free vibration of the frame is measured. The model is excited with a sine wave at a frequency slightly higher than the natural frequency. Input motions are increased again until significant response is measured. The table motion is then stopped and the structure allowed to oscillate freely. The final test is an El Centro record with a time scale factor of 1.414. For the bare frame only, each of the three tests are conducted at four levels of post-stress applied to the columns. The frames are tested at post-stressing forces of 13.3, 26.7, 40.0, and 53.4 kN (3, 6, 9, and 12 kips). For frames with infill, only the 53.4 kN post-stressing will be used.

The frame or infill are damaged by subjecting the system to increasingly stronger earthquake ground motions until significant cracking develops. It is the intent to destroy the infill, not the frame, and to change the response by cracking the frame or infill without yielding the reinforcement. In order to achieve the necessary forces to crack the system, the time scaled El Centro record had to be filtered to remove the large displacements induced by low frequency motion. A high pass filter with a cutoff frequency of 2 Hz was used to produce the necessary seismic record. This input motion was used at increasing levels until significant damage was observed. Random vibrations are input after each test to measure change in model response. After cracking the bare frame, the model is subjected to the same three tests at each of the four levels of column stress as before cracking to evaluate the effects of cracking on bare frame response.

2.4 Initial Observations.

The Pre-cracking response behavior was as expected for a linear elastic system. The natural frequency for all levels of post-stress remained at approximately 11.6 Hz. The structure showed very good single degree of freedom response. Damping was approximated by both log decrement method from the free vibration curves and the half-power method from the random vibration curves with good agreement. Damping for the uncracked model for all levels of post-stressing averaged of 2.2%. Response to the El Centro record was also consistent for all levels of post-stressing.

The structure was tested with the filtered El Centro record at four levels. The first two tests gave a peak acceleration response of .43g and 1.08g respectively with little indication of damage. The third test showed a peak acceleration of 1.62g and a peak displacement of 3.0 mm (0.12 in). Random vibration and free vibration tests indicated a significant shift in the natural

frequency. The fourth test was performed at an input level 25% higher than test 3. Peak acceleration was measured at 1.72g with a peak displacement of 4.8 mm (0.19 in). Again, random and free vibration tests indicated a significant change in natural frequency as shown by the transfer function magnitudes in Figure 5, and the model was considered sufficiently damaged.

Initial cracking on the structure had an effect on both stiffness and damping. Damping calculations by both log decrement and half-power method show that the damping coefficient change from 2.2% to 4.3%. Working backwards through the equations of motion for a damped system using the damping and natural frequencies measured in the tests show that the stiffness of a single frame changed from 82.0 to 44.8 kN/cm (46.8 to 25.6 kips/in).

After cracking, the model was again subjected to the random vibration, free vibration, and unfiltered El Centro tests at the same input level run on the pre-cracked model at the four levels of post-stressing. The post-cracking behavior was much different then the pre-cracked behavior. A slight increase in natural frequency with increasing post-stress was observed. Again, both the log decrement and half-power methods for computing damping give good correlation. While the values for damping of the cracked frame vary, there is no correlation with post-stress values. The average value for the coefficient of damping in the cracked frame is 4.3%. Calculation of frame stiffness gives more consistent results. The drop in stiffness after cracking is significant, and both pre and post-cracked results indicate a slight trend towards increased stiffness with increased post-stressing, which was expected.

Most important for structural engineers is how the shifts in stiffness and damping affect the behavior of the structure during an earthquake. Typically, increased damping is considered beneficial for energy dissipation. In this case however, the effect of the decreased natural frequency caused an undesirable shift towards the predominant frequencies of the El Centro N-S seismic record. The result was a significant increase in peak acceleration and displacement of the cracked model.

3. Lightly Reinforced Concrete Frames

3.1 Objective.

The objective of this research program is to evaluate the effects of the vulnerable details identified above on overall behavior of nonductile concrete frames by experimentally assessing the capacity of scale models drawn from two prototype structural systems. With this knowledge we may better assess the complete capacity of the frame systems. Additionally we will test the effectiveness of viscoelastic (VE) dampers to improve the behavior of concrete frames under earthquake loading. The program is being worked cooperatively with Professors Wood and Foutch at the University of Illinois. It will be conducted in three phases. In the first phase, currently ongoing, tests of scale model frame subassemblages will be conducted to study their behavior and assess their capacities. A test of a scaled three story nonductile structure will be conducted in phase two. The third phase of the project will be devoted to technology transfer. Conventional strengthening schemes will be compared with upgrading using VE damper technology and the cost-effectiveness of different solutions evaluated. The following sections of this paper will focus on the first phase of the program, assessing the capacity of the existing frames through testing of frame subassemblages.

3.2 Test Specimens.

A total of four specimens representing the critical second floor joint of two lightly

reinforced concrete frame systems will be constructed. Two 6/10th-scale models of a typical beam/slab/column joint have been tested on the USACERL Biaxial Shock Test Machine. Two 1/2-scale flat plate/column joint specimens will be tested this summer on the shaketable. Geometric similitude between the prototype and test specimens was maintained. Subassemblage members were reinforced to produce the same demand/capacity ratios as those of the prototype elements.

The beam/slab/column test specimen measures 243.8 cm by 284.5 cm (8 ft-0 in. by 9 ft-4 in.) in plan and is 182.9 cm (6 ft) tall. The column is 18.4 cm (7.25 in.) square. The slab is 7.6 cm (3 in.) thick and the beam extends 16.5 cm (6.5 in.) below the slab. (Figure 6) Reinforcement details of the subassemblage follow ACI 318-51 code provisions. The longitudinal column reinforcement of the model consists of 4 - #5 bars; the bars are spliced 20 d_b at the top of the slab. Transverse reinforcement is provided by #2 ties spaced at 17.8 cm (7 in.) o.c., the minimum column dimension, along the height of the column. The ties have 90 degree hooks. Longitudinal beam reinforcement is 4 - #3 bars top and bottom. Bottom bars were discontinuous and extend 8.9 cm (3-1/2 in.) into the column. This extension provides similitude with the positive moment capacity of the prototype structure based on available bond strength. Where bars were continuous in the prototype structure, such as beam centerlines and midheights of columns, bars were hooked in the test specimen to ensure development. The transverse reinforcement in the beams was provided by #2 bar open stirrups spaced 7.6 cm (3 in.) o.c. arranged typical of gravity load conditions; they did not extend the full length of the beam.

The flat plate/column specimens have nominal dimensions of 304.8 cm by 304.8 cm (10 ft-0 in. by 10 ft-0 in.) and are 182.9 cm (6 ft) high. The column is 18.4 cm (7.25 in.) square and the slab is 9.2 cm (3-5/8 in.) thick. (Figure 7) The reinforcement consists of two mats of #2 perpendicular bars at the top and bottom of the slab. Standard spacing within the column strip was 7.6 cm (3 in.) o.c. and 12.7 cm (5 in.) o.c. in the middle strip. Column reinforcement was the same as that of the beam/slab/column model.

To maintain gravity stresses and produce the appropriate reactive mass in the models their mass was augmented by steel weights. A total of 26.6 kN (6 kips) was added to the beam/slab/column model, 13.3 kN (3 kips) at the top of the column and 13.3 kN (3 kips) on the slab. Inflection points of the columns and beams were assumed to occur at their mid-height and mid length respectively. The test fixture provides for rotation at these points. (Figure 8)

The joint region of each model is instrumented to measure curvature of the beam/slab, curvature of the column above and below the beam/slab and shear in the joint and column. Global displacements are measured from a fixed reference frame at the top of the column, slab and base of the column. Accelerations are measured at the top of the column, slab and base in the direction of load and at the slab in the transverse direction.

3.3 Test Procedure.

White noise and free vibration tests are conducted on each model to determine the characteristic frequency and damping of the model at various points in the test program. The El Centro 1940 N-S record, time scaled by a factor of 2.0, is used to excite the model at increasingly greater magnitudes to produce damage in the model. The specimens are tested to failure. The effects of structure geometry and configuration, yield and collapse capacities and energy dissipation characteristics are evaluated. A viscoelastic damper is incorporated into the second model of each type and the influence of the increased viscous

damping on the elastic and inelastic dynamic response of the structure investigated. Parametric studies will assess the differences in performance of the models with and without dampers.

3.4 Initial Observations.

Tests of the beam/slab/column specimens with and without dampers have been conducted. The failure sequence of the lightly reinforced joint was as expected; first, the discontinuous positive reinforcement slipped from the joint. This was associated with a period shift in the model. As the tests continued a hinge formed at the base of the upper column, indicating slip in the column lap splice, and the joint region of the beam/column deteriorated increasingly. The tests were stopped to prohibit failure of the now unstable upper column. Damage to the joint also made the structure extremely flexible. Capacity of the joint was generally as predicted.

A preliminary review of the data from the tests with the viscoelastic dampers shows reduced accelerations and displacements of the structure as compared with the specimen without dampers for the same input excitation. After testing with the dampers, they were removed from this model and it was tested at the peak excitation of the first model. The failure mode was similar but more dramatic.

Further study of the data from both tests is required to evaluate the response of the subassemblages, the improved response due to dampers and the effect of the test sequence on model response. Similar tests will be conducted this summer on slab/column subassemblages with and without dampers. More detailed analyses will be compared with experimental results. The work summarized above constitutes the first phase in a three phase program. Results of this work will be used to develop a better understanding of the behavior of the structures and to develop simplified design approaches for upgrading lightly reinforced concrete frames to improve their seismic behavior.

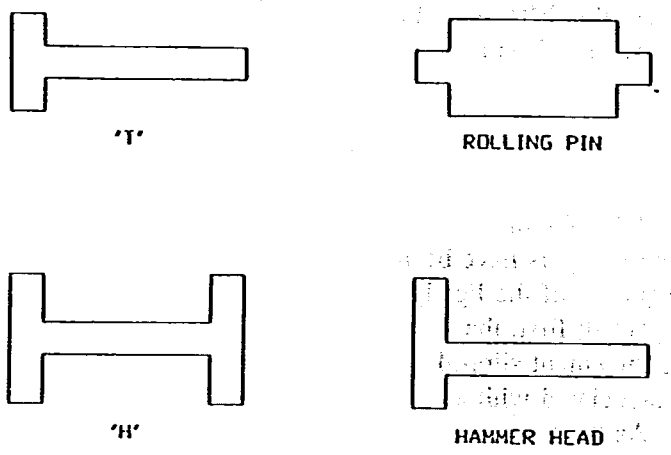


Figure 1. Standard building configurations.

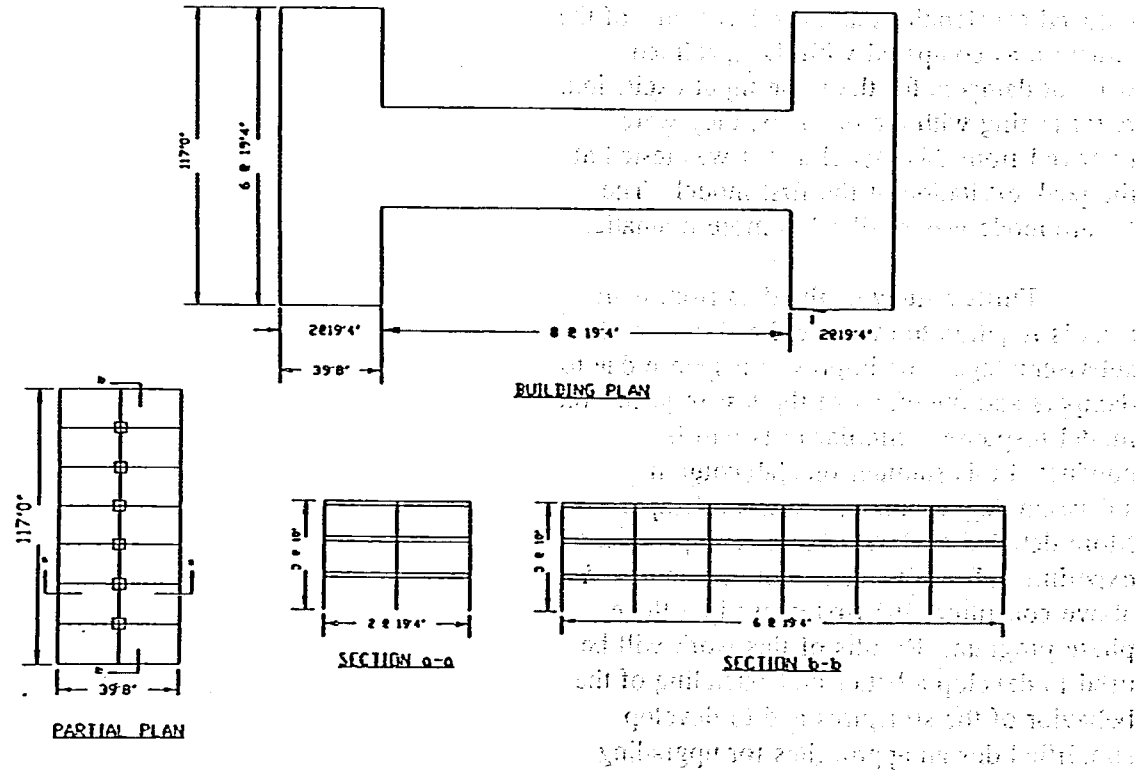


Figure 2. Typical "H" building dimensions.

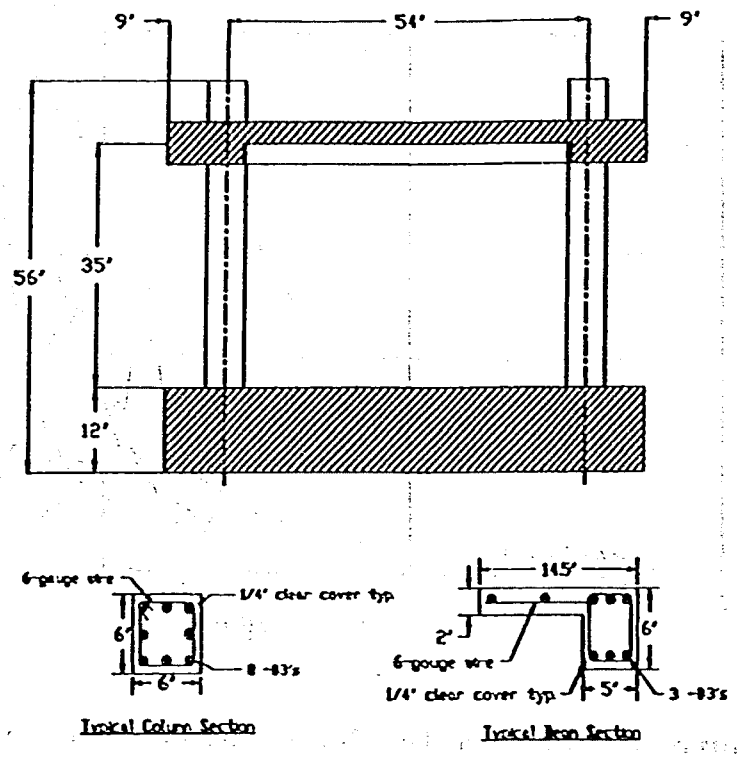


Figure 3. Strong frame dimensions and details.

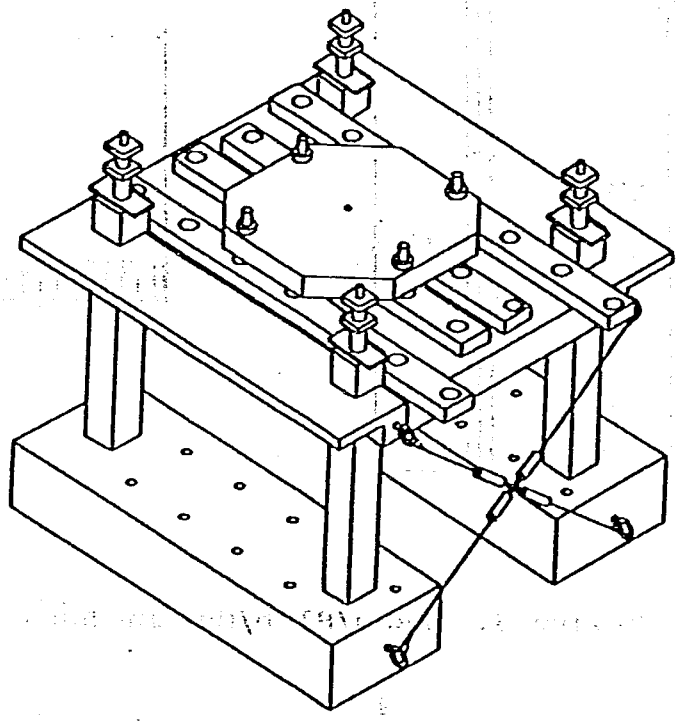


Figure 4. Assembled model with masses and bracing.

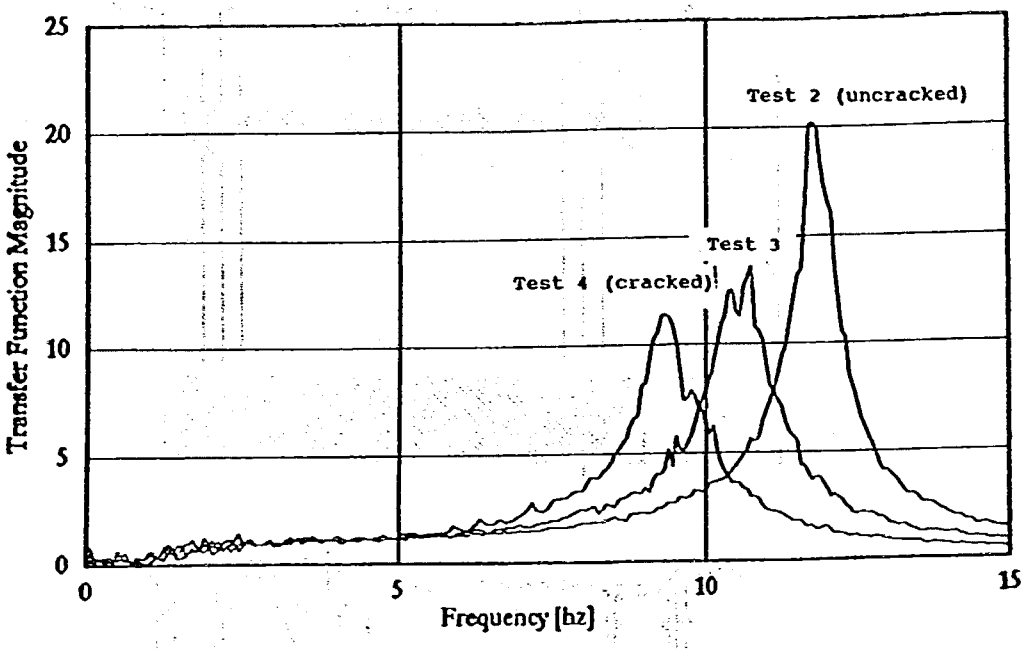


Figure 5. Change in the response due to cracking.

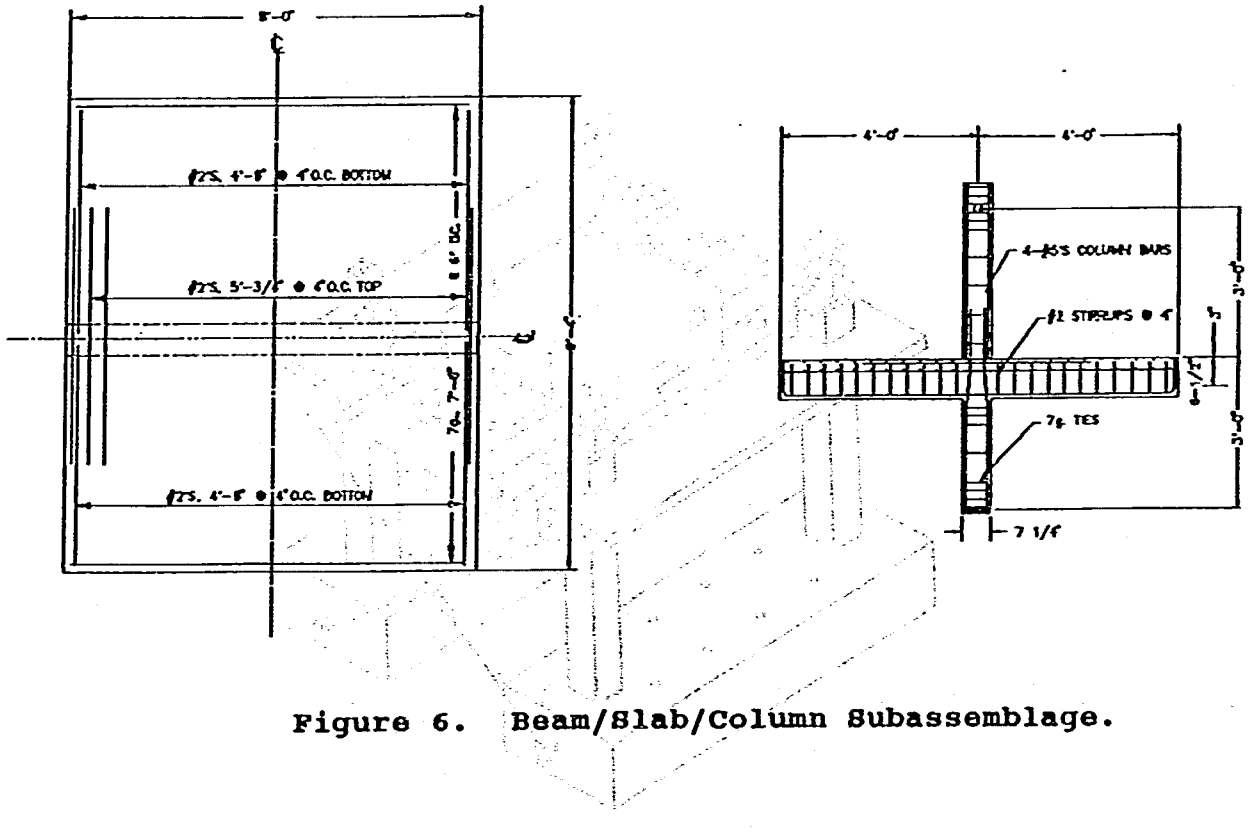


Figure 6. Beam/Slab/Column Subassemblage.

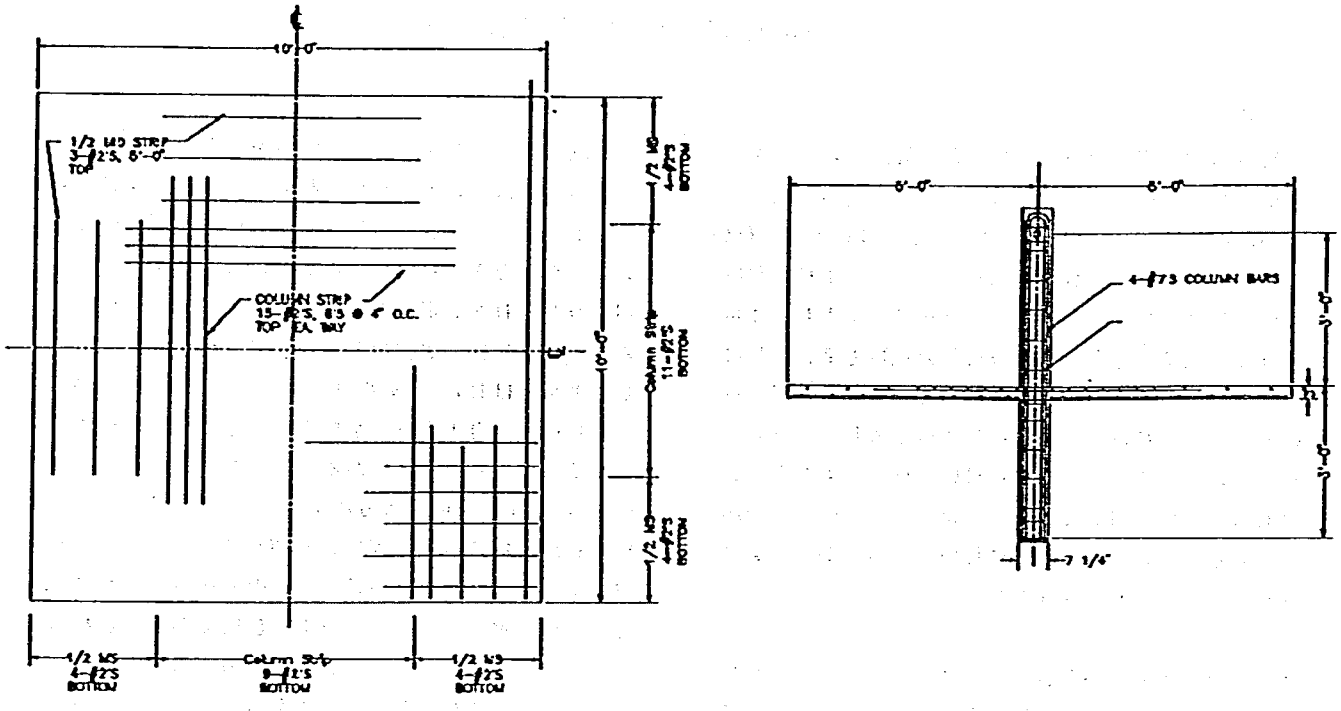


Figure 7. Slab/Column Subassemblage.

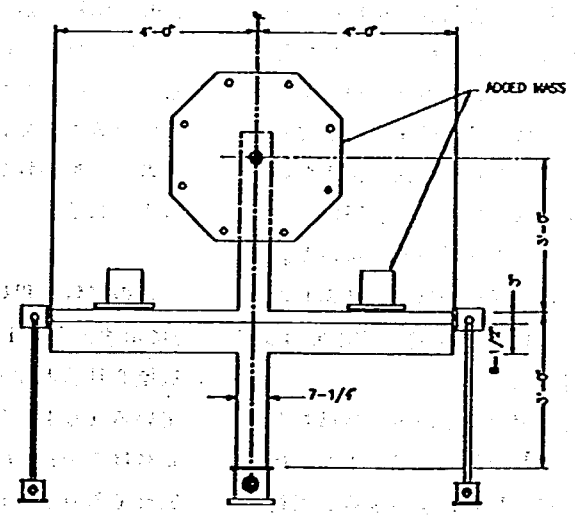


Figure 8. Test Configuration.

RC柱の振動台破壊実験

笹崎親宏[3]　小川信行[2]　御子柴正[4]　大八木則夫[1]

（要約）

防災科学技術研究所大型振動台が導入したデジタル制御方式に付いて述べ、導入に際し、実施した縮尺率1/3程度の鉄筋コンクリート柱の破壊実験の概要に付いて示す。この破壊実験では鉄筋コンクリート柱の剪断補強筋（フープ筋）の間隔をパラーメータとし、４種の柱の試験を行った。また静的加力試験も２種の柱について行った。これら試験に於いて、破壊モードに付いて、静的試験と動的試験結果に若干の相違が見られた。また、荷重・変形曲線に付いては動的ものと静的なものはほぼ近似している。また、Ｐ－δの効果が顕著に見られた。また、鉄骨に付いてであるがこれをコンピュータシミレーションで確認した。

（キーワード）　振動台、デジタル制御、アナログ制御、鉄筋コンクリート、振動台破壊試験、静的加力試験、Ｐ－δ効果

1. はじめに

振動台は免震・制震、液状化、各種耐震設計コード確認等に使われている。これら耐震試験の中には地震動の高い再現性を求めるものがある。弾性域の試験等の再現可能なものでは試加振を行い入力修正ができるが、液状化試験、構造物の破壊試験では試加振ができず、従来の逆周波数特性を用いた制御方式が使えない。防災科学技術研究所の大型振動台は、最近、DSPボードを使い、振動台の制御特性のさらなる改善を試みた。この改善制御方式（ここではデジタル制御と呼ぶ）の効果を見るため模型（縮尺率1/3程度）の１層鉄筋コンクリート柱の破壊実験を行った。

鉄筋コンクリート柱の終局耐力を求める試験は数多く行われているが、ほとんど静的加力試験、もしくは擬動的試験によっている。静的試験による場合、振動方程式の速度項を、無視あるいは、仮定しなければならない。速度抵抗すなわち減衰定数は加振レベルの上昇により増加するものであり、加振中に於いても一定のものではないと考えられ、できるならば動的な試験を行うべきである。ここでは、上記の振動台破壊実験により実際の動的応答を起こし、鉄筋コンクリート柱の終局耐力ならびに破壊モードを調べることを目的とした。

2. 試験

試験は鉄筋コンクリート柱の種類に合わせ４回行った。内２回は従来のアナログ制御、あと２回はデジタル制御で加振した。この他にも鉄骨柱の試験も行っている。今回導入したデジタル制御のためのシステムは、パソコンとＤＳＰボードから構成され、デジタル制御とアナログ制御の選択はキーボードで行う。アナログ制御の場合はH.Sデジタルコントロラー部をバイパスして、単に振動台入力波形を発生器として、このシステムを使っているケースである。制御システムの概要をFig.1に示す。今回、入力に使った地震波形は１９６８年十勝沖地震の八戸ＥＷで、0.4Hz～50HzのFFTによるバンドパスフィルターを通したものである。Fig.2はその振動台加速度波形の主要部１０秒間である。目標加速度波形を最上部に、アナログ制御の波形を２段目、デジタル制御を最下部に示してある。この図については両制御とも５００GALの最大加速度を目標にした。これら加振が振動台サーボ弁容量限界（振動台速度限界75cm/sec±5%）に近い加振であるがアナログ加振は良い再現性を示している。デジタル加振はアナログ加振ほどの再現性を示していない。このデジタル制御方式は系の制御特性を改善し、かつロバスト性を系に付与するのが目的であり、見かけ上の油柱振動数10Hzから13Hzに改善されている。今後、コントローラーにサーボ弁の非線形特性を考慮するなど、一層の改善が必要と考えられる。

この試験で用いた鉄筋コンクリートの試験体はFig.3に示すように１層の２ｍ×２ｍのボックスフレームで、破壊を目的とするため２７．８tonの屋根部重量を有している（なお、振動台の重量は180tonであり、試験体重量はその1/5である）。屋根部の梁は鉄骨製の高さ450㎜であり、柱に比べ充分に高い剛性を有する。柱は４本あり、130㎜角の鉄筋コンクリートで長さは850㎜

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であり、両端に固定部を有している。屋根梁及び基礎梁への固定は両部とも4本の長尺ボルトで行った。1本の柱に加わる軸力は6.9tonで応力にすると40kg/cm²である。柱主筋は16mm異形鉄筋を4本で、フープ筋は6mm異形鉄筋である。4種の柱のフープ筋間隔はθ50mm、θ105mm、θ170mm、θ280mmである。コンクリートの圧縮強度試験は230kg/cm²～300kg/cm²、鉄筋引張試験の弾性限界値は3600kg/cm²であった。なお、屋根重量部は加振により落下する。これを受けとめるため鉄骨フレーム（SAFETY FRANE）を試験体内側に組立、このフレームの上に緩衝材を敷いた。落下量は5cmから10cmである。

計測は屋根部の加速度と、柱の変形量を求めるため、柱脚部と同じ動きをするSAFETY FRAMEと屋根部の相対変位について行った。なお、変位計はリールタイプのポテンショである。ビデオによる記録を各ケースに於いて行った。

静的加力試験では引張方式のオイルジャッキを屋根フレームの左右に2台づつ取付けた。なお、27.8tonの屋根荷重は掛かったままである。繰り返し荷重のレベルは変形角にして1/1000、1/200、1/100、1/50、1/20と増加させ、最終的には崩壊させた。

3. 静的加力試験結果

θ280mmとθ50mmの2種の鉄筋コンクリート柱に付いて静的加力試験を行った。θ280mmの試験では、1/100の加力でクラックが柱中央部に入り、1/50の加力で斜めの剪断クラックの幅が広がり、第1構面の柱2本の内部鉄筋が座屈し、鉄筋とコンクリートは分離した。

θ50mmの試験では柱頭・柱脚部に1/50、1/25の加力で曲げクラックが入り、1/5の加力で圧壊が急速に進み、ついに、屋根重量部がSAFETY FRAMEに着座した。θ280mm、θ50mmの荷重・変形曲線をFig.5、6に示す。なお、縦軸の値は柱1本当たりの荷重である。θ280mmの試験終了後の4本の柱の状況をPhoto 2に示す。

4. 振動台試験結果

試験体の固有振動数は、4種の柱で僅かに異なるものの4．5Hz程度であった。減衰は5％弱である。崩壊寸前の固有振動数は約3Hzになっている。4種の試験体とも加振波形の主要部に入った所で崩落している。Fig.4，7は各試験の柱フープ筋の配筋図と応答加速度と水平層間変位のリサージュである。加速度応答に屋根重量部の質量を乗ずれば荷重・変形曲線に対応する。

θ280mmのケースでは、初期剪断クラックが急激に成長し、3本の柱は柱頭部近くで折れ、残りの1本は柱脚近くで折れた。この柱クラック状況をPhoto 3に示す。

θ170mmのケースでは、初期クラックはθ280mmの場合と同じように剪断タイプのものであった。しかし、θ280mmのときより半波だけ僅かになが く持ちこたえ、逆方向に変形した。この柱クラック状況をPhoto 4に示す。

θ105mmの場合では、θ170mmのときよりなが く持ちこたえた。柱頭・柱脚に曲げクラックが生じ、こららクラックが成長し、被りコンクリートの剥離を伴って、崩壊した。この柱クラック状況をPhoto 5に示す。

θ50mmの場合は、θ170mmのときよりさらになが く持ちこたえ、明確な荷重・変形曲線を描くのが確認できた。柱頭・柱脚に曲げクラックが生じ、多く見られる崩壊形を示した。この柱クラック状況をPhoto 6に示す。

5. P－δ効果

この試験に於いては、Fig. 7の加速度・変形曲線を見て分かるように、屋根重量の影響が如実に現れている。すなわちP－δ効果である。鉄筋コンクリートの振動台試験では充分なループを描く前に崩壊してしまったので、予備試験として行った鉄骨柱で行った振動台実験の結果に付いて考察する。この試験で用いた柱はH形鋼（100mm×100mm×12mm×8mm）で高さ1mである。その他の条件は鉄筋コンクリートの場合と同じである。

ここで梁剛性無限大としたFig. 8の単純モデルを考える。このモデルの方程式を立てると次のようになる。

$$m(\ddot{v}+\ddot{v}_g)-m(\ddot{u}+\ddot{u}_g+g)\frac{v}{h+u}+k_s v=0$$

$$m(\ddot{v}+\ddot{v}_g)\frac{v}{h+u}+m(\ddot{u}+\ddot{u}_g+g)+k_a\left(\sqrt{(h+u)^2+v^2}-h\right)\frac{\sqrt{(h+u)^2+v^2}}{h+u}=0$$

ここで、第1式のk，vの項を柱の水平復元力特性f（v）で置き換える。このf（v）をFig. 9のようにモデル化する。この式でシミレーションを試みる。ここでX1=1cm，X2=1.3cm，BK1=9ton/cm，BK2=7ton/cm，BK3=0.2ton/cm，BK4=5ton/cm，h=80cm，m=0.027ton·sec²/cm，k_s=750ton/cmとした。減衰は上下応答に対し

0.01、水平応答に対し0.04とした。Fig. 10はP－δを考慮したシミレーションケースであり、Fig. 11はhを無限大とした場合で、P－δを無視したケースに対応する。振動台入力として上下入力があるが、これはノイズとして生じたものである。この2図を見ても分かるようにこの試験結果を説明するには屋根重量を考慮にいれたP－δ解析を行う必要があることが分かる。

6. 結論

この試験によりつぎの結論を得た。

・鉄筋コンクリート柱の変形能ならびにエネルギー吸収能はフープ筋の数を増すことにより改善されることが動的実験により確認できた。また、

・言いるP－δ効果による耐力の見かけの低下が確認された。

・デジタル制御は振動台周波数特性を改善するが、波形精度の点で努力が必要である。

謝辞

当試験に関し、東京大学生産技術研究所柴田碧教授の貴重なご助言をいただいた。また、実験実施、試験体設計製作に於いて東急建設（株）に協力をいただいた。ここに深く謝意を表します。

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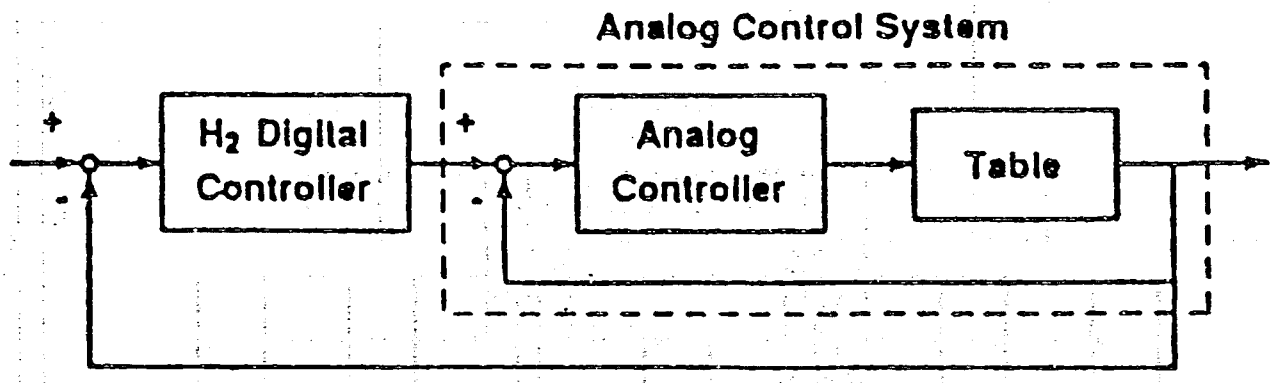


Fig. 1 Outline of Shaking Table Control

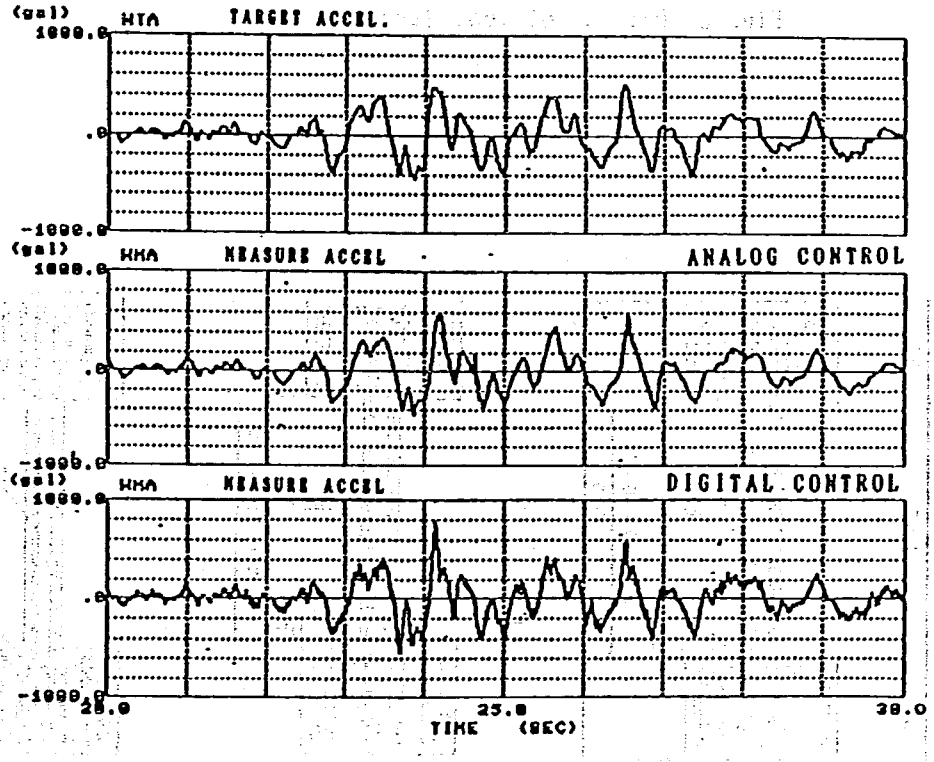


Fig. 2 Shaking Table Responses

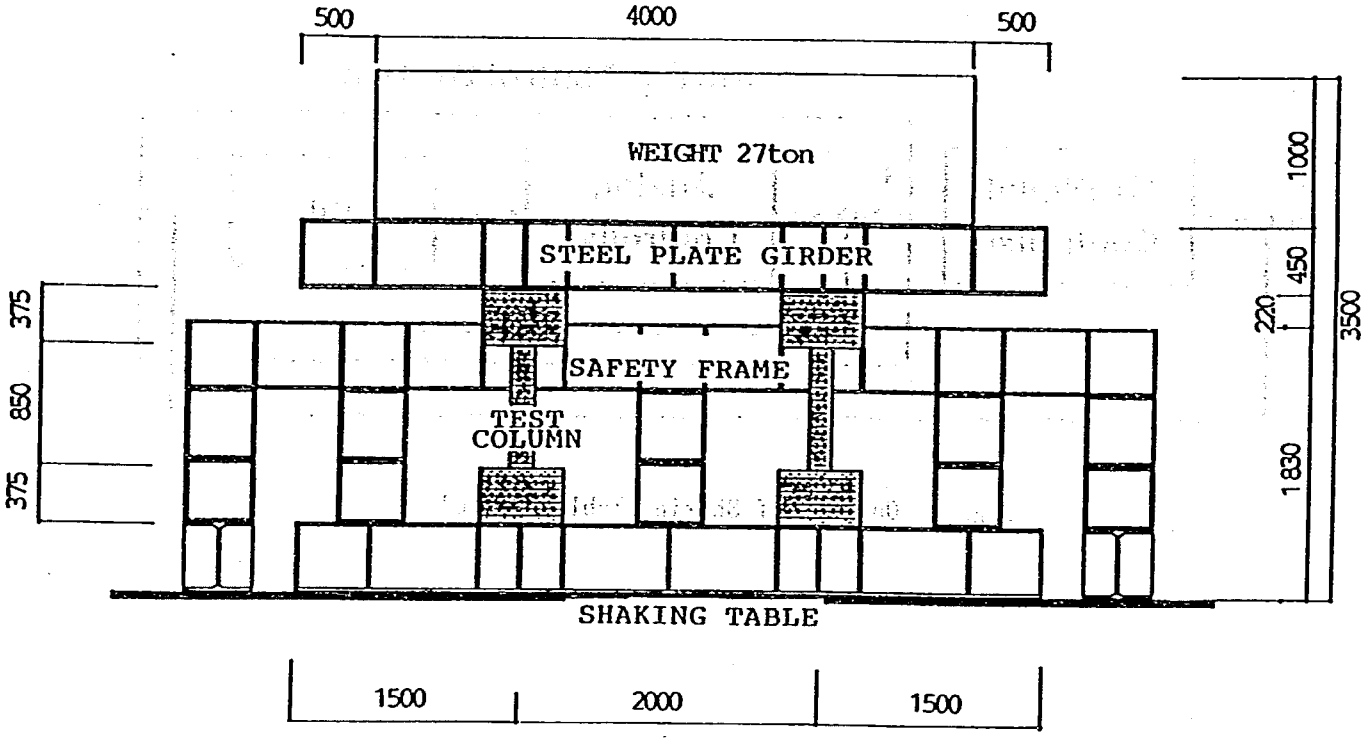


Fig. 3 Outline of Test Structure

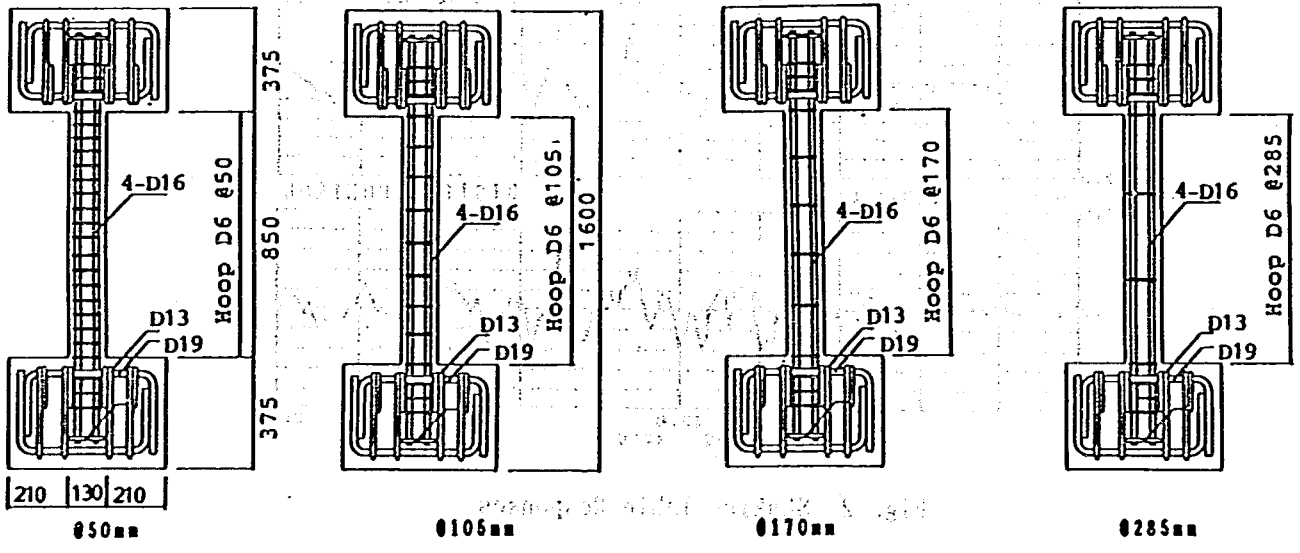


Fig. 4 Reinforcement Bars

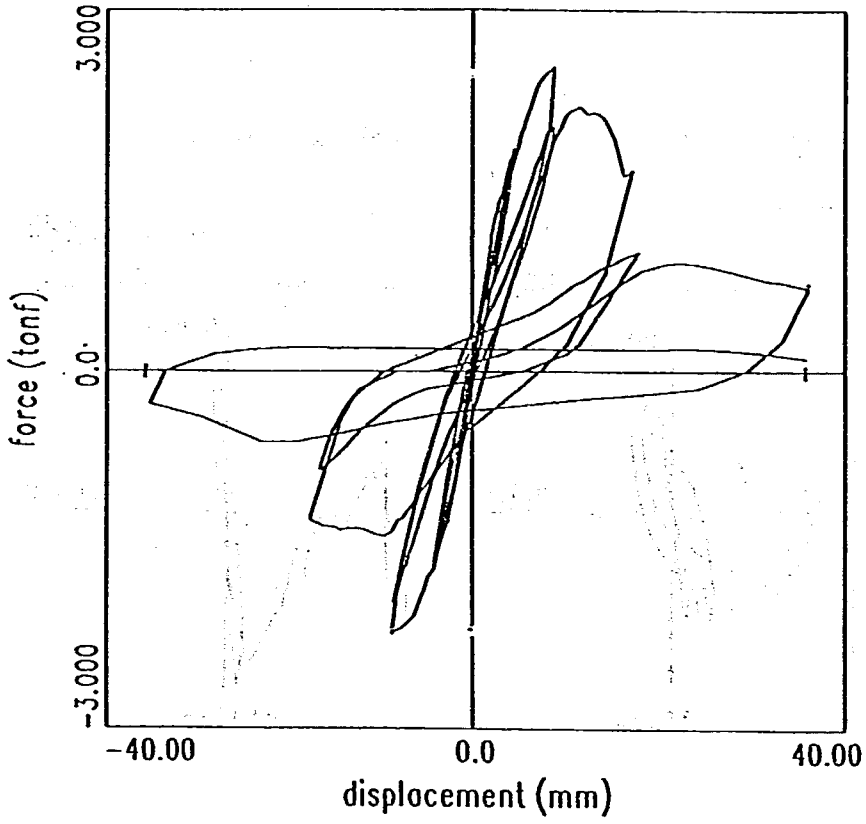


Fig. 5 Ø285mm Static Loops

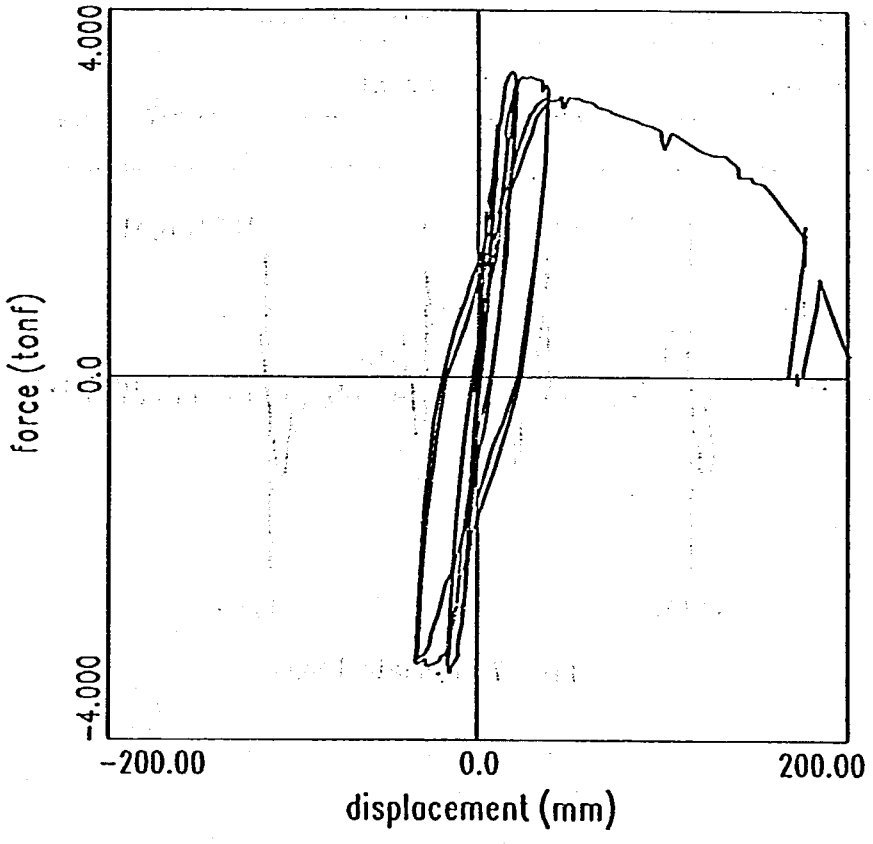
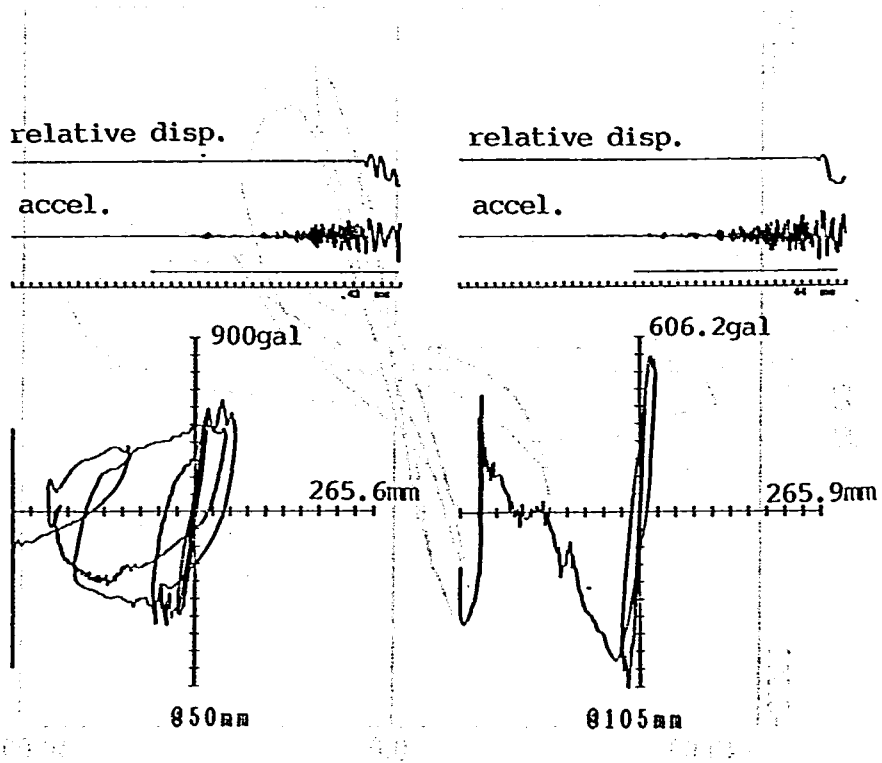


Fig. 6 Ø50mm Static Loops



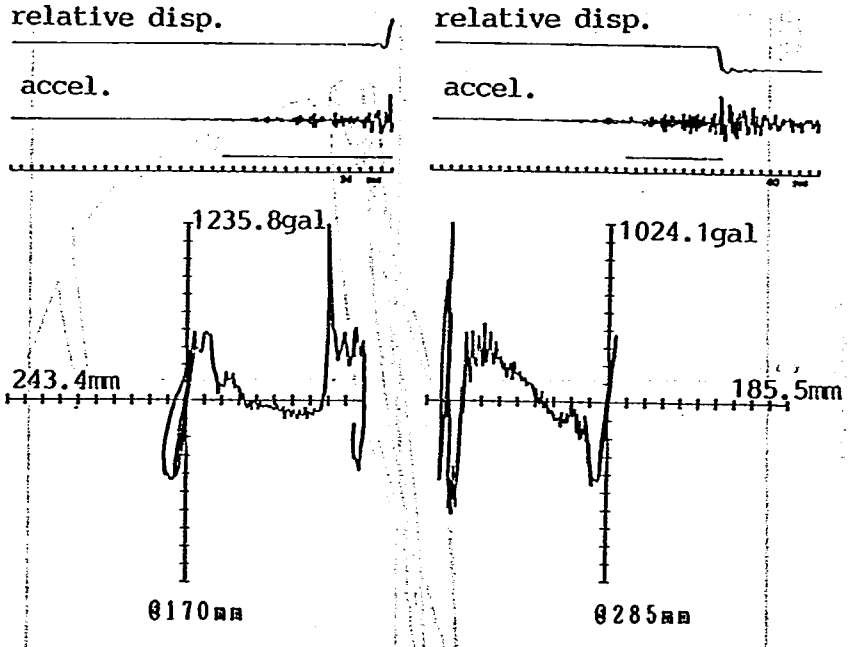
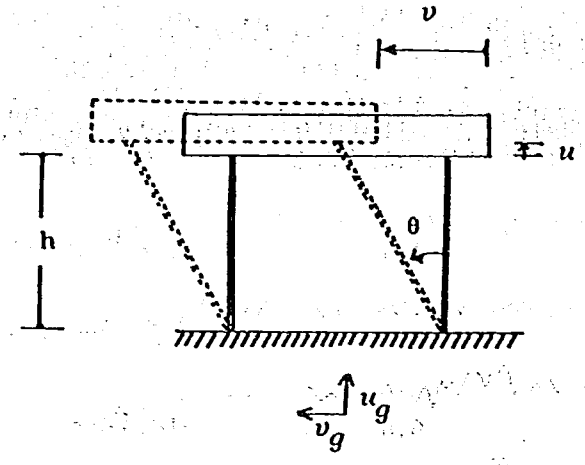


Fig. 7 Dynamic Loops



E:YOUNG MODULUS
I:GEOMETRICAL MOMENT OF INTERIA OF COLUMN
m:MASS OF TEST STRUCTURE
A:SECTION AREA OF COLUMN
Q:SHEAR RESISTANCE OF COLUMN
N:AXIAL RESISTANCE OF COLUMN
F_v:HORIZONTAL EXCITATION FORCE
F_u:UP-DOWN EXCITATION FORCE
$k_b=2\cdot12EI/h^3$, $k_a=2\cdot EA/h$

Fig. 8 Analysis Model

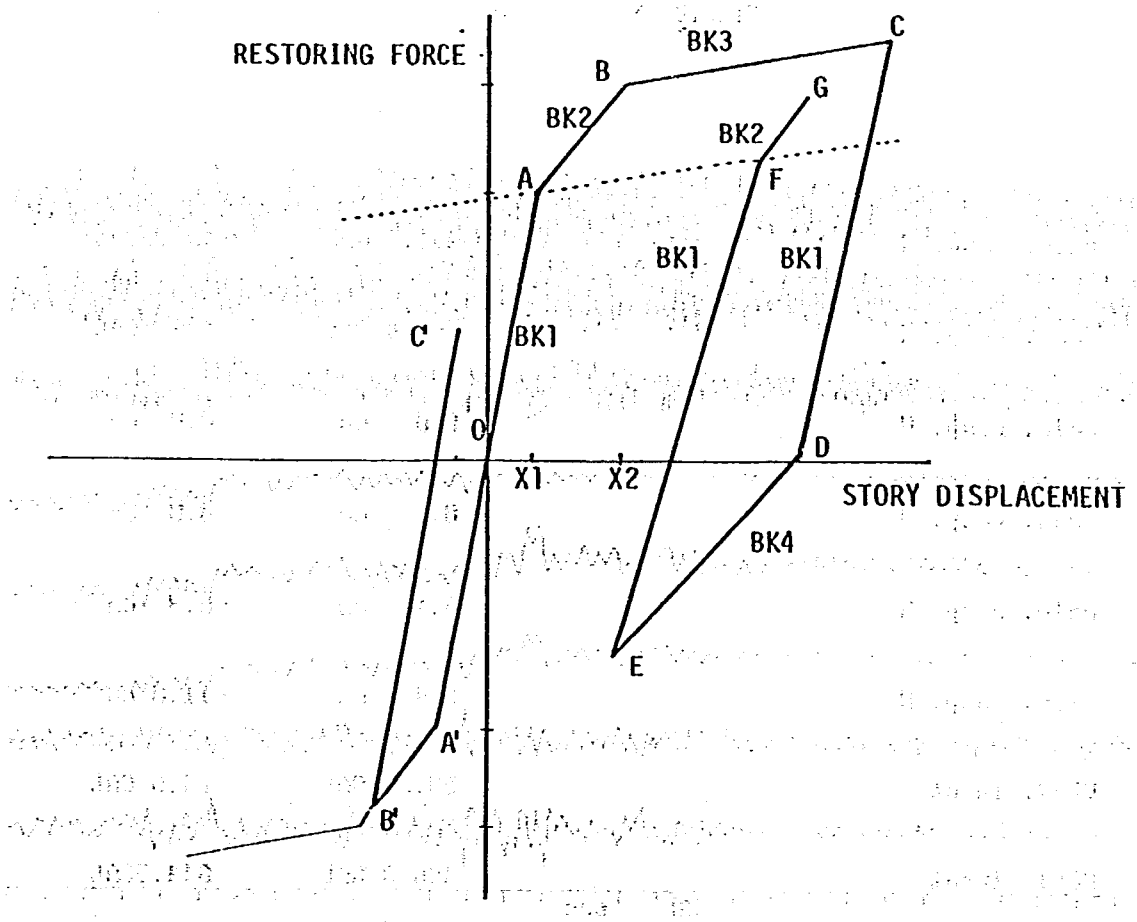


Fig. 9 Restoring Force Model

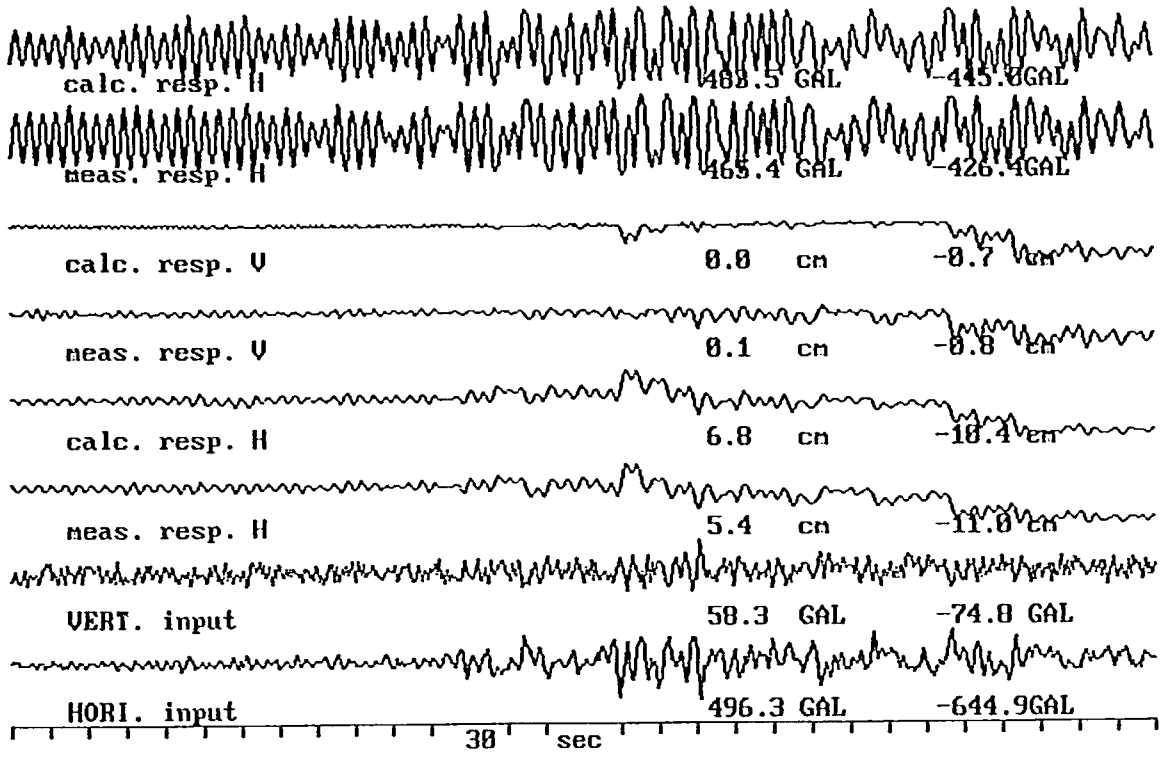


Fig. 10 With P − δ

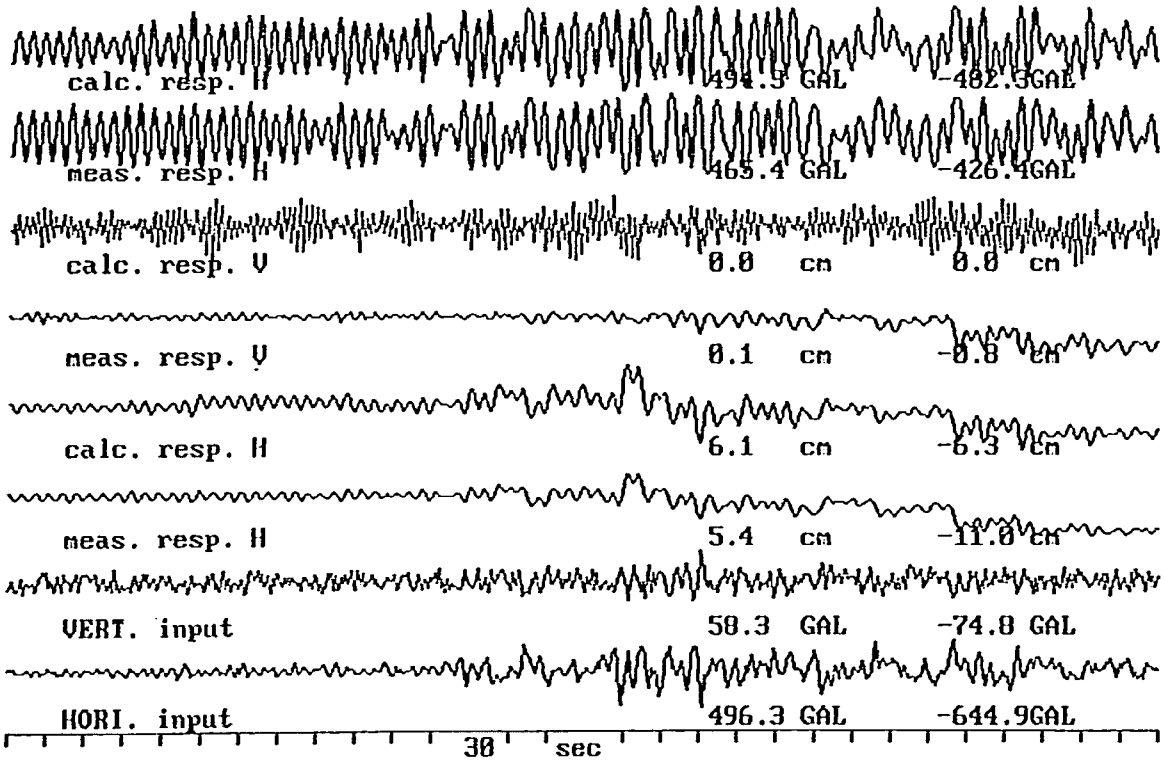


Fig. 11 Without P − δ

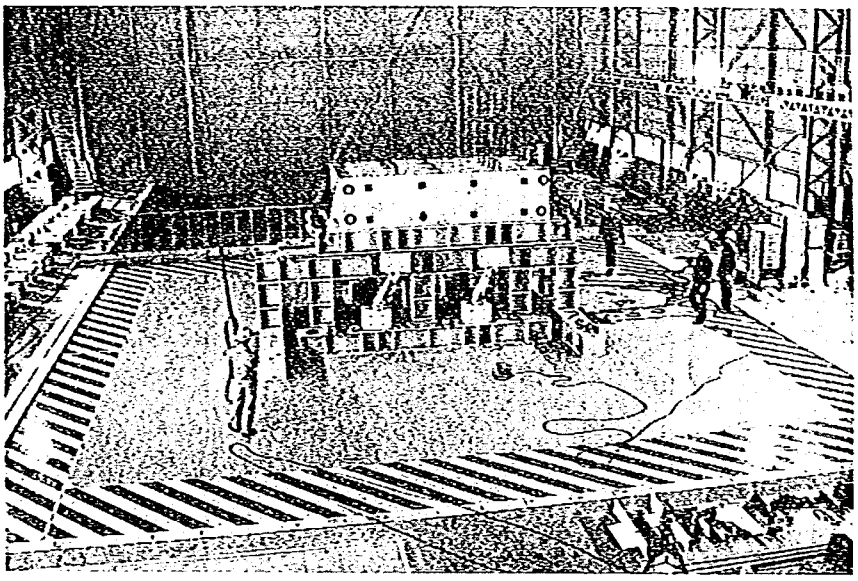


Photo 1 Over view of Test

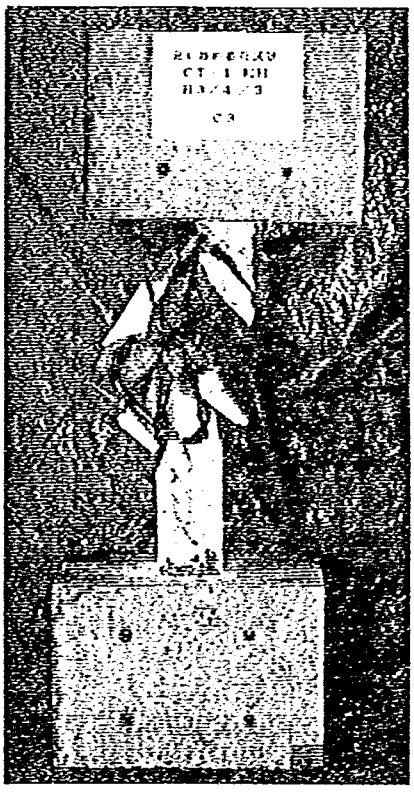


Photo 2 Ø285㎜ (Static)

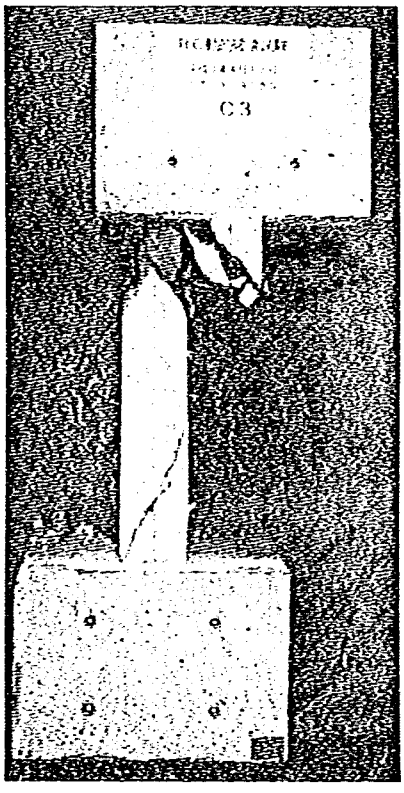


Photo 3 Ø285㎜ (Dynamic)

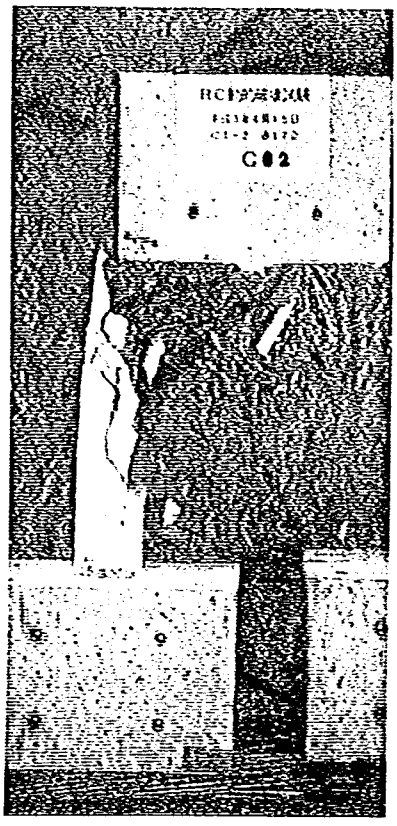


Photo 4 Ø170㎜ (Dynamic)



Photo 5 Ø105㎜ (Dynamic)

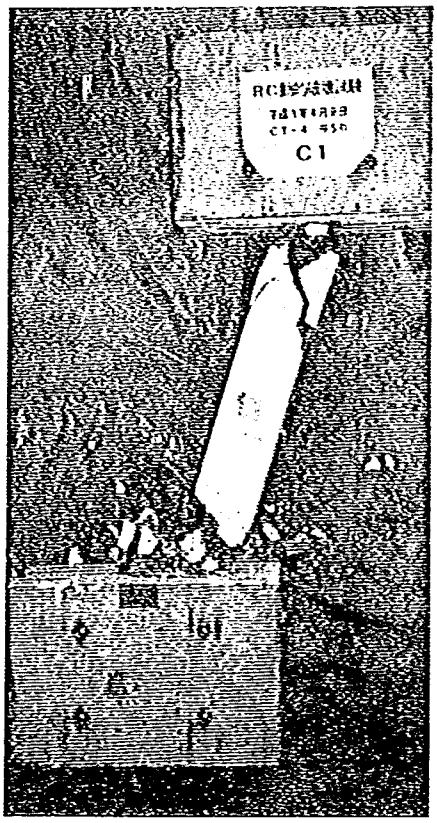


Photo 6 Ø50㎜ (Dynamic)

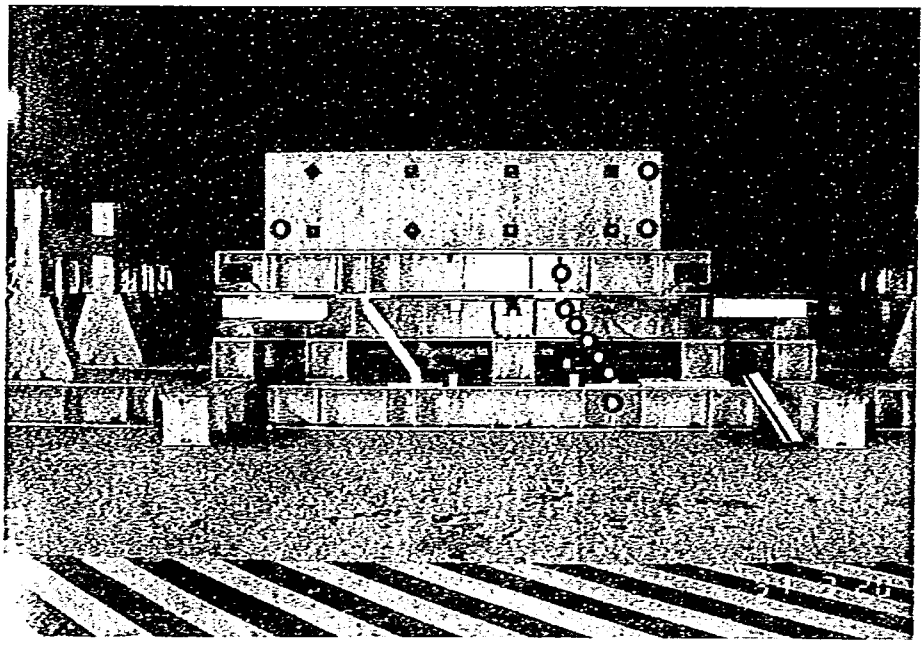


Photo 7 Preparatory Test (Steel Column)

A Hollow Clay Tile Wall Seismic Performance Program Overview

by

J. E. Beavers, W. D. Jones, and W. C. T. Stoddart[2]

ABSTRACT: An overview of a multiyear hollow clay tile wall (HCTW) program being conducted by Martin Marietta Energy Systems, Inc., at the Oak Ridge Y-12 Plant,[3] for the U.S. Department of Energy is presented. The purpose of the HCTW program is to determine the load capacity of unreinforced infilled HCTW buildings when subjected to earthquakes. Progress to date tends to indicate that extensive retrofit of such structures may not be warranted in low-to-moderate seismic zones.

1. INTRODUCTION

Many structures in the United States, especially the central and eastern United States, are constructed of unreinforced masonry. One type of unreinforced masonry, hollow clay tile (HCT), was used extensively as an infill during building construction throughout the United States, especially during the early to middle 1900s. As a result, the U.S. Department of Energy (DOE) has a significant inventory of buildings with infill walls constructed of unreinforced HCT. These buildings do not meet today's seismic standards for new construction.

Bennett and Flanagan (1992) conducted an extensive literature review that revealed no relevant research on the performance of hollow clay tile walls (HCTWs) under seismic loading. To determine the seismic performance of HCT buildings, the Center for Natural Phenomena Engineering (CNPE) of Martin Marietta Energy Systems, Inc. (MMES), is conducting an HCTW testing and analytical program for DOE. The program was initiated in the spring of 1990 and includes in situ, laboratory, full-scale building, and shake table tests; nondestructive evaluation (NDE) development; and analytical studies. The program is focused on the goals of providing information and guidance in the use of this information for assessing the behavior and, if required, identifying cost-effective methodology for retrofitting these buildings.

Nineteen types of in situ and laboratory tests are being performed, ten of which are both in situ and laboratory. The total number of tests to be performed is approximately 110 in situ and 190 laboratory. The NDE program consists of developing techniques to determine HCTW construction quality and infill boundary conditions. The analytical portion of the project consists of research on existing analysis methods and on the development of analytical evaluation techniques for buildings and components.

Building analysis consists of modeling and analyzing entire structures. Potential analysis techniques are either equivalent static analyses

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[1] Presented at the Tenth World Conference on Earthquake Engineering, July 19-25, 1992, in Madrid, Spain.

[2] Center for Natural Phenomena Engineering Martin Marietta Energy Systems, Inc., Oak Ridge, Tennessee, 37831.

[3] Managed by Martin Marietta Energy Systems, Inc., for the U.S. Department of Energy under contract number DE-AC05-84OR21400.

(such as the UBC code Rw factors) or dynamic methods, such as time history analysis. Modeling of the HCT infills may need to be simplified, using, for example, equivalent struts. Component analysis involves developing an appropriate constitutive model for the HCT and an appropriate idealization. All components that are tested, such as prisms, in-plane infills, and out-of-plane infills (air bag tests), will be analyzed, and parametric studies will be performed. It is expected that this approach will lead to the development of simpler idealizations that can be used in the analysis of entire building structures.

Following the Loma Prieta earthquake of October 18, 1989, a significant number of buildings constructed of HCT in the San Francisco area were found to be damaged, including the city hall of Oakland, California. In addition, those that were not damaged or experienced little damage are being examined for the need for retrofit. The results of this HCTW program will provide significant insight into the performance of such structures and provide a methodology for evaluating HCT buildings that better represents their performance. In fact, in low-to-moderate seismic zones, the results may indicate that, although many such structures were not designed for seismic loads, major retrofit may not be required, as might be indicated from using conventional analysis techniques in the evaluation process.

2. SPECIFIC GOALS AND BUILDING CODES

As noted above, the general goal of the HCTW program is to develop recommendations for assessing the behavior of buildings constructed of infilled, unreinforced HCTWs. The program focuses on HCTW buildings located in zones of low-to-moderate seismicity, although the results are directly applicable to high seismicity zones. If infilled, unreinforced HCTW building construction can be shown to have inherent capacity above that normally assumed by design codes, retrofit may not be required in low-to-moderate seismicity zones if a risk-based approach is adopted. A risk-based approach is based on the principle that, while the building may not meet today's codes and standards requirements, the expected seismic performance is reasonably understood and has sufficient margins of safety. To help accomplish these goals, testing and improved understanding are required.

A second principle that has an impact on the issue of whether or not to retrofit in low-to-moderate seismicity zones is the cost increment for retrofitting. In many cases, the cost for retrofitting a building to a higher seismic capacity rather than to a lower seismic capacity is not much more than the cost for retrofitting to the lower seismic capacity; hence, the incremental cost may be very low. For example, the cost for installing a 1-in.-diameter (25.4-mm) bolt required for the retrofit of a building in a moderate or high seismic zone is basically the same cost as that for installing a 0.5-in.-diameter (12.7-mm) bolt in the same building for retrofit in a low seismic zone. Thus, the most significant cost impact for retrofitting buildings in low-to-moderate seismic zones is not the seismic capacity level of retrofit, but the decision, or requirement, to retrofit.

Building codes in the United States generally specify that the capacity of unreinforced masonry, due to its brittle nature, be limited to a stress less than the stress that causes cracking. However, the overall seismic capacity of an infilled masonry building panel can be an order of magnitude higher than its load capacity when the first crack occurs or as indicated by baseline design codes. This leads to a key proposition in the risk-based approach to a "no retrofit philosophy" for existing construction in low-to-moderate seismic zones. This proposition is that, even though HCTWs may crack when earthquakes occur, overall stability (no collapse) will be maintained. Thus, significant cracking of infilled walls is being

described by the authors as acceptable damage for buildings in low-to-moderate seismic zones. Therefore, the principal basis of the research and development program on HCTWs described in this paper is to investigate and develop understanding and computational methods for determining post-cracking behavior, including ultimate load capacity, of infilled, unreinforced HCTW building construction.

3. HCTW PROGRAM DESCRIPTION

The HCTW program at MMES is divided into four elements of work in a classical engineering work breakdown structure (WBS), shown in Figure 1.

WBS 1.2 is HCTW testing which represents the part of the program that includes static, in situ, and laboratory testing of HCTWs and wall components. WBS 1.3 is analytical modeling and analysis, which consists of research of methodologies, pre- and post-test analyses, and the development of evaluation methodologies that incorporate the results of all research efforts. WBS 1.4 is the seismic shake table facility, which is designed to show that the evaluation methodologies selected, based on static testing, are conservative when tested dynamically. Thus the evaluation methodology selected will provide a better, but still conservative, seismic performance estimate of HCTW buildings. The final element of the HCTW program is WBS 1.8, NDE research and development. For proper evaluation of an existing HCTW building, it is imperative that the existing condition of the HCTWs and the boundary conditions between the HCTWs and the structural framing be determined. Thus, a significant amount of effort in the HCTW program is the establishment of NDE techniques that can correlate controlled laboratory-constructed specimens to in situ wall construction and that can be relied on to properly define key parameter inputs to the evaluation methodologies.

4. DESCRIPTION OF HCTWS

The specific infilled, unreinforced HCTWs studied in this program are constructed of multi- or single-wythe HCT, usually 8 in. (203.2 mm) or 12 in. (304.8 mm) in width, using running bond, with all cores laid horizontally. Double-wythe walls in this program are constructed of 4-in. (101.6-mm) and 8-in. (203.2-mm) HCTs staggered to give overlapping head joints, as shown in Figure 2. The 4-in. (101.6-mm) and 8-in. (203.2-mm) tiles are also staggered from course to course. In this type of construction, no continuous collar joint exists; however, in each row, the 4-in. (101.6-mm) and 8-in. (203.2-mm) tiles are separated by a 0.5 in. (12.7 mm) to 1 in. (25.4 mm) gap. In forming the full wythe bed joint, some mortar has fallen into this gap. There is no vertical or horizontal reinforcement in the walls. The HCTWs are placed within the steel and concrete framing with varying offsets to the framing centerline depending on the particular building. In some cases, HCT pilasters, integrated with the continuous HCTW, are actually framed around a column as the HCTWs transverse the length of a building.

5. PREVIOUS HCTW PROGRAM ACTIVITIES

Aspects of the HCTW program have been going on since 1989; however, the program was not fully developed, as shown in Figure 1, until mid-1990. Beavers, Bennett, and Flanagan (1991) discussed an earlier version of the program and referred to some test results, while Fricke and Jones (1990, 1991) described the results of some of the earlier tests of element WBS 1.2, defined as a subelement WBS 1.2.1, push-test. Nine push-tests to measure the shear strength of the mortar bed joint were reported. The average shear strength was found to be 99 psi (683.1 kPa), with a standard deviation of 32.5 psi (224.3 kPa). Butala, Jones, and Beavers (1991) have described some of the work in WBS 1.3.2,

pre- and post-test analyses in preparation of testing activities of WBS 1.2.14, wall-column bending. By examining analytically the potential implications of story drift on the out-of-plane performance of such walls, they found that building drift in the out-of-plane direction imposed more bending stress on a wall panel than did inertial forces on the wall panel itself. This stress caused horizontal cracking to occur at a "g" level less than 5% (5%g). They also reported that the cracking would occur along several mortar joints, such that the steel framing would act independently at a frequency of 0.75 Hz; therefore, the inertial forces acting on the wall would be small, and it would remain stable and standing. The question posed here is, How much in-plane strength remains, given the out-of-plane horizontal cracking? This concern will be further addressed by testing in WBS 1.2.14.

Chua (1991), using existing analysis methodologies, conducted a parametric study of boundary conditions, Poisson's ratio, and other parameters to determine a best approach to estimate ultimate out-of-plane behavior of infilled HCTWs. As noted by Beavers, Bennett, and Flanagan, Chua found that for static out-of-plane loads, the modulus of rupture and the modulus of elasticity had the most significant effect on first panel cracking, while Poisson's ratio and shear moduli had little effect.

6. RECENT HCTW PROGRAM RESULTS

Under the HCTW testing element WBS 1.2, one major subelement is the air bag test, identified as WBS 1.2.7. An air bag test is the application of out-of-plane loading to an infilled wall using a pressurized air bag (Dawe and Seah 1989). The first air bag test (Fricke, Huff, and Jones 1992) was conducted in October 1991. The test was conducted on a 8-in. (203.2-mm) single-wythe HCTW 28 ft (8.53 m) wide by 12 ft (3.66 m) tall infilled within a steel frame of W14X142 columns, a W30X108 overhead beam, and a concrete floor slab. This test was conducted on the first floor of a five-story building. A schematic of the test setup is shown in Figure 3, with the actual wall and corresponding reaction frame shown in Figure 4.

For design purposes, conventional methods using code allowables and a static analysis would generally be based on one-way action in the short (vertical) direction. This procedure would predict a uniform loading limit equivalent to an inertial load of approximately 0.1g. However, as discussed by Fricke, Huff, and Jones, the test results revealed an equivalent inertial load capacity of around 3g, thirty times their results obtained from the conventional approach. These researchers also looked at other alternative conventional evaluation methods and compared the results to their test results. In all cases, the test results revealed a minimum in situ out-of-plane seismic capacity of 13 times the least conservative conventional method. The basic reason for the underestimated capacity resulting from conventional methods is the lack of consideration of two-way arching action that actually occurs in infilled walls. The load deflection curve for the central point of the wall is shown in Figure 5.

More recently, Flanagan, Bennett, and Barclay (1992) conducted three laboratory in-plane tests, on full-scale infilled HCTWs. These tests were part of a sixteen-test program under subelement WBS 1.2.9, designed to study failure mode, shear wall performance, load transfer mechanism, energy absorption, and hysteretic behavior of HCTW infilled frames having various frame to infill stiffness ratios. The walls were 8 ft (2.4 m) in height and width, constructed with single-wythe 8-in. (203.2-mm) HCTs, placed between the flanges and against the column webs, as is much of the construction of DOE facilities. For each test, the overhead beam size was held constant at W12X35 while the column sizes for each test varied (W10X12, W10X30, and W10X45).

For determining the in-plane capacity, conservative conventional analyses usually result in defining that capacity as being the load at which the first shear crack occurs in the infilled wall. The results of the three tests when applied to a first floor interior wall of a typical three-story DOE building showed that the equivalent in-plane seismic capacity was approximately 0.3*g* at first panel cracking. Applying the ultimate capacities of the tests to this typical building indicated an equivalent in-plane seismic capacity of approximately 0.8*g*. A typical failure mode at the completion of each test is shown in Figure 6, while Figure 7 shows a typical hysteretic curve of the in-plane load versus deflection.

Again, the analysis of these tests showed that conventional analysis techniques are inadequate to estimate the ultimate performance of unreinforced HCTW structures of the type described herein.

7. OTHER TEST RESULTS

As shown in Figure 1, unit block tests (WBS 1.2.3), bond wrench tests (WBS 1.2.11), and prism compression tests (WBS 1.2.4) are also being conducted in support of the overall program. Fifteen unit block tests (ten in compression parallel to cores and five splitting tensile strength perpendicular to cores) have been conducted with an average net compression stress of 5539 psi (38,194 kPa) and a standard deviation of 634 psi (4,391 kPa). The average splitting tensile strength was 351 psi (2420 kPa) with a standard deviation of 106 psi (731 kPa). The results of prism compression tests showed an ultimate capacity of 228 psi (1573 kpa) when loaded perpendic-ular to the cores.

Preliminary bond wrench tests on the air-bag-damaged wall have shown an average ultimate tensile capacity of the bed joint mortar to be 16 psi (110 kPa). As the HCTW testing program continues, additional tests and corresponding results will be reported.

8. SUMMARY AND CONCLUSION

The HCTW program is a multiyear testing and analytical effort to address the seismic performance of steel and concrete frame structures with infilled HCTWs. The first year of an approximate three-year program has been completed. Results to date support the assertion that infilled HCTW structures can have significantly more inherent strength than conventional analysis and code allowable stresses would imply, and must be treated as structural systems.

Some progress has been made in developing alternative analytical methods, but more test data are needed to support current results. The program is being documented, with technology transfer as one of its goals. As further testing is conducted and analytical techniques verified and/or developed, results will be published. Based on the preliminary results of the HCTW program, it is believed that many unreinforced HCTW buildings in low and moderate seismic zones may not require retrofit, when evaluated using a risk-based approach. Further testing and analytical development are expected to confirm this assertion.

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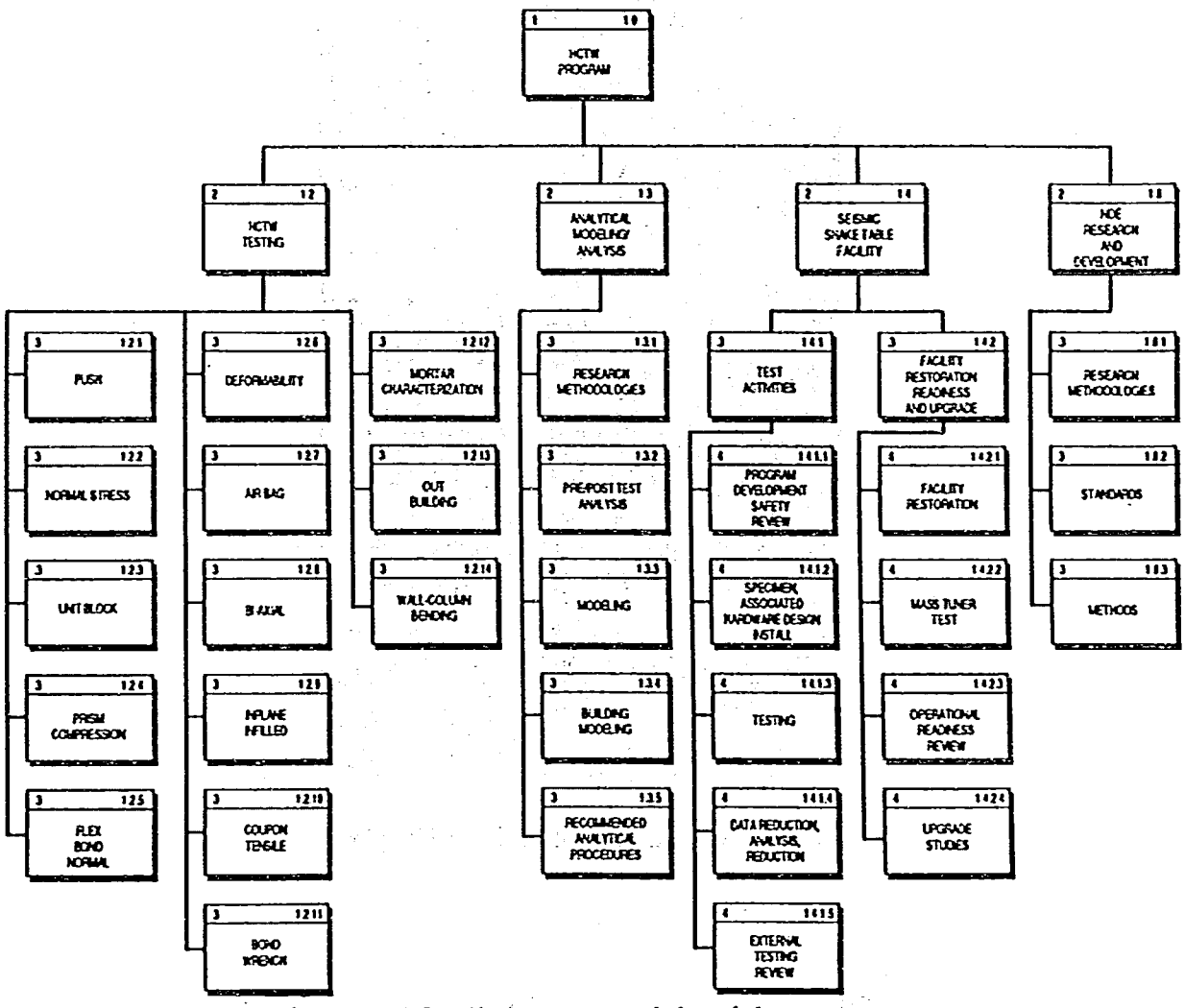


Figure 1. HCTW program work breakdown structure.

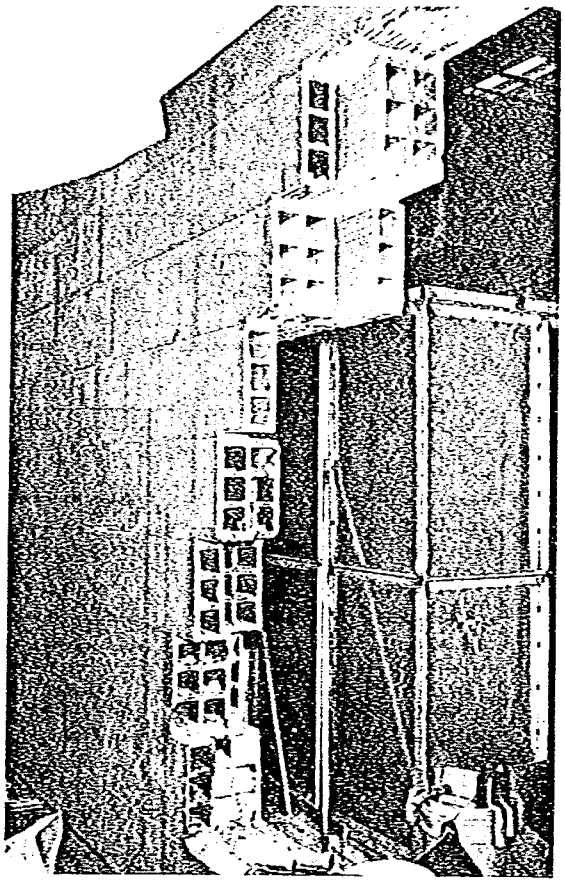


Figure 2. Cut-section of an infill HCTW.

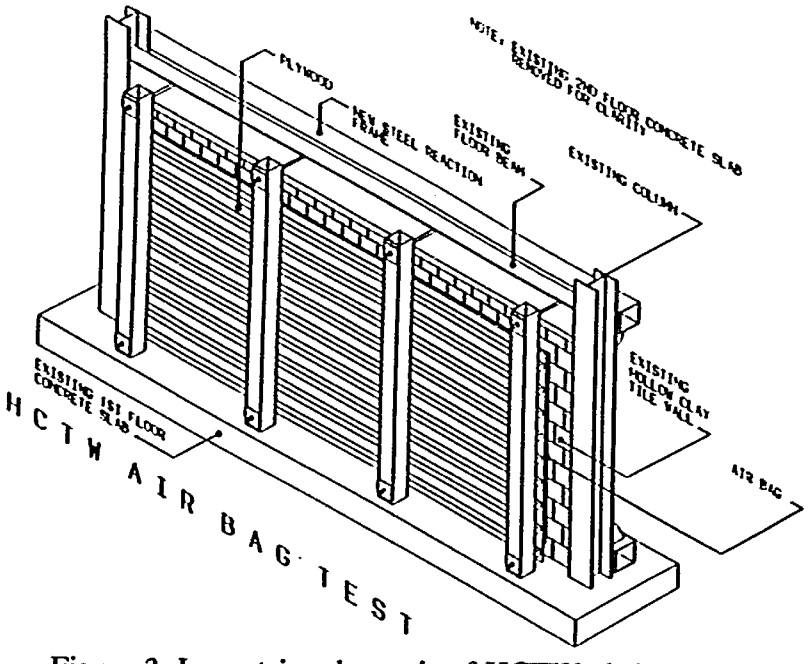


Figure 3. Isometric schematic of HCTW air bag test.

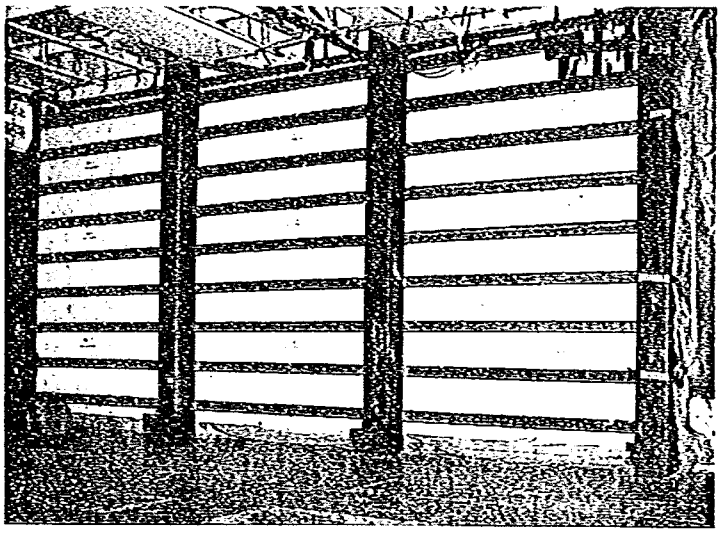


Figure 4. Air bag test HCTW with loading frame.

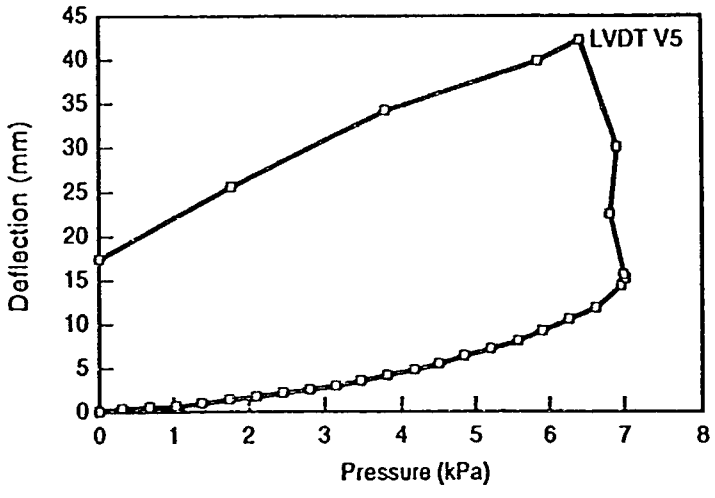


Figure 5. Air bag test HCTW central point load-displacement curve.

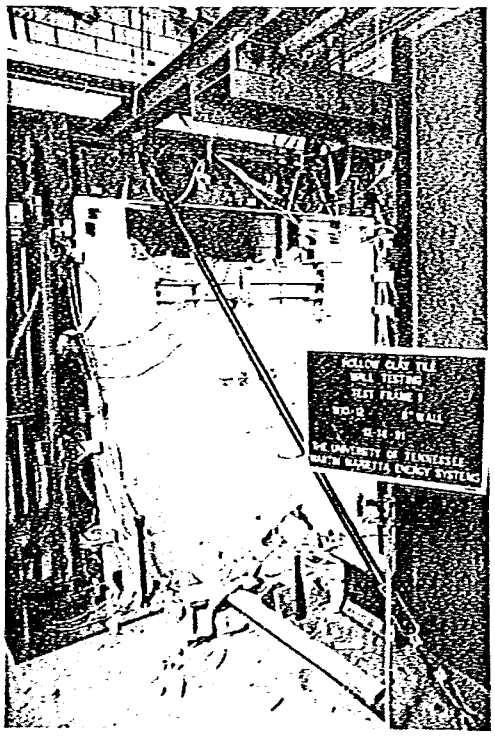


Figure 6. In-plane laboratory HCTW test cyclic failure mode.

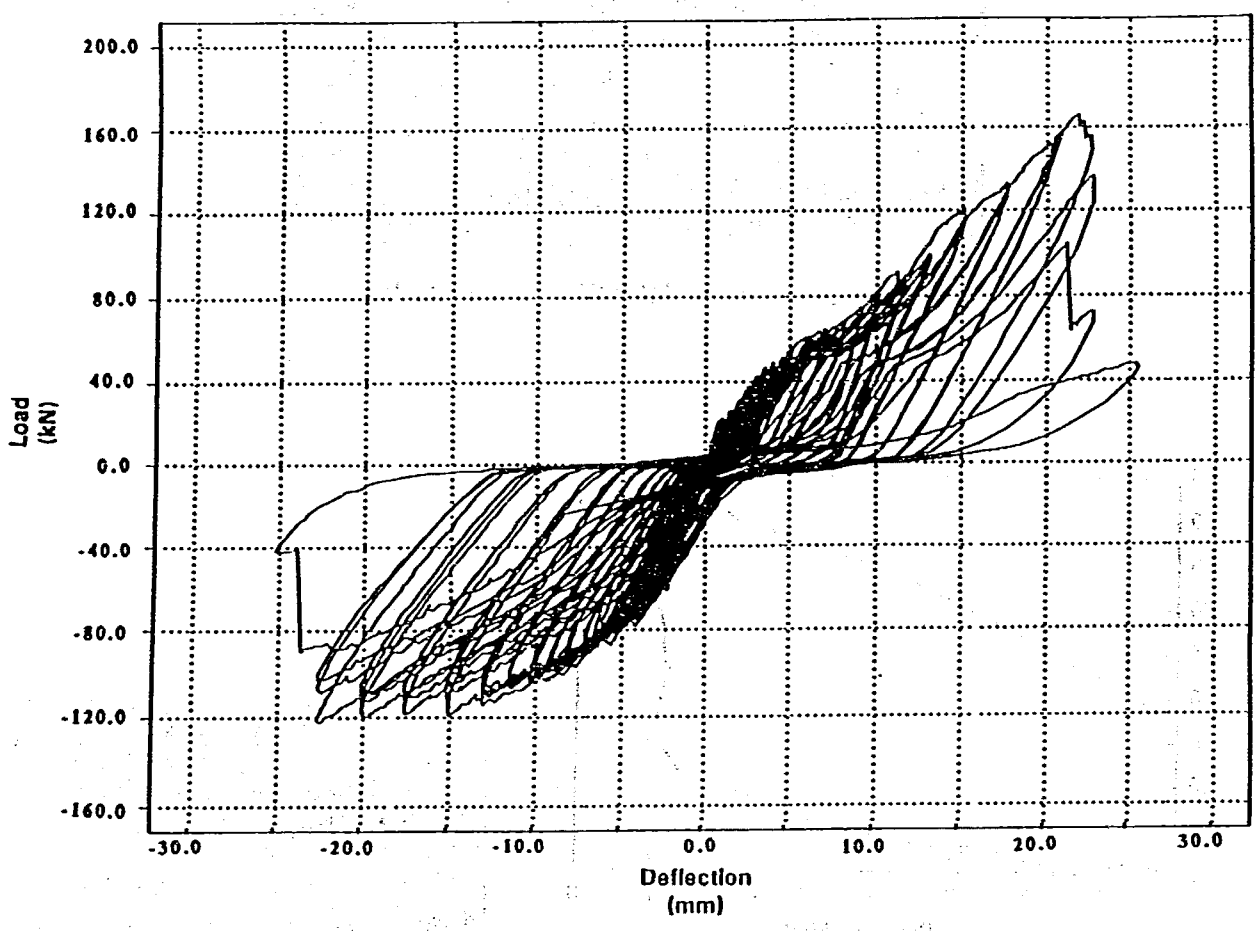


Figure 7. In-plane laboratory HCTW test load-displacement hysteretic curve at top of wall.

構造計算用プログラムの評定・指定制度

(THE VALUATION AND DESIGNATION SYSTEMS FOR STRUCTURAL CALCULATION PROGRAMS IN BUILDING ENGINEERING)

大谷圭一[1)]

要　約

最近の数十年間の間に，研究・開発・設計・現場等の建築の全ての分野で電子計算機の利用が急速に浸透している。さらに，電子計算機利用の普及は将来より拡大するであろう。これらの状況を背景として，電算プログラムの評定委員会は，プログラムを審査し，その質と信頼性を確認する目的で，日本建築センターに1974年に設置された。この委員会では汎用的かつ確認申請用の構造計算書作成のための一貫構造計算プログラムを取り扱っている。評定済みプログラムに対する建設大臣の指定という行政的手段が1977年に開始された。この評定・指定制度は世界に例を見ないユニークなものである。この制度の設置により，プログラム作成者，利用者，行政担当者それぞれが恩恵を受けることとなった。本文はこれらの制度の現状について報告するものである。

キーワード：プログラムの評定，プログラムの指定，構造計算プログラム，確認申請制度，建築基準法

1，序　論

全ての建築の世界，特に構造計算や日影計算などの分野，での電子計算機の利用は３０年以上に達している。更に全ての仕事に於て電子計算機の利用はますます拡大していくものと考えられる。

これらの状況を背景として，構造計算プログラムに対する評定委員会が，コンピュータを利用して作成するための構造計算プログラムの質を保持し，特定行政庁での確認申請業務の迅速化を図る目的で，1974年に日本建築センターに設置された。その後，評定を終了したプログラムに対する建設大臣の指定制度が行政的施策として，1977年に開始された。

この評定・指定制度は画期的なものであり，諸外国に例のないものであったので，この制度には多くの国から強い関心が寄せられた。本文は構造計算プログラムに対する評定・指定制度の現状について報告するものである。

2，確認申請制度

日本では建物を設計・建設しようとする時には，「建築基準法」の定める所に従うことが規定されている。即ち，建物の建設開始以前に，設計者は設計図書（図面，構造計算書等）を確認申請のために特定行政庁に提出することが義務付けられている。特定行政庁では受理した設計図書が「建築基準法」に定める諸規定に照らして妥当なものであるかをチェックし，妥当なものであれば，建設許可を与えることとなっている。しかしながら，高さ６０ｍを超える建築物や特殊な技術を採用している建築物に対しては，建築大臣が個別に建設許可を与えるといった，前述の一般的な手続きとは別のシステムも用意されている。

確認申請制度により設計図書の妥当性をチェッする方法は，日本の建築行政の特徴の一つである。

3，評定・指定制度を取り巻く環境

前述したように，「建築基準法」は設計した建物に対して，確認申請を受理することを規定している。近年，電子計算機を使用して構造計算書を作成するケースがどんどん増加している。確認申請に携わる建築主事（行政担当者）は，プログラムの中身を「ブラックボックス」として受理し理解出来ないままに，構造計算書の妥当性を検討しなければならなくなってきている。更に，コンピュータや通信網を利用したデータ通信や，パーソナルコンピュータの普及により，プログラムの内容を検討出来ない第三者の計算機利用が増加してきている。構造技術におけるコンピュータ利用の混乱を避け，かつ，プログラムの質の確保に対する適切な方法の導入について社会的要求が高まってきた。

これらの状況を背景として，建設省はプログラム評定作業について検討する調査委員会を1971年に日本建築センターに設置した。1972年6月に発表されたこの委員会の報告に従って，「電算プログラム評定委員会」

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が，プログラムを調査し，その質的内容を確認し，その評定結果を直接的に建築行政に反映させることを目的として，同じく日本建築センターに設置された。この評定委員会の活動は1974年1月に開始された。

また，建設省は評定を終了したプログラムに対する建設大臣の指定制度を1977年に開始した。この指定制度は評定制度を強化するものであり，行政事務での取扱に直接寄与するものであった。この指定制度の実現は評定制度の開始当初から切望されていたものである。

評定済みプログラムの所有者は建設省の定める手続きに従って，建設省に申請すると，建設大臣からプログラムの指定を受けることとなる。従って，指定を受けたプログラムは二つの性格を有することとなる。すなわち，第一は評定によりプログラムの適用範囲とその計算方法が確認され，第二に論理的正確さが建設大臣から容認されたという点である。この施策により評定制度の位置づけは建築行政の一環として定義されたこととなった。また，プログラムの作成者，行政担当者，評定委員会委員，プログラムの利用者それぞれの立場が明確にされることとなった。併せて，これらの制度の効果として，建物の安全性の向上，プログラムの質的進歩に寄与することとなった。

4．評定・指定制度の効果と内容

これらの制度の考えられる直接的効果は次のようである。

1）プログラム作成者は行政担当者やプログラム利用者に対して，プログラムの信頼性について説明する必要がなくなった。
2）プログラムの利用者は，評定済みプログラムを最初の利用から，プログラムの信頼性について特段の注意を払うことなく使用可能となった。
3）行政担当者はプログラムの信頼性についてチェックを要せず，対象建物の性能についてのみ検討を加えればよくなった。

これらの評定制度の効果は，全ての分野から期待されていたものである。今日では，この評定制度は社会的道具として建築の社会に定着しており，特に特定行政庁からは非常に効果的な制度であると評価されている。

これらの期待されている効果に応えるために，評定委員会において次の項目について審査を行っている。

1）プログラム自身のなかに構造計算の結果に影響を及ぼすようなエラーがないか。
2）プログラムの誤用　（その防止法及び対策）
3）計算結果の誤用　（その防止法と対策）

委員会での評価にあっては，プログラムの適性について，その内容の説明，計算の再現やその他のテストを行うことによって，確認を行っている。この評定委員会では，構造計算の一般的利用及び構造計算書作成のための利用に供する一貫利用プログラム（パッケージプログラム）を対象としている。この評定作業は，プログラムの有する著作権やノウハウを扱うことになるという大きな問題を抱えているので，委員会の委員は大学，国立研究所及び行政庁の職員から構成されている。

5．評定・指定制度の現状

これらの制度の開始された当初は，評定対象はいわゆる汎用コンピュータ用プログラムに限定していた。評定委員会では，私的組織内での利用（プライベートユース），公開された組織での利用（パブリックユース）の双方を扱っている。また，委員会はプログラム作成者と異なる組織によるプログラムの運用の妥当性についても，評定を行っている。

建築基準法施行令，特に構造計算に係わる部分，が1981年7月に大幅に改定された。この時までに２２のプログラムが評定を完了していた（また，いくつかのプログラムについては指定も完了していた）。しかしながら，建築基準法施行令の改定に伴い，上述の２２のプログラムの評定・指定の効力を失効させた。

建築基準法施行令の発効により，汎用コンピュータ用の新たなプログラム（上述の２２のプログラムを含む）への評定作業が再開された。高性能を有するパーソナルコンピュータの利用がますます一般化してきたのに伴い，建設省の了解の基に，評定委員会ではパーソナルコンピュータによるプログラムについても，評定対象とすることを1982年3月に決定し，具体的な作業を同年7月に開始した。

現在では，プログラムの一部変更や機能追加を含む２５０件の案件について，評定作業を完了している。（表１，図１，２参照）全体として，プライベートユース，パブリックユース，パーソナルコンピュータ用プログラムを含む１０１の異なるプログラムについて評定を終了し，８５の組織が評定済みプログラムをその構造計算の仕事に運用している。しかしながら，パーソナルコンピュータ用プログラムと保有水平耐力の計算の部分に対しては，未だ指定制度の対象外となっている。そのため，６８の組織，４７プログラムのみが，建設大臣からプログラムの指定を受けているのが現状である。

6．結　論

電子計算機利用の普及，建築確認申請という建築行政の特性を背景として誕生した，建築技術の構造計算プログラムに対する評定・指定制度は，今日では建築の世界にしっかりと定着したものとなっている。評定を終了したプログラムの数は１０１に達し，指定を受けているプログラムは６８例になっている。

しかしながら，この評定・指定制度も次のような問題点を抱えていることも認識しておかねばならない。

１）プログラム変更に対応するこれらの制度の有効な対処法

２）より効率的な評定作業推進の具体的方策の立案

関係者の努力と協力により，これらの評定・指定制度がますます有効に機能し，発展していくことを期待している。

なおまた，建築の評定制度の設立が契機となって，高圧ガス施設の耐震設計用プログラムや測量の解析プログラムなどに見られるように，他の分野でも同様の制度が設置された。

Table 1. THE STATE OF VALUATION AND DESIGNATION SYSTEMS (February, 1991)

	Valuation		Designation
Public use	17	186	10
Private use	55		37
Personal Computer	29		
Program Employment	85		21
Modification or Addendum	64		
Total	250		68

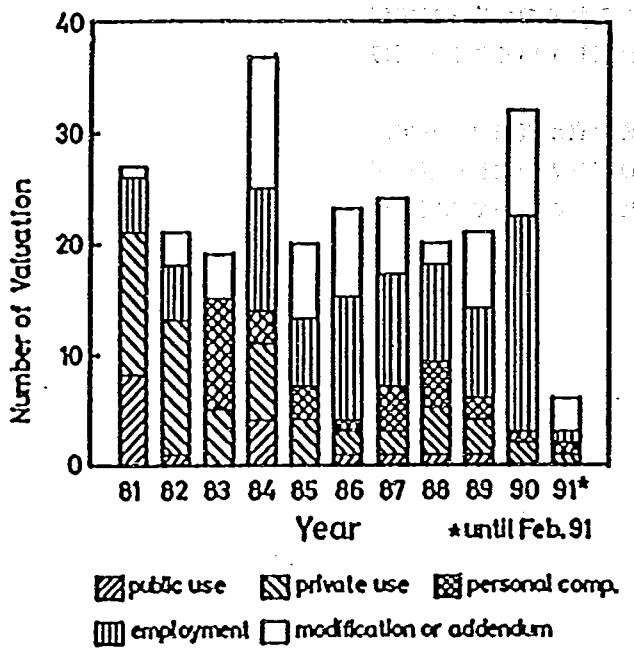


Fig. 1. THE YEAR TREND FOR THE NUMBER OF VALUATED PROGRAMS

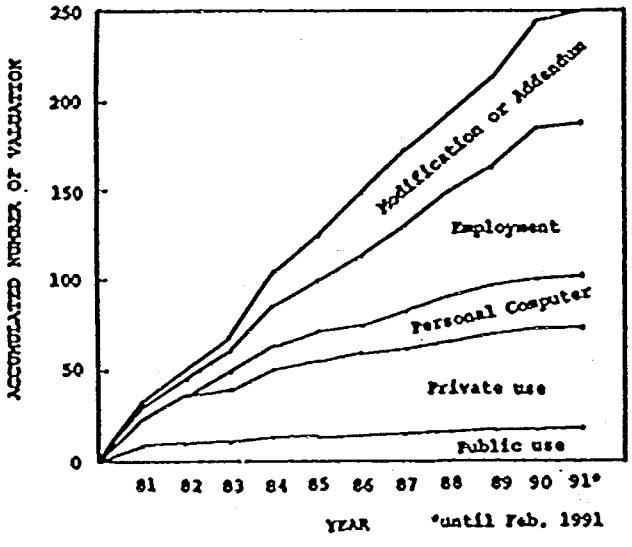


Fig. 2. THE YEAR TREND FOR THE ACCUMULATED NUMBER OF VALUATED PROGRAMS

Highlights of the 13 March 1992 Erzincan (Turkey) Earthquake

by

Mehmet Celebi*

ABSTRACT

The March 13, 1992 M_s = 6.8 Erzincan earthquake in Turkey is highlighted here. The epicenter of this earthquake was located 7.7 km from the eastern end of the North Anatolian fault. The strong motions recorded in Erzincan had peak ground accelerations of approximately 0.5 g, accompanied by a pulse of 2 seconds. The duration of the earthquake was 7 seconds. This earthquake caused collapse of about 150 buildings—mainly to 4–5-story reinforced, concrete-framed buildings with infill walls. This damage, which is discussed, can be attributed to non-compliance with seismic codes.

KEY WORDS: earthquake; North Anatolian fault; magnitude; acceleration, code, spectra.

1. BACKGROUND INFORMATION

At the eastern end of the North Anatolian fault in Turkey, a magnitude 6.8 earthquake occurred on March 13, 1992 at 19:19 local time (17:19 UTC). National Earthquake Information Center (NEIC) reported the depth at 28 km and the epicentral coordinates as 39.706°N and 39.570°E. The epicenter is 7.7 km SE of the center of the City of Erzincan, the coordinates of which, from the Gazetteer (1984), are 39.75°N and 39.50°E. The City of Erzincan has a population of 91,000 plus approximately 30,000 military personnel. Official count of lost lives was 800. However, unofficially, as many as 3,000 persons may have lost their lives during the earthquake.

The City of Erzincan is located on the edge of an alluvium basin lying on an alluvium fan at an elevation of 1,150 m. It is bordered in the north and south by high and steep mountain ranges, some reaching altitudes of 3,800 m. The geology of the basin can be best described as 100 m of interbedded sand and gravel overlying lacustrine sediments. The depth to bedrock in the basin is estimated to be between 1,200–1,500 meters. At Erzincan, the depth to bedrock is estimated to be 300–500 meters. Borehole logs are available only for the top 200–225 meters.

The North Anatolian fault, well known for its similarity to the San Andreas fault in California, is a right-lateral strike-slip fault. The recurrence intervals of large earthquakes are also similar to the San Andreas (California) recurrence intervals. The last large earthquake of 1939 leveled the City of Erzincan of that time, killing 39,000 people. The 1939 earthquake had a magnitude 8.0 and epicentral coordinates of 39.7°N and 39.7°E (Ambraseys, 1970). Since 1939, there have been 60 magnitude 6 and larger earthquakes along the North Anatolian fault

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(Ambraseys, 1970). Ambraseys states that historically, there are quiet periods of seismicity of approximately 150 years. A general map showing the North Anatolian fault, the location of the City of Erzincan is provided in Figure 1. The unique topography of the epicentral area and the basin of Erzincan is shown in Figure 2.

As a consequence of the 1939 earthquake, the City of Erzincan was relocated to the north of its original location to its current location (towards the hills) with the idea that the new buildings would be on firmer ground and shallower depth of alluvium. Furthermore, in the new city, the number of stories was limited to 3—a restriction, which, in the 1970s and 80s was slowly forgotten and abandoned. At the time of the 13 March 1992 earthquake, the tallest building in the city was 7 stories. Only 2 engineered buildings (the Railroad Station Building and the Military Headquarters Building) from the pre-1939 era survived the 1939 earthquake and also the recent earthquake.

It was reported that several buildings that collapsed during the 13 March 1992 earthquake were repaired after sustaining severe damage during a smaller (M = 5.6) 1983 earthquake.

TYPICAL CONSTRUCTION

Typical construction in the City of Erzincan is of reinforced concrete frame with infill walls, generally of hollow-tile masonry construction. The infill walls vary in height, sometimes up to window levels and sometimes as piers extending between floor to ceilings. The roofs are generally of wood-truss with clay tiles. Gable end walls, generally not braced, are also constructed with hollow tile masonry. Shear wall construction is seldom used. There is some adobe (mostly single-story) and stone-masonry construction.

The majority of the inventory of buildings in Erzincan is 4–5 stories. A significant percentage of the inventory is single-story houses, although there are many 2–3-story buildings also. Very few of the buildings in the City of Erzincan are 6–7-story buildings.

There are also some typical industrial buildings (steel frame of sugar refinery, saw-tooth, roof-type reinforced concrete cotton textile mill and a new plant of precast gable) in the Erzincan basin.

DAMAGE

The City of Erzincan had most of the engineered structures in the earthquake stricken Erzincan Basin and vicinity and it suffered the most damage. Other population centers that experienced damage were smaller towns and villages where the majority of buildings were not engineered. In a small scale, the basin and the damage patterns in Erzincan due to the earthquake resemble the Mexico City (Michoacoan) earthquake of 1985. Figure 3 shows a typically pancaked four-story building and Figure 4 shows a typical reinforced concrete frame building in the state of collapse, its infill walls spalled out and plastic hinges developed at the column ends due to insufficient and/or incorrectly implemented shear reinforcement. On the other hand, a single-story, partially timber-reinforced adobe

building located in downtown Erzincan (Figure 5) survived the earthquake without any damage.

Typical samples of damage are illustrated in Figures 6 to 10. The general view of Unsan Flour Mill, approximately 10 km east of Erzincan, is seen in Figure 6. The details of the short column clearly indicate insufficient and inappropriately placed shear reinforcement (Figure 7). Similarly, the four-winged Military Hospital, damaged extensively beyond repair, is seen in Figure 8. The hospital lost one of its wings, lost one story in one of the two central wings, and sustained heavy damage to the remaining two wings. This was one of the three hospitals in Erzincan. All three hospitals were out of service as a result of the severe damages during the earthquake. The L-shaped wing of the seven-story Social Security Hospital totally collapsed and the State Hospital was severely damaged.

Some of the very few steel structures in Erzincan, at the Sugar Refinery, were heavily damaged. In Figure 9, the main refinery building is seen. The infill walls of the refinery sustained heavy damage. The stack of the calcination unit of the refinery collapsed (Figure 10) damaging part of the main refinery building in the process.

Approximately 150 buildings either totally collapsed or were damaged beyond repair.

STRONG-MOTIONS RECORDED

The Earthquake Research Center (ERC) of the General Directorate of Disaster Affairs of the Ministry of Public Works and Settlement (Republic of Turkey) operates and maintains the strong ground motion network of Turkey. The network, initiated in 1973, consists of 69 SMA-1 type accelerographs and 36 Wilmot seismoscopes. In the earthquake stricken area, there are three tri-axial strong-motion accelerographs. All three stations recorded the 13 March 1992 main shock. The relative locations of these three stations (Erzincan, Refahiye and Tercan) are seen in Figure 1. The analog films were recovered immediately after the earthquake and processed to be digitized at ERC headquarters in Ankara. These three tri-axial records (as digitized by ERC) are seen in Figure 11.

The record from Refahiye has peak accelerations of 0.07 g (NS), 0.07 g (EW) and 0.04 g (vertical). The records from Tercan have clear peculiarities (no clear P and S waves, etc.) that are not yet clarified; therefore, no discussion of this record will be made. Both records possibly need careful re-digitization and processing.

The record from within the City of Erzincan has peak accelerations of 0.39 g (NS), 0.49 g (EW) and 0.24 g (vertical). The record is significant because (a) it is one of the few near-field recorded strong-motions for a damaging earthquake, and (b) it shows a prominent pulse of approximately 2 seconds duration. Using the ERC digitized uncorrected accelerograms, baseline correction and filtering (high-pass Butterworth filter at 0.125 Hz and order 4) was done at the United States Geological Survey (USGS) offices at Menlo Park, California to obtain the velocity and displacement time-histories seen in Figure 12. From the acceleration time-history, the duration of the earthquake is estimated as approximately 7 seconds calculated on the basis of the 5–95% rule,

applied to the integrated square of the recorded acceleration time-history (Figure 13).

The acceleration response spectra of the (NS, EW and vertical components) of the earthquake for 5% damping are shown in Figure 14. The EW spectrum clearly identifies the predominant peaks at approximately 0.2, 0.3, 0.65 and 2 seconds. Of these, the 2-second period is the source frequency and the others are attributable to the site frequencies, possibly incorporating basin reflection effects. The record shows significant energy at approximately 0.2–0.35 seconds (clearly within the range of the frequencies of the 4–5-story reinforced concrete framed buildings with infill walls).

In Figure 15, the time-histories and the corresponding Fourier amplitude spectra of the dominant direction (and its orthogonal component) of the Erzincan ground accelerations are shown. The dominant direction of 128° is calculated using the recorded NS and EW acceleration records and minimizing their cross-variance. This is accomplished by the relationship:

$$\phi = 0.5 \tan^{-1} \{2\sigma_{12}/(\sigma_1^2 - \sigma_2^2)\}$$

where σ_1^2 and σ_2^2 are the variances of the recorded horizontal orthogonal motions (NS and EW directions) and σ_{12} is their cross-variance. When the NS and EW records are rotated by ϕ, the orthogonal components in the dominant direction of the recorded acceleration time-histories are determined. The significance of the 128° is that the strike of the North Anatolian fault near the Erzincan region is also approximately the same angle. This may possibly be a strong reason as to why the shaking in Erzincan was apparently stronger in the direction of the fault and possibly adversely affected the fate of some of the buildings.

CODES

At the time of the 13 March 1992 Erzincan earthquake, the seismic resistant design code in effect in Turkey was the nationwide code enacted in 1975 entitled "Specifications for Structures to be Built in Disaster Areas." The complete code is included in the previous as well as the latest issue of the "Earthquake Resistant Regulations: A World List," a publication of the International Association for Earthquake Engineering (1988). The basic design philosophy of this code is elastic. This particular code recommended a seismic coefficient of 0.1 for the highly seismic zone. The 1975 code was advanced for its time as it included a spectral coefficient that incorporated the period of vibration of the underlying soil layers. Such a factor entered into the UBC code in 1976.

Since the damages to engineered structures in Erzincan are very much related to the implementation of the code provisions, significant highlights of the 1975 code are summarized below. Also, the new draft code (not yet enacted) will be highlighted to provide a comparison with the 1975 code.

1975 Code

Method of analyses for aseismic design of buildings is based on the equivalent lateral static forces applied at each floor level (as in most other codes including the Uniform Building Code). The code stipulates that, in the absence of rigorous dynamic analyses, the equivalent lateral static force

concept be accepted as the minimum requirement for all reinforced concrete or steel framed structures that are not higher than 75 meters above the foundation level and that have regular load bearing systems. The provisions are also applicable for masonry buildings, chimneys, towers and elevated tanks.

The lateral load is calculated by:

$$F = CW$$

where

C = seismic coefficient $= C_0 K S I$

and

C_0 = seismic zone coefficient,

K = structural type coefficient,

S = spectral coefficient,

I = building importance coefficient.

The structural type coefficient and the building importance coefficient are similar to those in the various versions of the UBC. For one- and two-story buildings, $S = 1$, $K = 1$ and for all masonry buildings $S = 1$.

Some of the significant and unique aspects of the provisions of the 1975 code are:

a) Seismic Zone Coefficients are based on the Seismic Zoning Map. The map divides Turkey into four seismic zones and has been issued in 1972 also by the Ministry of Public Works and Resettlement following a ruling by the Council of Ministers. The map (shown in Figure 16) clearly follows the North Anatolian Fault (Figure 1) as the major Seismic Zone 1 region including the City of Erzincan. The largest coefficient adopted for Zone 1 is 0.1.

b) Spectral Coefficient (with a maximum value of 1.0) is based on the formula:

$$S = 1/[0.8 + T - T_0]$$

where

T = natural period of the structure,

T_0 = predominant period of the underlying soil.

Compared with codes of other countries in 1975, the Turkish Code was one of the first to include the spectral coefficient into the code. UBC included spectral coefficient in 1976.

Reinforced Concrete structures are constructed as "ductile systems." There is no ductility reduction factor (as reflected by the low seismic zone coefficient of 0.1 for Zone 1. Detailing specifics for reinforced concrete structures recommend confinement regions for both the uppermost and lowermost sections of columns and for beam-column joints. One significant deficiency is the lack of provision or recommendation for using deformed reinforcing bars. Again, based on observations of damaged or collapsed buildings, except for a very few cases (less than 10), only undeformed bars were used.

New Draft Code

At the time of the earthquake, a draft of a new version of the code was being circulated for final review. The proposed draft brings significant changes and approaches to the seismic design provisions, particularly in the calculation of the seismic coefficient (C) in the equivalent lateral static force formula $F = CW$:

$$C = C_0 IS/R\,.$$

The coefficient C_0, in the draft code, is proposed to be 0.4 for the seismic Zone 1. The spectral coefficient is revised as:

$$S = 1/[1.0 + T - T_0]^{2/3}$$

The coefficient R is now called the structural type factor and incorporates both the K factor of the 1975 Code and the Reduction Factor of the UBC 1991. For example R is 3.3 for reinforced concrete-framed structures and 5.0 for reinforced concrete-framed structures with "enhanced ductility." Assuming all other coefficients as 1.0, then, $C = C_0$ (new)$/R$ effectively is 0.08–0.12. Comparison with $C = C_0(1975) = 0.1$, indicates that for the structures with the ductility provisions similar to that of the 1975 Code, the coefficient C is increased by 20%. For "enhanced ductility" C is decreased by 20%.

CONCLUSIONS

One of the important factors in the fate of the collapsed buildings during the 13 March 1992 Erzincan earthquake was non-compliance and lack of implementation of the seismic design code rather than significant deficiency in the design code. The 1975 Code provided sufficient and pragmatic detailing provisions for reinforced concrete-framed buildings; if followed, it would probably have prevented the collapse of many of the lost buildings. The damages suffered during this earthquake should be a reminder of the vulnerability of many underreinforced concrete buildings in the mid- and eastern United States.

The repair and retrofitting methods applied in Erzincan and vicinity should be closely monitored to derive lessons about their effectiveness during future earthquakes. During this recent earthquake, such an opportunity was lost because some of the buildings that performed well, that were lightly or heavily damaged or that totally collapsed during the 1992 event had been repaired following the 1983 earthquake. Due to lack of documentation of the various retrofit and repair techniques, many of the lessons that could have been learned were simply missed.

It is imperative that dissemination and outreach of new techniques and modern design details of new construction as well as retrofitting of existing structures must be conveyed to the practicing engineers and contractors.

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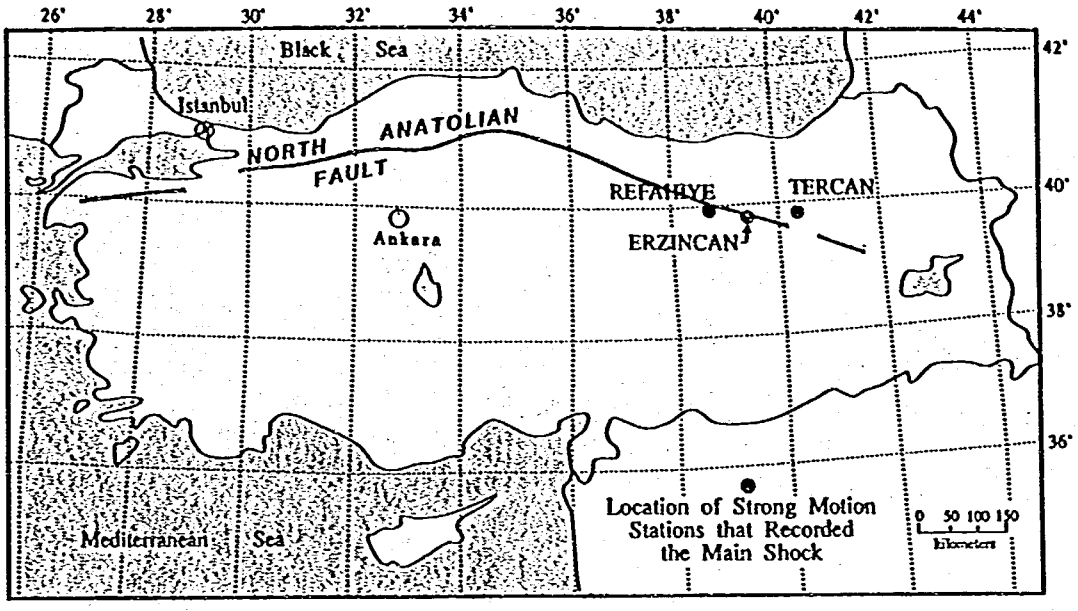


Figure 1. General map of Turkey, the North Anatolian fault and the location of strong-motion stations (at Erzincan, Tercan and Refahiye) that recorded the earthquake.

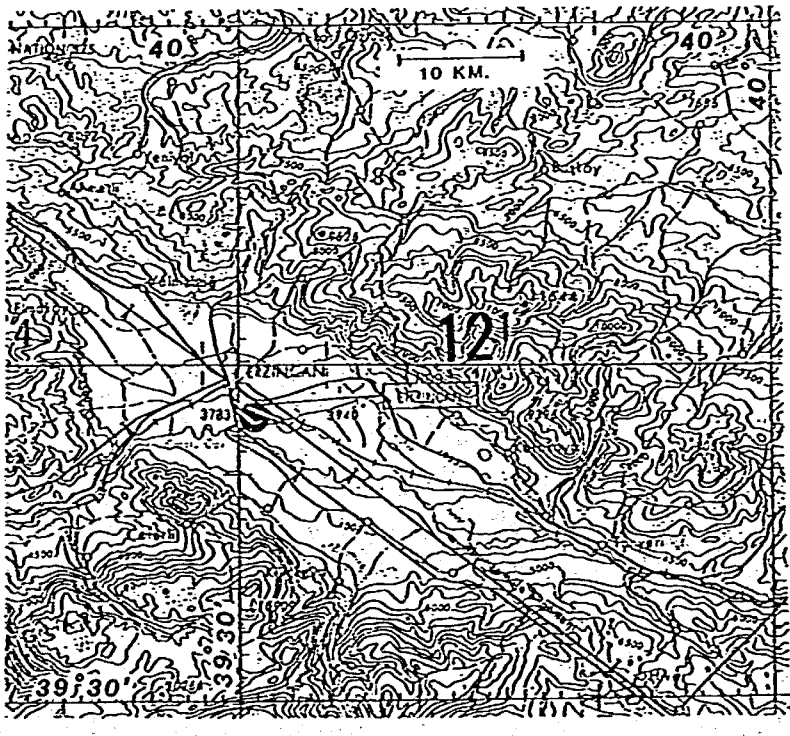


Figure 2. General topographical map of the Erzincan basin (contour intervals 500 ft.) [Source: Defense Mapping Agency, Aerospace Center, St. Louis, Mo., 1989]

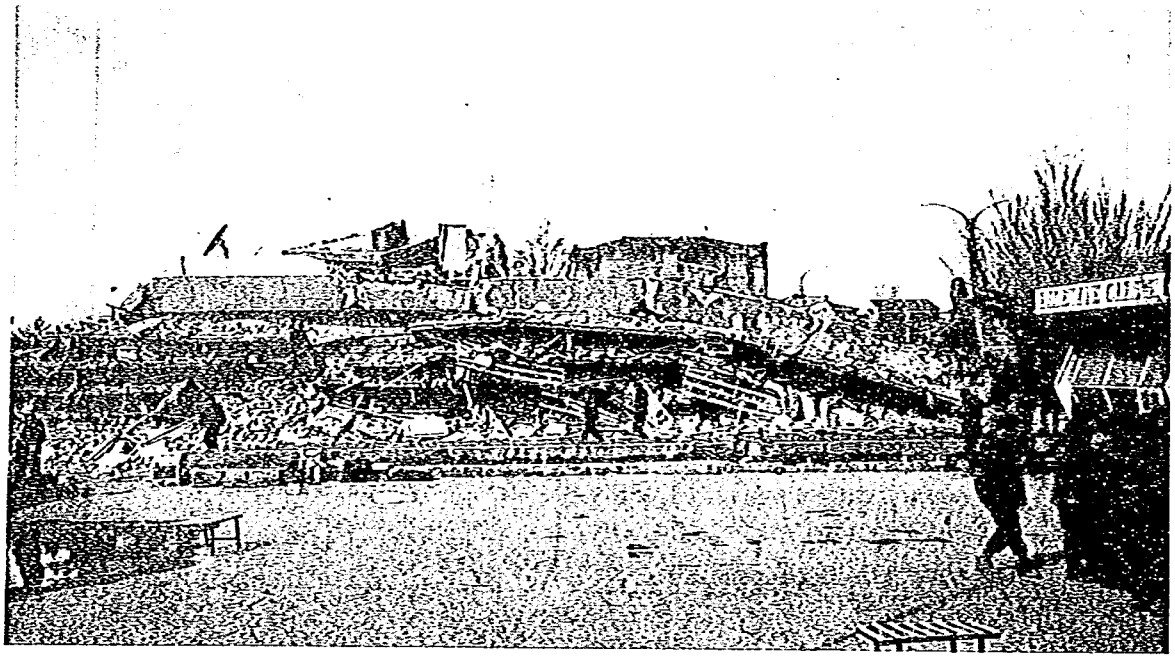


Figure 3. Typically pancaked building in Erzincan.

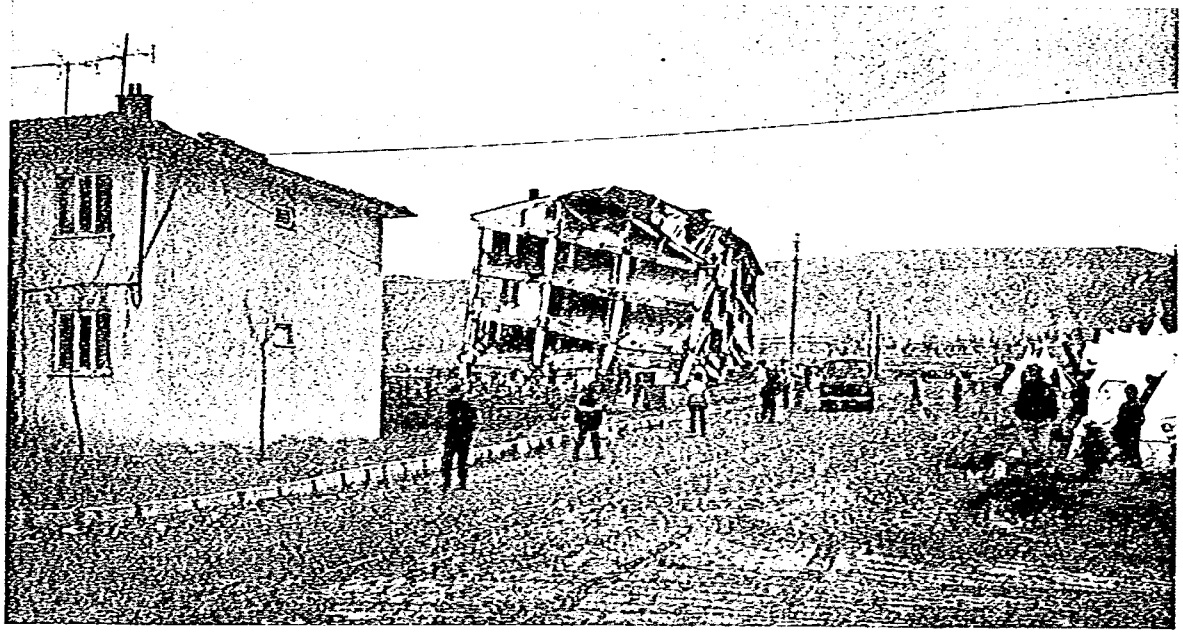


Figure 4. Typical damaged building exhibiting collapse mechanism.

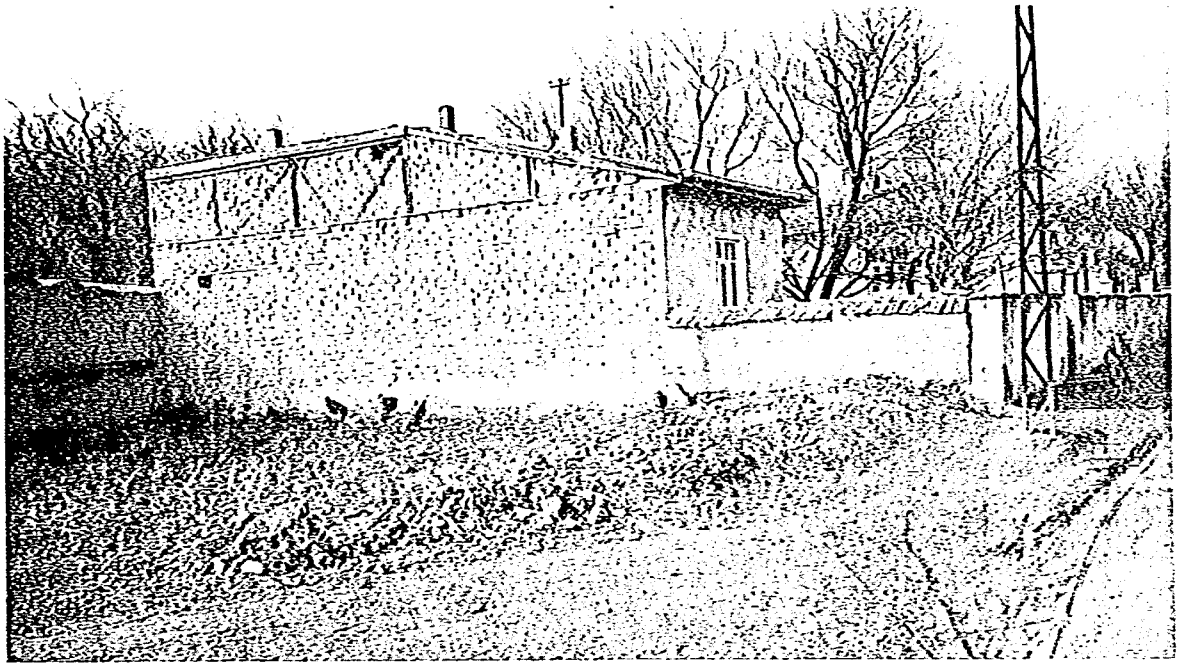


Figure 5. Undamaged single-story adobe building in Erzincan.

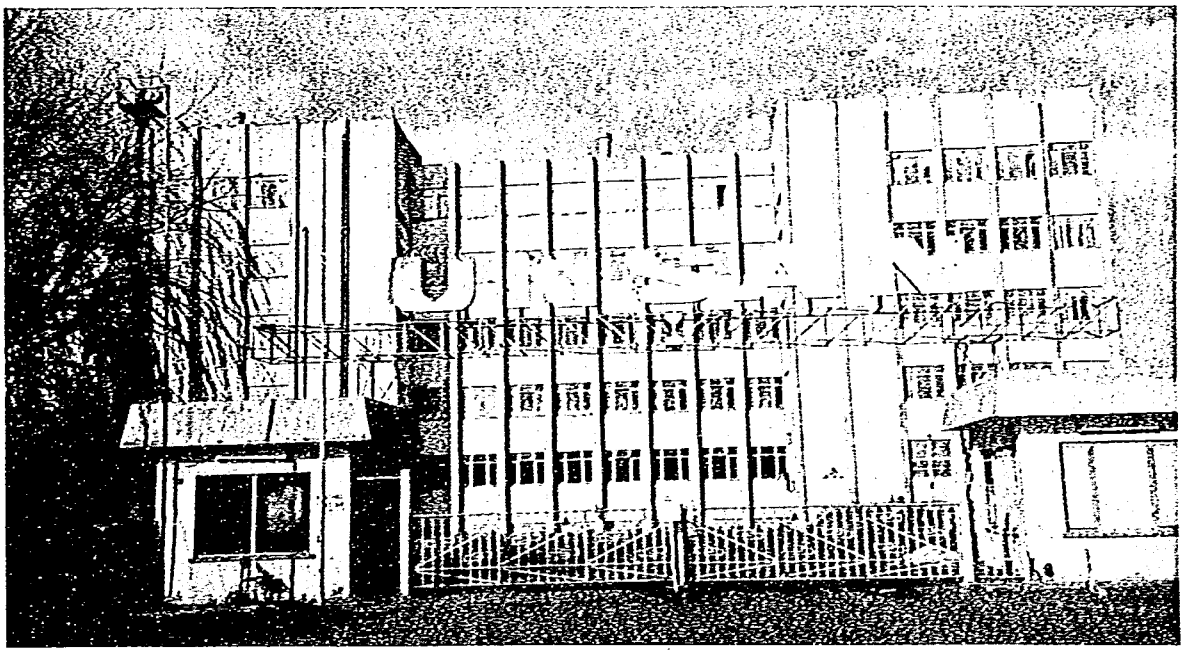


Figure 6. Unsan flour mill.

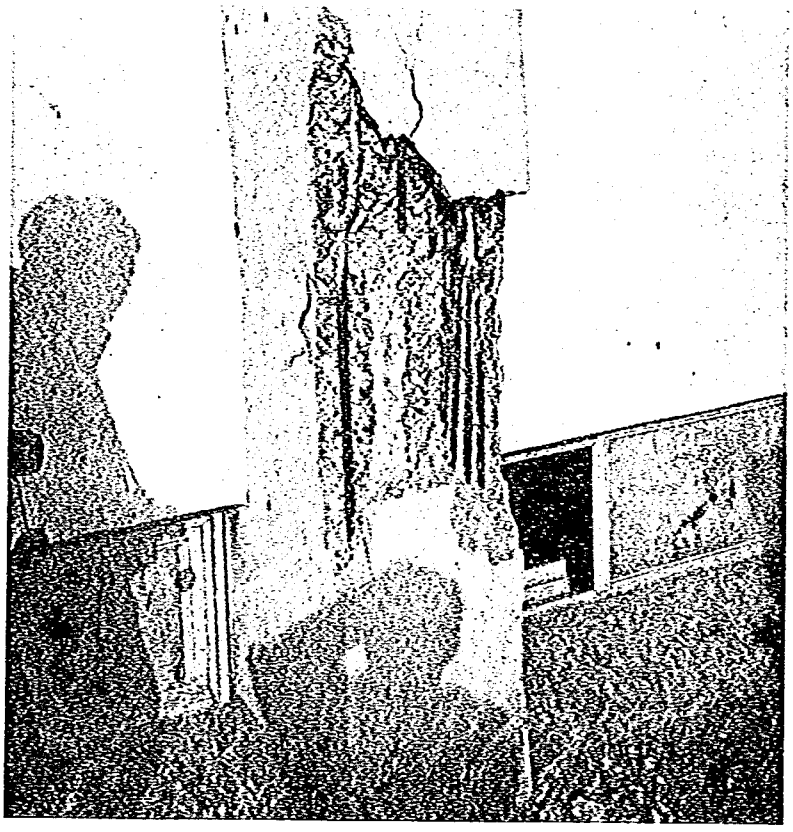


Figure 7. An external column of Unsan flour mill exhibiting short column and deficient detailing lack of shear reinforcement.

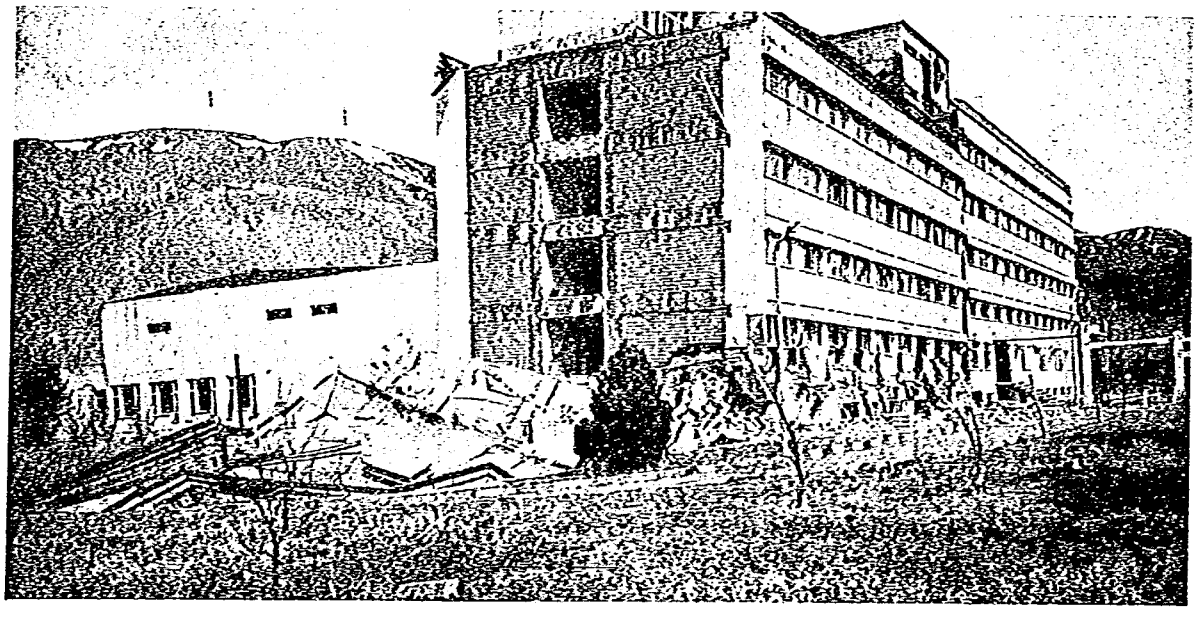


Figure 8. The Military Hospital—lost one wing, lost a story of one of the two middle wings and sustained heavy damage to the remaining two wings.

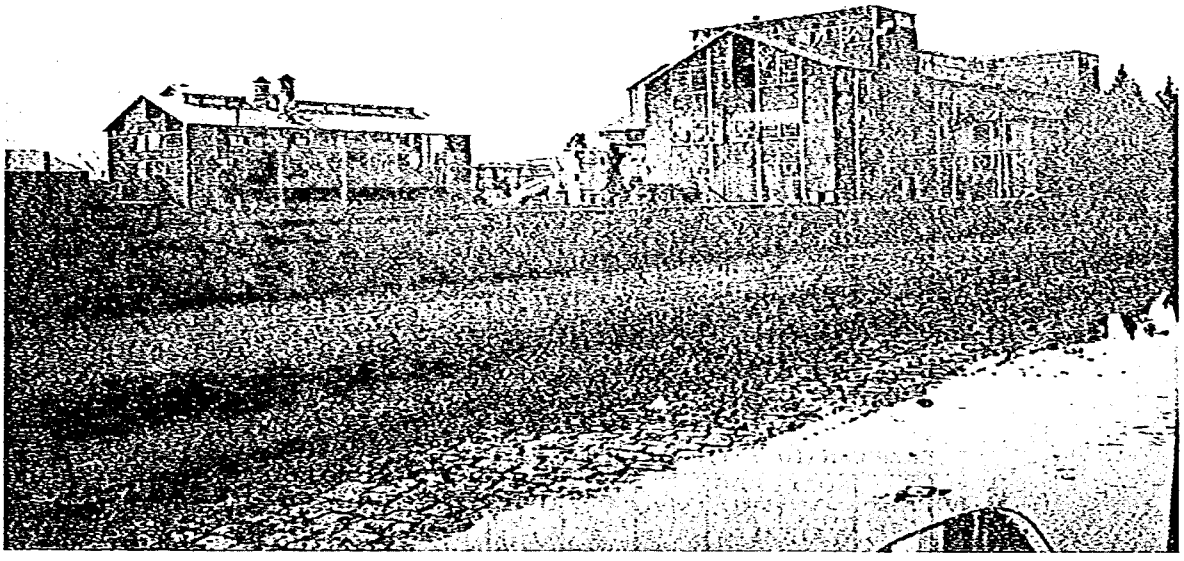


Figure 9. General view of the main sugar refinery buildings.

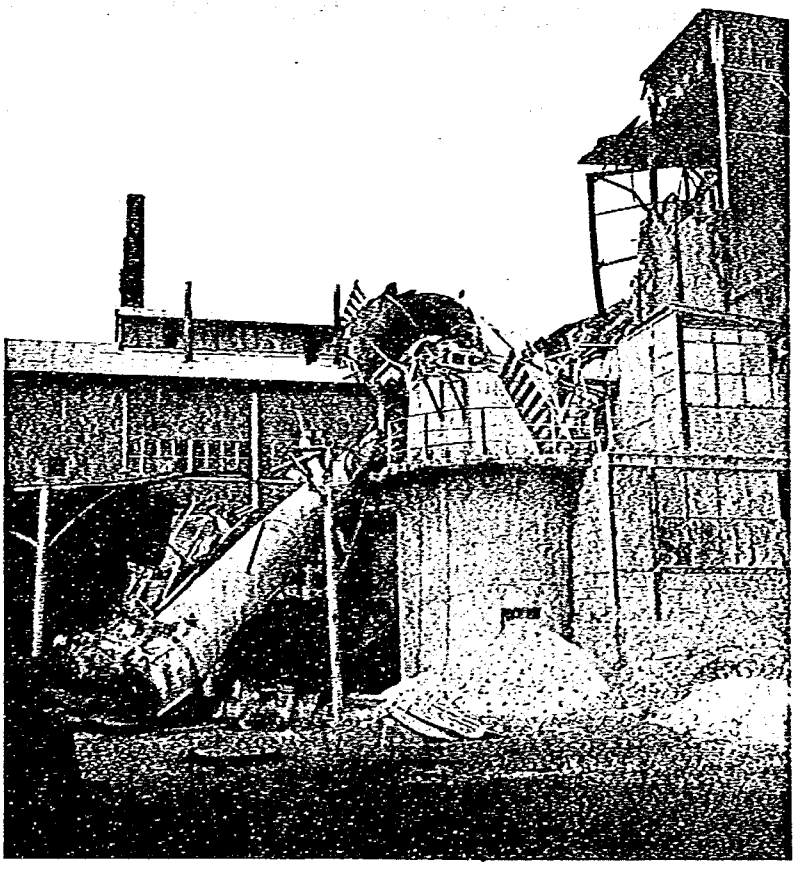


Figure 10. Collapsed stack of the calcination unit of the sugar refinery.

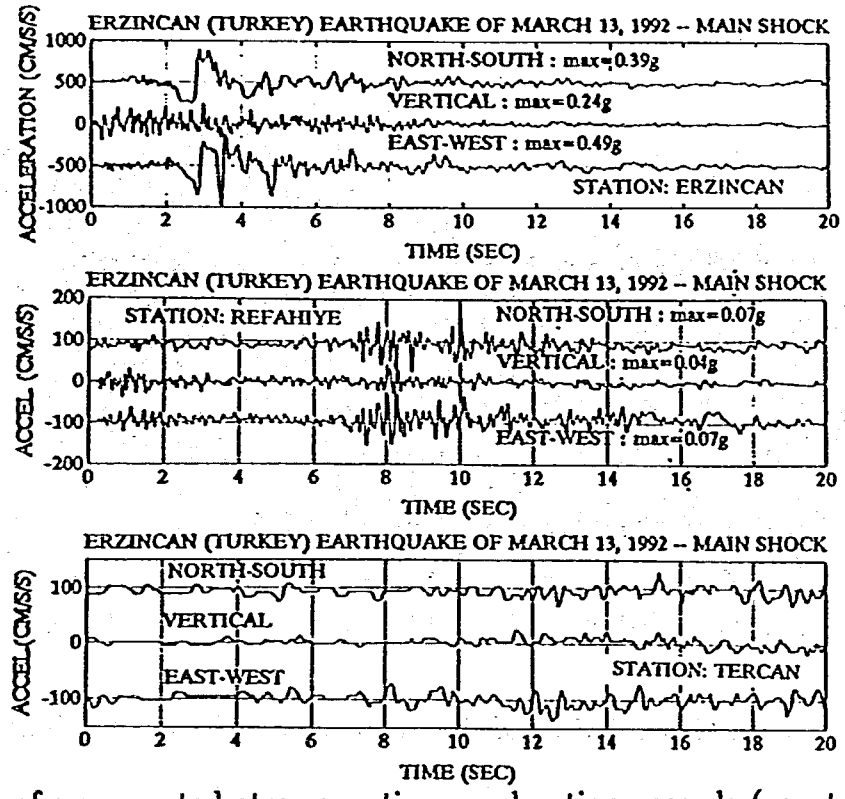


Figure 11. Plots of uncorrected strong-motion acceleration records (courtesy of General Directorate for Disaster Affairs of the Ministry of Public Works and Settlement, Republic of Turkey).

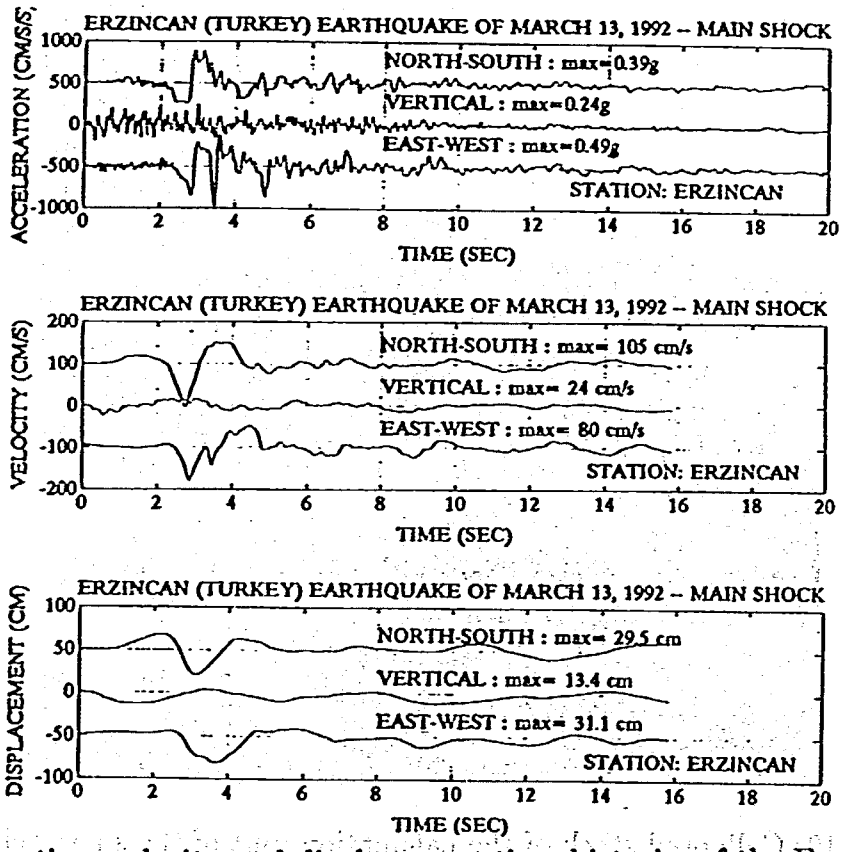


Figure 12. Acceleration, velocity and displacement time-histories of the Erzincan record.

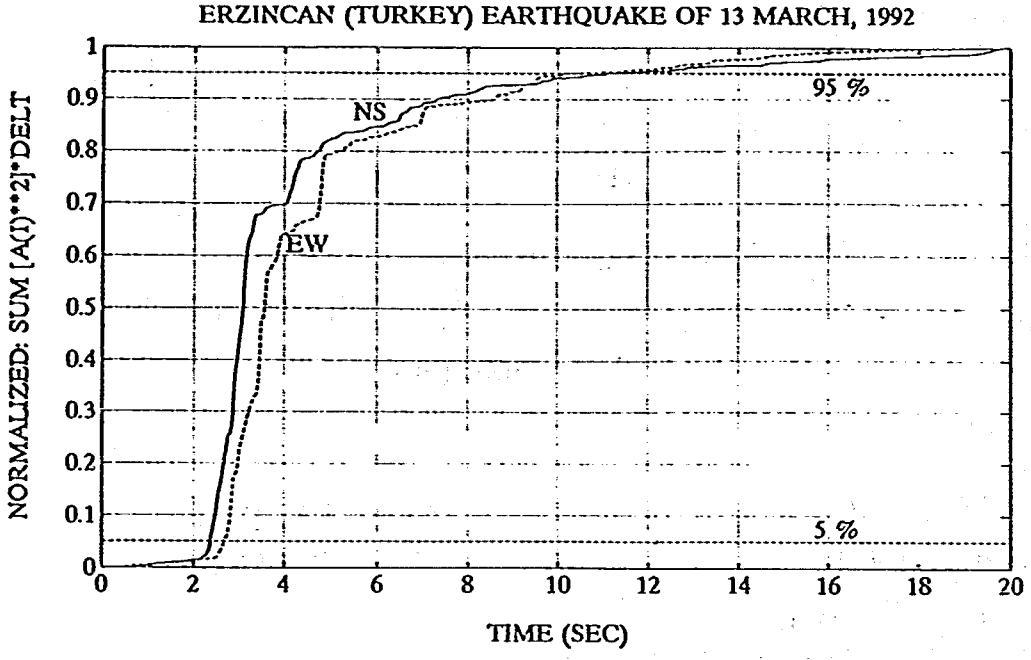


Figure 13. Plot of normalized summed squared (NS and EW) acceleration records.

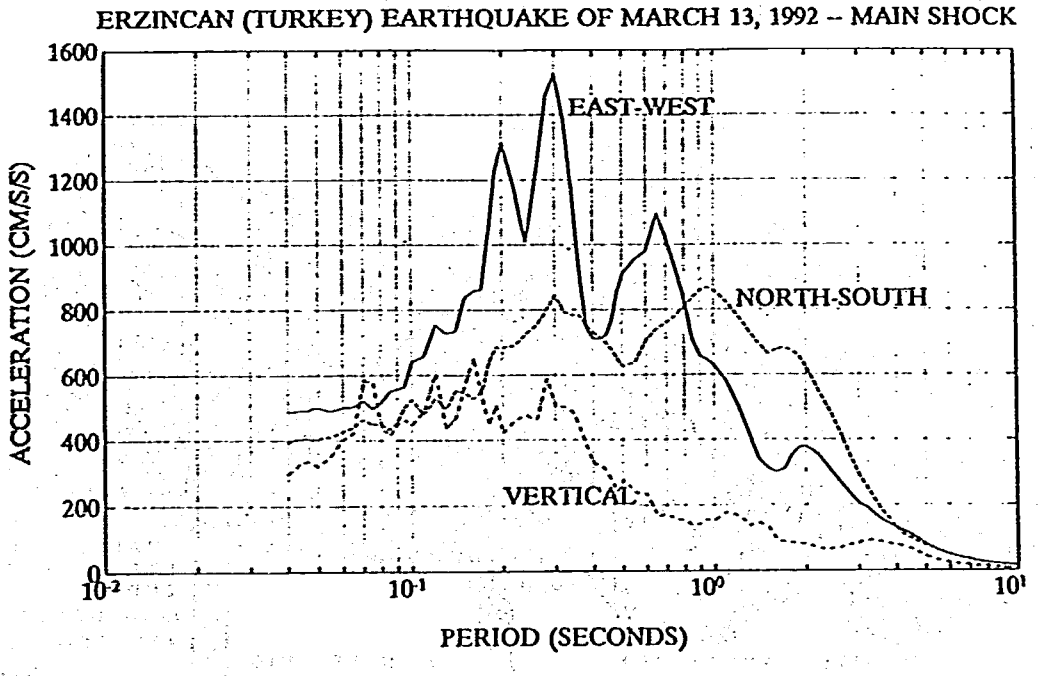


Figure 14. Acceleration response spectra (5% damping) of the Erzincan record.

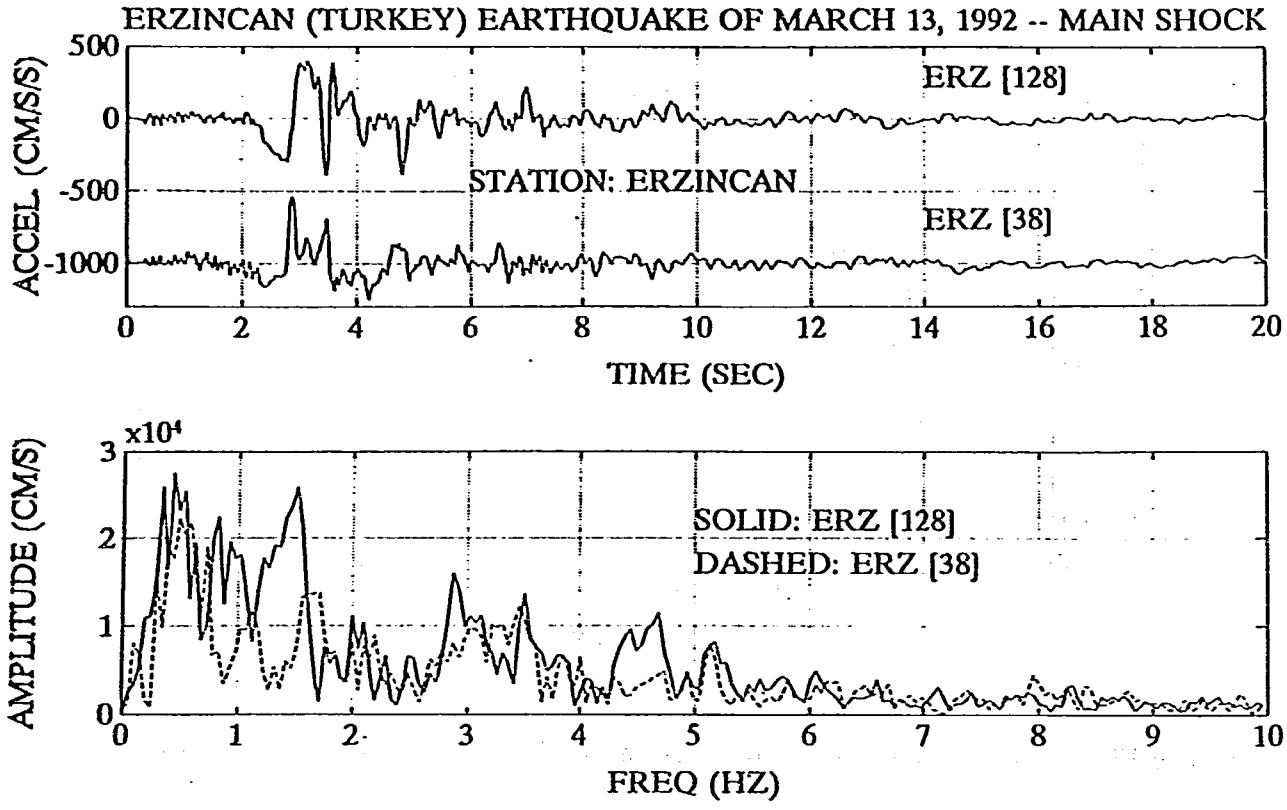


Figure 15. Horizontal components of the Erzincan acceration time-histories rotated to the dominant direction and corresponding amplitude spectra.

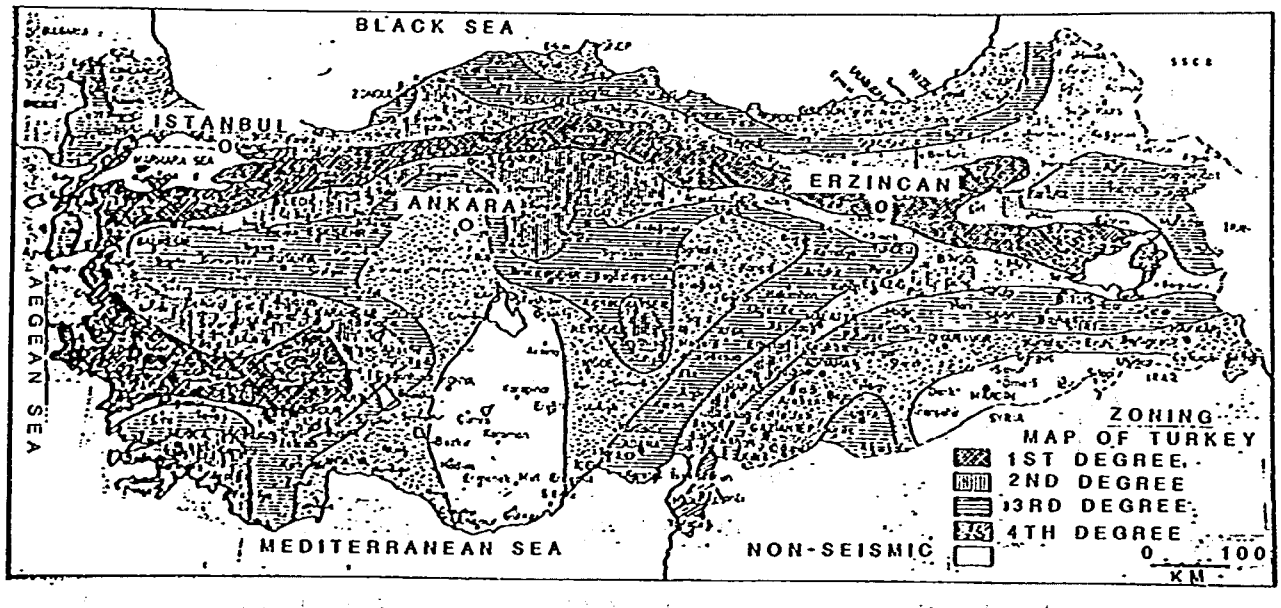


Figure 16. Earthquake zoning map of Turkey.

Developing a Seismic Safety Program

by

Donald G. Eagling, P.E.*

ABSTRACT

Although the field of seismic safety is in the domain of the engineering profession, funding to carry out a seismic safety program is normally derived from the budgets that managers (or owners) need to carry out the missions of their organizations. Consequently, it is generally the manager rather than the engineer who must make most of the key decisions which are needed to carry out an effective seismic safety program. It is also the manager who must resolve issues of risk management which accompany seismic hazards and involve life safety, damage control, potential evacuation costs, potential liabilities and cost-benefit considerations. It is therefore very important that facility managers have a broad understanding of the comprehensive nature of an effective earthquake program as well as an awareness of the socio-political, legal and economic risks that often impede the progress and success of such programs.

This paper provides practical guidance for managers who must carry out a seismic safety program including advice for selection of an earthquake safety consultant, the main elements of a balanced program, and risk management techniques for avoiding common pitfalls and achieving expeditious results.

KEYWORDS: risk management; seismic safety.

1. INTRODUCTION

The technical aspects of earthquake safety are normally within the domain and control of the engineering profession and academia. On the other hand, the cost of earthquake safety is a drain on the very budgets that managers (or owners) of facilities need to carry out the missions of their organizations. This is true whether the enterprise is funded privately or by government sources. Consequently, the responsible manager must be heavily involved in the planning, scheduling and funding of an earthquake safety program if it is to be successful. Thus, it is very important that managers receive practical professional advice so they can carry out a cost effective program and properly manage the associated risks. In the end, the responsibility for decision must lie with the manager, but good communication and mutual trust with a practical earthquake engineering consultant is essential to provide the manager with an extension of expertise in this specialized field. On the other hand, the engineering profession has a public obligation to see that earthquake safety is achieved at minimum cost and not lost in the process of lengthy and costly investigation and voluminous reporting.

This paper provides advice for the owner or manager who must carry out an earthquake safety program from the perspective of an engineer who has had such an experience as a facility manager.

The scope, depth and focus required to carry out a satisfactory program will vary considerably with the age of a facility, the risk involved and the quality of design which was applied during its construction history. For a new and growing facility the focus will be on design and construction. For an older facility the need to evaluate existing conditions and prioritize projects for abatement of seismic hazards will receive most attention. For the majority of sites, however, a balanced program will be most effective in preventing further development of new hazards while reducing the backlog of old ones.

Those structural engineers who are experienced in earthquake engineering and have reviewed a number of facilities, both in government and

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private enterprise, have found a wide variety of serious seismic deficiencies that the owner or manager were unaware existed. This is not unusual even in areas of the country where seismic design provisions have been part of the building code for many years.

East of California, few conventional buildings in the United States have been designed for earthquakes, even where there has been a history of earthquakes of sufficient intensity to damage buildings. At those locations where the potential for seismic destruction exists along with a legacy of hazardous buildings and contents, the prospect of carrying out a comprehensive earthquake safety program is indeed challenging.

The owner or manager who is responsible for a facility is usually unfamiliar with earthquake engineering and may tend to look for answers in techniques that are more sophisticated than are required to solve the actual problems in earthquake safety. The approach to solutions of these problems can become so academic, ponderous, and expensive that abatement of the seismic hazards simply doesn't get done in a timely manner.

In recent years, the state-of-the-art in seismology, geo-technical theory, and the dynamic analysis of structures has progressed tremendously. Spurred on by the need to resolve questions in seismic safety for nuclear power plants, the field has become very compartmentalized and specialized. The great strides made in these specialties have contributed significantly to the field of earthquake engineering and public safety. Unfortunately, it is easy for the responsible manager or plant engineer to "fall into a crack" between these experts who quite naturally tend to resolve seismic questions based on their own specialties.

Often more time and money are expended analyzing the problems in earthquake safety than would be needed to design practical solutions to these same problems. To gain a better perspective it is important to understand that most problems found in existing construction are a result of not implementing what has been known and observed about earthquake-resistant construction for a long time. Structural engineers who have observed and studied earthquake-damaged buildings are able to diagnose hazardous deficiencies in existing buildings rather easily and efficiently, often without complex calculations. They know which building types are hazardous. They also know standard methods for rehabilitating them.

2. SELECTING AN EARTHQUAKE SAFETY CONSULTANT

The most important thing a manager can do to initiate an effective and economical earthquake safety program is to hire an experienced earthquake engineer as an earthquake safety consultant who is strong on design and tends to keep analysis straight-forward and simple. Occasionally, there is good reason to apply structural dynamics to provide a better understanding of a complex problem, but not often. The earthquake engineer, working closely with the manager, should advise on selection of other consultants, define their scope of work, coordinate their work with the overall program and ensure that the owner is not victimized by unnecessary or impractical studies.

Selecting the right structural engineering firm to counsel a practical way through the complexities and pitfalls which can befall earthquake safety programs is the most important decision a manager can make.

This is also true when selecting geo-technical consultants. The level of sophistication in state-of-the-art techniques for predicting the intensity of ground shaking is intimidating. There is a strong tendency for both consultants and clients to believe the predictions to be more accurate than history shows they are. This tendency may lead participants to spend more money and time than the exercise is worth. The illusion of security thus developed is apt to be in direct proportion to the degree of sophistication applied.

Even when structural dynamics is to be employed, the selection of an effective ground-motion input can be a relatively simple matter. There is little to be gained by exhaustive site studies because history shows that the prediction of ground motion is indeed an inaccurate science. The inaccuracies of input often can be accommodated in good structural design.

Potential earthquake engineering and geo-technical consultants should be asked to explain

in simple terms how they expect to approach the project, what techniques will be applied and what they expect to find. They should provide examples of previous work, names of clients, and cost history. The owner should be sure that the lead consultant selected has strong design experience and has made field investigations of earthquake damage.

3. THE BALANCED PROGRAM

An effective earthquake safety program is analogous to an effective lateral-force-resisting system; it should have no weak links. Several basic precautions should be taken in establishing a program; of primary importance are the following five:

First, make certain that planned new buildings are not being inadequately designed while the process of earthquake safety review of existing buildings is underway. This is a profound admonition, but the possibility of its happening is real. It can be avoided by using plan-check or third-party review, prior to construction, to ensure that the calculations and designs of new structures and rehabilitation projects are done properly to resist earthquakes. It is embarrassing for a manager to find that a newly designed and constructed building is worse than an old one, but it happens.

Design criteria should be clearly defined and simple to use. Standards and model codes are readily available and should be used. Complex approaches or criteria should not be applied unless the need is clearly established. Criteria should be practical. At most sites, many building modifications, experimental setups, equipment installations, cabinetry, nonstructural components and other projects are designed regularly by architects, mechanical engineers, designers, and other nonstructural engineers who do not have sufficient background in seismic analysis and design. Nevertheless, if the criteria are simple and straight-forward these minor subprojects will be designed and built with adequate earthquake resistance. The more significant structural problems should be handled by registered structural engineers. Also, research or production facilities using hazardous materials should be carefully reviewed for seismic safety by a structural engineer. A professional engineer's stamp and signature as well as a third-party review should be required.

Second, the site should be reviewed for geologic and seismic hazards. Potential conditions that are inherently hazardous in ground shaking should be identified. These may include the following:

1. Unstable slopes and existing landslides;
2. Areas of low-density saturated granular soils subject to densification and subsidence;
3. Areas of low-density saturated granular soils subject to liquefaction;
4. Areas where sensitive clays may be subject to strength loss under heavy ground shaking;
5. Areas where flooding would occur due to failure of an up-slope levee or dam.

Active faults should be identified and a geologic map of the site developed.

The site review need not be rigorous in detail unless the potential hazards pose a high risk for an existing building or lifeline. If a new building or improvement is planned, the specific siting can be examined in more detail. The main point is to recognize potential hazards and take them into account. For example, it would be folly to permit the typical one-third increase in allowable soil bearing capacity for seismic loading in an area of sensitive clays subject to strength loss under ground shaking. The initial review should be quite broad and superficial in character. It is important that it be carried out by a geologist or soils engineer who understands the nature of soil dynamics, preferably one who has experience with earthquakes. Generally, except for fault rupture, each of the potential hazards that may exist can be mitigated through standard stabilization practices, or by simply avoiding them in the case of new construction. Sometimes fault movement can be accommodated, or the effect of fault movement mitigated, if it is known where surface ruptures are likely to occur.

Third, survey and evaluate all existing buildings and structures to determine their earthquake safety ratings. A structural engineer experienced in earthquake damage investigation should do the job. The assessment should be kept simple. A basic requirement is to find out what has to be done to insure that each building has a predicable lateral-force-resisting system. The job of rehabilitation should be started one step at a time, reducing liability on a priority basis.

Given a limited budget it is important to determine which buildings will have the greatest benefit for the money spent for improvements in life-safety and property damage. Of course, practical risk management must also address those socio-political issues which encompass and plague earthquake safety programs and pose questions of public and personal liability. Practical risk management is discussed in the next section of this paper.

Fourth, make an earthquake-hazards survey of the contents each building including operations, equipment, hazardous materials storage and nonstructural building elements. Obvious hazards, such as loose-item overhead storage, should be corrected by building managers or operations supervisors. Most operational hazards are obvious to one simply observing the scene and imagining an earthquake taking place. Tipping hazards, such as storage cabinets, tall files, library shelving, and similar furnishings, should be braced or anchored. Tie-downs should be installed on plant equipment such as transformers, emergency generators, tanks, elevator drives, fans, motors, and similar units. Apply a simple and judgmental priority system to use limited resources economically.

Fifth, develop an emergency plan to recover from a destructive earthquake. Apply the scenario technique to develop a probable model for the aftermath of an intense earthquake. Use those department heads who will have to handle the recovery to lead the planning effort. Reduce obstacles to recovery by eliminating obvious hazards and ensuring that the supplies and equipment that will be needed in an emergency will in fact be available. Lifelines, such as water supply lines, power systems, storm and sanitary sewers, transportation and communications systems also should be surveyed with earthquakes in mind. The consequences of possible facility losses can be mitigated by careful emergency planning, and the potential for loss of a given facility reduced by "hardening" the lifelines that would likely be in jeopardy during an earthquake.

Self-help planning, preparation, and training should be key elements in any emergency response plan for earthquake safety. Make sure that building managers and operations supervisors understand this fact and let them take part in the preparation of local emergency plans.

In the process of administering an earthquake safety program, a manager will encounter differences of opinion among the engineers and consultants involved. These differences reflect the perspectives of experts and tend to give the managers some insight into the practical state-of-the-art. It also reminds us that there often is more than one answer to a given problem. When a problem is particularly difficult and costs, risks, or liabilities are high, it is unquestionably worthwhile to get more than one opinion.

The question arises, how does a manager resolve technical differences of opinion between two consultants on subject-matter about which the manager feels inadequate.

The best answer lies in the manager's usual role, that of managing the multi-disciplinary functions of a technical complex such as a major research and development laboratory or a sophisticated production facility. The development of good communication and mutual trust with a practical earthquake engineering consultant will provide the manager with an extension of expertise in this specialized field much as it does in any other specialized field under his or her management. In the end the responsibility must lie with the manager and it is important to realize ahead of time that technical differences of opinion are apt to arise about earthquake safety management. A good earthquake engineering consultant will be interested in the goal of practical earthquake safety rather than earthquake engineering for its own sake. This extension of expertise through selection of wise counsel is a challenge that most managers face in other facets of their responsibilities.

When questions regarding technical differences of opinion or criteria persist, it is important that they are resolved by some due process within a technical framework that will stand the test of future technical and legal review. The designer and the manager should be reasonably protected by the due process involved assuming each fulfills his professional responsibilities satisfactorily.

4. RISK MANAGEMENT

Risk management from the manager's point of view is an important part of a comprehensive

earthquake safety program. It is, of course, inherent in the establishment of lateral force criteria for the design of buildings and equipment including research facilities and experimental set-ups. Earthquake codes establish *minimum* requirements for life safety and essentially provide protection against collapse. Damage control is not a prime objective. Many code provisions, such as the prescribed limitations on "drift" (deflection based upon prescribed lateral forces), have the effect of reducing damage, but one must realize that the real earthquake will cause deflections much greater than "code deflections." When damage control is an important consideration, the design must account for these larger deflections. This protection is not inherent in the Code. One approach is to analyze a structure with the objective of predicting or estimating the location and extent of damage that will probably result from a major earthquake. *In this way, additional attention to design detail can be applied specifically in the area of concern to "buy insurance" against damage for little extra cost, an example of good risk management.*

Liability is certainly a legitimate concern in risk management. Often, however, this concern is translated into a legalistic solution, rather than a practical solution which actually mitigates the seismic hazard. For example, the Code is generally not retroactive so it is not legally incumbent upon the "responsible official" to upgrade an existing building to current standards. Nevertheless, some earlier editions of the Code made it "legally" possible to design and construct a hazardous building. A non-ductile reinforced concrete frame building is a good example. A decision not to review an existing building because it was once designed to Code is a legalistic solution to avoid liability, but it does nothing to mitigate any seismic hazard that may exist. This legalistic position under the protective umbrella of the Code is becoming more difficult to assume because the engineering profession is now much more aware of hazardous buildings which have been "built to Code." One cannot be certain to avoid liability by remaining ignorant of the hazard. The legal issue may well be whether or not such a building is commonly known to be hazardous by the profession.

On the other hand, the risk of liability should be managed carefully when structural hazards are revealed as a result of seismic safety surveys and reports. For example, if a building is reported to be a collapse hazard, the official responsible for the safety of the occupants should take steps to mitigate the hazard. It is important to actively seek funds to abate the hazard and to inform the occupants that the building is deficient. While these steps will not guarantee immunity, failure to take them certainly increases liability.

The problems of funding rehabilitation work are unusually difficult and the solutions time consuming. The period during which the hazard may continue to exist increases the risk of liability.

A public agency cannot legally go out of business; therefore, it cannot spend its available funds so heavily for rehabilitation that it cannot fulfill its prescribed mission. This fact weighs against liability, but does not provide immunity. In the event that a damaging earthquake results in litigation, the pertinent issue is: what funds were available to the responsible official and what were they used for? With this in mind, one approach to managing the risk of liability is to take the basic risk-reducing steps which can be identified immediately (assuming the hazard cannot be easily abated in a short item), then follow due process to find a permanent solution to the problem. Examples of emergency steps are: adding temporary supports, reinforcing structural joints, installing epoxy grouting, removing potential hazards, or even reducing the occupancy loading. Normally, some emergency funding can be found for such purposes while the more time-consuming tasks of evolving a permanent solution and developing adequate funding take place. The important point is that responsible action (within constraints) must take place if liability is to be minimized.

It is likely that a seismic safety survey will turn up a number of structurally deficient buildings and facilities. This has been the common result where such reviews have been carried out. For years, many buildings were designed with non-ductile, reinforced-concrete frames that were then permitted by Code, but are now known to have poor seismic resistance. Many older buildings have no formal or predictable lateral force resisting system. Sometimes building alterations have reduced or destroyed the resistance incorporated in the original design. The point is, a seismic safety survey will likely present the responsible official with a multiplicity of hazards and risks to manage.

It is important to mitigate risks on the basis of priority, but it is even more important not to get bogged down in a complex series of studies or a methodology which slows the process of abatement. A simplistic priority system based upon some due process and responsible professional judgment is sufficient. As with the Richter scale for measuring the magnitude of earthquakes, it is not as important that the result is accurate as it is that relative size (or priority) is easily and quickly established.

The same selection principle should be applied when seismic safety surveys are initially carried out. That is, the priority system for the sequence in which buildings and facilities are surveyed should be simplistic and direct. Obvious problems, possible collapse hazards and high risk facilities such as those with toxic dispersible contents, should be reviewed early. Emergency recovery facilities, high occupancy buildings and lifeline facilities should also be early on the list. It is important that the survey should not be held back by an academic approach to the multiplicity of potential hazards and the complexity of the problem. The recommendation is to keep the approach simplistic, rely on good professional judgment and move forward.

One stumbling block to seismic safety is the ever present concern for accurately estimating the intensity of the potential earthquake that a given site might experience. For various reasons, political, academic and psychological, the immediate, basic need to find out whether a brace is indeed missing unfortunately too often becomes secondary to guessing how big the earthquake is going to be. Commonly, the problems uncovered in a seismic survey have less to do with lateral force criteria to be applied than with other design deficiencies such as missing links, brittle members or connections, lack of continuity or just poor construction. When a building or facility is found deficient, the size of potential seismic input is only one of the considerations which may be brought to bear on the corrective measure. Usually there will be ample time to develop detailed lateral force criteria after the real problem is revealed.

The recommendation is: don't delay the seismic survey in order to study the potential seismicity of the site. Experience shows that this approach is not good risk management.

The design criteria for new buildings are rather well established in the latest Codes, but this is not true for rehabilitation work. Here good risk management requires more careful consideration. The lateral force provisions in the Code provide good guidelines for rehabilitation design, but often, lateral force resistance is only part of the problem. As discussed, brittleness, lack of continuity or redundancy, deflections, poor detailing, poor workmanship, and many other possible deficiencies may exist. From the standpoint of risk management, it is even more important than in the design of new facilities that the designer of rehabilitation work give particular attention to a complete seismic diagnosis and criteria development.

Once it is determined that a building has a serious structural deficiency that must be corrected, another kind of problem is often present. The building may have other Code deficiencies by current standards that are not central to the main hazard. That is, the main structural deficiency may be a collapse hazard, but the other Code deficiencies may not present life safety hazards. The question may then arise: will the responsible official (or the engineer who designs the rehabilitation work) be put in a position of liability if the design does not correct all the deficiencies by current Code standards? Often it is not economically feasible or even good risk management to correct everything. For example, funding may be used to correct two collapse hazards rather than spend it all on one building to bring everything up to Code.

The recommendation is to achieve the most life safety for the funding available, but mitigate possible liabilities for the design professional by careful due process. For example: a "criteria board" can be established consisting of professionally knowledgeable members such as the rehabilitation designer, the plan checker (preferably an independent consultant), and the "building official" (the in-house person responsible for design and construction).

The authority to set seismic criteria should then be officially delegated to this "expert" group. Court judgments in the United States have held that the responsible authority will be immune from liability for acting in a discretionary manner if this person has the authority and knowledge to do so. The authority must be properly delegated and

"knowledge" implies professional judgment in the practice involved. The rehabilitation design engineer will be reasonably protected by the due process involved assuming he fulfills his design responsibilities satisfactorily.

High on the list of effective risk management techniques related to seismic safety is the so-called "third party plan check" that has been mentioned earlier in this paper. This independent plan check, together with proper field inspection of construction (whether new or rehabilitation work), is highly recommended as one of the best ways of assuring seismic safety in structural design and construction.

In "earthquake country," engineers generally believe the buildings they design will be subjected to a damaging earthquake during the lifetime of the structure. They don't consider the hazard to be one that in all likelihood will not occur. They are also aware that the earthquake is the real *master inspector*. If there is a gap in the lateral-force-resisting system, the consequences will be more serious than statistical. This point of view has a very positive effect upon risk management as applied to design.

In areas where earthquakes are very rare events, it is difficult for engineers to take them seriously, particularly in the design of conventional structures. It is a fact that few buildings in the midwestern and eastern United States have been designed for earthquake safety, even in those areas where it is well known that damaging earthquakes have occurred in the historic past. Many buildings are constructed of unreinforced unit masonry, one of the building types particularly susceptible to earthquake damage and collapse. A great deal of progress could be made by simply avoiding the use of unreinforced unit masonry in new construction.

In "earthquake country," the choice of criteria for seismic design can be a relatively simple matter, if one believes the great earthquake is imminent. In this case, the design earthquake can be taken relatively close to the maximum credible earthquake. This might correspond to 0.2g base shear using the static equivalent design approach or to 0.8g using ground spectra acceleration for an inelastic dynamic analysis. This design approach is based on what is known as the "minimax decision," since it minimizes the maximum losses in the future.

In areas of the country where the potential for a great earthquake exists, but the probability is extremely low, the choice of criteria can be very difficult. The maximum credible earthquake is not a practical choice for the design of most conventional structures in such areas. *A more fundamental consideration relates to the decision whether or not to design for seismic forces at all. However, considerable seismic resistance can be achieved for very little extra cost by simply applying the principles of equivalent static lateral force design and making sure that the system is continuous and ductile.* The insurance available in such a minimal approach is a true bargain in risk management. The lateral-force factor to be utilized is of secondary importance. However, if *Building Code* values are used, one has some assurance that this choice is properly coordinated with other design provisions of the Code.

5. CONCLUDING COMMENTS

This paper is based upon experience with the earthquake safety program at the Lawrence Berkeley Laboratory (LBL) where 34 buildings were strengthened over a 20 year period following the San Fernando Earthquake in 1971. As a consequence of that experience, LBL published a "Seismic Safety Guide" in 1983 for the U.S. Department of Energy (DOE). It was written for those managers of DOE facilities (and engineers) who had little background in earthquake engineering. Its guidelines include hazard identification and evaluation, site planning, the evaluation and rehabilitation of existing facilities, the design of new facilities, lifelines, operational safety, emergency planning and the management of risks and liabilities.

Since 1983 great progress has taken place in earthquake engineering. Also, a number of damaging earthquakes have taken place where structures were designed to resist them. As a result a wealth of new information has become available. A new edition of the "Seismic Safety Guide" will be published in 1993 to incorporate and interpret this new material so that the managers of DOE facilities can achieve a comfortable level of understanding for the decisions they must make relative to seismic risk management.

The format for the "Guide" will follow the original edition. Each chapter will be written by an experienced professional for an audience composed of managers or engineers with little background in earthquake engineering. Comment and advice from the facility manager's perspective will be provided in the foreword to each author's section. For those interested in a much more detailed treatment of this paper's topic, the new "Seismic Safety Guide" will be published in the Spring of 1993.

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Desk-Top Model Buildings for Dynamic Earthquake Response Demonstrations

by

A. Gerald Brady*

ABSTRACT

Models of buildings that illustrate dynamic resonance behavior when excited by hand are designed and built. Two types of buildings are considered, one with columns stronger than floors, the other with columns weaker than floors. Combinations and variations of these two types are possible. Floor masses and column stiffnesses are chosen in order that the frequency of the second mode is approximately five cycles per second, so that first and second modes can be excited manually. The models are expected to be resonated by hand by schoolchildren or persons unfamiliar with the dynamic resonant response of tall buildings, to gain an understanding of structural behavior during earthquakes. Among other things, this experience will develop a level of confidence in the builder and experimenter should they be in a high-rise building during an earthquake, sensing both these resonances and other violent shaking.

KEY WORDS: building models; construction; dynamic; earthquake; resonance; vibration.

1. INTRODUCTION

The physical characteristics and construction of desk-top model structures that demonstrate earthquake shaking effects have probably been described in many research thesis reports. The models here, though, are primarily hands-on displays where the participant provides the shaking input by hand in the form of a sliding to-and-fro motion, of an easily held block of wood acting as the foundation for the structure. The models are suitable for schoolteachers in regions of high seismicity to acquaint children with the kind of motion they might experience if they are in a tall building during an earthquake, rather than a low-rise school building, and to give children a perspective of how the high-rise buildings they are familiar with in their cities shake during earthquakes in ways that are understood, and have therefore been accounted for, by the structural engineering profession. The models are also suitable for showing this behavior in a simple fashion to people outside the seismic engineering profession.

Almost every university or institution with a strong earthquake engineering component will have had, at some time, a motor-driven exhibit shaking a platform in the north-south and east-west directions simultaneously while a structure bolted to the plat-

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form illustrates its seismic response. A more current form for this display is a computer program where earthquake magnitude and epicentral distance can be entered, together with structure height and construction material. Detailed structural behavior can be shown on a color screen, and catastrophic collapse provides audience excitement. The models described in this paper, however, are not complicated, and their hands-on feature provide a different experience. It can actually be difficult to find the perfect frequency and thus the perfect timing to excite, slowly and steadily, a flexible or resilient structure into one of its vibrational resonances. Once learned, the ability to excite these model resonances remains, as does the ability to resonate other structures like fishing rods, handrails, and small flagpoles, for example.

The models show how a building resonates in a particular direction when the ground shakes in a steady vibrational to-and-fro motion in the same direction. The frequencies at which this happens have been calculated first, and the models are designed and constructed so that the frequencies are within the range that the operator can readily provide, namely from ½ to 5 cycles per second (cps). This range includes the resonant frequencies of some typical tall buildings (Housner and Brady, 1963). The models show how quickly a resonating mode can be excited and how little input amplitudes are needed to maintain excessive, highly visible, structural amplitudes.

There are two basic models described here, representing the following buildings:

A. Strong column–weak beam (bending beam) buildings: Buildings whose columns are far stronger and stiffer than the floors and floor beams so that the cantilever bending characteristics of the columns are unaffected by the presence of the floors.

B. Weak column–strong beam (shear beam) buildings: Buildings whose floors and floor beams, being far stiffer than the columns, remain flat and horizontal during vibrations.

The columns are forced into slight *S*-bends between floors.

There are many versions and combinations of these two that permit modeling:

1. Buildings that are different in height.

2. Buildings which throughout their height, behave as though both types A and B above have an effect.

3. Buildings whose lower stories have such massive columns that type A is in effect there, but whose upper story columns have less strength and type B is in effect.

4. Buildings whose stiffness is increased by the addition of shear walls, sections of exterior wall reaching the entire height, and whose wall direction is the same as the direction of shaking.

5. Buildings that are not symmetric in plan with respect to the shaking direction, and which therefore twist slightly, along a vertical axis, when shaken.

Placing two structural models on the same base can provide an illustration of the pounding of adjacent structures not vibrating in unison.

Although a fundamental structural resonance might have a low resonant frequency, a higher resonance (or overtone or harmonic

in musical terms), can be excited at close to six times (for type A) or three times (for type B) the fundamental frequency, depending on the structural type, and the models are planned to be able to show this.

Construction of the models is based on wood blocks of suitable size to represent horizontal floors, and strips of thin sheet aluminum to represent vertical columns whose flexibility or stiffness can be adjusted by specifying different widths and thicknesses. They have various low amounts of damping, depending on the joint details, but damping is assumed insignificant in the analysis. There is no effort to incorporate specific energy dissipation capability in the models, although some exists in all moving joints. The result is that resonant amplitudes of motion are large, particularly if, by mistake, input amplitudes are maintained at high levels once resonances are achieved. This can lead to damage in the form of weakened screwed joints, permanently bent columns or column fatigue failure unless reasonable care is taken.

2. SPECIFICATIONS

Because these models are hand-held and excited into resonance by shaking of the base with a steady vibrational to-and-fro motion, the highest expected resonant frequency is limited by the highest frequency able to be provided by the user, about 5 cps for most people. The base must be small enough to be grasped between thumb and fingers because the highest frequency resonances require the user to overcome the inertia of the model which builds up in proportion to the square of the frequency, and to overcome the tendency to overturn.

For ease of transportation, the height of the tallest models is limited to 2 ft.

Figure 1 illustrates the two basic models and construction details for the joints. Figure 1(A) represents two vertical cantilevers rigidly attached to the base and able to deform by bending under the action of horizontal inertia forces when the base is shaken. Horizontal deflections are representative of the columns of tall buildings whose columns at all elevations are much stronger or stiffer than the floors and floor beams at the same elevations. The floors, where they meet these columns, bend readily, and their contribution to the bending resistance of the joint is insignificant. The column bends, throughout its height, as though the floors have no effect. In the type A model, the floor bending effectively takes place in the joint itself, whereas in real structures it is distributed over the total extent of the floor.

Figure 1(B) illustrates the model of a building with stiff, unbending floors. During vibrations, the floors remain horizontal and the joints do not rotate, forcing the columns in the vicinity of the joints to be vertical. In between floors, the columns bend into very slight S-bends.

The sketches adjacent to the model of Figure 1(A) show two deformed shapes of the basic 2ft-high vertical cantilever model as it vibrates in its first and second modes. The base is shown having zero motion. During an earthquake, or, in this case, during hand excitation, the base is moved at the same frequency that the model exhibits. With models having insignificant damping, the base motion required at resonance has surprisingly low amplitude. The most effective technique, once the resonant frequency is

established, is to vibrate the base slightly ahead of the structure. The second mode of model A has a theoretical frequency 6.27 times faster than the fundamental mode. A model with half the height, also sketched, has a fundamental frequency four times the fundamental mode. Calculations for frequencies, given specific member sizes, may be found in the Appendix.

Adjacent to the model of Figure 1(B) are the deformed shapes of the rigid floor/ flexible column model as it vibrates in its fundamental and second modes. The second mode has a frequency approximately three times the fundamental. A model of half the height has a fundamental frequency two times that of the fundamental mode. Calculations for frequencies may be found in the Appendix.

3. CONSTRUCTION

The construction of these models has been made simple by the selection of hardware that is readily obtainable. The columns are cut from aluminum sheet in thicknesses of 0.031" (or 1/32") and 0.010", obtainable from a metal dealer and cut on a shear at the dealer or local sheet-metal shop. The floors are from 1" × 2" pine moulding, smoothed four sides (finished section 3/4" × 1 1/2"). The bases for those models with no aluminum attached to the longer dimension may be wider, for stability against overturning, for example 1" × 3". Models with vertical shear beams for stiffening require bases the same size as the floors above. Short lengths (1/4") of plastic or rubber hose, inside diameter 1/8", are required for the hinges of model type A. They hold the floors in position between the columns, threaded on the #8 roundhead wood screws. Washers under the heads of the #8 sheetmetal screws in model type B are built up to an outside diameter of 1/2" to ensure proper bending of the columns just above and below these connections.

The base of every model is 4" long, presenting a convenient size to grasp. Floors having hinges are 1/4" shorter for each end that is hinged, so that the type A model has 3 1/2" floors. The floors of the Type B model are 4" long. Column, floor and hinge detail for these and other models listed in the Introduction can be seen in Figure 1.

4. CONCLUSIONS

These models are simple to make and are challenging to excite into resonance. They fulfill the desired goal of providing a hands-on experience for the public in tall building dynamics. Seeing the response of the models to "ground" motion may help allay the fears of persons experiencing an earthquake.

5. APPENDIX

5.1 Buildings with strong columns and weaker floors

Figure 2(a) shows the $x-y$ coordinate system for a cantilever beam, placed vertically, with constant physical characteristics: moment of inertia I, mass density ρ, cross-sectional area A, and Young's modulus E. Neglecting shear distortions and rotary inertia (see, for example, Hurty and Rubinstein, 1964), the deflection $y(x,t)$ satisfies

$$\frac{\partial^4 y}{\partial x^4} + \frac{\rho A}{EI} \cdot \frac{\partial^2 y}{\partial t^2} = 0 \qquad (1)$$

Figure 2(b) shows two such cantilevers representing columns, tied together through floor beams with connections allowing free

rotation of the beams. Representing the floor mass as m_f, total floor mass M_f, and assuming the floor mass is uniformly spread over the height l, equation (1) becomes

$$\frac{\partial^4 y}{\partial x^4} + \left[\frac{2\rho A + M_f/l}{2EI}\right]\frac{\partial^2 y}{\partial t^2} = 0 \quad (2)$$

Looking for a natural mode solution

$$y(x,t) = y(x)\sin\omega t \quad (3)$$

where ω is a modal circular frequency, leads to

$$\frac{d^4 y}{dx^4} - \beta^4 y = 0 \quad (4)$$

where

$$\beta^4 = \left[\frac{2\rho A + M_f/l}{2EI}\right]\omega^2 \quad (5)$$

Looking for a solution of the form

$$y = e^{\lambda x} \quad (6)$$

where λ is complex, and using the boundary conditions

$$y(0) = y'(0) = y''(l) = y'''(l) = 0 \quad (7)$$

yields

$$\cos\beta l \cosh\beta l = -1 \quad (8)$$

Solving numerically, for example by trial and error with a calculator,

$$\beta l = 1\cdot 875\,,\ 4\cdot 694\,,\ \ldots \quad (9)$$

Substituting in equation (5),

$$\omega_n = \frac{1}{l^2}\sqrt{\left[\frac{2EI}{2\rho A + M_f/l}\right]}\cdot N_n, n = 1,2,3\ldots \quad (10)$$

where

$$N_1 = 3{\cdot}516, N_2 = 22{\cdot}035, \frac{N_2}{N_1} = 6{\cdot}27 \quad (11)$$

Selecting $l = 24''$, $b = 1.5''$, $d = (1/32)''$, $m_f = 0.1$ *lb./g*, $h = 4''$; and using $E = 10 \times 10^6$ psi, $\rho = 0.0975$ *lb./g/in*^{3}, $g = 386.4$ in/s^2, $I = bd^3/12$, $A = bd$, $M_f/l = m_f/h$, yields

$$f_1 = 0.9 \text{ cps}\,,\ f_2 = 5.7 \text{ cps} \quad (12)$$

and the frequency of a building of half this height is

$$f_{l/2} = 3\cdot 6 \text{ cps} \quad (13)$$

5.2 Buildings with strong floors and weaker columns

Figure 3(a) and (b) show two structures of height l under the action of a horizontal shearing force F, which displaces the top of the structures a distance y_t. Figure 3(a) is a vertical shear beam with cross-section area A, shear modulus G, and mass density ρ. Figure 3(b) represents an N-story building whose floors are strong and remain horizontal, forcing the weaker columns to be vertical in the vicinity of the beam–column joints. The two columns have Young's modulus E, and each has moment of inertia I. The known wave equation governing the dynamic displacements of (a) can be transferred to the structure (b) provided their stiffnesses under the action of the force F, and densities, are equivalent.

For the vertical shear beam (see, for example, Hurty and Rubinstein, 1964),

$$\frac{F}{A} = G\frac{y_t}{l} \quad (14)$$

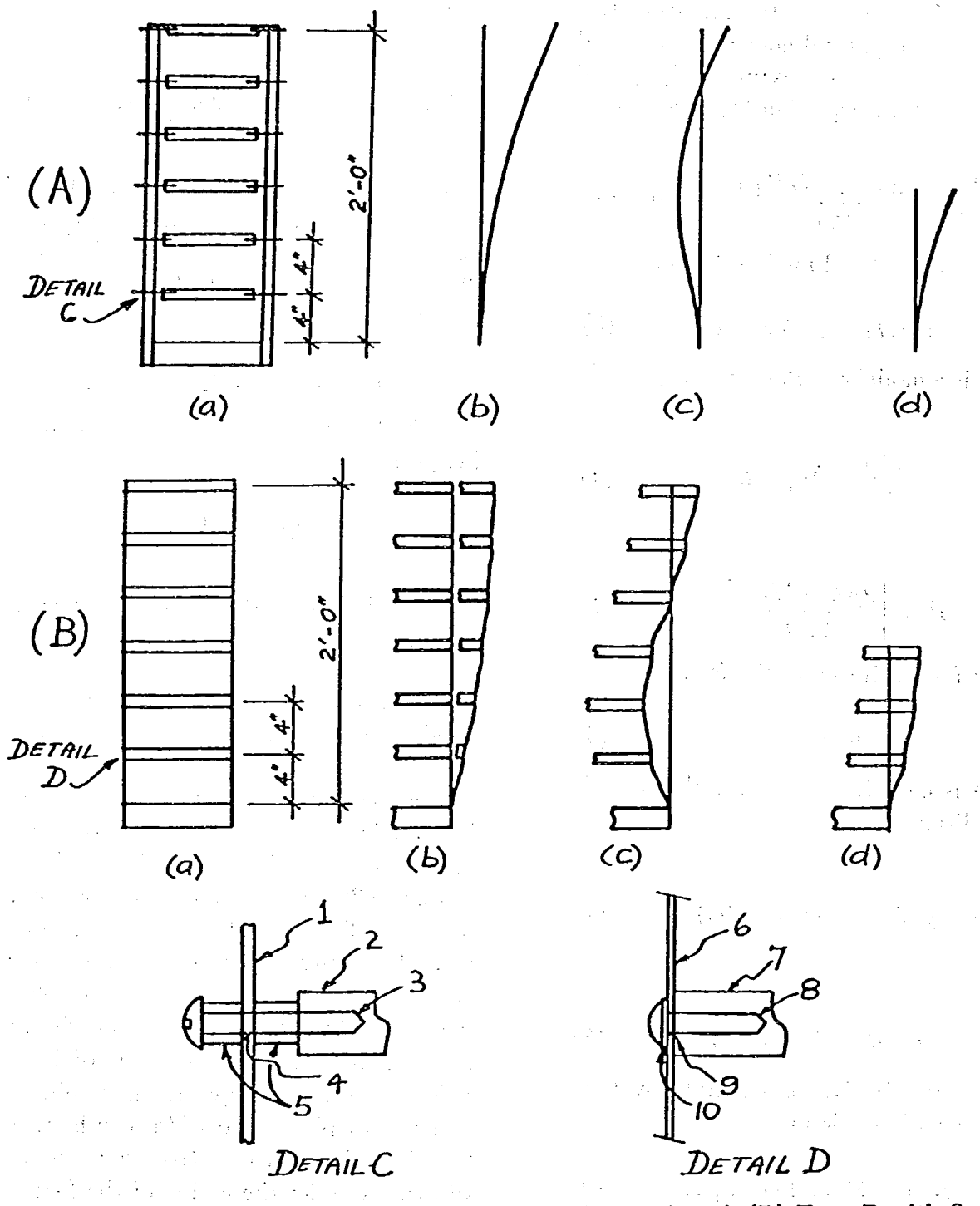


Figure 1. (A) Model type A with wood floors pinned at each end; (B) Type B with floors rigidly connected to weaker columns—(a) general view, (b) first mode shape, (c) second mode shape, (d) first mode shape of half-height model. Details: 1, 0.031" × 1½" aluminum column; 2, 1" × 2" × 3½"-long wood floor; 3, #8–1¼" roundhead wood screws, two per hinge, ¾" apart; 4, 13/64" hole allowing adequate clearance; 5, ¼" length of plastic tube, ⅛" I.D.; 6, 0.010" × 1" aluminum column; 7, 1" × 2" × 4"-long wood floor; 8, #8–1¼" roundhead sheetmetal screws, two per joint, ½" apart; 9, 11/64" hole; 10, #8 and ¼" washers.

For the N-story building,

$$F = \frac{24EI}{h^3} \cdot \frac{y_t}{N} \qquad (15)$$

Equating stiffnesses,

$$GA = \frac{24EIl}{h^3 N} \qquad (16)$$

Considering free vibrations of the vertical shear beam,

$$\frac{\partial^2 y}{\partial x^2} - \frac{\rho}{G}\frac{\partial^2 y}{\partial t^2} = 0 \qquad (17)$$

Looking for a natural mode solution of the form

$$y(x,t) = y(x)\sin \omega t \qquad (18)$$

leads to

$$y''(x) + k^2 y(x) = 0 \qquad (19)$$

where

$$k^2 = \frac{\rho}{G}\omega^2 \qquad (20)$$

Looking for a solution of the form

$$y = A\cos kx + B\sin kx \qquad (21)$$

with boundary conditions

$$y(0) = y'(l) = 0 \qquad (22)$$

leads to

$$kl = (2n-1)\frac{\pi}{2}\ ,\ n = 1\ ,\ 2\ ,\ \ldots \qquad (23)$$

Substituting equation (23) into (20) yields

$$f_n = \frac{\omega_n}{2\pi} = \frac{1}{4l}\sqrt{\frac{G}{\rho}} \cdot N_n \qquad (24)$$

where

$$N_1 = 1\ ,\ N_2 = 3\ ,\ N_2/N_1 = 3 \qquad (25)$$

Transfer the stiffness from Figure 3(a) to 3(b) using equation (16), and transfer the density by equating the total mass (ignoring the mass of the thin aluminum columns compared with the floors):

$$\rho Al = M_f = Nm_f = \frac{l}{h} \cdot m_f \qquad (26)$$

Selecting $l = 24''$, $b = 1''$, $d = 0.01''$, $m_f = 0.1$ *lb./g*, $h = 4''$; and using $E = 10 \times 10^6$ psi, $g = 386.4$ in/s^2, $I = bd^3/12$; and substituting equations (16) and (26) into (24), yields

$$f_1 = 1 \cdot 4 \text{ cps}\ ,\ f_2 = 4 \cdot 2 \text{ cps} \qquad (27)$$

The frequency of a building of half this height is

$$f_{1/2} = 2 \cdot 8 \text{ cps} \qquad (28)$$

Figure 4 reproduces photos of some of the models and their mode shapes.

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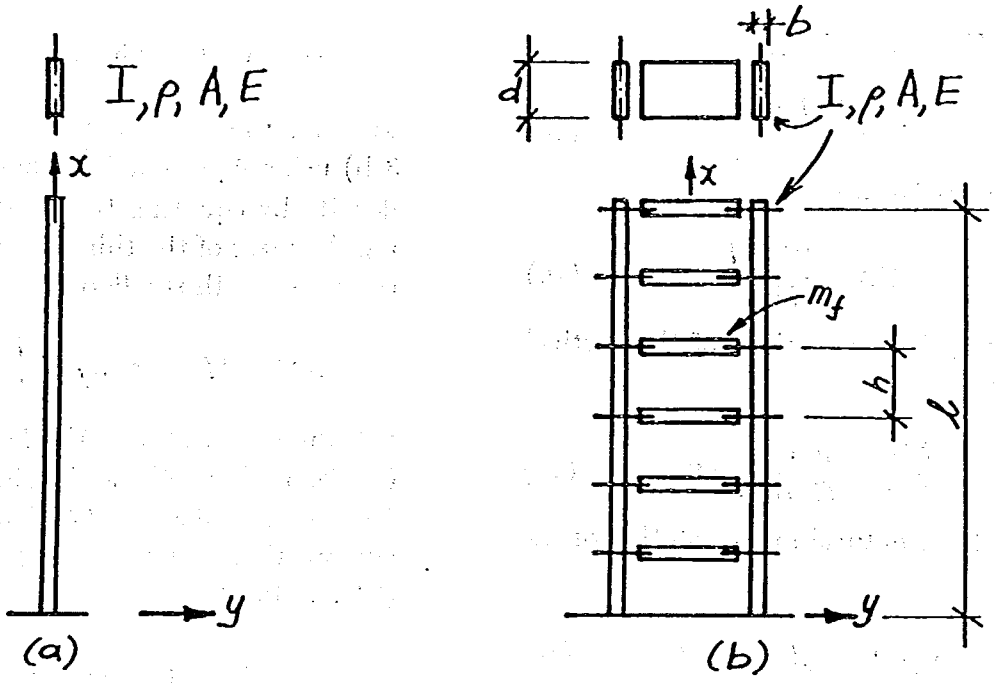


Figure 2. (a) Cantilevered bending beam placed vertically, (b) schematic for analysis of type A model.

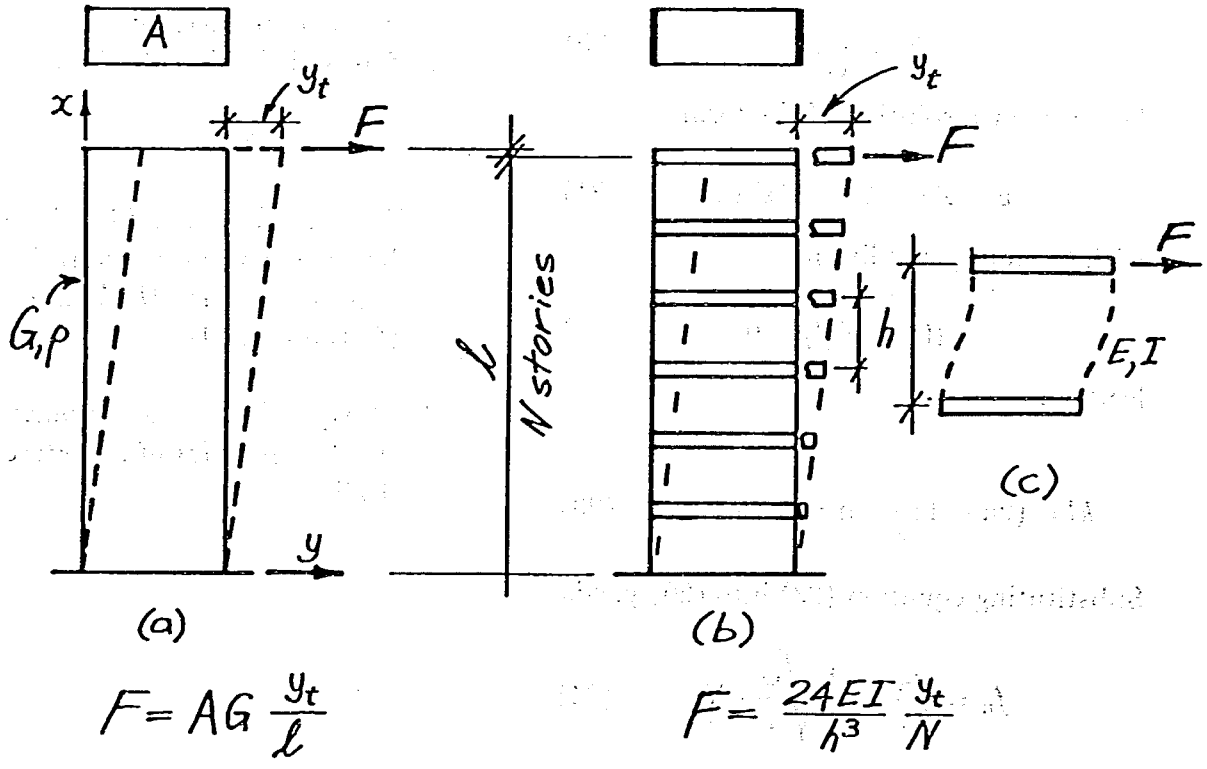


Figure 3. Two structures resisting horizontal forces by shearing action. (a) Cantilevered shear beam placed vertically; (b) schematic for analysis of type B model; (c) interstory detail.

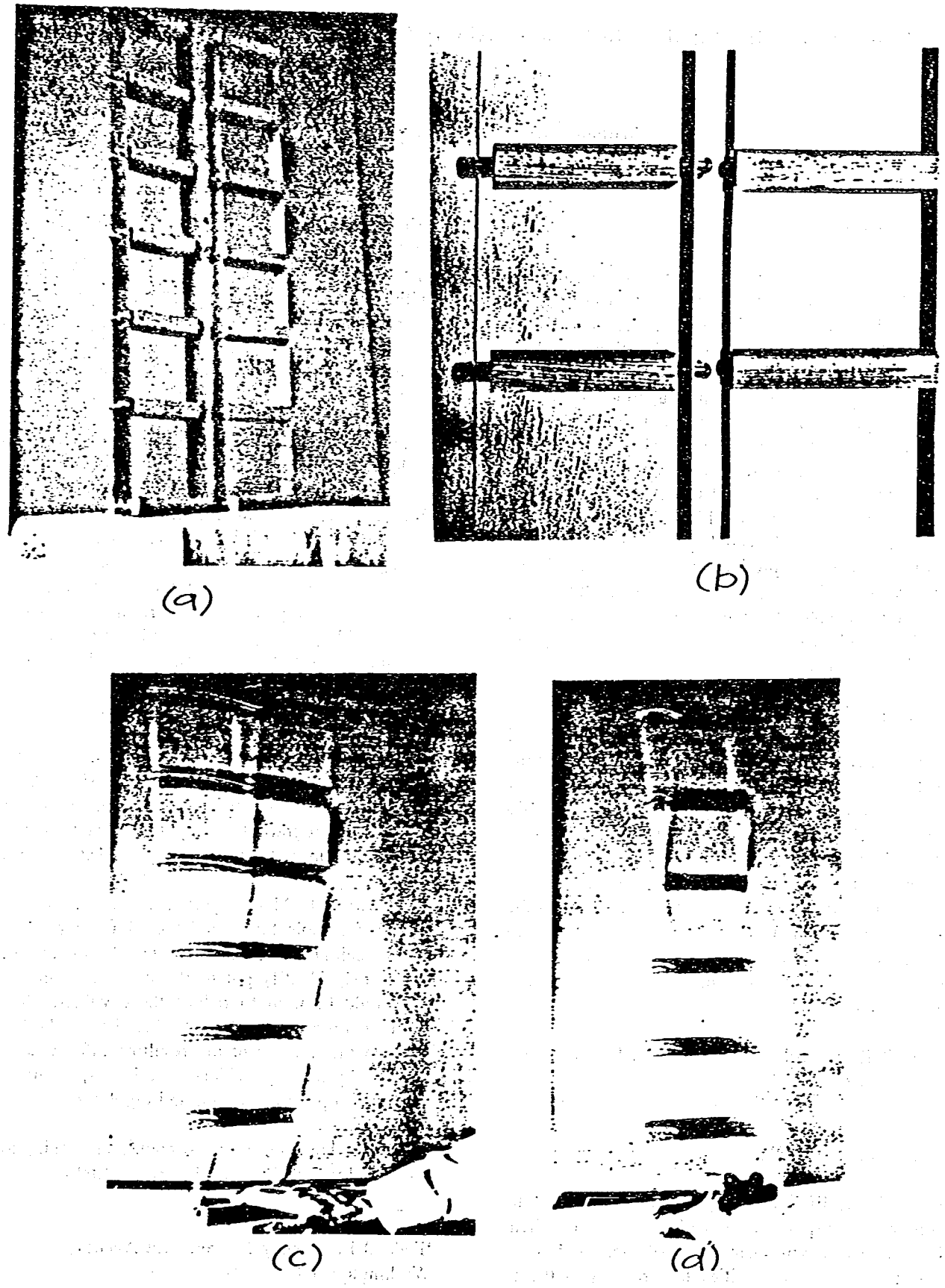


Figure 4. (a) Type A and B models; (b) floor and column detail; (c) first mode of type B model, frequency 1.9 cps; (d) second mode of type B model, frequency 5.6 cps.

The Impacts of Siting Lifelines in Close Proximity Upon Their Earthquake Vulnerability

by

William Bivens[a] and Phillip Lowe[b]

ABSTRACT

The Federal Emergency Management Agency (FEMA) has initiated a program to examine the potential increases in lifeline earthquake vulnerability due to their close placement or collocation. To date, the study has focused on developing a practical engineering approach that builds upon available data and analysis techniques for evaluating individual lifelines or lifeline components. The methods have been applied in California at the Cajon Pass, which is located north and east of Los Angeles. The site lifelines analyzed included: highways, railroads, and their bridges; natural gas bulk transmission pipelines; refined petroleum product transmission pipelines; high voltage electric power transmission lines; and fiber optic telephone communication lines. The San Andreas fault crosses at approximately the midpoint of the Cajon Pass canyon. During the study, improved methods to analyze the vulnerability of buried pipelines and bridges were developed. The study found that a fragmented regulatory control system, environmental concerns about disturbing the surface conditions, and the economics of right-of-way acquisition and lifeline construction led to over 100 separate lifeline collocations within the study area. Depending upon the lifeline, collocation impacts were estimated to increase the delay to restore lifeline temporary service from 20 to 60% compared to the restoration time if no collocation were assumed to exist. FEMA now intends to extend the study to examine other locations in the U.S. and to refine the analysis methods.

KEYWORDS: collocation, earthquake, damage, impacts, lifelines

1. INTRODUCTION

In the U.S., lifelines are presently being sited in "utility or transportation corridors" to reduce their right-of-way environmental, aesthetic, and cost impacts on the communities that rely upon them. The individual lifelines are usually designed, constructed, and modified throughout their service life. This results in different standards and siting criteria being applied to segments of the same lifeline, and also to different standards or siting criteria being applied to the separate lifelines systems within a single corridor. Presently, the siting review usually does not consider the impact of proximity or collocation of the lifelines on their individual risk or vulnerability to natural or manmade hazards or disasters.

There have been cases when some lifeline collocations have increased the levels of damage experienced during an accident or an earthquake. For example, water line ruptures during earthquakes have led to washouts which have caused foundation damage to nearby facilities. A transportation lifeline failure has led to the subsequent failure of a collocated fuel pipeline, and the resulting fire caused considerable property damage and loss of life. Loss of electric power has restricted, and sometimes failed, the ability to provide water and sewer services or emergency fire fighting capabilities.

In response to these types of situations, the Federal Emergency Management Agency (FEMA) is examining the use of such corridors, and FEMA has initiated a program to study the impact of siting multiple lifeline systems in confined and at-risk areas.

The overall FEMA project goals are to develop managerial tools that can be used to increase the understanding of the lifeline systems' vulnerabilities and to help identify potential mitigation approaches that could be used to reduce those vulnerabilities. Another program goal is to identify methods to enhance the transfer of the resulting information to lifeline system providers, designers, builders, managers, operators, users, and regulators.

In the initial project of this program[1], a screening tool was developed to identify the critical lifeline

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[b]INTECH Inc, 11316 Rouen Dr., Potomac, MC 20854

collocation locations and the conditions that makes them critical. The probable time required to restore the lifeline service after an earthquake was used to measure the impact of collocation compared to an assumed no collocation situation.

The analysis methods were based on using existing data bases and lifeline vulnerability to earthquake evaluation methods so that a practical, engineering approach could be developed. During the study, improved methods to analyze buried lifelines and to account for soil liquefaction, as well as improved methods to analyze potential bridge vulnerability to an earthquake event were developed. The resulting analysis tools have been successfully applied to the lifelines sited at the Cajon Pass, California. These lifelines included: highways, railroads, and their bridges; natural gas bulk transmission pipelines; refined petroleum product transmission pipelines; high voltage electric power transmission lines; and fiber optic telephone communication lines.

2. CONCLUSIONS

A very useful screening tool has been developed. The tool can be used to identify the critical lifeline collocation locations and the conditions that make them critical. It can be used to identify areas of technical uncertainty and poor siting practices, and its use can identify important research and development opportunities.
At the Cajon Pass it was found that:

- ground movement due to landslides and liquefaction are more important than ground shaking for estimating the vulnerability of most lifelines. Bridges are the exception. For them, ground shaking can be a critical part of the estimation of their vulnerability;

- fuel pipeline failures had the greatest impact on other nearby lifelines during the immediate recovery period after an earthquake;

- current siting practices for fiber optic cables indicates that more severe telephone communication failures than have been experienced in past earthquakes can be anticipated in future earthquakes when fiber optic systems provide the basic telephone service (compared to the current hard wire systems);

- lifeline siting practices do not fully consider the potential impact that other nearby lifelines could have on the lifeline being evaluated;

- communication, electric power, and fuel pipeline lifelines can generally be analyzed as a set of discrete collocation points. The restoration of service at one location is generally not impacted by the restoration work at other collocation points, because the repair equipment and personnel can access the work site from a number of alternative paths. Thus, the critical restoration of service time is the longest time estimated for any one collocation point in those lifelines;

- transportation lifeline restoration of service is highly dependent on sequentially repairing the lifeline damage along the lifeline itself, as heavy equipment access to the next damage location often requires use of the transportation system itself;

- some collocation can produce benefits, as the nearby parallel location of a similar lifeline can provide the detour or access route to the damaged sections of the other lifeline. However, intersecting lifelines generally result in the failure of one lifeline increasing the risk of failure of the lifelines it crosses; and

- additional applications of the analysis methods are needed to assure that the results obtained are not in some way unique to the Cajon Pass and also to see if additional improvements in the analysis methods can be developed.

3. THE ANALYSIS METHODS

The analysis used a four Step approach: 1, data acquisition; 2, calculation of individual lifeline vulnerability; 3, calculation of collocation damage; and 4, interpretation of the collocation impacts on the individual lifeline systems.

3.1 Data Acquisition

In this step all the required information on the lifeline routes, methods of construction, materials of construction, location of the collocation points, collection of the topographic and geologic data, identification of probable landslide and liquefaction locations, performing an earthquake analysis to determine the location of the regions of constant Modified Mercalli Indices (MMI), and the plotting of the data with respect to the lifeline locations are performed. In the first application, the data was

input into a PC computer and analyzed and plotted using commercially available computer aided design (CAD) programs. The MMI isobars were developed using the methods of Evernden[2]. This method was used because it has been verified by comparison with historical earthquakes, it incorporates the local sediment conditions into the model and the sediment conditions are generally available in the U.S. Geological Survey geologic data base, it is easy to use, and it is readily available to other researchers. When this was performed for the Cajon Pass, 103 separate collocation locations that required further analysis were identified in the 220 square kilometer study area (approximately 90% of the collocation locations were found in a 65 square kilometer region along the foot of the Cajon canyon). A number of the lifelines were located in the expected fault zone of the San Andreas fault.

3.2. Calculation of Individual Lifeline Vulnerability

Damage matrices[3] were used to estimate the earthquake damage to each individual lifeline at each collocation point. The analysis was performed as if the lifeline were an isolated lifeline at that location. The damage matrices related the percent damage and the probability of that damage to the MMI index. The analysis also accounted for earthquake-induced landslides, and to a lessor extent earthquake-induced soil liquefaction. In the method applied at the Cajon Pass, improved damage matrices were developed for determining liquefaction impacts on buried lifelines[4]. In addition, an improved screening method to estimate the shaking damage to bridges was developed. It was based on adapting bridge retrofit requirements analysis methods developed by the California Department of Transportation[5].

The important items that are calculated for each lifeline at each collocation point are the estimated damage state (or percent damage) and the estimated time to restore the lifeline to the needed service level. Both of these parameters can be calculated using the matrices and tables of Rojahn[3].

3.3. Calculation of Collocation Damage

This analysis step builds upon the results obtained from the two previous Steps. A lifeline zone of physical influence was defined to describe the physical region in which one lifeline could impact other nearby lifelines. Then, based on the individual lifeline damages calculated in Step 3.2, and the zone of influence, a collocation damage scenario was defined. The damage matrices of step S.2 and the restoration of lifeline service tables of Step 3.2 were used to calculate the percent of damage and lifeline restoration of service time for each lifeline for the assumed collocation damage scenario.

One of the easier direct impacts to hypothesize is that the collocation conditions will lead to an increase in the damage state of one or both of the collocated lifelines (if there are more than two collocated lifelines this applies to all of them). It is easy to understand the damage state, as it relates to a physical condition. Because the individual lifeline damage states assuming no collocation are known, those values can be used to help understand how the lifeline could impact another nearby lifeline. If, for example, light damage of a pipeline had been calculated, it would be expected to cause no direct change in the damage state of a nearby bridge. However, if the bridge had been estimated to collapse, it would be reasonable to estimate that within the bridge's zone of influence it would lead to failure of the pipeline (this example also illustrates that the impacts are not necessarily reciprocal).

As another example of how collocation impacts on damage state can be estimated, consider the condition of a pipeline and a fiber optic conduit hung from a bridge. The earthquake vibration may not be enough to cause serious damage to the bridge or to the pipeline or conduit if they were not collocated with each other. However, the vibrations may cause the anchors holding the heavy pipeline to the bridge to fail. As the pipeline sags (but does not fail) it could fall onto the lower conduit, causing it to fail. The collocation damage state hypothesis would then be: no impact on the bridge; a small increase in damage state of the pipeline to account for the work required to rehang the pipeline; and catastrophic failure of the fiber optic conduit.

Special attention should be given to the collocation of fuel carrying lifelines with other lifelines that have the ability to provide an ignition source. The resulting fire and/or explosion could lead to significant collocation damage. Similarly, broken pipelines which eject fluids could lead to foundation erosion problems that would result in increased damage to nearby lifelines.

3.4. Interpreting the Collocation Impacts on the Individual Lifeline Systems

It was established that the most realistic measure of the impact of collocation was the "most probable incremental change in the restoration of service time" for each lifeline. This is defined as the product of the probability of the collocation damage occurring times the incremental increase in restoration of service time. The incremental increase in restoration of service time is the restoration time calculated for the collocation damage state minus the restoration time calculated for the lifeline at the same location but assuming no collocation existed.

There are several ways to estimate the change in the probability that damage will occur, none are exact and there are no statistics available from the literature on earthquakes. However, there are some insights available to guide the analysts.

If the probabilities for damage to two lifelines, assuming no collocation conditions, are P1 and P2, they represent an upper bound on the probability that a collocation damage would occur. For example, if the probability that lifeline 1 would fail is P1, and it is known that if lifeline 1 fails it will cause, with a 100% probability, damage to lifeline 2, then the probability that lifeline 2 receives collocation damage is also P1 (e.g., P1 x 100%). Similarly, the upper bound on the probability that lifeline 2 has damaged lifeline 1 is P2.

As a practical matter, the collocation damage likely will be less, since there is seldom a 100% chance that the collocation damage scenario will occur. A useful measure of the probability that the collocation event has occurred is the product of the two probabilities that the single independent events that were used to develop the collocation scenario have occurred (the independent events are the estimate of the damage state of each lifeline assuming there was no collocation). In the present case, that is found by multiplying P1 x P2. The product can be interpreted as follows. It represents the increase in probability that the two independent lifeline damages will occur during the same initiating event. If both events must occur before the collocation damage scenario can take place, then it is a measure of the probability of the collocation damage scenario.

The actual probability that the collocation event will occur should be a number between the numerical limits of P1 and (P1 x P2) for having lifeline 1 cause additional damage to lifeline 2, and P2 and (P2 x P1) for having lifeline 2 cause additional damage to lifeline 1. It is recommended that for calculational purposes, the product P1 x P2 be used to characterize the hypothesized collocation damage scenario.

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Nonlinear Research Needs for Concrete Gravity Dams

by

Robert L. Hall* and Wayne G. Johnson*

ABSTRACT

The Corps of Engineers' Engineer Technical Letter (ETL) 1110-2-303 outlines a sequence of analyses of concrete gravity dams subjected to the maximum credible earthquake (MCE). The first step is a two-dimensional linear-elastic response spectrum or time-history analysis with five percent damping. This analysis may include the effects of hydrodynamic loads, foundation flexibility, and absorption of the reservoir bottom (Chopra, 1978). For this step, five percent of critical viscous damping is assumed unless tensile stresses exceed fifteen percent of f'_c. If the tensile stresses exceed 0.15 X f'_c, adjustments are made in the damping to account for some cracking of the concrete. These different levels of damping combined with different concrete strengths are assumed to produce a conservative and appropriate procedure.

Nonlinear dynamic calculations were performed to evaluate guidelines of the ETL. The analyses revealed that existing tools generally produce reasonable results; however, the studies demonstrated the need for further development of nonlinear analysis tools. Limitations presently exist in the modeling and assumptions of material properties (Fenves, 1987), damping assumptions, and effects of water cavitation and intrusion into cracks (Dowling, 1987). To ensure the safety of concrete dams subjected to strong ground motions, further research is needed on nonlinear analysis methods and the corresponding parameters that govern these complex geometric and materially nonlinear models.

1. INTRODUCTION

The Corps presently supports the program SDAM (Cole and Cheek, 1988) and an acceleration-time-history program EAGD-84 (Fenves and Chopra, 1984) for the analyses of concrete gravity dams. These codes are supported through the numerical maintenance modeling program. Both codes assume linear-elastic material response and can be used to perform the seismic analyses according to procedures outlined in ETL 1110-2-303.

Mlakar (1986) evaluated the presently accepted procedure by performing a nonlinear analysis of three different dams of different heights subjected to two different earthquake records. The earthquake ground motions and the reservoir-structure dynamic interaction were modeled with the ADINA 84 finite-element code. For the cases investigated, the simpler analytic models seem to be conservative for evaluation of concrete gravity dams.

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However, the elastic procedures incorrectly located the region of maximum cracking (Mlakar, 1986).

Fenves (1987) evaluated this ETL by calculating the nonlinear response of the Pine Flat Dam subjected to a scaled horizontal Taft ground motion for five different load cases. These five load cases considered a full and empty reservoir and peak acceleration varying from 0.18 g to 0.45 g. The results demonstrated that cracks can form but remain stable under low amplitude ground motion; however, under large amplitude ground motion, Fenves found that the cracks propagate across the entire cross-section of the studied monolith.

2. DETAILED STUDIES

Mlakar's research was funded by the U.S. Corps of Engineers' Structural Research Program in order to evaluate the current ETL 1110-2-303. This evaluation was done by performing nonlinear analyses of three nonoverflow-gravity dam cross-sections which were selected to characterize the Corp's population of concrete gravity dams. The N65W and vertical components of the 1966 Parkfield, California earthquake recorded at Temblor No. 2 Station (CIT File Nos. B037-1 and B037-3) were used for each analysis. The records from this earthquake were chosen because the peak ground acceleration and frequency content are representative of strong ground motions. The nonlinear analyses were performed using the general purpose finite-element code ADINA. The constitutive behavior of the concrete was described by Bath and Ramaswamy (1979). The hydrodynamic loading of the reservoir was modeled by adding concentrated nodal masses on the upstream face corresponding to the distribution described by Chopra (1978). All foundations were assumed rigid.

The shortest section analyzed represented the Richard B. Russell Dam on the Savannah River. This structure is 185 ft high and has a modulus of elasticity of three million psi. The finite-element grid contains 65 elements and 247 nodes providing 594 degrees of freedom. The two-dimensional cross-section experienced no nonlinear behavior when subjected to the Parkfield ground motion. The ground motions were then tripled in order to determine the nonlinear response of this gravity dam. Figure 1 shows the cracked regions of the dam due to tripled Parkfield accelerations.

The second gravity dam modeled has a height of 300 ft, but it is not representative of any particular dam and is labeled "Standard Dam." The finite-element model for the dam consists of 594 degrees of freedom. The modulus of elasticity for the mass concrete was assumed to be three million psi. The ADINA analysis showed two cracked zones on the downstream surface at the elevation of slope change and at a slightly lower elevation (Figure 2). These cracks progress to the upstream surface. An examination of the ETL procedure for this situation reveals that no indication of cracking above the base of the dam would be realized.

The third dam analyzed was the Dworshack Dam on the Clearwater River in Idaho. This gravity dam is 638 ft high and it has an estimated modulus of elasticity of five million psi. This structure was chosen to represent the tallest dam owned by the Corps of Engineers. The finite-element model of this dam has 80 elements and 363 nodes with 726 degrees of freedom. The

nonlinear analysis indicates a cracked zone which transect the cross section of the elevation of change in downstream slope. Cracking also initiates at the upstream edge of the base and soon stabilizes (Figure 3). At later times the cracking propagates from the downstream face at the elevation of the slope change. The ETL procedure indicates that cracking is expected through the base of the structure.

Fenves performed a nonlinear seismic analysis of the tallest (400 ft) nonoverflow monolith of the Pine Flat Dam. The finite-element model of the structure consists of 162 nodes comprising 136 quadrilateral elements with a total of 315 degrees of freedom. The finite element model of the impounded reservoir water extends upstream 1,200 ft from the dam and consists of 224 constant-pressure elements. By modeling the reservoir, the analysis accounts for the interaction between the reservoir and dam. Stiffness proportional damping was used to provide five percent critical damping of the fundamental frequency and a modulus of elasticity of 3.25 million psi was used. The S69E component of the 1952 Taft ground motion was taken as a horizontal component acting in the upstream-downstream direction. The peak ground acceleration from these ground motions was 0.18 g.

When the reservoir was empty, the Taft ground motion had to be scaled up by a factor of 2.5 to an acceleration of 0.45 g in order to initiate cracking. However, when the reservoir was assumed full (381 ft), the nonlinear analysis indicates cracking from unscaled Taft ground motions. The cracking occurs at the head of the dam where a stress concentration exists because of the assumed rigid foundation. When ground motions were scaled to 0.27 g, cracks are shown to propagate further along the base. When ground motions were scaled to 0.36 g, cracking also occurs on the downstream slope where the change in the slope occurs. The extensive cracking led to a numerically unstable solution which indicates that stability analyses are needed. Figure 4 gives a summary of these results.

The ETL procedure was followed using the computer program EAGD-84. Five load cases were investigated for the Pine Flat Dam. Table 1 summarizes the five cases and results. For cases 1, 3, and 4 the maximum tensile stresses were less than 638 psi (.15 f'_c), so no cracking of the monolith was presumed. Cases 2 and 5 were performed again assuming 7-percent viscous damping which equates to 14-percent hysteretic damping (Table 2). Case 2 resulted in maximum tensile stress of 563 psi at the downstream face which exceeds 0.10 X f'_c and, according to the ETL, a crack should be assumed at two locations and sliding stability analyses performed for the portion of the dam above this plane. For Case 5, the 0.10 X f'_c criteria was exceeded on the upstream and downstream faces as well as at the heel. The ETL criteria again require stability analysis to be performed.

These nonlinear analyses clearly indicate the importance of the nonlinear response of the Pine Flat Dam to these ground motions. The ETL criteria appeared to be reasonably accurate for these five analyses. However, the procedure needs further investigation because there is no theoretical basis for assuming that tensile cracking results in increased energy

dissipation which is implied by greater damping ratios.

3. NONLINEAR RESEARCH NEEDS

The Panel on Earthquake Engineering for Concrete Dams Committee on Earthquake Engineering, Division of Hazard Mitigation, through the National Research Council has recently completed a publication entitled, "Earthquake Engineering for Concrete Dams: Design, Performance, and Research Needs" (NRC, 1990). This publication presents details for needed research in the nonlinear seismic response of concrete dams. The following are the items which should be addressed in future research programs:

3.1 Material Testing of Mass Concrete

Further testing of mass concrete under dynamic loads is needed. It is needed to determine tensile cracking of the mass concrete under multiaxial stress states which represent the in-situ strain paths in a concrete dam during a seismic event. These tests must quantify the effects of strain rates, concrete mixtures, and aggregate size. Concrete samples should be a mixture of cores from actual dams and carefully prepared laboratory samples.

3.2 Development of Materials Models for Concrete

Once the test data are available, realistic numerical models for tensile cracking under dynamic loads can be developed. These models must allow for multiaxial stress states, strain rate effects, shear transfer by aggregate interlock and criteria for tensile cracking and propagation of cracks. The studies must include smeared-crack approach, fracture mechanics principles, cracking-consistent damping, and the discrete-crack approach.

3.3 Modeling of Other Nonlinear Mechanisms

Since the limiting tensile strength of the concrete is across lift surfaces, it is important to develop construction joint models. These models will redistribute the forces across a joint as the two surfaces open and close. The degradation of the concrete across the joints must be modeled due to the damaging number of loading cycles from an earthquake.

3.4 Numerical Procedures for Computing Nonlinear Response

The numerical material and joint models can then be incorporated into a finite-element program with time integration for the equations of motion or other discretization methods for solving dynamic nonlinear equations. The procedures must also include interaction with the impounded water, flexible foundation rock, and the reservoir-bottom absorption. These procedures will need to be refined to take advantage of vector and parallel processes in the latest computers.

3.5 Parameters and Detailed Response Studies

With the development of accurate numerical procedures, parametric studies can be performed to determine the sensitivity of the nonlinear response to the parameter describing the nonlinear models. These studies would identify the significance of tensile cracking and joint opening with respect to either failure or dynamic response. Finally the following factors should be determined: effects of ground

motion characteristics, water compressibility, foundation flexibility, reservoir bottom absorption, and modeling issues.

3.6 Dynamic Testing of Dam Models

Testing of dam models is essential for verifying nonlinear numerical procedures. These tests may require association with international augmentation testing and use of facilities such as a large earthquake simulator recently installed at the research laboratory of the Ministry for Water Conservancy and Hydroelectric Power in Beijing, China.

3.7 Identification of Design Criteria

Accurate design criteria based on numerical and experimental studies could then be developed. Nonlinear seismic analysis may not become a standard practice in design but will certainly define the tensile strength of the structure and the post-cracking stability.

3.8 Investigation of Earthquake-Resistant Design Measures

These nonlinear capabilities will allow for the investigation of innovative measures for increasing the seismic safety of concrete gravity dams. These tools could be used to study effects in the geometry of dams, jointing schemes, and joint materials to dissipate energy.

4. CONCLUSION

Nonlinear capabilities are important in determining the seismic stability of concrete gravity dams. The capability to develop a complete program as described above is beyond single capability of the Corps of Engineers. However, the Corps can strategically utilize the research being performed through the National Science Foundation and other organizations to answer these complex issues and develop the necessary tools and criteria. Nonlinear analysis will provide the necessary insight for the development of reliable cost saving earthquake resistant designs.

5. ACKNOWLEDGMENT

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COL Larry B. Fulton, EN, was the Commander and Director of the U.S. Army Engineer Waterways Experiment Station during the preparation of this paper. Dr. Robert W. Whalin was Technical Director.

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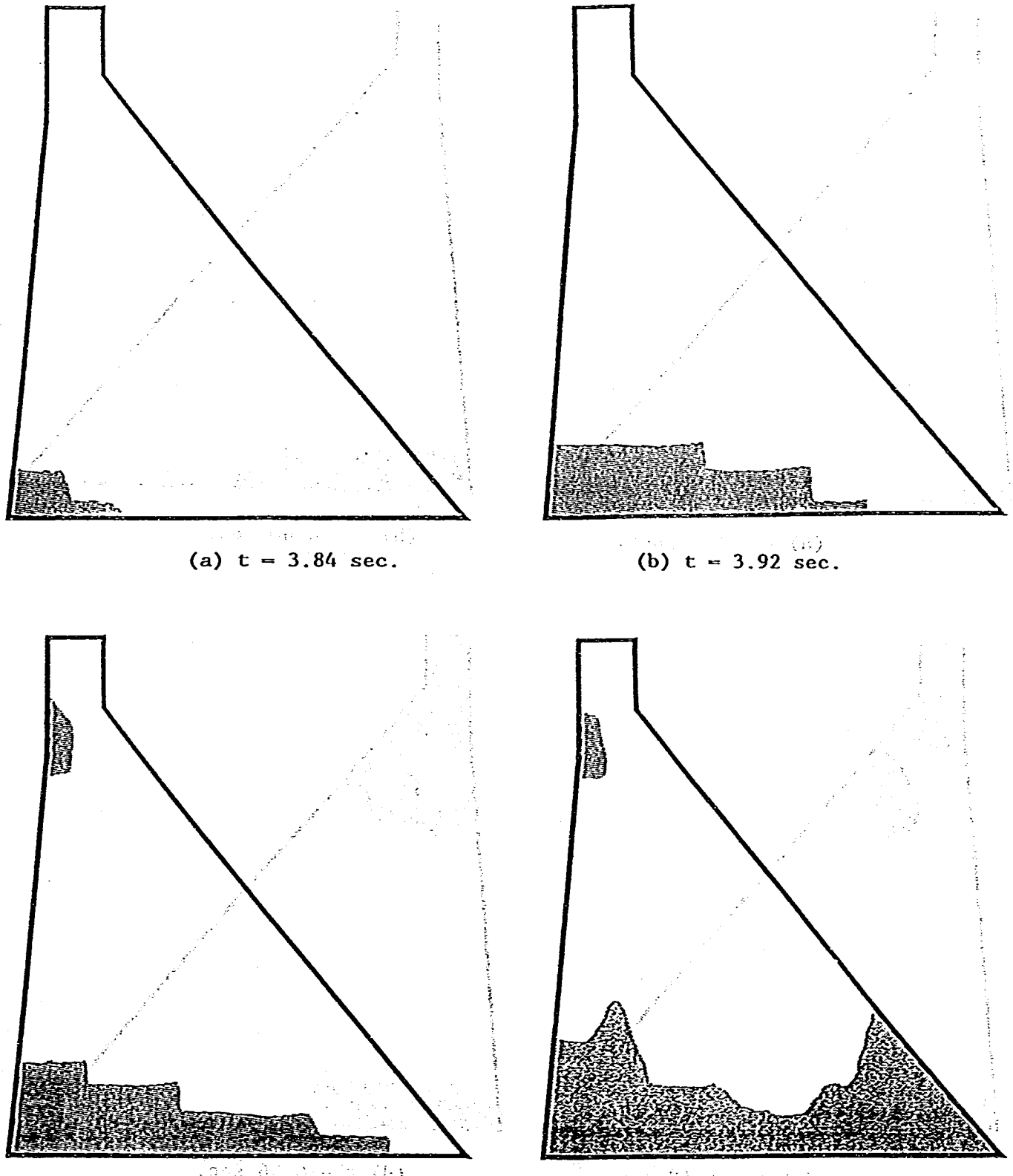
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Table 1. Maximum tensile stress in Pine Flat Dam due to scaled horizontal Taft ground motion from first linear dynamic analysis.

Case	Water	a_g (g)	μ_f	Max. Stress (psi) Up	Down	Heel
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Empty	0.18	0.10	130	256	4
2	Empty	0.45	0.10	466	666	271
3	Full	0.18	0.10	217	250	361
4	Full	0.27	0.10	415	363	539
5	Full	0.36	0.10	510	580	718

Table 2. Maximum tensile stresses in Pine Flat Dam due to scaled horizontal Taft ground motion from second linear dynamic analysis.

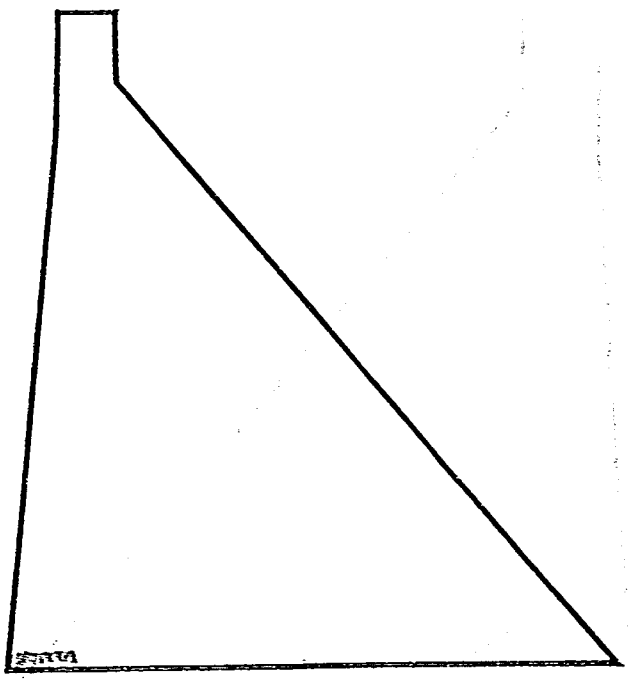
Case	Water	a_g (g)	μ_f	Max. Stress (psi) Up	Down	Heel
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Empty	0.18	---	---	---	---
2	Empty	0.45	0.14	383	563	198
3	Full	0.18	---	---	---	---
4	Full	0.27	---	---	---	---
5	Full	0.36	0.14	459	479	642



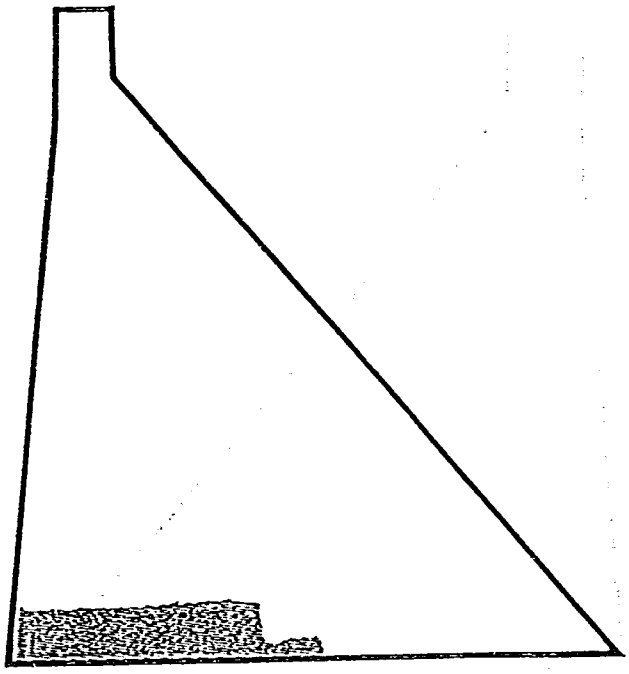
(a) t = 3.84 sec. (b) t = 3.92 sec.

(c) t = 4.02 sec. (d) t = 4.04 sec.

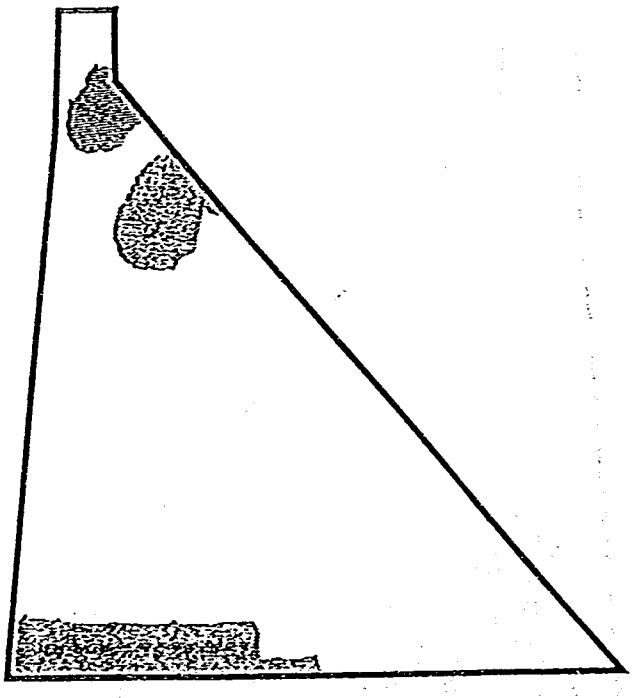
Figure 1. Cracked zones of Russell Dam with tripled Parkfield loading.



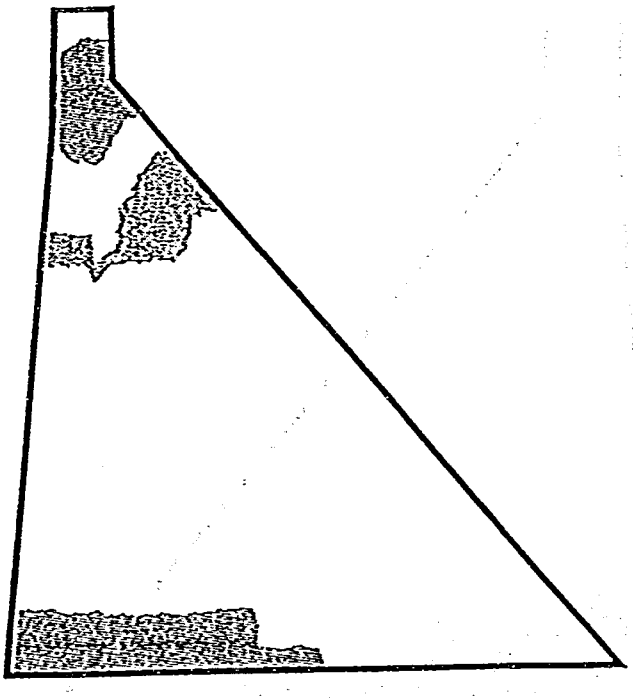
(a) t = 4.00 sec.



(b) t = 4.40 sec.

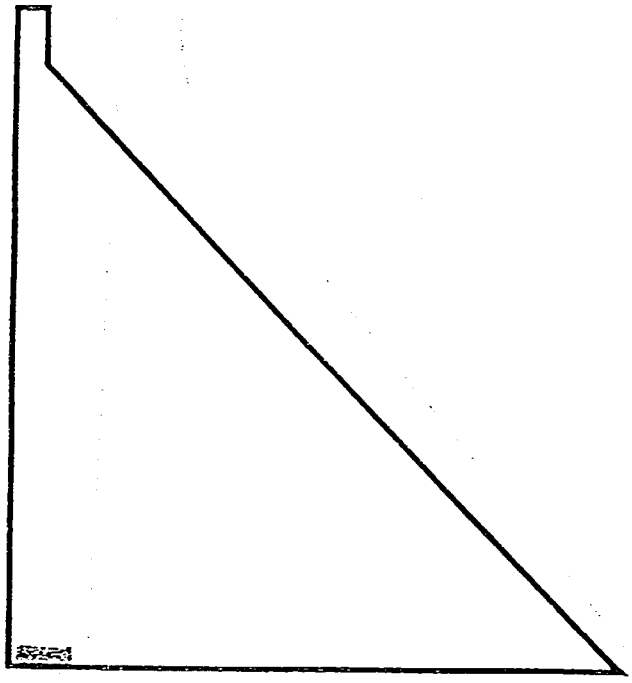


(c) t = 4.48 sec.

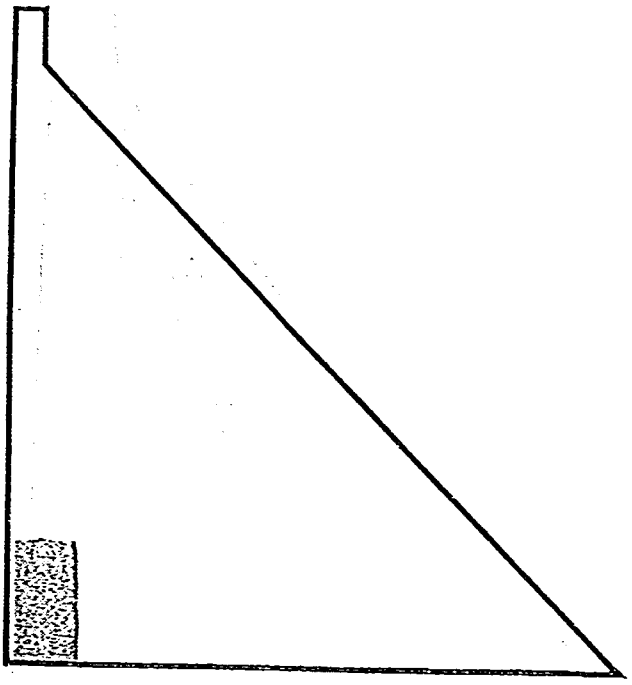


(d) t = 4.56 sec.

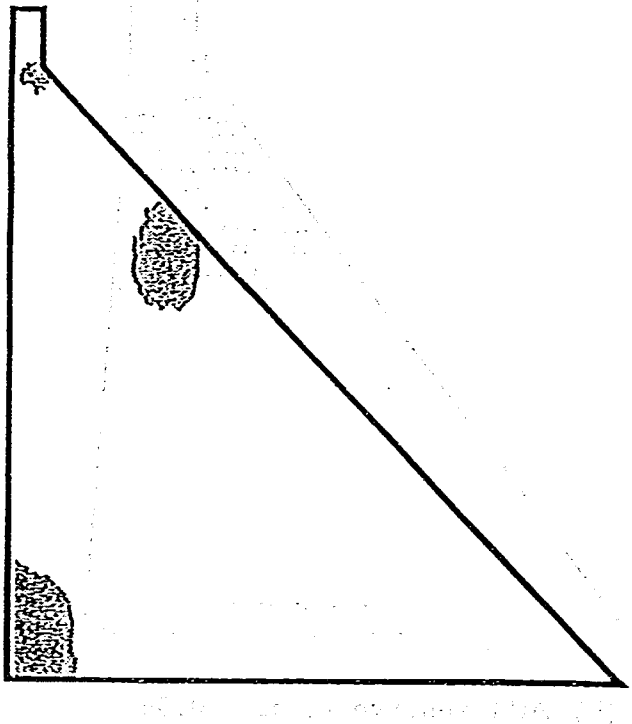
Figure 2. Cracked zones of Standard Dam.



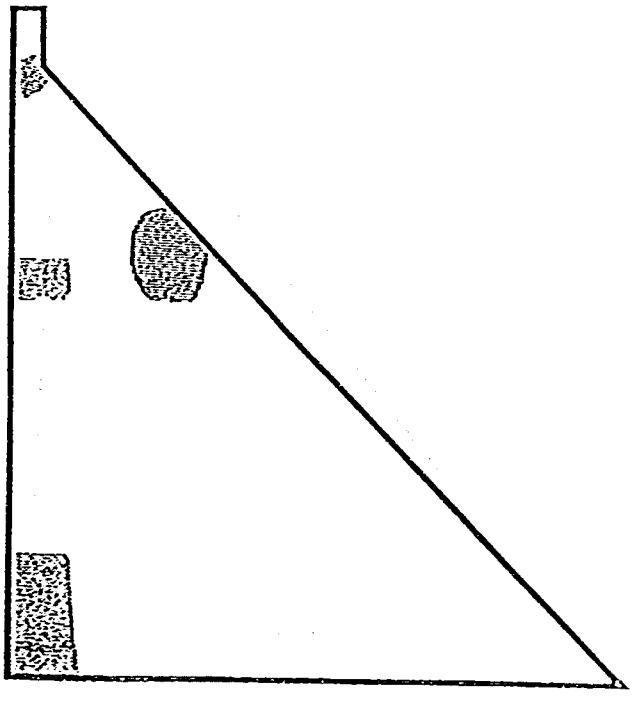
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(b) t = 4.16 sec.

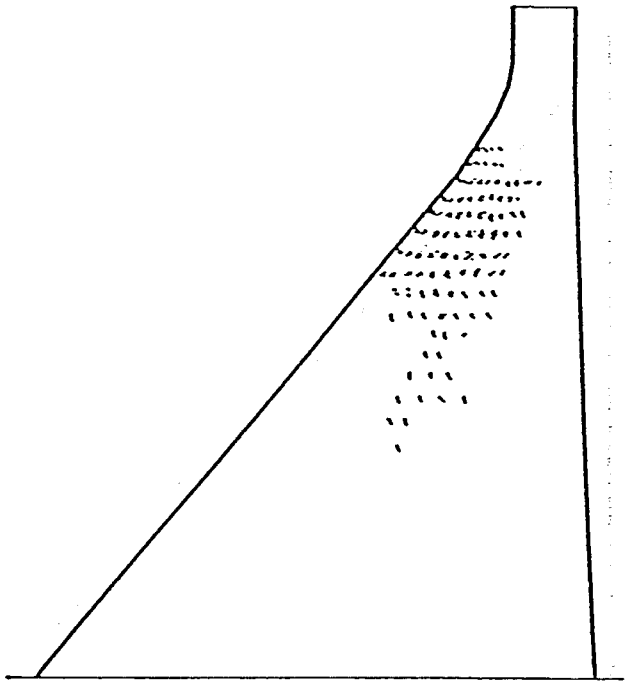


(c) t = 4.24 sec.

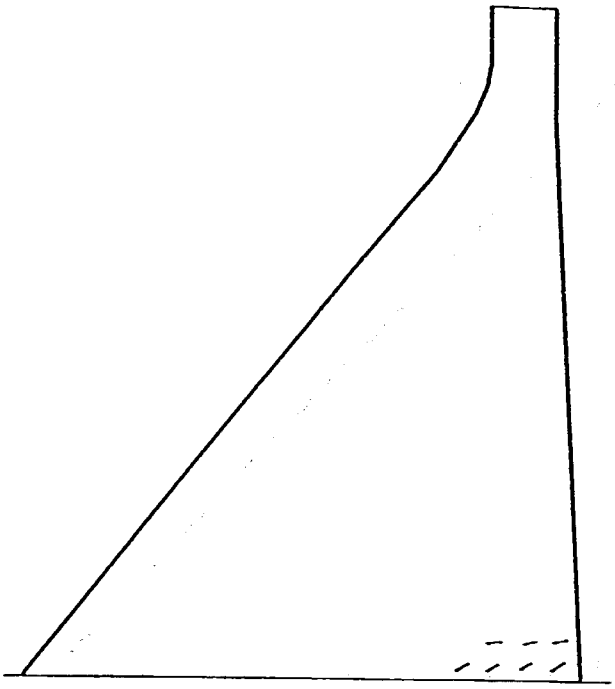


(d) t = 4.40 sec.

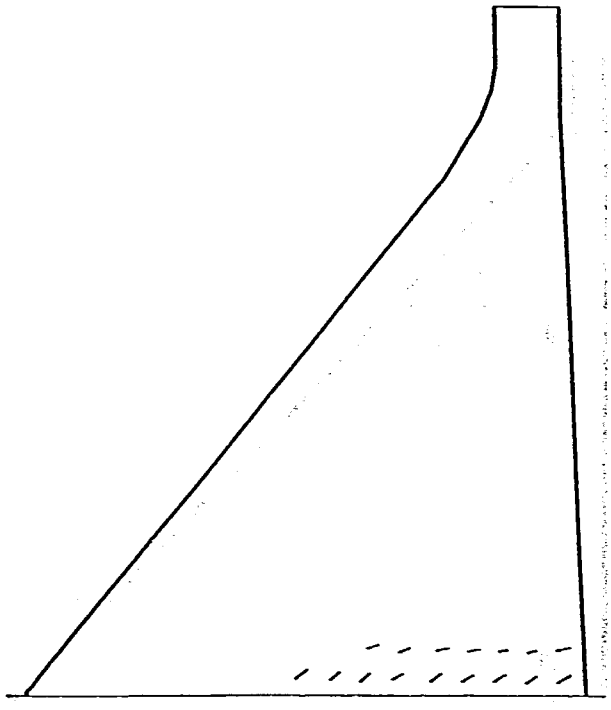
Figure 3. Cracked zones of Dworshak Dam.



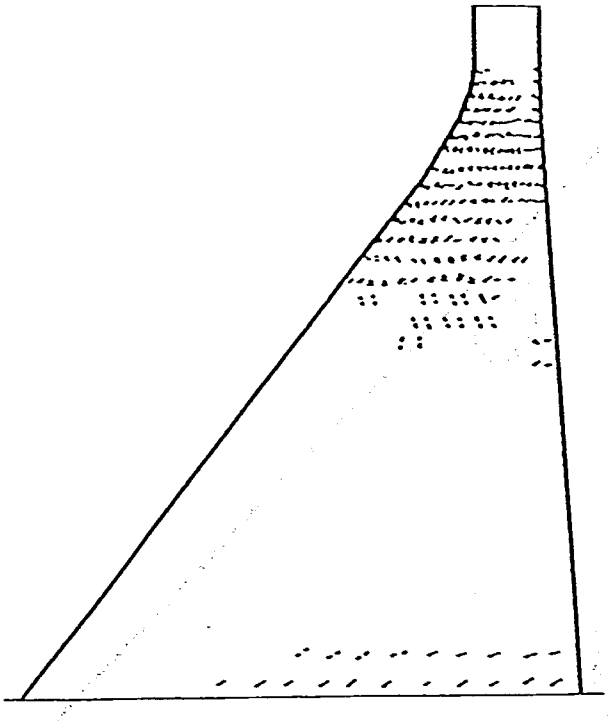
(a) Empty reservoir, a_g = 0.45 g



(b) Full reservoir, a_g = 0.18 g



(c) Full reservoir, a_g = 0.27 g



(b) Full reservoir, a_g = 0.36 g

Figure 4. Tensile cracks in Pine Flat Dam due to Taft ground motion scaled to peak ground acceleration a_g.

Seismic Risk Assessment Methodology for Water Systems

by

Craig E. Taylor* and Stuart D. Werner**

ABSTRACT

In this paper, we discuss some of the methodological highlights of the five-year voluntary project resulting in the American Society of Civil Engineers (ASCE) monograph entitled Seismic Loss Estimates for a Hypothetical Water System. Highlights of this project include application of the monograph in progress to regional loss studies and discussions of valuation methods, seismic attenuation models, analysis of liquefaction-induced ground failure potential, analytic models for estimating losses from given input earthquake intensities, and probabilistic modeling of earthquake activity and of risks. Based on the results of this project, we make recommendations on future steps to be undertaken in research, the development of a manual, and the integration of earthquake risk methods into system planning.

KEYWORDS:

1. INTRODUCTION

In 1986, the Seismic Risk and the Water and Sewage Committees of the Technical Council on Lifeline Earthquake Engineering (TCLEE) of ASCE undertook a project to demonstrate how direct earthquake loss estimates can be evaluated for a water system. Two years after project inception, a paper on the project progress was presented at the Ninth World Conference on Earthquake Engineering in Tokyo (See Taylor, et al., 1988). Four years after project inception, a paper was delivered on project progress at the annual ASCE convention (see Taylor et al., 1990). Since the state-of-the-art still requires much improvement, this paper explained why the monograph was not designed to be a manual: After five years of effort involving about forty participating engineers, the monograph entitled Seismic Loss Estimates for a Hypothetical Water System was completed (see C. Taylor, ed., 1991).

To develop primary loss estimates, the following steps were undertaken:

- Exposure Definition. The characterization of a hypothetical demonstration water system, including component definition, spatial definition and replacement cost estimation.
- Seismic Hazard Identification and Definition. The characterization of seismic hazards including earthquake sources, regional attenuation patterns, local strong ground motion amplification factors, and local susceptibilities to permanent ground failures.
- Component Loss Algorithm Development. The development of component loss algorithms (or, for other types of studies, component reliability or fragility models), and
- Seismic Risk/Loss Calculation. The calculation of expected direct system losses (or, for other types of studies, the calculation of system reliability and/or secondary losses) and of various breakdowns on these losses (e.g., by

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earthquake source, by component type, by locations in high liquefaction susceptibility zones).

The project implementation began with the assignment of specific project steps to lead engineers, with a group identified for assistance and review. Accomplishment of these tasks was given very wide latitude, with an emphasis on statements of potential difficulties and how potentially innovative approaches can be used to overcome these difficulties. Project coordination made it possible for project steps to be taken so that they fit into an overall framework--a demonstration earthquake loss analysis and resulting monograph being the ultimate goals.

In the remainder of this paper, we first outline some of the highlights of the project and then discuss some future steps that might be taken to improve seismic risk applications to water systems.

2. SELECTED PROJECT HIGHLIGHTS

The highlights of the project result from special efforts made by voluntary participants in for what was many a new context: an earthquake risk analysis. Highlights include:

- The use of the draft monograph in progress as material for regional earthquake loss studies
- The revelation that uniform valuation methods would greatly assist in water system planning as well as in exposure definition for earthquake risk analysis
- The clarification of selection criteria for attenuation functions and of the need for improved modeling of the probability of liquefaction-induced ground failure
- Attempts to overcome the use of subjective estimates of losses through analytic modeling of filtration plants and pipelines, respectively, and
- Use of probabilistic modeling of earthquakes so that potential losses of earthquake scenario selections are overcome.

After reviewing these highlights, we make recommendations on future steps that might be taken.

The aforementioned project highlights will be discussed in turn. Those highlights discussed at length previously will be discussed here only briefly.

2.1 The Benefit to Required Earthquake Loss Studies

When the TCLEE project began, many of the participants were not experienced in all steps needed to perform an earthquake loss analysis for a water system. However, as the project evolved, many participants not only fulfilled their tasks successfully but also became better acquainted with the overall steps needed to undertake a seismic risk analysis. At least three regional earthquake loss studies resulted from the experience of project participants demonstrate the diverse ways how the collaborative TCLEE project has increased the awareness of practitioners of the benefits of the state-of-the-practice methods illustrated in the monograph. These include earthquake loss evaluations of the Seattle water system (Ballantyne et al., 1990), the water system and other utilities in the City of Everett, Washington (Ballantyne et al., 1991), and of the water system in the City of Portland (see Wang et al., 1991).

2.2 The Role of Valuation (Cost Estimation Procedures)

Although technological development and research attention has rightly focused on ways to facilitate inventory development for lifeline networks, the greater emphasis in this project turned out to be the relative absence of uniform valuation methods for water system components. The experience of project participants in water system engineering and planning was

demonstrated in their sensitivity to the uncertainties involved in estimating replacement and repair costs. Often overlooked in scientific and engineering risk studies, these uncertainties can easily yield 30 percent variations in some earthquake loss estimates--at least for specific components. (A more complete discussion of this issue is provided in Taylor et al., 1990.)

2.3 The Role of Ground Attenuation and Liquefaction Probability Models in Seismic Risk Analysis

Although covered elsewhere in the growing earthquake literature, the monograph contains general criteria for the selection of suitable functions for modeling how earthquake ground motions attenuate with distance from the earthquake source. For the specialist as well as the interested engineer, these criteria are expounded in terms of:

- types of variables, including the dependent strong-motion parameter to be used in conjunction with component loss models and the independent parameters such as earthquake magnitude selected, distance from source to site, and local site effects.
- range of variables for application and empirical evaluation (e.g., accuracy of distance estimates, lower and upper earthquakes magnitudes considered, and selection of valid strong-motion records)
- functional form, or form of the equation applied and statistically evaluated.

The models used to assess liquefaction-induced ground failure hazards are based on an extrapolation from standard in-situ methods rather than on general liquefaction probability matrixes. Adaptation of in-situ methods, whether to critical components directly or to microzones through extrapolation from existing boring log and geological data, potentially permits this element of seismic risk analyses to be improved through research designed to examine further these standard methods. As one example, the current problem of how earthquake magnitude (or, somewhat more precisely, the number of strong ground motion cycles) affect liquefaction probabilities is a problem for seismic risk analysis as well as a typical research problem for standard methods.

2.4 The Role of Component-Loss Models in Seismic Risk Analyses

The latitude that was provided task leaders produced noteworthy results with respect to the development of component loss models. These models represent loss (as a percent of replacement value) as a function of some measure of earthquake intensity (e.g., Modified Mercalli intensity, peak ground acceleration, peak ground velocity, etc.). Response to this task typically required clarification of the component-loss model to be developed and, as expected, a lively discussion of what notion of intensity was suitable.

Resulting component loss models were developed from (a) readily-available existing models i.e., the Applied Technology Council (ATC-13) models), (b) empirical data and/or engineering judgment and/or (c) analytic methods. In many cases, as with water filtration plants, past earthquakes provide only scant evidence of potential earthquake damage. Furthermore, this evidence is only rarely documented in terms of dollar loss relative to replacement value, and it is often applicable to filtration plants whose seismic design provisions were very different from those of the plants analyzed in the TCLEE project. In other cases, as with pipelines, investigators have made significant strides in compiling damage data in earthquakes.

Given the dearth of empirical damage or loss data, the uniqueness of some structures, the need to explain damage through analysis, and the subjectivity of readily-available models, task leaders in two topic areas developed analysis methods in order to arrive at component loss models. Those two topic areas were filtration plants and pipelines, respectively. For this

presentation, we shall discuss very briefly the application of analytic methods to filtration plants.

Analytic methods consist of the use of dynamic response calculations, actual earthquake data, and/or experimental data to establish algorithms that interrelate dynamic response with potential earthquake losses. A wide range of dynamic response descriptors and input motion intensity descriptors can be used in these methods as long as (a) the input motion intensity descriptors can be correlated with the potential earthquakes simulated for the water system; and (b) the dynamic response descriptors can be used to establish meaningful relationships between dynamic response and estimated losses.

The TCLEE Monograph contains an example of the application of an approximate analytic method to estimate a loss model for a chemical feed building. This method consists of (a) the development of design spectra for the building, based on empirical compilations of ground response data for different geologic conditions; (b) estimation of a force-deformation relationship for the building that incorporates the building's yield limit and its ultimate capacity; (c) use of these results to obtain approximate dynamic characteristics of the building in terms of its natural period and damping ratios at yield and at ultimate; and (d) estimation of the potential damage to the building as a function of the ground acceleration, by using the design spectra from Step (a), the approximate dynamic characteristics of the building from Step (c), and certain simplifying assumptions that relate the degree of damage to the amount of inelastic deformation estimated for the building at each ground acceleration level.

2.5 Probabilistic Procedures for System-wide Loss Calculation

A major emphasis of the monograph was the use of probabilistic rather than deterministic procedures for calculating potential earthquake-induced losses in geographically dispersed networks. At a minimum, probabilistic modeling incorporates loss estimates from an entire suite of representative earthquakes. In contrast, deterministic modeling considers only a small sample of earthquake scenarios. Whether or not probabilities are somehow assigned to these scenarios, enormous biases enter into the selection. For those who are well-versed in earthquake risk analysis, it becomes possible to select the earthquake scenarios that provide the desired loss result, whether high or low. In the monograph, we avoid these potential biases through the use of probabilistic methods initially developed as recently as the mid-1980's.

3. RECOMMENDATIONS FOR FURTHER RESEARCH AND APPLICATION

The state-of-the-art in assessing the earthquake risks to water systems can still benefit from improvement in a number of areas, as implied for instance by the discussion of highlights. That discussion yields the following research recommendations:

- the need for continuing review and improvement of valuation procedures, especially as the ability to transfer data improves
- the ongoing need for improvements in all geoscience models, including those of earthquake activity sources and rates, attenuation models, and models of the response of sites owing to local soil conditions; the need for improved models to estimate liquefaction probabilities was apparent in this project
- the strong need to replace subjectively derived component loss models with models based on experience and/or analysis.
- the need for a continuing re-examination of how parameter variability affects loss estimates (this includes continuing progress in clarifying the variability of individual parameters).

- compilation of data to assist in the future development of models for calculating secondary losses.

Already earthquake loss and risk estimation procedures have been used in:

- regional earthquake loss estimation studies including water systems
- studies to develop guidelines for state regulatory activities with respect to culinary water facilities, and
- studies designed to assist in developing loss prevention programs for specific systems or subsystems.

Although these earthquake risk studies are clearly providing important insights for planning, design, and retrofit of water systems, there is a need to further develop and refine the current risk modeling and analysis procedure. The monograph itself was designed first for the audience of participants themselves and second for an audience of other potential investigators interested in current methods and difficulties in earthquake loss estimation. Within the limited scope of this paper, only suggestions can be made of how to further this technology transfer. A manual could be a next step, especially if some of the above research issues that outline the step-by-step seismic risk analysis process are addressed. Assignment of a working group to define further possible ways to effectively apply seismic risk analysis to water systems could be another step. Along these lines, earthquake risk methods can be incorporated into current National Institute of Standards (NIST)/Federal Emergency Management Agency (FEMA) efforts to determine how to develop and implement earthquake risk reduction strategies for lifeline networks.

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課題Ⅱ　耐震工学　討議

（４）Seismic Design Considerations for Jordanelle Dam
(J.Wilson*, P.Hensley, W.Engemoen and F.Mclean)

山　　口（Ｑ）　動的応答解析を行なう際の物性値と解析法はどうなっているか？

Wison　（Ａ）　一定の試験方法があり、ダム各層の密度、せん断波速度を求める。この物性値をもとに応答解析を行っている。

山　　口（Ｑ）　鉛直応答の計算を行っているが、それに対する安全性評価上のクライテリアはあるのか？

Wilson　（Ａ）　どこが危険かの判断基準を検討中である。

（５）Consideration on Evaluation of Seismic Force Acting on Embankment Dams and Its Simplified Prediction Procedure
T.Fujisawa, N.Yasuda, N.Matsumoto and K.Yamabe (Y.Yamaguchi*)

Franklin（Ｑ）　１次元と２次元解析との差は１５％程度と、両者の対応はよい。これは非線形性、粘性減衰の影響か、１５％程度の差が一応の目安か、１次元解析でも値はかなり大きいと思うが？

山　　口（Ａ）　詳細はわからない。

Kimbol　（Ｑ）　非線形解析が必要である。大型のダムでは物性がわからない。最低基準としてはコンサバーティブとなる。

（６）Resolving Uncertainties in Seismic Hazard Model
J.K.Kimball*

（Ｑ）　震源評価が重要と指摘されたが、それを限定すれば精度があがるのではないか？　地域によって異なるため歴史的記録と照合して、地域特性を決める必要があるのではないか？

Kimball（Ａ）　決める必要がある。

川　　島（Ｑ）　図４中記号の○、△は実測データか？

Kinball（Ａ）　統計処理を行ったものである。

川　　島（Ｑ）　記号の意味を聞いている。

Kimball（A）　歴史的資料である。

（7）　Earthquake Alert System Feasibility Study
Harben、P.*

川　島（Q）　このシステムが実現すると、例えば、サンフランシスコ湾岸地震では、どの位の時間がセーブできるのか？

Harben（A）　断層によって異なるが、ロマ・プリエータ地震のようなものであれば、30秒程度の時間がセーブできる。

（8）　Guideline for Seismic Design of Large Underground Structures
Asakura, H.*, Kawashima, K., and Sugita, H.

Ｌｅｗ（Q）　岩盤中のトンネルについては耐震設計は不要か？

朝　倉（A）　岩盤中であれば、変形が小さく、耐震設計は不要である。

Ｌｅｗ（Q）　この耐震計算法は、径の小さいパイプ類にも適用できるか？

朝　倉（A）　適用可能である。

Ｌｅｗ（Q）　ＮＩＳＴが現在進めているライフライン施設の耐震設計法の整備に関連が深く、関心があるので、このガイドラインが印刷できたら入手したい。

朝　倉（A）　現在、印刷準備中であるので、印刷後、送付する。

（10）　Design Criteria for Base Isolation of Building
Shenton, H. Tripp.*

Katona（Q）　鉛直方向の荷重について考慮しているか？

Shenton（A）　鉛直方向の入力は考えていないが、転倒などの慣性力に対しては考慮している。

北　川（Q）　上部構造はどのようなモデル化になっているか？

Shenton（A）　モデルとしては論文の３章に示すように塑性骨組である。

北　川（Q）　コード耐力100%及び２５％（基礎固定）のとき、破壊状態はどうか？

Shenton（A）　解析ではコード耐力５０％のときは１００％と同じ状況、R／Cでは２５－５０％で１００％と同様な状態である。

Abrol（Q）　４階建てでT=2.5secのモデルがあるが、Tが2.5secを超えた場合はどうなるか？

Shenton（A）　通常、免震システムの固有周期は２－３sec となるようにモデルを設定する。

（１１）Menshin Design Manual for Highway Bridges
K. Kawashima*, K. Hasegawa and H. Nagashima

Wright（Q）　パフォーマンスクライテリアは実際に製作可能な免震装置の特性をどう考慮しているのか？

川　島（A）　設計する方はクライテリアとしては厳しい方が良いが、それに対してデバイスの製作ができなければどうしようもない。メーカーサイドの立場からの人間と設計サイドの立場からの人間が何回も話し合ってクライテリアを決めた。結果的には現状では製作可能で、その中でかなり上のレベルをねらっている。従って、このクライテリアを満足する免震支承は現状でもかなりある。今後のよりレベルの高い免震支承の開発に対するメーカーの努力を求めるためには、クライテリアとしては高いレベルを示した方がよい。

Cooper（Q）　免震設計は、設計と同時に耐震補強にも使用できる。耐震補強として免震設計を使用する場合の桁変位のコントロールのクライテリアは？

川　島（A）　もちろん、耐震補強にも適用可能である。耐震補強では、橋脚の耐力に合わせて、固有周期と減衰性能を定めなければならない。あまり剛性の高い（固有周期の短い）免震支承を使用すると、桁の変位は小さくて都合がよくても橋脚の耐力はオーバーすることになる。従って、桁の変位と橋脚の耐力の両者を考慮して免震設計を適用することになる。

Ｌｅｗ（Q）　ジョイントを大きくすると騒音が大きくなることがないか？

川　島（A）　その通りである。都市部では、騒音が重大な問題となっており、過度に長周期化して、ジョイントのガップを大きくすることはできない。したがって、長周期化と桁の応答変位はトレードオフの関係になる。

Ｌｅｗ（Q）　免震橋では、支承としては、ゴムタイプがよいのではないか？

川　島（A）　走行性の点からはゴムタイプがよくても、耐久性の点から問題があるようだ。日本ではいろいろなタイプのジョイントが開発されているが、まだ、不十分であり、ジョイントの取り替えが交通渋滞の大きな原因の一

つとなっている。従って、ここでは、フィンガージョイントをベースにして２方向に移動可能なジョイントを開発したが、これについては、免震設計以前の問題として、今後、さらに技術開発が必要である。

（１２）Friction Controllable Bearings for sliding Base Isolation System
Shinozuka, M.

Abrol（Q）　耐火について考慮しているか、また、信頼性はどうか？

篠　塚（A）　耐火についてはやっていないが、信頼性テストは今やっている。

Abrol（Q）　温度の上昇は？

篠　塚（A）　温度は常時アルゴリズムの一環として検知している。

Ｌｅｗ（Q）　プレッシャーに関して検知システムは？

篠　塚（A）　まだ室内実験の状況で、トリガーはつけていない。

Cooper（Q）　Ｐｒｅｓｕｍｅのコントロールについて。Ｆｉｇ．１のセンサーをすべて使っているのか？

篠　塚（A）　速度センサーのみをコントロールに使っている。

Brady（Q）　Fail-safe Mechanismの観点から、シールが破れたときの対応はどうか？

篠　塚（A）　現在のシステムでは、一つのFailが系全体に影響する。このため、現在の研究課題としており、日本のメーカーと共同で対処する予定。

（１３）State-of-Art and Building Structure with Seismic Response Control System in Japan
Kitagawa, Y*. and Midorikawa, M , BRI

大　谷（Q）　アクテイブ・ダンパーは本当に有効なのか？

北　川（A）　付加質量が１％であるとストロークは１０ｍになるが、ＴＭＤシステムを使用すれば、このストロークを短くすることができるようになる。社会的なコンセンサスが得られれば、アクテイブ・ダンパーを使用するようになると思うが、今は、これらのシステムをエンカレッジする状況である。

Liu　大谷さんの質問は適切なポイントをついており、理にかなったものである。また、北川さんの答も的を得ている。しかしながら、大谷さんの質

問は無視できないが、新技術の開発には何等の制約をつけずにアクティブ・コントロールのプログラムを進めたい。

（１５）Significant Results from Standardization Work on Rubber Bearings for Base-isolated Buildings
Ohashi, U. and Yamanouchi, H.*, BRI

Celebi（Q）　免震支承の標準化に米国のデータを使用しているのか？

山　内　（A）日本のメーカーのデータだけを使用して基準化を図っているが、米国のデータとの比較は意義あるものと思う。

（１６）Effect of Temperature on Dyanamic Behavior of a Base-isolated Bridge
Nakano, O.*, Taniguchi, H and Nishi, H., HDB

Celebi（Q）　免震支承の実験では、寒暖の繰返しによる経年変化の影響を考慮したのか、また、その影響は？

中　野　（A）一つの供試体を使用して各温度で実験を行ったので、経年変化の影響は考慮していない。もし、影響が現れたならば、リプレイスすれば良いのでは。

川　島　（C）日本では、ラバー支承の寒冷地での使用実績には長い歴史があり、その問題は起きていない。ただし、免震支承については、メーカーの製品検査の方法を今後検討する
必要がある。

北　川　（Q）寒冷地での積層ゴムの特性に関し、温度変化による剛性の影響については？

中　野　（A）確かに、この問題が当初から予想されたので、この実験を行った。その結果、予想以上に剛性の変化があることが実験結果より判明した。それで、この影響が、橋梁全体系に与える度合いを調べたが、必ずしも、その度合いで影響が橋梁全体系では生じないことも今回の解析より明かになった。

（１７）Daynamic Centrifugal Model Tests of Embankments on Liquefiable Ground
Koga, Y., Takahashi, A. (Koseki, J.*)

北　川（A）液状化により地盤の固有周期は長周期側に変化すると思うが、このような測定を実施したか？

古　　関（A）　液状化に至る過程での固有周期の測定は、実験的には困難である。ただし、剛性が減少するので、固有周期は当然長周期側に変化するものと考えている。不規則波加振時に見られた、液状化後の突然の加速度応答はその影響によると考えている。

Hynes　（Q）　遠心力模型実験では、地盤の固有周期は、実際の盛土地盤に対応した値であったか？

古　　関（A）　装置の制約により、固有周期の測定は行なうことができない。

Hynes　（Q）　土槽端面の影響を低減するために、何か処理を行なったか？

古　　関（A）　特別な処理は行なっていない。実験結果を説明できるような解析手法を開発し、これを端面の影響のない実地盤に適用することにより、対応したいと考えている。

（18）Earthquake-Induced Landslides in the Santa Cruz Mountains
Mary E. Hynes*

古　　関（Q）　対策工の検討を行なっているか？

Hynes　（A）　対象地域が広大なので全体的な対策は不可能である。特定の施設について対策すること、及び開発規制等の規制を行なうことは可能と考えている。

大　　谷（Q）　Newmarkのすべりブロック解析では、地下水位の影響を考慮した地すべり範囲の予測を一般にできるのか？

Hynes　（A）　地震時のすべりは、通常地下水面以深で生じる。応答加速度の増幅特性に及ぼす地下水位の影響は小さい。

（19）Study on Estimation of Lateral Ground Flow by Soil Liquefaction and Its Influence on Piles
Tokida, K., Matsumoto, H., Towhata, I., and Sasaki, Y. (Koseki, J.*)

Hynes　（Q）　杭の実験で、地盤の固有周期を測定したか？

古　　関（A）　たぶん測定しているので、後日資料を送りたい。

川　　島（Q）　前記の質問をした背景を教えてほしい。

Hynes　（A）　杭の有無が地盤の固有周期に及ぼす影響を知りたい。

川　　島（C）　実験では、たぶん測定していないと思う。御質問の内容は、杭と地盤との

動的相互作用として研究されている分野である。これについては、多数の研究があるので、その資料を後日送りたい。

Ｌｅｗ　（Ｑ）　地盤流動の研究で、杭以外を対象としたものがあるか？

川　　島（Ａ）　上水道・下水道パイプについては、地震後に掘り起こして調査した結果がある。抵層木造住宅、ＲＣビルについては、地震前後の航空写真を比較して調査した結果がある。これらの研究は、主に東海大学の浜田教授らが中心となって実施している。

（２０）Horizontal Bearing Capacity of Pile Foundations Based on Horizontal Loading Tests
M. Okahara, S. Nakatani, Y. Kimura, and S. Takagi (M. Fujiwara*)

Cooper（Ｑ）　図－７における地盤条件および杭の諸元は同一か？

藤　　原（Ａ）　種々のケースを行っており、土質条件は論文中に示してある。

Cooper（Ｑ）　土木研究所における試験の他に大スケールの試験結果もあるか？

藤　　原（Ａ）　建設現場において実物の杭１本について実験している。

H.S. Lew（Ｑ）　荷重－変位曲線におけるReasonable Displacement:δは如何なる値を言うのか？

藤　　原（Ａ）　構造物に有害な変形が発生しないという条件として日本の基準で一般に示されている杭径の１％以下とした。

（－）Seismic Requalification of Offshore Structures　（論文なし）
C. Smith*

川　　島（Ｑ）　プラットフォームの杭の根入れ長は如何にして決めるのか？

Smith（Ａ）　横方向荷重に対してはアンカーワイヤーで支持されているので、この構造物に対して支配的である鉛直荷重によって決める。

Brady（Ｑ）　ニューオリンズのプラットフォームにおいてオイラーバックリングの可能性は無いのか？

Smith（Ａ）　浮力が作用しており、その心配はない。

（２１）Seismic Effects on Port and Harbour Structures

H. Tsuchida and S. Noda*

Hays（Q）　図－８において上下動比がばらついている原因はなにか？

野　田（A）　異なる地震、地盤および観測地点のデータによることが一因であるが、その他は不明である。

Smith（Q）　サンタバーバラの水深４０ｆの海底の地震動が陸上の値より小さかったが、日本でもそのような事例があるか？

野　田（A）　海底地震観測のデータが無いので、そのような比較例は持っていない。しかし、数値解析で両者の相違を検討した例はある。

（２２）　Worldwide Seismic Risk Management Program-Progress Report
Hays, W.　(Alegermissen, T.*)

大　谷（C）　日本への協力依頼があったが、日本ではWWERM project に入力すべきデータは十分にあるので、具体的方法を検討することとしたい。

Ｈａｙｓ（A）　我々の作業は世界中のSeismic Risk Assessment Map のジグソーパズルを埋めていくことなので、ぜひ御協力を願いたい。

川　島（C）　日本ではRisk Analyses に関して多くの研究成果を有しているので、相互に持っているものの比較等をStarting Pointとしながら、作業部会Ｅの中で活動して行きたい。

（２３）Implementation of Executive Order 12699, "Seismic Safety of Federal and Fedrally Assisted or Regulated New Building Construction"
Wright, R.*

川　島（Q）　ActとExecutive Orderの違いは？

Wright（A）　Actは議会にて決定されたものであり、E.O.は大統領から連邦政府への命令である。

川　島（Q）　Executive Orderは民間へは効力が及ばないのか？

Wright（A）　直接的には効力はないが、間接的な行政指導等により効力は及ぶものである。

川　島（Q）　Lifelineについてはどうなっているのか？

Wright（A）　1977のActは1990年に修正され、Lifelineについても耐震対策を講じるこ

とが求められている。本年６月末には議会に検討結果を報告することになっているが、時期的にはややむずかしいと考えている。

大　谷（Q）　既存建物については？

Wright（A）　1994年12月迄に検討することとなっている。

Ｌｅｗ　（Ｃ）　Actは2-3年ごとに改訂が行なわれており、1990年の改訂が最新版である。

（２４）CORPS Earthquake Engineering Research Program
Roper, W.*

山　口（Q）　水じめダムのモデル化では、明確なZoningは不可能では？

Roper　（A）　図に示したものは単純化したものであり、より詳細なモデル化のためにはより多くの研究が必要であると判断している。

山　口（Q）　動的解析に用いる材料定数はどのようにして得ているか？

Roper　（A）　コアボーリングの試料等から決定しているが、パラメトリックな検討も行なっている。

山　口（Q）　米国には３つの水じめダムがあるとのことだが、Sardis Damがもっとも高地震地帯にあるのか？

Roper　（A）　Sardis DamはNew Madrid Fault Zoneから100Km位にあり、最も地震危険度が高い。他の２つについても地震危険度評価を行なっている。

（２５）Seismic Performance of Reinforced Concrete Structures Utilizing High-Stregth Materials
T. Murota, H. Hiraishi, T. Kaminosono, H. Shiohara and H. Fujitani
(Y. Kitagawa*)

Wright（Q）　1000Mpaの鉄筋にはストレーンハードニングがないが、ダクタイルな挙動が得られるのか？

北　川（A）　論文の２．２章で材料テストを行っている。高強度鉄筋の各種テストによれば、部材角で１／８０までは問題がなかった。

Ｌｅｗ（Q）　図２．４のコンクリートのＳ－Ｓカーブを見ると、ピークと破壊の間のデータが無いが？

北　川（A）　後で調べて、報告する。

（２６）Dynamic Testing of Reinforced Concrete Frame
P.A. Brady and S. Sweny*

Celebi （Q） 最新のコードでは、ダクタイル名材料を規準にしているが、ノンダクタイルな材料については、どう取り扱うのか？

Sweny （A） 工兵隊としては、ダクタイルな材料を基本に考えている。

Celebi（Q） ノンダクタイルな材料の補修・補強方法についてのデータは？ また、現在のノンダクタイル部材に関する変形量の情報は？

Aweny（A） 手持ちの情報の中からデータがえられると思う。変形については、1980年代中期に開発した方法で解析できると思う。

Shinozuka（C） 米国では、ＮＳＦのサポートで補修・補強に関するプロジェクトが始まっており、また、工兵隊にも同様なプログラムがある。ダイナミック・ダンパーを用いた補修方法も研究されている。

（２７）Shaking Table Test Collapse Tests of Reinforced Concrete Columns
N. Oyagi, N. Ogawa* and T. Minowa

Colebi （Q） 最新のコードでは、ダクタイル名材料を規準にしているが、ノンダクタイルな材料については、どう取り扱うのか？

大 谷（A） せん断補強筋の密度を変えてダクタイルからノンダクタイルの実験を行っている。ノンダクタイルのものについては、動的実験で補強法を調べようとしている。

Ｌｅｗ（Q） 日本では、せん断補強筋のスペースは50㎜と規定しているのか？

大 谷（A） 十勝沖地震で、せん断補強筋の規定が変わった。

Ｌｅｗ（Q） 実験では、せん断補強筋のスペースが280㎜とされているが、これは現実的な値か？

大 谷（A） 規定よりもかなり大きなスペースのものである。

Shinozuka（Q）Ｐ－デルタ効果（図１０と図１１）について、両方の図で何が異なるのか？

小 川（A） 図１０では崩壊が進む傾向がみられる。まだ、前段階的な解析で、これから検討したい。

（２８）A Hollow Clay Tile Wall Seismic Performance Program Overview
Beavers, W.D.*, Jones, W.D. and Stoddart, WCT.

Cooper（Q） コードでは0.1gの耐力を要求しているのに対し23倍の耐力を得たとのことだが１PSIで20分の荷重保持は何によって決まっているのか？

Beavers（A） 載荷、定常化、計測等によるものがある。

Cooper（Q） 補修は必要ないが危険が存在するとしているが、その危険度とは何か？

Beavers（A） life safetyを目的とするなら軽微なクラック等は許容されると思う。被害の許容範囲はその予測が必要である。

（３０） Preliminary Report on Reconnaissance of Turkish Earthquake
Celebi, M.*

北　川（Q）コードは良かったようであるが、技術移転に問題はあったのか？

Celebi（A） 技術移転にも問題はあったと思う。トルコでは大学を出るとすぐに仕事に就くという背景がある。

住　吉（Q）災害建物のチェック法について問題があったのでは？

Celebi（A） 設計は良かったが、施工品質管理に問題があったと判断している。Recommendationを含めた報告書がＥＥＲＩから出版される予定である。トルコの技術者もこの点は認識しており、各種の調査を実施中である。今後の改善が期待される。

大　谷（Q）震央距離が最短の強震記録はどこで得られているか？

Celebi（A） 複数の記録が10km以内で得られており、良好なNear Field Recordが収集された。

課題III. 高潮・津波

１７９２年眉山の地すべりにより発生した津波の数値計算

後藤智明*・高山知司**

要　約

１７９２年に発生した島原雲仙岳の山体崩壊による有明海の津波に関する数値計算を行い，津波の特性を調べた．その結果，海底地形の影響による津波の集中と発散など興味深い津波特性が明かとなったので報告する．

キーワード：土砂崩れによる津波，津波，数値計算
雲仙岳，島原湾

１．はじめに

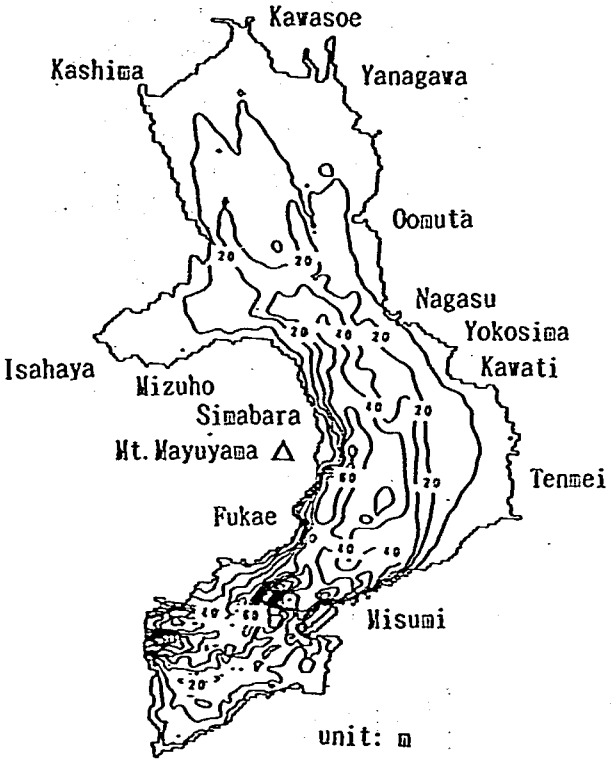
島原雲仙岳の火山活動が活発となり，火砕流・土石流または山体崩壊などにより津波が発生する可能性が危惧されている．１９７２年には，火山活動に伴う地震動で眉山が崩壊し，土砂崩れとなって島原湾に流入し，大津波を起こしている．この津波により，島原半島で人命約１０，０００名が失われ，対岸の熊本沿岸でも約５，０００名の人が亡くなっている．

一般に，津波の発生原因としては，地震による海底地盤の変動，火山活動（噴火・爆発），地滑りそして核爆発が考えられる．７世紀以降にわが国沿岸付近で発生した津波の記録を調べてみると，発生事例の約９５％が地震に起因する海底地盤の変動により発生した津波であり，残りの約５％が火山活動を含めたその他の原因である．したがって，約２００年前に発生した島原湾の津波は少数の例にあたり，一般的な地震による海底地盤の変位に起因する津波と様相が異なる．

本文では，１７９２年の有明海の津波の数値シミュレーションの結果を報告する．

２．津波の数値計算法

計算に用いた方程式は，海底摩擦を考慮した非線形長波理論

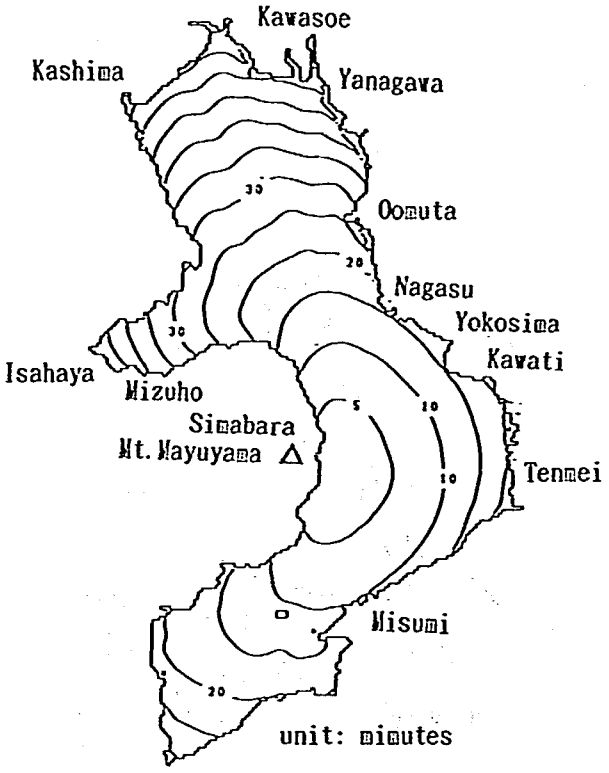


図－１　島原湾・有明海の水深分布

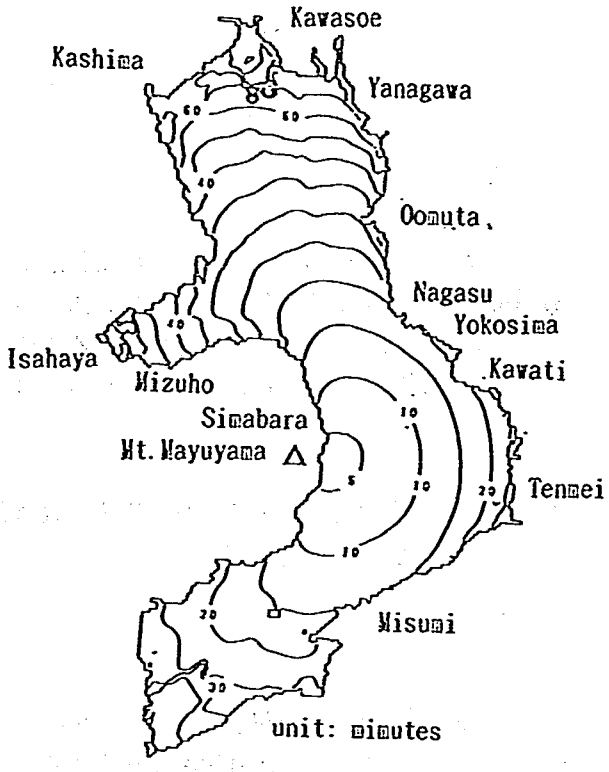
$$\frac{\partial \eta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \qquad \cdots\cdots (1)$$

$$\frac{\partial M}{\partial t} + \frac{\partial}{\partial x}\left[\frac{M^2}{D}\right] + \frac{\partial}{\partial y}\left[\frac{MN}{D}\right] + gD\frac{\partial \eta}{\partial x} + \frac{g\eta^2}{D^{7/3}}MQ = 0 \qquad \cdots\cdots (2)$$

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図－２　第１波先端の到達時間



図－３　第１波ピークの到達時間

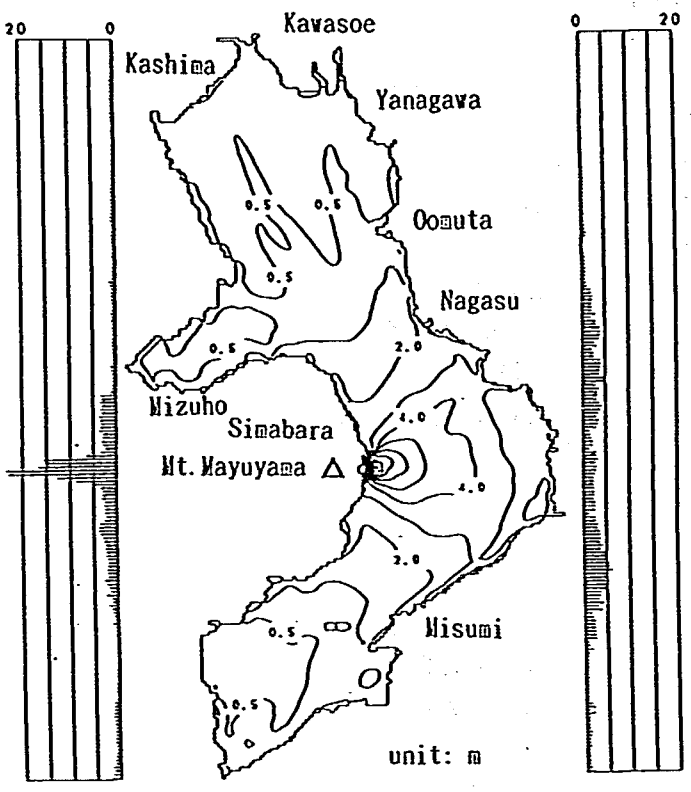
$$\frac{\partial N}{\partial t}+\frac{\partial}{\partial y}\left[\frac{N^2}{D}\right]+\frac{\partial}{\partial x}\left[\frac{MN}{D}\right]+gD\frac{\partial \eta}{\partial y}+\frac{g n^2}{D^{7/3}}NQ=0 \quad \cdots\cdots (3)$$

である．ここに，（ｘ，ｙ）は静水面に沿った空間座標，ｔは時間座標，（Ｍ，Ｎ）は（ｘ，ｙ）方向の線流量，ηは水位変動，ｇは重力加速度，ｈは静水深，Ｄは全水深，ｎはマニングの粗度係数（０．０２５を採用）である．式（１）から（３）をスタッガード・リープ・フロッグスキームで差分化し計算している．

対象とした領域は，早崎海峡から奥の有明海および島原湾全域であり，図－１に対象領域の水深分布を示している．なお，この図は満潮位に関する水深分布である．

計算格子は全域で４００ｍとしている．一般的に，津波の数値計算では，数値計算上の誤差を小さく抑えるために津波の一波長を２０分割程度以上できる計算格子を採用する必要があり，これ以上粗い計算格子で計算すると数値的なエネルギーの分散がおこり津波波高が小さく計算されることが分かっている．島原半島から対岸熊本県の海岸の平均的な水深は約３０ｍである．発生する津波の周期は１０分程度であり，波長に換算すると約１０ｋｍである．したがって，計算格子が４００ｍであるので一波長が２５格子で近似されることになり，上記の条件を満たしている．

開境界としては，自由透過条件を適用した．海岸線の境界においては，完全反射の条件を用いている．ただし，干潟に関しては，津波の遡上および遡下を考慮している．本計算では，早崎海峡と柳ノ瀬戸で津波を外海に自由透過させている．対象領域内には，その他に本渡瀬戸，三角ノ瀬戸など八代海と結ぶ海峡があるが，これらの海峡は計算格子４００ｍに比べ狭いため



図－4　津波の最大水位分布

完全反射の条件を適用した．

津波発生時の潮位は計算対象の全域に関して島原湾の平均的な満潮位Ｔ．Ｐ．２．５ｍを与え，計算の初期条件としては静水すなわち海水の流動が一切ないと仮定した．

津波の発生条件としては，相田[1]の海中への土砂崩れの流入モデルを採用した．総土砂崩れ量は，太田[2]が推定した崩壊量4.6x10⁷m³の土砂量が４分間で海中へ流入するものと仮定し計算した．また，崩壊範囲としては，太田の推定した崩壊範囲から土砂の流出範囲を設定した．

３．計算結果

３．１　津波の到達時間

津波の第一波先端の到達時間分布を図－２に示す．図は津波発生時を原点として５分ピッチの等到達時間線で描いている．なお，津波の第一波先端の到達時刻は，水位が１ｃｍ以上変動した時点と定義している．

対岸の熊本県側に津波の第一波が到達する時間は，三角付近が最も早く津波発生後１０－１５分である．熊本港へは２０－２５分程度で到達し，有明海の湾奥部は５５分後である．

図－３は津波の第一波ピークの到達時間を示している．場所により多少異なるが第一波ピークは第一波先端の到達してから５分後となる．

３．２　津波の水位

図－４は津波発生から２時間までの各計算地点の最大津波水位をｍ単位で示したものである．図の両側の棒グラフは，東海岸と西海岸の海岸線に沿った津波の最大水位を表している．なお，津波水位はすべて静水

位からの津波の高さを意味しており，津波の絶対水位はこの津波水位に潮位を加える必要がある．

図から，発生域の津波は，浅い海域に多量の土砂が流入してくるため非常に高い水位となることがわかる．島原半島沿岸に沿った津波の波高分布は，発生域から離れるほど津波水位が低くなる傾向が見られるが，安徳から大崎の海岸で多少高くなる．

対岸の熊本県側では，長洲から菊池側河口までと緑川河口から三角までが周辺領域に比べ津波水位が高くなる．過去の津波による熊本県側の人的被害が長洲から菊池川の範囲の沿岸および緑川から三角の範囲の沿岸で多い傾向とよく一致する．

3．3　津波の波形

島原半島沿岸の津波波形に関しては，第1波も大きいが対岸からの反射波である第2，3波も同程度以上である．したがって，第3波までが防災上重要となると考えられる．熊本県側に関しては，第1波が大きく．第2波以降は第1波に比べ小さく，遠浅海岸であるため波形が多少前傾化し，段波やソリトン分裂の可能性が考えられる．有明海沿岸に関しては，勢力が弱まり，湾奥部の津波高さは島原半島対岸の高さに比べ1／10程度以下となる．

4．おわりに

1792年の雲仙岳山体崩壊に伴って発生した津波に関する数値計算結果について報告した．今後は，陸上遡上を含む詳細な数値計算により詳細な津波の挙動を明らかにする予定である．

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Empirical Simulation of Storm Histories for Coastal Design

by

Martin C. Miller[1] and Robin D. Reinhard[2]

ABSTRACT

Coastal erosion and flooding caused by hurricanes have tremendous potential for economic damage and loss of life in populated areas. The U.S. Army Corps of Engineers is frequently asked to determine the feasibility of coastal protection projects through evaluation of the economic benefits of project alternatives. The development of design criteria for water level, waves, and beach profile change require the evaluation of data collected over long time periods, or the generation of data using tested computer models. A methodology has been developed at the U.S. Army Engineer, Waterways Experiment Station, Coastal Engineering Research Center for determining project feasibility by the application of several environmental models. The economic evaluation depends on the application of a statistical resampling technique known as the "bootstrap" which allows calculation of expected values of an outcome by sampling, with replacement, from a limited historical data set. This method has been applied to a potential project at Panama City Beach, Florida.

KEYWORDS: modeling; water level; simulation; "bootstrap"

1. INTRODUCTION

Many coastal engineering design problems require estimates of extreme events such as wave height and water levels. Estimates of normal and extreme conditions are also required to justify projects that protect coastal property. Comparisons are made between the consequences of the occurrence of the design storm with and without the protection project in place. The economic benefit of a coastal protection project during the design event must exceed the loss that would be experienced if the protection were not in place. In the Gulf of Mexico and along the southeastern U.S. coast, extreme waves and water levels are caused by hurricanes. Along the mid-Atlantic coast, extremes may be caused by either hurricanes or northeasters, and along the northeast coast, northeasters provide the storms of record. The coastal response to these events is extreme shoreline erosion with consequent flooding, property damage and possible loss of life. The U.S. Army Corps of Engineers is often requested to design measures that will protect the coastal property from the effects of these extremes.

The statistical methods used to estimate the extreme value of an environmental variable are well known (Borgman and Resio, 1982; Goda, 1978; Haring, *et al.*, 1976). Data are collected or hindcast over a long period of time and systematic procedures are used to plot the data according to some probability distribution.

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The resulting curve may be extrapolated to some selected return interval to determine the magnitude of the parameter selected. Though a data record of three to five times the maximum return interval to be estimated is usually recommended, in most U.S. coastal areas the historical database is not adequate to represent the full range of possible occurrences of storm effects at a specified site. Future storms may pass at different coastal crossing locations than are present in the historical database and the storms may have different characteristics of central pressure, radius to maximum winds, forward speed, duration, etc. It may, therefore, be necessary to augment the historical data set with additional storms that could have occurred at the site given the random nature of events. This paper briefly describes a statistical procedure which may be applied to augment the historical database of hurricanes with hypothetical storms that could have occurred at the site.

2. BACKGROUND

A study has recently been conducted by the U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC) for the U.S. Army Corps of Engineers Mobile District to examine the feasibility of providing a protective sand berm along an eighteen mile section of Panama City Beach, Florida (Figure 1). This beach is heavily developed over much of its length, with single family homes and condominiums built along the waters' edge. Inlets at each end of the beach provide access to small boat harbors and quiet coastal lagoons. The beach is susceptible to flooding from hurricane related storm surge, which could result in great loss of life and property. Beach and flood protection is considered necessary, but the recreational and aesthetic value of the area must also be preserved. The protective sand berm concept was determined to be appropriate to provide protection from flooding and to preserve, to the maximum extent, the natural scenic and recreational ambience of the area.

In order to determine the economic justification for the project and to effectively design the berm, it is necessary to evaluate the degree of protection it will provide during storms of various magnitude that may impact the site. From the designer's standpoint, it is desirable to know the return period of the storm that would be protected against for the selected design. From the economic standpoint, project justification requires that the benefit derived from the selected level of protection outweigh the cost.

3. GENERAL APPROACH

The primary goal of this study is to provide an estimate of the economic damages which might occur at the Panama City Beach area due to a tropical storm or hurricane. The methodology used to obtain these estimates consists of a many phased approach which relies on the application of several numerical models. Available data on the historic storms affecting the study area were collected. A subset of storms, called the "training set," was chosen which is representative of the full range of possible storm conditions. Numerical models which estimate the winds, water levels, waves, and beach profiles were run for each member of the training set to produce variables of interest, or "responses." A Gaussian "nearest-neighbor interpolation" relationship was developed which allowed the response for a storm to be determined from the training set storm characteristics without having to simulate the responses of all of the numerical models for every storm. One hundred statistical simulations of 50 years each were produced. For each year of a simulation the number of storms was set using a Poisson process. The responses for each of these storms was determined by the random "nearest-neighbor bootstrap" technique. The simulated sequence of storm responses were used to determine the return period levels of the storm response variables. The simulated sequences were also used in the economic model to determine the storm damages and comparisons

were made between conditions with and without the planned improvement.

4. STORM SELECTION

Tropical storm data used in this study was obtained from the National Hurricane Center (NHC) (Jarvinen, *et al.*, 1984) and CERC's Wave Information Study (WIS) data set (Abel, *et al.*, 1989). The latter contains much of the basic information of the former but has been augmented with additional details in order to hindcast the waves in selected storms. The NHC set covers the years 1886-1989. The significance of this data set is it's completeness, which makes it valuable for determining the frequency of different storm characteristics. The WIS data set covers the years 1956-1975. Hurricanes Juan and Kate, both of which made landfall near Panama City Beach in 1985, were added later. Only storms which exceeded the severity threshold of wind speed of at least 34 knots within 75 nautical miles of Panama City, Florida were selected.

The meteorological database included 34 storms which occurred over the past 105 years. The set of 34 historical storms was expanded by running each of the storms at four random tide phases, to obtain response variables for a total of 136 storms. Meteorologic data for 31 additional storms was also gathered. These 31 hypothetical storms did not actually occur as described but are based on the characteristics of the historical Gulf of Mexico storms. The hypothetical storms were chosen so that the entire range of storms which could impact the Panama City Beaches study area were represented. The total storm set contains 167 events (136 historical and 31 hypothetical storms).

A subset of the total storm database, referred to as the training set, was established to determine the relationship between storm parameters and environmental responses. The range of storm types found in the training set should reflect the likely range of storms to be encountered. Storms were chosen for the training set based on the ability of the set to cover the characteristics of a full range of storm types. This was done by working backward through hurricane records and eliminating storms that were near duplicates of each other. The training set included 24 storms from the set of 136 historical storms plus the 31 hypothetical storms.

Storm parameters were determined both at the time of closest approach and at the time of landfall. The set of storm parameters which was chosen to describe the storms statistically are: (1) the relative phase of high tide and storm surge maximum, (2) the relative phase of tidal change and storm surge maximum, (3) maximum wind speed in knots, (4) central pressure deficit in millibars, (5) radius to maximum wind in nautical miles, (6) forward speed in knots, (7) course angle of the storm track in degrees and (8) closest point of approach distance from the study area in nautical miles.

5. HYDRODYNAMIC MODELING OF STORM EVENTS

Several numerical models were used to establish the design criteria for the Panama City Beach area and to provide the data necessary for the economic evaluation of project feasibility. Most of these were developed by CERC engineers or by contractors under the direct technical supervision of CERC.

The hurricane wind-field was produced using the Standard Project Hurricane Model (SPH). This is a parametric model which represents a steady-state hypothetical hurricane defined by a set of interrelated parameters such as eye location, central pressure deficit, track angle, forward speed, radius to maximum winds, etc. (Cialone, 1991). The model produces wind components and surface atmospheric pressures. This model was run for each storm on a grid covering the Gulf of Mexico. Results of the model were used to determine water level and waves.

Total water level was obtained from ADCIRC

(ADvanced CIRCulation model) (Luettich, *et al.*, 1991; Westerink, *et al.*, 1991). This model is a time-stepping finite-element model containing the effects of tidal forcing, surface wind stress, atmospheric pressure variation, and bottom friction. A major advantage of the finite-element method is that the computational mesh can be refined to represent the details of local topography in the area of interest. Figure 2 shows the grid for the area of the entrance to St. Andrew's Bay. The ADCIRC model was tested against tidal data and against the measured storm surge produced by two hurricanes (Kate and Juan) impacting Panama City Beach (Miller, *et al.*, 1991). No tuning was done to the surge model, though some adjustments were required for the SPH to correct for landfall effects.

The waves were determined using the SHALlow water WaVe model, SHALWV, which is a time-dependent, spectral hurricane wave-field model. It has been previously tested against a large number of hurricane wave measurements and used to model sixty-five hurricanes along the Gulf and East coasts of the U.S. (Jensen, *et al.*, 1983). The wave spectrum at a point is represented by an array of discrete frequency and direction components. At each time step of the model, the wave energy is first propagated in the grid, taking into account refraction, diffraction and dispersion. The wave energy in each frequency-direction component at each grid point is then recalculated, taking into account energy input by winds, energy dissipation by breaking, and energy transfers within the spectrum through wave-wave interaction processes.

The results of the wave and surge stage frequency analysis were used as input for the numerical simulation model Storm-Induced Beach Change (SBEACH) (Larson and Kraus, 1989; Larson, *et al.*, 1990). SBEACH was used to evaluate the cross-shore transport of beach and dune materials exposed to varying waves and water levels under hurricane storm conditions. The model calculates sand transport produced by breaking waves, simulates erosion of dune and berm beach fills, and develops the evolution of the equilibrium beach profile including the offshore bar formation. Longshore transport was not modeled. Four representative beach profile lines along the 18 miles of beach were evaluated using the SBEACH model. The selection of the representative surveys were based on property structural value, beach profile shape, dune elevation and shape, longterm erosion rates, and grain size. Four responses (1) maximum linear feet of recession during the storm event measured from zero National Geodetic Vertical Datum (NGVD), (2) maximum water level above NGVD, (3) wave height at a depth of 49 feet, and (4) wave period were obtained from SBEACH to be used in the economic analysis.

6. DEVELOPMENT OF RETURN PERIOD FOR COASTAL STORM IMPACTS

The purpose of running a training set of storms was to determine a relationship between the storm parameters and the response variables, which are values for the storm determined using the numerical models for waves, water level, and beach erosion. Determination of this relationship removes the need for further runs of the numerical model by allowing determination of the water level, waves, erosion, etc. for a given site directly from the storm parameters.

The responses for the 112 storms which were not run through the numerical models are estimated from the responses of the training set storms with a Gaussian Nearest-Neighbor Interpolation. The Gaussian interpolation determines a weighted average of the responses of the training set of storms. For the purposes of determining the relationship, a measure of similarity between storms is required. This measure, called "distance" in this study, should possess the quality that the responses of storms converge as the distance between them decreases. The n storm parameters used can be considered the coordinates of an n-dimensional space in which storm i is represented by a vector, c_{ik}, where k = 1, n.

The difference, or distance, between vectors i and j , d_{ij} is the sum of the squares of the difference between each component of each vector

$$d_{ij} = \sum_k (c_{ik} - c_{jk})^2 \qquad (1)$$

This is the square of the usual Cartesian difference of two vectors.

For this study, different storm parameters are weighted differently for evaluating the distance between two storms. For instance, storms with similar distances and maximum wind speed will produce effects more alike than would storms similar only in radius to maximum wind and tide phase. Therefore, each parameter can be normalized by its root mean square (RMS) value, and assigned a scaling radius, R_k , and weight, W_k , based on the importance of the parameter. The RMS values are used as a scale factor around the mean value to normalize the range of the parameters due to differences in units. The distance then becomes

$$d_{ij} = \frac{\sum_k W_k \left[(c_{ik} - c_{jk})/R_k \right]^2}{\sum_k W_k} \qquad (2)$$

where the tilde indicates optional norming. For this study, all parameters were normalized, and all R_k were set to unity (Borgman 1991). All W_k were set to unity for most parameters. The W_k were set to ten for maximum wind speed at closest approach and distance of closest approach, set to five for relative phase of high tide and storm surge maximum, and set to three for relative phase of tidal change and storm surge maximum. The values were chosen so the parameters that better describe the similarities between storms are given higher weights. Thus, wind speed of the storm, its distance, and its tidal phase of arrival are considered most important in this study.

The response values, r_{ik} , for the k-th response of the i-th historic storm, not part of the training set, are then given a weighted average value of all training set storms

$$r_{ik} = \frac{\sum_{j_t} w_{ij} r_{jk}}{\sum_{j_t} w_{ij}} \qquad (3)$$

where the j_t are the j-s of training set storms only. The weights w_{ij} are set by a bell-shaped ("Gaussian") curve

$$w_{ij} = \exp\left[-\frac{\pi d_{ij}^2}{D_i^2} \right] \qquad (4)$$

where D_i is an effective width of the bell equal to twice the average distance of the four nearest training set storms to storm i .

If the mean frequency of storms is known, the Poisson distribution can be used to determine the number of storms in a given period. The Poisson distribution is given by

$$\Pr(s;\lambda) = \frac{\lambda^s e^{-\lambda}}{s!} \qquad (5)$$

where $\Pr(s;\lambda)$ is the probability of having s events in a period in which λ is the mean frequency of events per period. For this study, the interval is 1 year and $\lambda = 0.3238$ year^{-1} (34 historic storms / 105 years). As a result, the most likely event for a given year is no storms ($\Pr(0; 0.3238) = 0.7234$). The number of storms in a given year of a simulation is chosen using a random number generator combined with a Poisson probability distribution.

Once the number of storms in a year of a simulation has been found, it is necessary to determine the characteristics of each of those storms. This is done by a nearest-neighbor bootstrap method developed by Borgman (1990 and 1991). Bootstrap methods are based on resampling of observed data (Efron 1982). Multiple sequences of events can be developed

from a single sequence of observed data by drawing random samples (with replacement) from the observed data using the program HBOOT. The multiple sequences can then be used to estimate various statistics of the population of which the observed data is a sample.

To determine the responses of a simulated storm, one historic storm is selected at random from the total storm set. Since all historic storms have equal historic probability (each occurred once), storms selected at random will have the same probability density function (pdf). The responses of the new simulation storm are set to a weighted average of the selected storm and its four nearest neighbors, with the weights determined at random. This averaging procedure makes it possible for the simulation to have storms which have never occurred, but which are relatively similar to the storms which have occurred. It also allows an almost infinite number of hypothetical storms to be generated since storms with different responses will be simulated using the same reference storm.

The results from the statistical model are then used as input for the economic damages model. Data for the determination of the cost-benefit ratio for the proposed project were developed from 100 simulations of 50 years each of the model HBOOT, which is based on the nearest-neighbor bootstrap technique described above and elaborated upon in the references. The rationale for the use of multiple simulations is that tropical storms (including hurricanes) are relatively rare and have varied characteristics so no single 50-year simulation would be adequate to determine the risks, costs, and benefits associated with the project. By determining the cost-benefit ratios for multiple 50-year simulations using the bootstrap, one can determine the expected value of the 50-year cost-benefit ratio, as well as obtain a measure of the uncertainty of the calculations.

Since the economic model used must be given the year of occurrence of each event in a 50-year simulation to allow discounting of damages to a reference year, the 100, 50-year simulations give the number of storms each year and the magnitude of the response variables for each. For a given 50-year simulation, the economic damages can be found for each storm, discounted to the reference year, and summed to give a damage value for that simulation. The hurricane and storm damages were evaluated using a time series simulation combined with a composite damage reduction analysis. The composite damages method computes the damages of a storm to each structure for the three basic categories: inundation, wave attack, and storm recession.

Once the structure and content damages from inundation, storm-induced recession and wave attack were computed, the maximum, or critical, damage was chosen for each. Beginning in the first year of the project life, the model determines if any storms occurred, calculates the damages for each structure and each storm, increments the shoreline by one year of long term change, then begins the calculations for the second year. When the model completes the calculations for one 50 year period, it begins again for the next 50 year period. Other factors that are considered are long-term erosion, rebuilding constraints, back bay flooding, downdrift impacts, existing structural impediments to erosion such as seawalls and bulkheads, and individual actions to protect against erosion.

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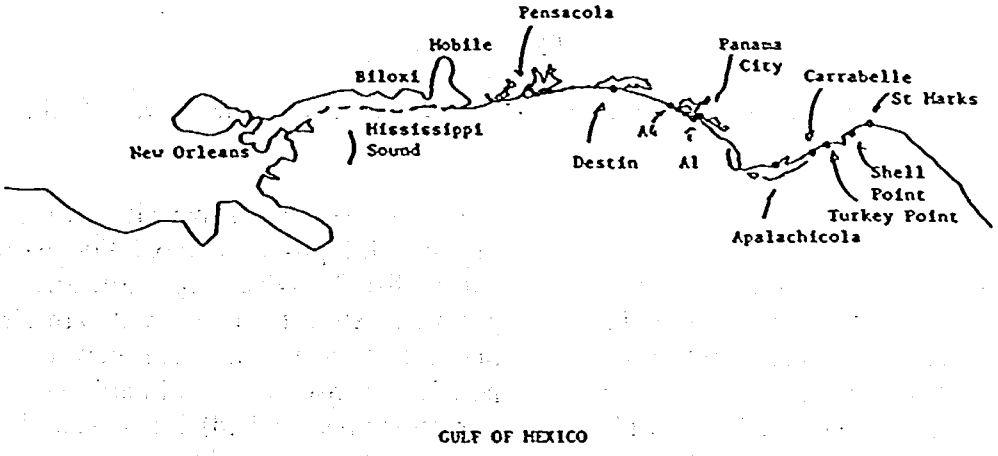


Figure 1. Northern Gulf of Mexico from Louisiana to Florida Bight. Study site is at Panama City.

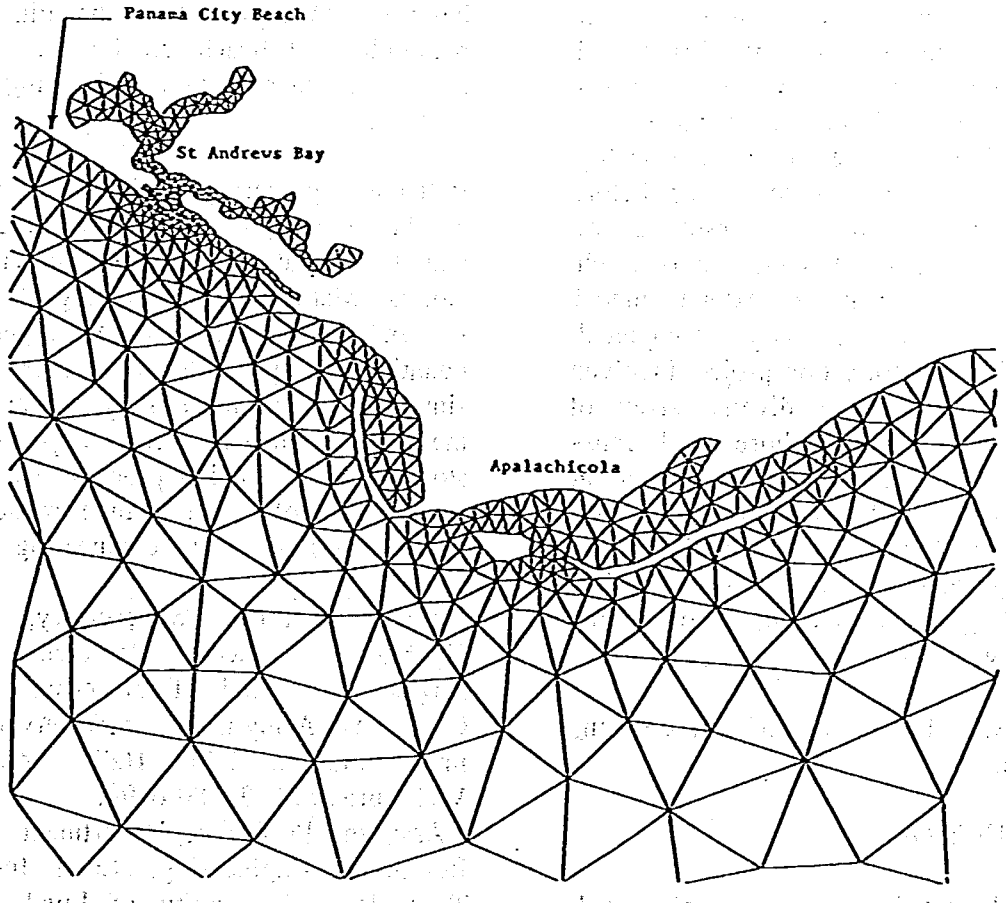


Figure 2. Details of finite element grid near Panama City, FL.

Beach Profile Response to Storm Surge and Waves

by

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ABSTRACT

The Corps of Engineers interest in storm-induced beach profile change stems from its mission in engineering shore protection projects. Beach fill is a popular method of shore protection, and a major design consideration is the degree of protection the fill provides. Mathematical modeling provides a quantitative tool to evaluate the response of natural beaches and beach fills to storms, and the SBEACH model was developed to fill this engineering need within the Corps. The paper summarizes development and application of the model, and ongoing research aimed at improving it. The SBEACH model simulates many of the important processes that characterize beach profile change during storms, and it has proven to be reasonably accurate as a quantitative predictor. However, applications have demonstrated the need for improving the model, and research efforts are under way to extend model capabilities. The recently completed SUPERTANK data collection project involved use of an extensive and diverse array of instruments to study cross-shore sand transport processes and beach profile change at prototype scale. Analyses of the data collected during SUPERDUCK will improve understanding of the complex storm erosion process and guide development of future model improvements.

KEYWORDS: beach; beach fill; erosion; model; storm.

1. INTRODUCTION

Beach nourishment is becoming the preferred method of shore protection because: (a) it is often the least expensive alternative, (b) use of sand fill provides flexibility, particularly when "hard" engineering structures such as groins, revetments, or seawalls are already in place, (c) some States are now establishing policies against the use of hard shore protection structures, and (d) it is a natural method of protection, providing an aesthetically pleasing environment and recreational benefits. However, recreational benefits can no longer be used to justify Federal participation in a beach nourishment project. Benefits derived from recreational use can be considered, but projects must be justified on the basis of storm-protection benefits, such as prevention of beach erosion and shoreline recession, and reduction in flooding and wave damage.

Reliable engineering methods are needed to aid in the design of protective beach berms and dunes that can withstand the impacts of storms. Mathematical modeling of beach profile and fill change provides a general and quantitative approach for representing the simultaneous influence of hydrodynamic, sediment and geometric factors important in the storm-induced erosion process. Kraus (1989) reviews use of beach change models in the coastal design and engineering process.

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Successful numerical simulations are limited only by the development of the underlying theory and availability of data to refine and validate the model. This paper reviews sand transport processes and beach profile change associated with storms, development and application of a mathematical model to simulate storm-induced beach profile change, and ongoing research aimed at improving both understanding of the governing processes and numerical model accuracy.

2. BEACH CHANGE DURING STORMS

Strong onshore-directed winds that occur during a storm result in the generation of waves that eventually break on the beach. Storm waves usually remove sand from the beach face and transport it both offshore and alongshore. Frequently, a linear bar is created in response to the tendency for offshore transport (linear in the alongshore direction). During the peak of the storm, when breaking wave heights are greatest, most beach erosion occurs and the linear bar is most pronounced. Figures 1a-1c show the evolution of a section of beach at the Coastal Engineering Research Center's (CERC) Field Research Facility (FRF) in Duck, NC, during the period 9-17 September 1985, in response to the passage of a northeaster storm (maximum significant wave height of 2.1 m) (from Howd and Birkemeier, 1987). Figure 1a shows the beach prior to passage of the storm, and Figure 1b shows the beach shape at the time of peak storm intensity. Note the presence and uniformity of the linear bar. During the storm sand can be transported in the alongshore direction at potentially high rates; however, beach profile evolution appears to be dominated by cross-shore processes. Figure 1c shows the same stretch of beach during the latter stages of the storm, as its intensity wanes. The beach begins to assume a very three-dimensional shape. However, the important point is that during the time of greatest beach erosion potential, beach change seems to be dominated by cross-shore processes and transport gradients.

Onshore winds and breaking waves associated with storms also increase the mean level of the water surface near the shore above what would normally be expected due to tidal action. Erosion caused by high-energy breaking waves is exacerbated by abnormally high water levels, which expose the previously subaerial portion of the beach berm and dune system to direct wave attack. As a result, considerable quantities of sand can be removed from the pre-storm berm and dune system and deposited offshore. If water levels remain well below the elevation of the dune crest, scarping of the dune can occur without significant overtopping of the dune. Under these conditions, if wave action persists long enough, severe erosion can occur and impact coastal, commercial, and residential structures (see Figure 2). In areas where no dunes exist, or when rising water levels approach or exceed the elevation of the dune crest, landward overwash of dune sand can occur in addition to offshore transport of foreshore sand. Because dune crest elevation varies considerably along the beach, overwash usually begins at low points in the dune system and overwash fans develop. As the water level rises to the predominant dune crest elevations, large-scale overwash can occur.

As storm wave energy subsides, wave conditions change from very steep wind waves to swell waves with low steepness (ratio of incident wave height to wave length). Swell waves tend to rebuild the beach, and the beach recovery process begins quickly following passage of a storm. Beach recovery is most noticeable in the form of an emergent foreshore berm. The newly formed berm is comprised of sand that was removed from the pre-storm foreshore, transported offshore under the action of storm waves, and then transported onshore under the action of swell

waves. Examples of this process are shown below for two New Jersey beaches.

The aforementioned cross-shore transport processes associated with storms occur over a short period of time (on the order of hours and days), but are of major engineering concern because of the potentially catastrophic nature of foreshore and dune erosion and inundation. Planning and design of shore-protection projects must consider potential damage produced by storms. Major factors that control cross-shore sediment transport and storm-induced beach profile change are: (a) beach profile shape prior to the storm, (b) grain size characteristics of the beach sand, (c) wave and water level climate during the storm, (d) presence of hard shore-protection structures, and (e) longshore non-uniformity in beach conditions.

3. STORM-INDUCED EROSION MODELING

All models of storm-induced beach profile change developed for engineering use have relied on the assumption that the beach profile evolves primarily in response to cross-shore processes. Also, these models have been based on some assumption about the shape of the profile as it evolves in response to varying wave conditions and water level. Early models were effectively time independent in predicting beach profile change from a pre-storm condition to an eroded condition following the storm (Edelman, 1972; Vallianos, 1974; Swart, 1976). Vellinga (1983, 1986) developed a simple mathematical model for estimating severe erosion of protective dunes impacted by storms that might be expected along the Dutch coast. Predictions from these models are independent of the temporal evolution of the storm driving forces (waves and water level); however, erosion is sensitive to the shape and duration of the storm hydrograph, particularly phasing between the storm-induced water level rise and the normal astronomical high tide. Wave properties also change during the storm, as mentioned previously. More reliable estimates of erosion will result if the time-dependent nature of the forcing functions is considered.

Realistic time-dependent simulation of beach and dune erosion was first accomplished by Kriebel (1982, 1986) and Kriebel and Dean (1985), who extended the static equilibrium beach profile concept of Bruun (1954) and Dean (1977). Kriebel and Dean (1985) demonstrated that their model produced reasonable results through comparisons with limited field data on dune erosion. Principal limitations of the model are: (a) weak dependence of predictions on incident wave characteristics (height and period); (b) somewhat unrealistic profile shape, in the sense of not being able to predict bar formation; and (c) limited capability to simulate accretion and berm buildup on the foreshore. Bars are a natural defense mechanism on beaches, and serve as a reservoir for sand that is transported offshore during the storm. It is important to reproduce this type of geomorphic feature.

Larson and Kraus (1989) and Larson et al. (1990) developed a mathematical model called "SBEACH," an acronym for Storm-Induced BEAch Profile CHange model, that overcomes limitations of previous models. A short description of the model follows.

The net direction of sand transport at any time during a storm is defined according to an empirical relationship between the deepwater wave steepness and dimensionless fall speed of the sand particles. These parameters best predicted transport direction observed in large-scale laboratory experiments and in the field, and frequently emerged in empirical correlations of dynamic and morphologic properties of the profile. Figure 3 shows the separation of erosional and accretionary events observed in large-scale wave tank experiments and 99 events in the field, along

with the empirical relationship used to differentiate between the two conditions.

The magnitude of the transport rate is calculated in four distinct zones across the beach profile, as depicted in Figure 4, analogous to zones of different wave properties identified in hydrodynamic studies of the surf zone (e.g., Svendsen et al., 1978; Basco, 1985). Empirical predictive relationships for the transport rate in each zone were developed using sequential measurements of profile change made in the laboratory during large-scale testing conducted by the U.S. Army Corps of Engineers (Saville, 1957; Kraus and Larson 1988) and the Central Research Institute of Electric Power Industry (CRIEPI) in Japan (Kajima et al., 1982). Sand transport rates in the broken wave zone are calculated based on the difference between the amount of local wave energy dissipation and the equilibrium energy dissipation level at which sand movement would cease (the equilibrium beach profile). In other words, the transport rate is proportional to the degree to which the beach profile is out of equilibrium. Available data support use of a linearly increasing transport rate in the swash zone, from zero at the limit of wave runup to the value calculated at the inner limit of the broken wave zone. In the breaker transition zone (between the breaker and plunge points), the rate increases exponentially from the value calculated at the outer limit of the broken wave zone, the plunge point. In the pre-breaking zone, the transport rate decreases exponentially from its maximum value at the break point. Empirical decay coefficients that depend on wave height and grain size were determined based on analysis of the large-scale laboratory data.

Expressions for predicting the position of the break point, the position of the plunge point relative to the break point, and the limit of wave runup are defined using formulations found in the technical literature, and are given in Larson and Kraus (1989). The numerical wave model of Dally et al. (1985) together with the cross-shore momentum equation are used to calculate the wave height needed to define the break point, energy dissipation, set up, and other wave-related quantities at discretized points along the profile.

An explicit finite-difference solution scheme is used to solve the mass conservation equation for sand using the calculated transport rates. The seaward limit of the model domain is set at the location where the calculated transport rate in the pre-breaking zone is negligible during the course of a storm. The shoreward limit is defined to be the limit of wave runup for the case of a sandy beach, or the transport rate is defined to be zero if a seawall or revetment is present. Avalanching is included to limit the steepness of the beach slope at the landward boundary. Typical time and space steps used in model simulations of storms are 5-20 min and 1-5 m, respectively. The water level and offshore wave conditions are input at each time step, and wave height across the shore and profile change are then calculated.

4. MODEL APPLICATION

The empirically based SBEACH model should be calibrated prior to its use in investigating beach fill response to storms, because main model parameters were established for a limited range of wave heights and periods, and grain sizes. Calibration requires high-quality data, including immediately pre- and post-storm beach profile data, grain size data, and the time history of wave and water level conditions during the storm.

Here, an example is given of model calibration using a data set from the Point Pleasant and Manasquan Beaches located on the south and north sides, respectively, of the Manasquan River jetties on the coast of New Jersey. Point Pleasant Beach is a relatively wide sandy beach with no active coastal structures except a 678-m long jetty to the north;

Manasquan Beach is a relatively narrow sandy beach with short groins spaced at approximately 200- to 300-m intervals. Details on the site are given in Larson et al. (1990).

The beach profile was surveyed to wading depth along eight transects spaced at approximate 200-300-m intervals at Point Pleasant and along nine transects midway between groins at Manasquan on March 27-28, 1984. A strong extratropical storm (northeaster) arrived on March 29, halting fathometer surveys that would extend the profile measurements to deeper water. The storm caused significant erosion; and, after the storm passed, profile surveys were repeated along the transects. The post-storm surveys taken on April 2-3, 1984, include 3-4 days of post-storm swell waves with low steepness. Water level changes were recorded hourly by a tide gage located inside the inlet, and wave height and period were calculated from measurements taken by a wave buoy located directly offshore from the inlet in 15 m of water. Water level and wave height and period between the times of profile surveys are shown in Figures 5 and 6.

A mean initial pre-storm profile was developed using an earlier measured subaqueous profile joined to the measured subaerial profile from 27-28 March. The mean subaqueous profile along the two beaches differed substantially, with the water depth along Point Pleasant profiles increasing monotonically with distance offshore, whereas the Manasquan Beach profiles had a subaqueous terrace located near the ends of the groins. A representative grain size of 0.5 mm was estimated from previously collected data. These beach and storm data were used as model input, and beach profile change for the storm was simulated. Calibration was determined based on a least-squares minimization between measured and simulated beach profiles.

Typical results showing measured and simulated profile changes at Point Pleasant and Manasquan are shown in Figures 7 and 8. Because of the small heights and long periods of the waves during the latter stages of the storm (after March 30), partial beach recovery undoubtedly occurred prior to the post-storm survey. This is evidenced by the presence of the foreshore berm in each figure. At elevations above the berm, predicted volumetric erosion is reasonable; and neglecting sand volume in the emergent post-storm berm, total foreshore erosion volume seems to be predicted very well. Bar position is also well-predicted at Point Pleasant but less so at Manasquan, perhaps because of the longshore influence of groins at Manasquan. Predictions of beach recovery following the storm are qualitatively correct, but quantitatively inadequate. Additional discussion of model results for Manasquan and Point Pleasant are discussed in Larson and Kraus (1991).

5. ONGOING RESEARCH

Results from this and many other applications of the SBEACH model have identified areas where further research is needed to improve the model. Better understanding of the relationship between the driving hydrodynamic forces and the entrainment and transport of sediment is needed to reduce the empiricism of the model. Another area of research involves determining the optimal way to include the effects of random wave breaking. The large-scale laboratory tests that produced data used to develop the model only involved generation of monochromatic waves. Monochromatic waves are a simplified representation of waves that impact most beaches. Other areas of much-needed research include study of the dynamics of wave impacts on berms and dunes, dune overwash, swash processes, and the post-storm beach recovery process.

Recently, a major laboratory project called "SUPERTANK" supported by several work

units in the U.S. Army Corps of Engineers Coastal and Dredging Research Programs was completed (Kraus et al., 1992). SUPERTANK focussed on cross-shore hydrodynamic and sand transport processes, and was designed to address the critical research needs outlined above. The 8-week project was conducted in the large wave tank facility of Oregon State University in Corvallis, OR (Figure 9). SUPERTANK drew participation of researchers from 20 Federal agencies, universities, and private companies in the United States, Denmark, Japan, The Netherlands, and Sweden.

SUPERTANK is believed to be the most densely and comprehensively instrumented nearshore processes data collection project ever conducted in the laboratory or the field. At the peak of data collection activities, the wave tank was instrumented with 16 resistance wave gages, ten capacitance wave gages, 18 two-component electromagnetic current meters, 34 optical backscatterence sensors (OBS), ten pore-pressure gages, three acoustic sediment concentration profilers, one acoustic-Doppler current profiler, one four-ring acoustic benthic stress gage, one laser Doppler velocimeter, five video cameras, and two underwater video cameras. Figures 10a and 10b show components of the instrument array (wave gages, current meters, and OBS sensors). These instruments provide detailed measurements of the nearshore hydrodynamics (waves and currents) and sediment concentration needed to quantify sand transport. Numerous fluorescent sand tracer studies were also conducted for the same purpose. During the experiments approximately 350 beach profile surveys were made to record the response of the beach profile to wave action. Surveys were made with an auto-tracking infra-red survey system call a geodimeter, which targeted a prism attached to a survey rod mounted on a moveable carriage. The survey system allowed accurate detailed surveys to be made in minimal time. Sixty-six different wave conditions were run for a total of 129 hours of wave action; 70% of the wave conditions involved random waves.

Test series focussed on the following: (a) beach erosion and accretion due random and monochromatic waves, (b) erosion in front of a seawall, (c) response of offshore subaqueous berms, and (d) dune and berm flooding and erosion. Figures 11 a and b show the condition of a dune before and during tests to investigate the dynamics of dune overtopping and overwash. Data return from the instrument array was remarkably high (99%). Preliminary results support many of the design procedures being used to investigate beach profile and fill response.

6. CONCLUSIONS

Cross-shore sand transport processes during storms are complex, particularly foreshore sand transport and the dynamics of dune erosion and overwash. The SBEACH model has been developed to quantify storm-induced beach profile change, and it successfully simulates many of the major morphologic responses observed during storms, such as foreshore erosion, dune and berm scarping, bar development, and post-storm recovery (with less accuracy). Because of the limited range of wave and beach conditions used to develop the model, calibration of SBEACH is recommended prior to application for project design. However, because of the rapid response of the beach to storm events and short time available to mobilize for data collection, it is usually difficult to obtain accurate data sets that are suitable for calibration purposes. Data from Duck, NC, and Point Pleasant and Manasquan, NJ, illustrate how quickly the beach responds. There are several areas in which the SBEACH model can be improved. For example, better descriptions of the effects of random waves on beach erosion and accretion and their relationship to sand transport are needed, as are improved descriptors of the dynamics of dune and berm inundation,

and subsequent erosion and overwash. Existing technology (laboratory facilities, computers, and instrumentation) allow complex and comprehensive investigations to be undertaken that will improve our understanding of the physics of sand transport and beach profile change. The SUPERTANK data collection project will provide valuable information for improving the SBEACH model and our understanding of storm-induced erosion.

7. ACKNOWLEDGEMENTS

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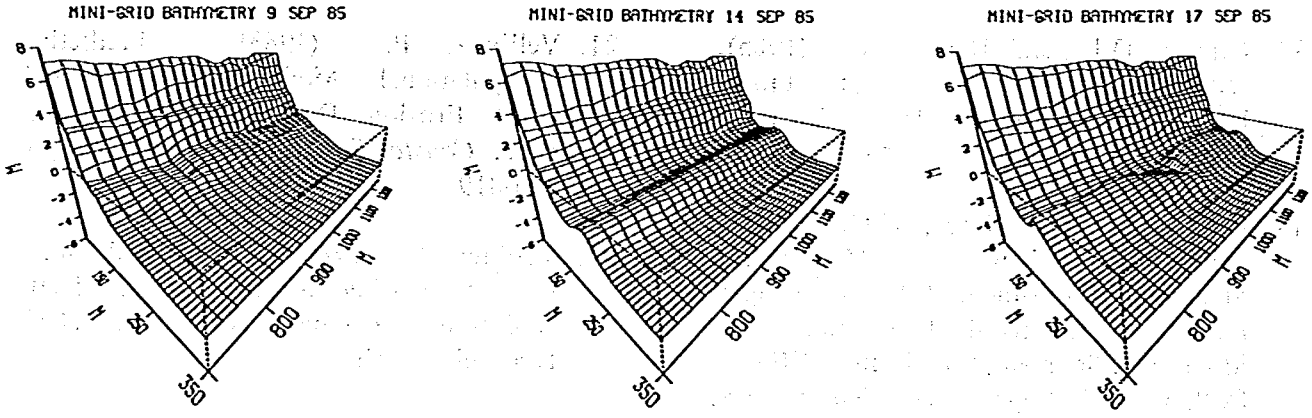
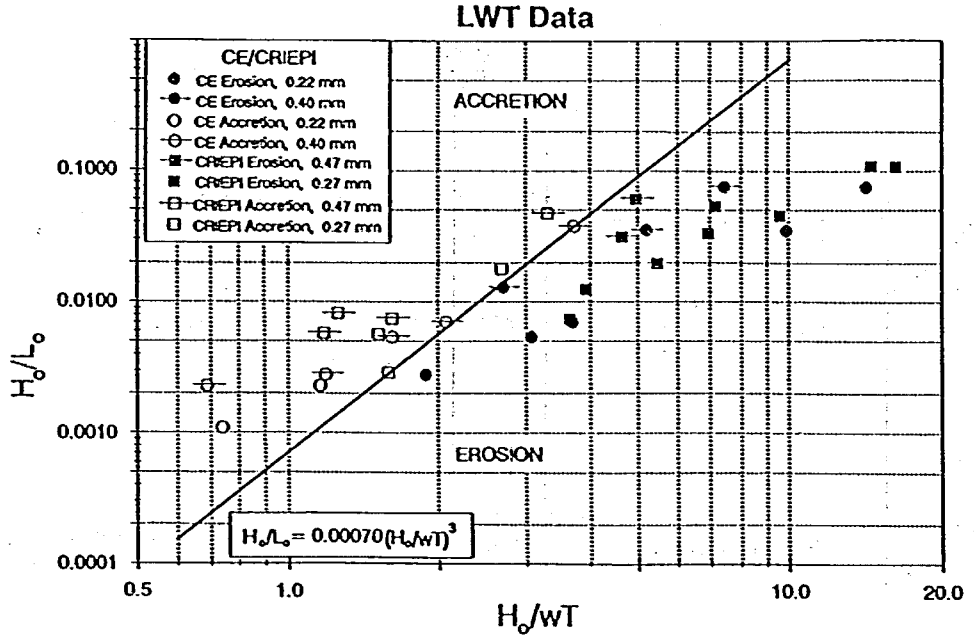


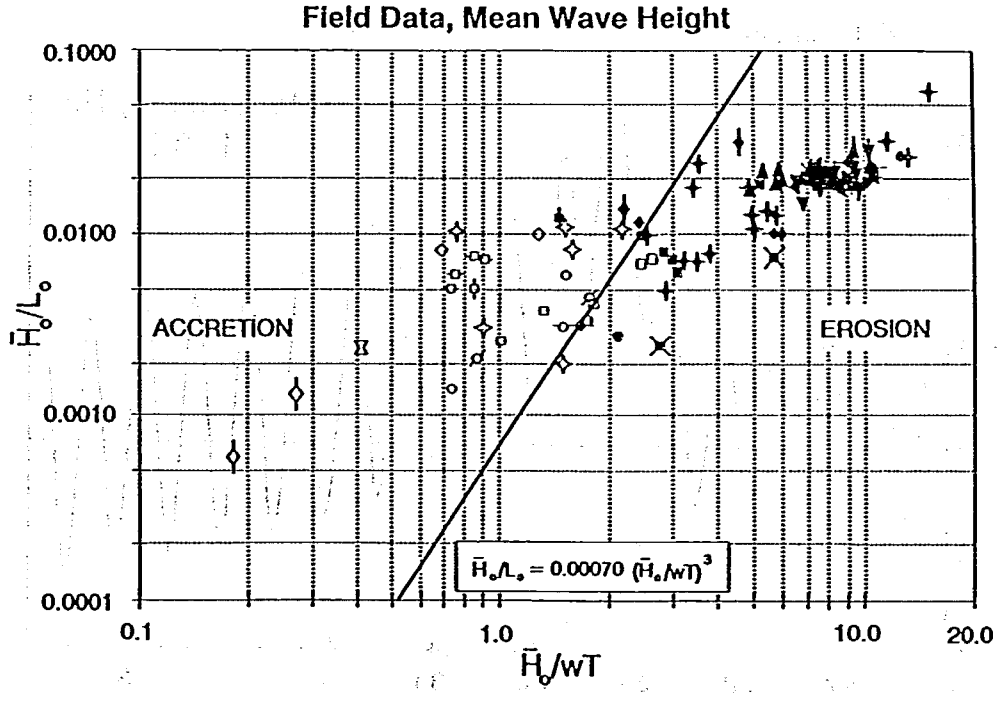
Figure 1. Beach change measured at the CERC FRF during September 1985 storm



Figure 2. Example of severe storm-induced erosion and structural damage



(a) monochromatic laboratory waves



(b) field conditions

Figure 3. Criterion for predicting beach erosion and accretion

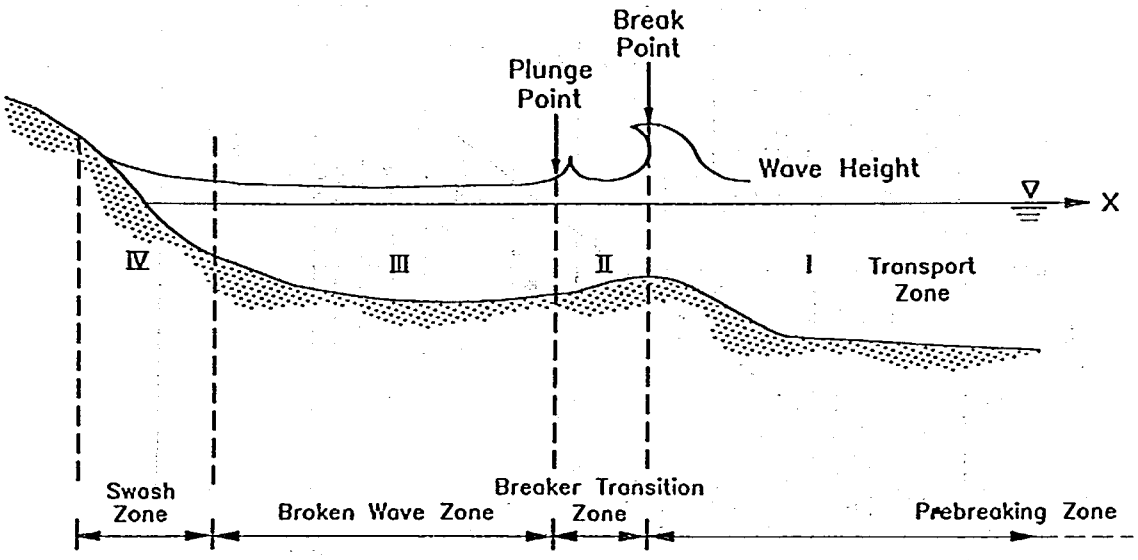


Figure 4. Regions of cross-shore sand transport

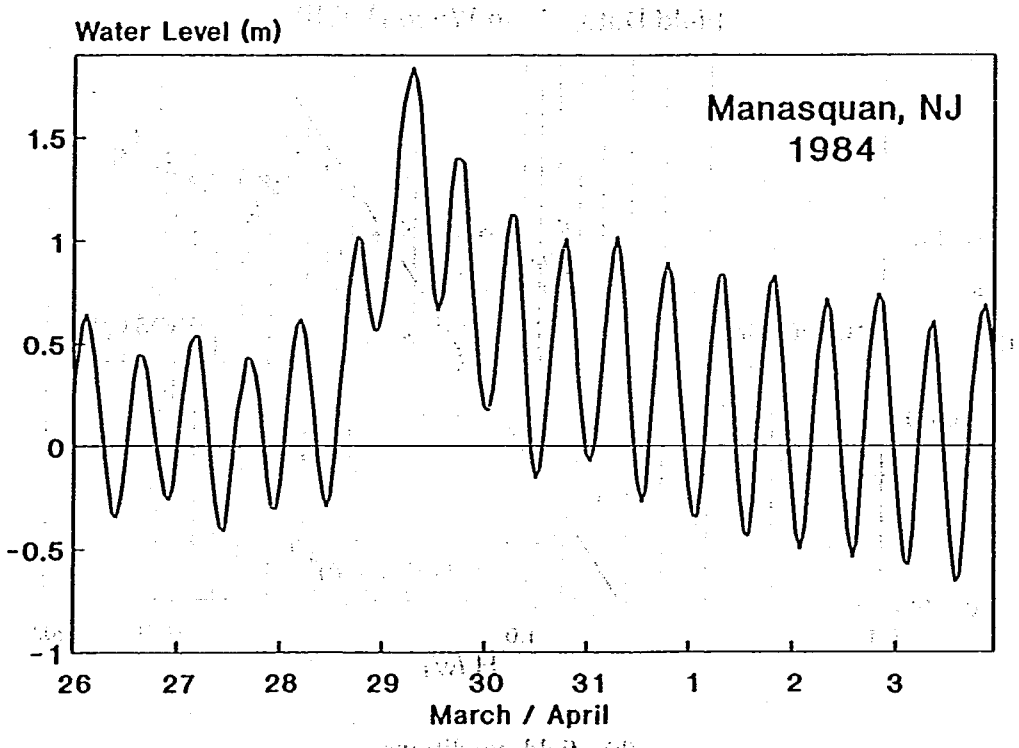


Figure 5. Water level measured at Manasquan Inlet during the March 1984 storm

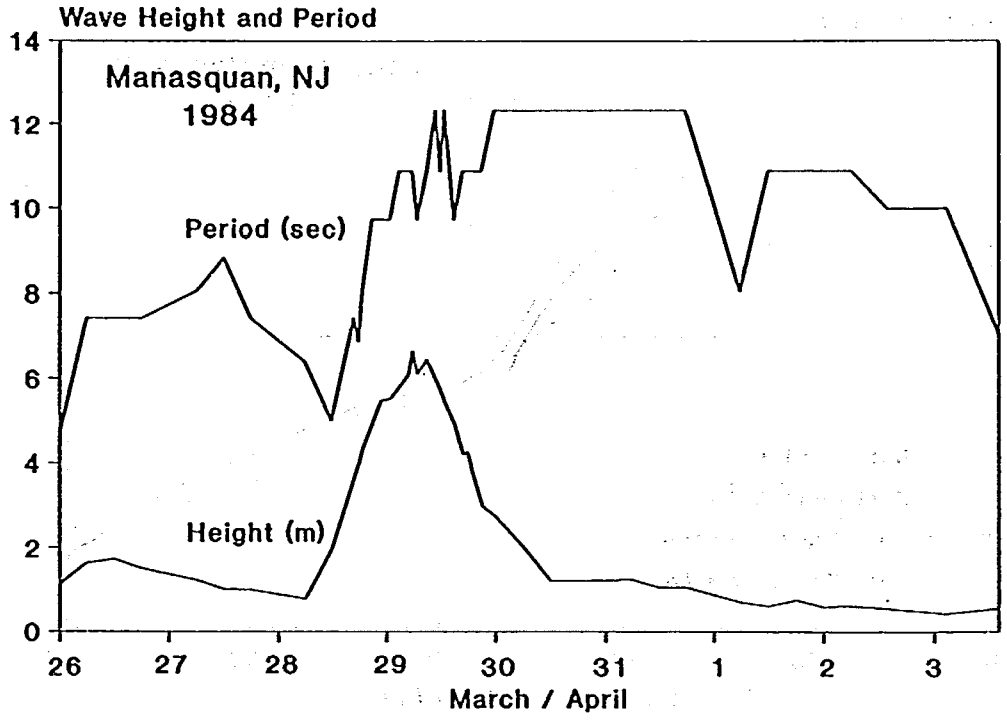


Figure 6. Wave height and period measured at Manasquan Inlet during the March 1984 storm

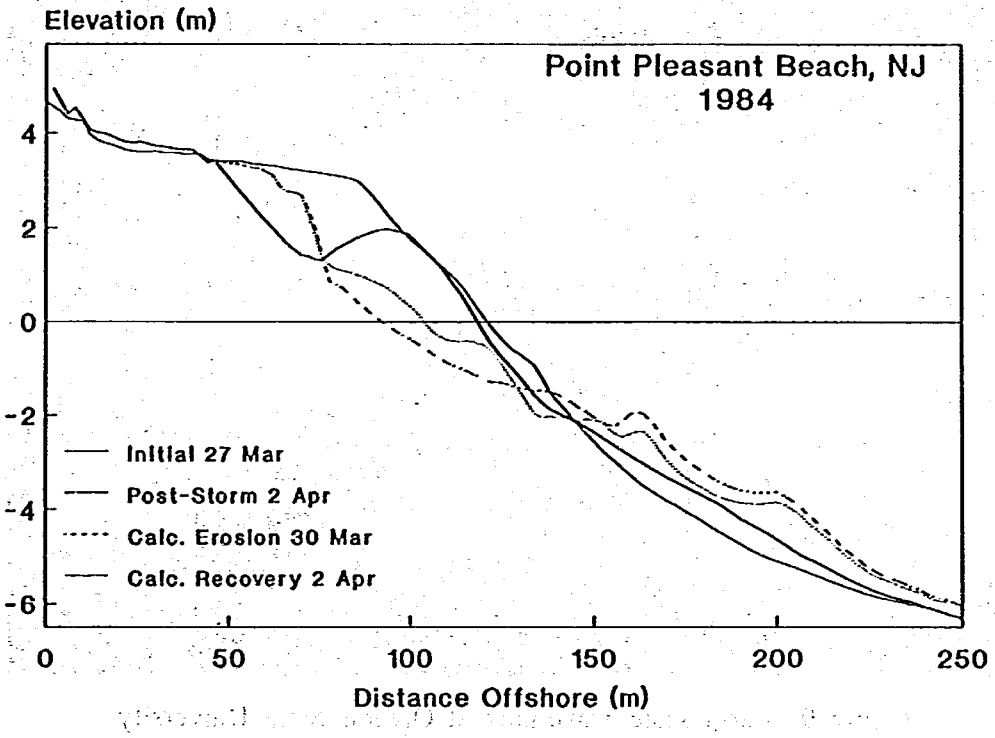


Figure 7. Storm-induced profile change at Point Pleasant Beach, New Jersey

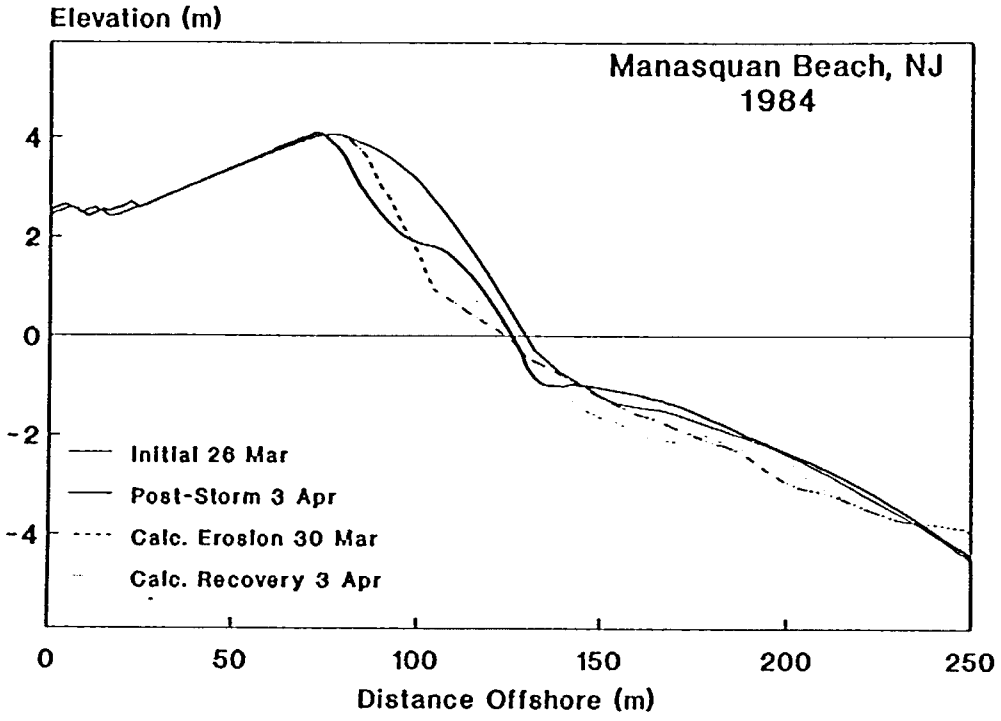


Figure 8. Storm-induced profile change at Manasquan Beach, New Jersey

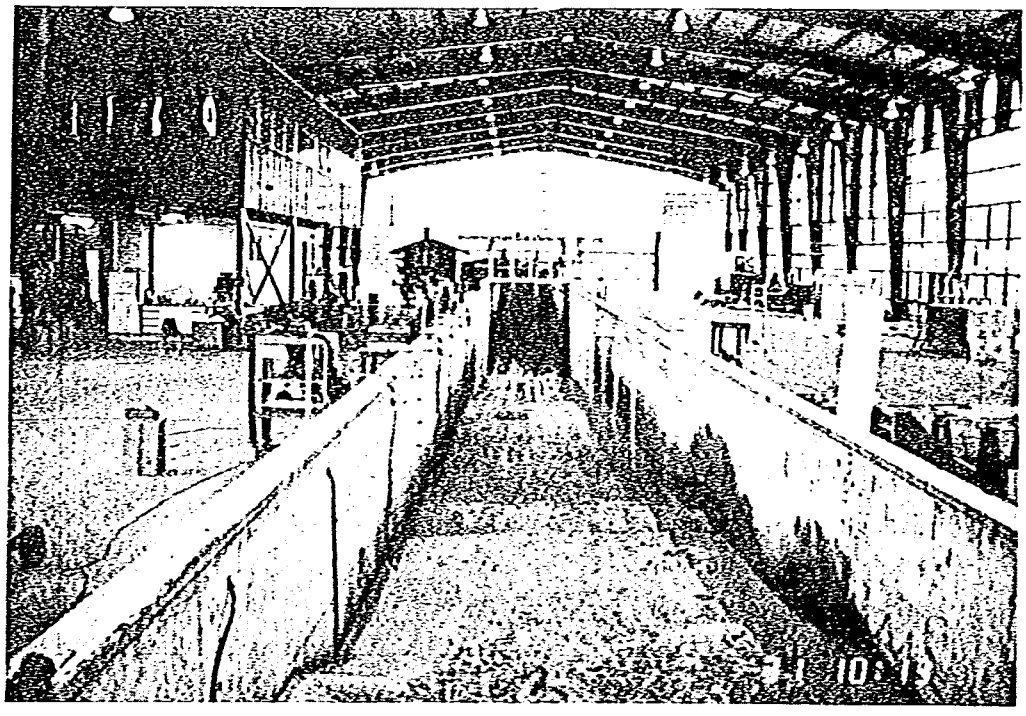
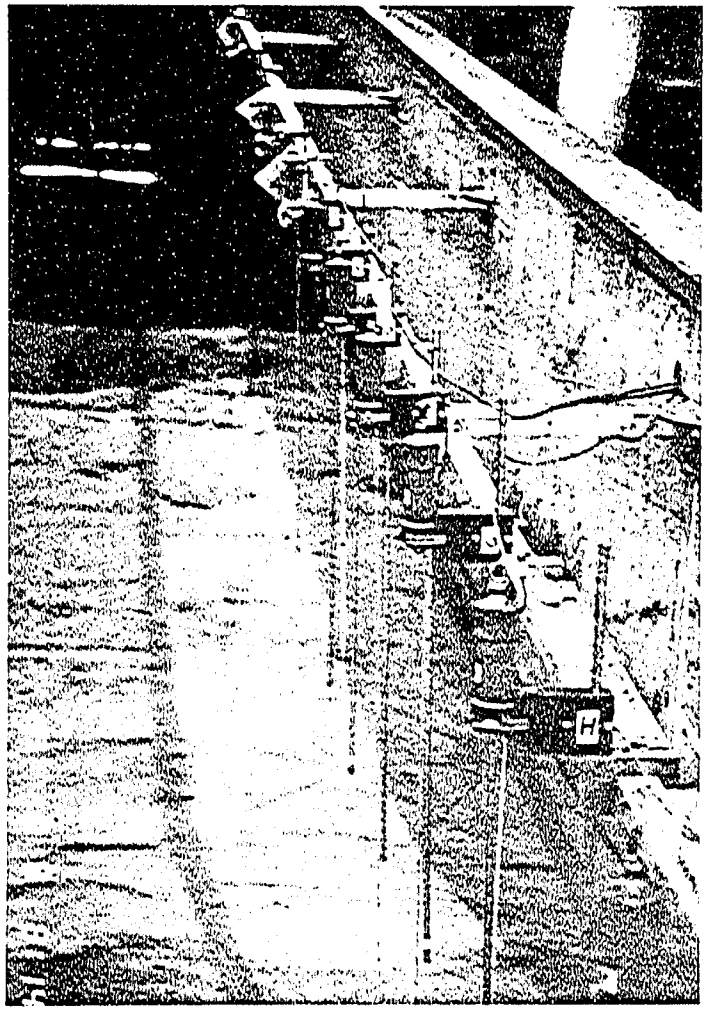
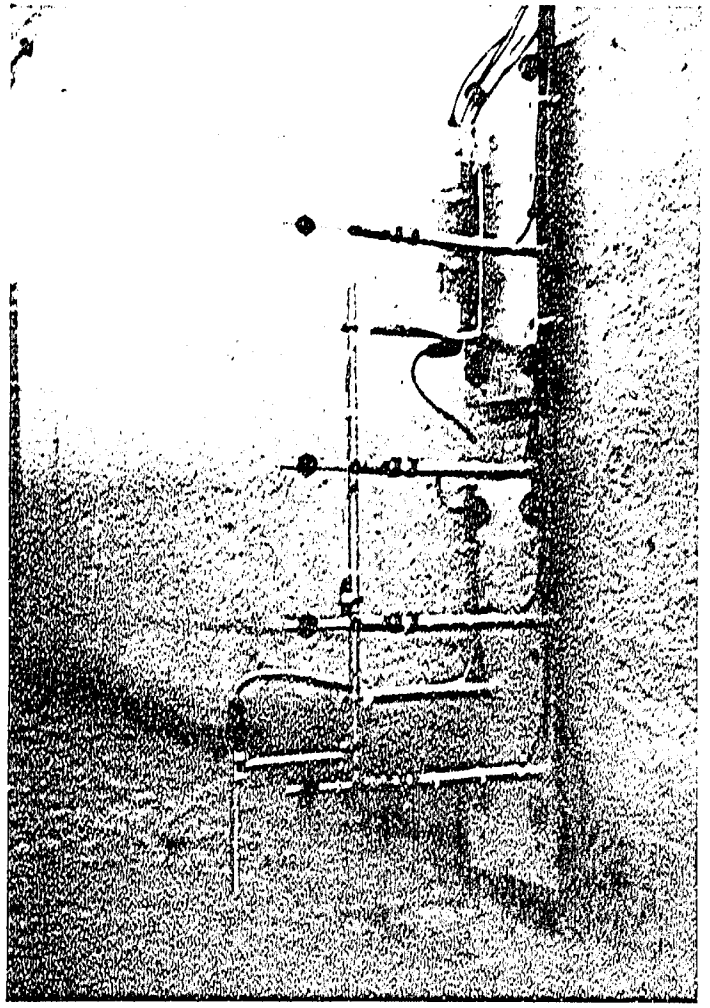


Figure 9. Large-scale wave tank at Oregon State University

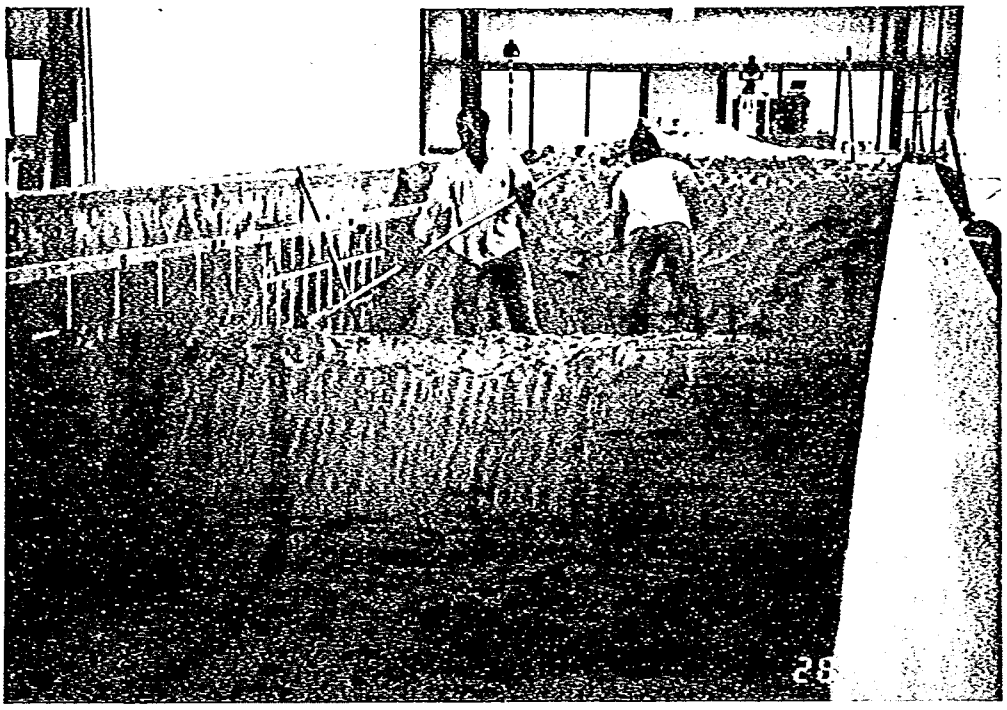


(a) capacitance-type wave gages

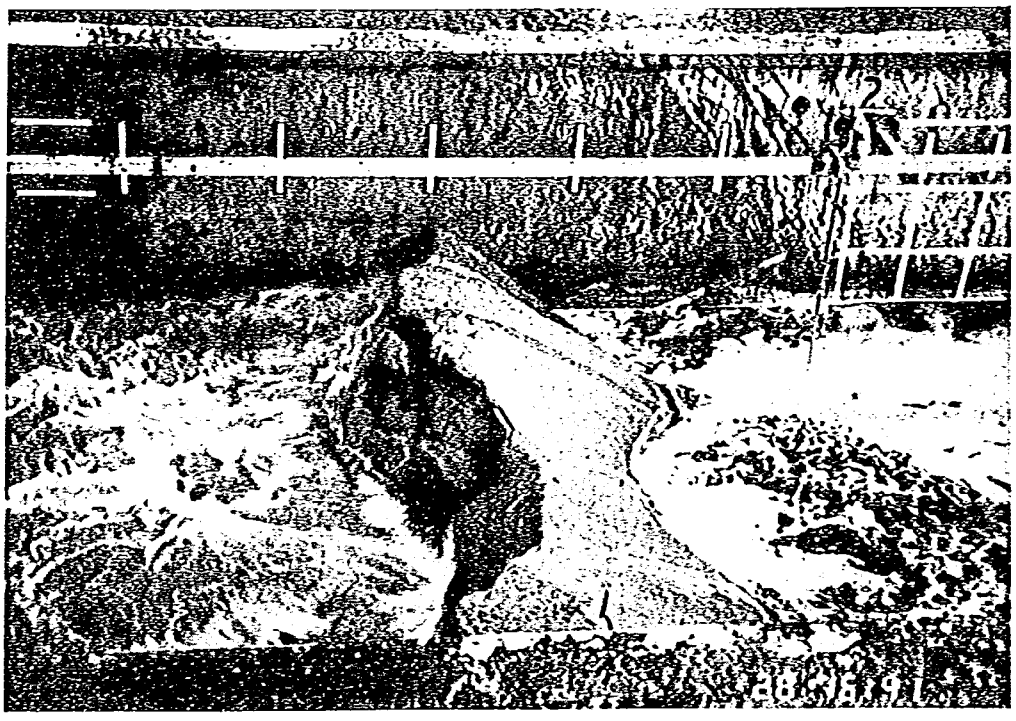


(b) electro-magnetic current meters and OBS

Figure 10. Instrumentation deployed in the large wave tank



(a) initial dune shape



(b) eroded dune

Figure 11. Photographs of dune erosion tests

課題Ⅲ　高潮・津波　質疑

（３６）A Numeral Simulation of the Tsunami Generated by Landslide of Mt. Mayuyama in 1792
Goto, C. and Takayama, T. (Noda S.*)

Wright　（Q）　一般的に言って歴史的（実測）データと数値計算による結果とは良い一致を示すものか？

野　田　（A）地震による津波については実測があり数値計算は良い一致を示す。しかしながら、地すべりによる津波については実測があまりなく、なんとも言えない。

大　谷（A）　日本には神社や寺に過去の地震や津波の記録、例えば、いつ、死者が何人か？　どれくらい木が倒れたかという詳細な記録がある。この記録から津波の到達距離を想定して数値計算の結果と合うかどうかを見ている。例えば、今、雲仙の普賢岳が問題になっているが、これについて古文書を捜し、検討を行っている。

Cooper（Q）　将来の予測ということに関してどのような仮定をおいているのか。例えば、最悪のケースの予測をたてているのか？

野　田（A）　将来のケースについては、最悪のケースのも想定するし、現実に実現の可能性の高いケースについても想定する。これらのケースを含め、パラメトリックに想定する。

Abrol（Q）　1792年以来大きな津波が現地で発生しているか？

野　田（A）　発生していない。

Brady（Q）　個人の生命や個人の財産が地震や津波により被害を受けた時には日本の中央政府はこれらに対して責任があるのか？

住　吉（A）　地すべりやガケ崩れについては防御策を設けたりしてその対策を実施している。台風や津波の被害については範囲が広範であり、全てに対しては対処できないのが現実である。

（３７）Empirical Simulation of Storm Histories for Coastral Design
Miller, Martin* and Reinhard, Robin D.

大　谷（Q）　ハリケーンの時には海岸線や海底の境界条件が変わってくると思うが、このような点を考慮しているか？

Miller（A）　海底面の粗度は一定とし、風によるせん断力を考慮して計算しているが、復間の事項については考していない。複雑な海岸線に対しては有限要素法のグリッドを細かくして対応している。

野　　田（C）　ハリケーンや津波があると海底地盤の変形があるが、ＳＢＥＡＣＨというモデルではこれを再現できるのか？

Cooper（Q）　将来の予測ということに関してどのような仮定をおいているのか。例えば、最悪のケースの予測をたてているのか？

Miller（A）　数値計算は現実に起った状況と起こり得る状況についておこなっている。Return　Periodは統計的な処理により求めている。従って最悪というケースは想定していない。

課題IV.　日米共同研究

日米共同大型実験研究　　日本側PRESSS レポート

日本における建築物の解析手法

日本設計株式会社
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複合構造研究官
中田　慎介

要約

日本においては、鉄筋コンクリート高層建物の動的解析には漸せんけい時刻歴応答課遺跡が要求される。動的応答解析には２つのモデルがあり、１つは質点系モデルであり、他の１つはフレームモデルである。質点系モデルは各階の重量を質点として分布させ、各界の曲げせん断バネは静的断塑性解析から求め、適切な減衰定数を用いる。静的断塑性解析では、水平力分布が仮定され、漸増解析が用いられる。フレームを構成する各部財は材単に剛塑性バネを有するモデルとされる。この復元力特性は曲げひび割れ、と降伏を考慮して決定される。通常は質点系応答解析の場合、このスケルトンカーブはトリリニアーであり、履歴特性として原点志向または連続デグレーティングシステムとする。（武田モデル）静的断塑性解析の場合も同様のモデル化が行なわれ、この質点系モデルの復元力特性はフレームモデルの動的応答解析にも使われる。

1序

日本における振動解析の手法としては、モーダルアナリシスを用いず、数値積分による時刻歴応答解析が行なわれている。この理由はレベル２と定義された大地震を想定して架構の弾塑性特性を検討することを対象としているためである。採用される地震動波形の種類としては、標準的な波形としてEL CENTRO 1940 NS およびTAFT 1952 EW、地域特性を表す波形としてTOKYO-101 1952 NS, OSAKA-205 1963 EW, HACHINOHE 1968 NS 等、長周期成分を含む波形としてHACHINOHE 1968 NS, EW等としている。原則として、これ等を含めて合計３種類以　上が採用されている。

地震動の強さはレベル１およびレベル２とし、地域における地震の活動等を考慮して決定されている。例えば、東京では、レベル１の地震動に対して最大速度25cm/sec以上（換算加速　度250cm/sec^2）、レベル２の地震動に対して最　大速度50cm/sec 以上（換算加速度500cm/sec^2）としている。レベル１に対しては、弾性応答解析が行なわれ、モード合成法が用いられている。レベル２に対しては、弾塑性応答解析が行なわれ、時刻歴応答解析が用いられている。しかし、建物の弾塑性挙動は不明の点が多く、モデル化や減衰に関して、色々な考え方がある。尚、鉄筋コンクリート造では、レベル１に対しても、ひび割れによる部材の剛性低下を考慮した弾塑性応答解析が用いられている。

2．質点系弾性地震応答解析

2．1　基本振動系モデル

（1）質点数

１層１質点とする。

（2）振動系モデル

・曲げせん断型モデル（図２．１）

弾性解析に用いたモデルの固有値、固有ベクトルを入力する。

・ロッキングースウエイモデル（図２．２）

地盤、杭を等価なロッキングバネおよびスウエイバネル置換し、上部構造は等価せん断バネとする。

（3）減衰

種　　類　　：　内部粘性型

$$[c] = \frac{2h_1}{\omega_1}[K]$$

減衰マトリックス　：　剛性比例型

減　衰　定　数　　：　１次０．０５（鉄筋コンクリート）高次の減衰定数は振動数に比例させる。

（4）数値積分

モード合成型を用いる。（通常５次まで）

2．2　復元力特性

（1）スケルトンカーブの形

Linear とする

（2）履歴法則

Linear とする。

3. 質点系弾塑性地震応答解析（1）

3．1 基本振動系モデル

（1）質点数

1層1質点とする．

（2）振動系モデル

・等価せん断型モデル（図3．1）

荷重増分解析による各層の層せん断力を層間変位で除した弾性を等価せん断剛性として、せん断型モデルに置換する。

・ロキングースウユイモデル

図2．2と同じモデルで、履歴特性が図3．1で示されるモデル．

（3）減衰

種　類　：　内部粘性型

$$[c] = \frac{2h_1}{\omega_1}[K]$$

減衰マトリックス　：　瞬間剛性比例型（各時刻における剛性マトリックス[K]）

減衰定数　：　1次0．05（鉄筋コンクリート）高次の減衰定数は振動数に比例させる。

（4）数値計算

線形加速度法を用いる。

3．2 復元特性

（1）スケルトンカーブ

Tri-linear とする。

（2）スケルトンカーブの設定方法

荷重増分解析より得られた各層の荷重ー変形曲線を折線置換する（図3.2）．

（3）塑性率の定め方

層の塑性率の基点は、スケルトンカーブの第1折点とする。

（4）履歴特性

Normal tri-linear とする。

3．3 荷重増分法による静的弾塑性解析

（1）外力分布

Ai 分布を上層ほど大きくなるように修正したものを外力分布とする．または、予備応答解析で得られたものを外力分布とする。

（2）解析モデル

平面骨組モデルで、材端は固定とし、曲げ変形、せん断変形、軸方向変形柱のみ）を考慮する。

（3）部材の復元力特性

部材は全て、完全弾塑性と仮定する。

（4）解析方法

荷重増分解析は次のように行なう。

(a) 地震荷重時の応力解析を行なう。

このときの層間変位を ${}_1\delta$ とする。

(b) 鉛直荷重時応力と地震荷重時応力から、全ての部材端の全塑性耐力係数βを求める。

(c) 全ての部材端の全塑性耐力係数βの最小値を ${}_1\beta$ とし、荷重変形折線の第1折点を

$$({}_1\beta \times Q_D,\ \beta \times {}_1\delta)$$ とする。

(d) 第1ステップのヒンジは、全ての部材の全塑性耐力係数βが $({}_1\beta + 0.1)$ 以下である点に発生すると仮定する。

(e) 全塑性耐力に達した部材端をヒンジとした架構の応力解析を外力 Q_D に対して行なう．このときの層間変位を ${}_2\delta$ とする。

(f) 第1折れ点に対応した応力を初期値として、増分応力による全ての部材端の全塑性耐力係数βを求める。

(g) 第2ステップのヒンジ発生点はヒンジの点のβの平均値が目標になるような範囲とする．この時のヒンジ発生点のβの平均値を ${}_2\beta$ とする。

(h) 第2折点は $({}_1\beta \times Q_D + {}_2\beta \times Q_D,\ {}_1\beta \times {}_1\sigma + {}_2\beta \times {}_2\delta)$ とする。

(e) から (h) を図3．4に示す．以下このステップを繰り返し第 折れ点は $(\Sigma {}_1\beta \times Q_D,\ \Sigma({}_1\beta \times {}_1\delta))$ とする。

（5）結果

各部材端のヒンジ発生とその時の荷重ステップが求まる各層の荷重ー変形曲線が求まる．保有耐力（最大地震荷重）が分かる。

4. 質点系弾塑性地震応答解析（2）

4．1　基本振動系モデル

（1）質点数

1層1質点とする．

（2）振動系モデル

・等価曲げせん断型モデル（図4．1）

荷重増分解析の結果より、柱の軸変形による全体曲げ変形成分を求め、層間変位から差し引いた残りの変位をせん断変形成分として、各層ごとに等価な曲げせん断棒に置換する．曲げ変形は弾性とする．

・ロッキングーウエイモデル

図2．2と同じモデルで、履歴特性が図4．1で示されるモデル．

（3）減衰

種　　類　：　内部粘性型

$$[c] = \frac{2h_1}{\omega_1}[K]$$

減衰マトリックス　：　瞬間剛性比例型（各時刻における剛性マトリックス[K']）

減 衰 定 数　：　1次0．03（鉄筋コンクリート）

高次の減衰定数は振動数に比例させる．

（4）数値積分

線形加速度法を用いる．

4．2　復元力特性

（1）スケルトンカーブ

Tri-linear（曲げ剛性は弾性とする．）

（2）スケルトンカーブの設定方法

荷重増分解析より得られた層間変位から曲げ変形成分を差し引いた残りの変形成分を層せん断変形として、層せん断力ー層せん断変形曲線を折線置換する（図4．2）．

（3）塑性率の定め方

層の塑性率の基点は、スケルトンカーブの第2折点とする．

（4）履歴特性

剛性逓減型Tri-linearとする．除荷時剛性低下指数γは0.40とする．

4．3　荷重増分法による静的弾塑性解析

（1）外力分布

A_i分布を修正したものを外力分布とする．または、予備応答解析で得られたものを外力分布とする．

（2）解析モデル

平面骨組みモデルで、柱・梁の部材モデルは、材端剛塑性回転バネモデルとし、曲げ変形せん断変形、軸方向変形（柱のみ）を考慮する．

（3）部材の復元力特性

柱・梁　　：　曲げひび割れ・曲げ降伏を考慮したTri-linearモデル（図4．3）

接合部バネル　：　せん断ひびれを考慮したBi-linearモデル（図4．4）

柱部材（軸力）　　：

引張側　Tri-linearモデル

圧縮側　Bi-linearモデル

（4）解析方法

荷重増分解析は、次の手順で行なわれる．

(a) 準備計算

(b) 材端変位～節点変位変換マトリックスの作成

(c) 部材の瞬間剛性マトリックスの作成

(d) 層の瞬間剛性マトリックスの作成

(e) バネルを1部又は全部無視する時は、層の瞬間剛性マトリックスを修正する．

(f) 骨組全体の瞬間剛性マトリックスの作成

(g) 骨組全体の瞬間水平剛性マトリックス（基礎固定の時の上部構造の瞬間水平剛性マトリックス）の作成

(h) 基礎のロッキング、スウエイを考慮した骨組全体の瞬間水平剛性マトリックスの作成

(i) 固有値計算・・・・・動的解析の最初の1回のみ

(j) 水平力増分を求める．

(k) 応答計算・・・・・水平変位の増分を求める．

(l) 材端変位および材端力の増分を求める．

(m) 材端変位および材端力の前回の値に今回の増分を加えて、今回の値とする．

(n) 部材の復元力特性に基づいて弾塑性判定を行ない、次回のステップの瞬間剛性を求める．

(o) 剛性変更があった時は (c) へ進む．剛性変更がなかった時は (j) へ進む．

（5）結果

各荷重ステップにおける各部材端のひび割れ、降伏、塑性率が求まる。

各層の荷重一変形曲線が求まる。

保有耐力（最大地震荷重）が分かる。

5．平面骨組地震応答解析

5．1　基本振動系モデル

（1）振動系モデル

平面フレームモデル（図5．1）

骨組を柱、梁、柱梁接合部、耐震壁などの構造部材に分割してとらえ、これらの各部材ごとに復元力特性を定義するものである。

（2）部材モデル

柱、梁部材の弾塑性部材モデルは、材端剛塑性回転バネモデルとする。

（3）減衰

種　類　：　内部粘性型

$$[c] = \frac{2h_1}{\omega_1}[K]$$

減衰マトリックス　：　瞬間剛性比例型（各時刻における剛性マトリックス[K']）

減衰定数　：　1次0．03（鉄筋コンクリート）高次の減衰定数は振動数に比例させる。

（4）数値積分

線形加速度法を用いる。

5．2　復元力特性

（1）スケルトンカーブ

4．3（3）で示されたもの。（図4．3、図4．4）

（2）履歴特性

Degrading tri-linear 最大点指向形モデル（Takeda モデル）（図5．2）。

5．3　弾塑性フレーム解析の流れ

4．3（4）解析方法の流れの中で、(j) の項を「地動の加速度増分」としたもの。

結論

日本においては、ここに紹介したフレーム応答解析により、部材の応答変形が許容塑性率以下におさまっているかというチェックにより耐震安全性の尺度としている。また、動的応答解析においても、応答層間変形が設定された値以下であることを確認して設計を進めている。

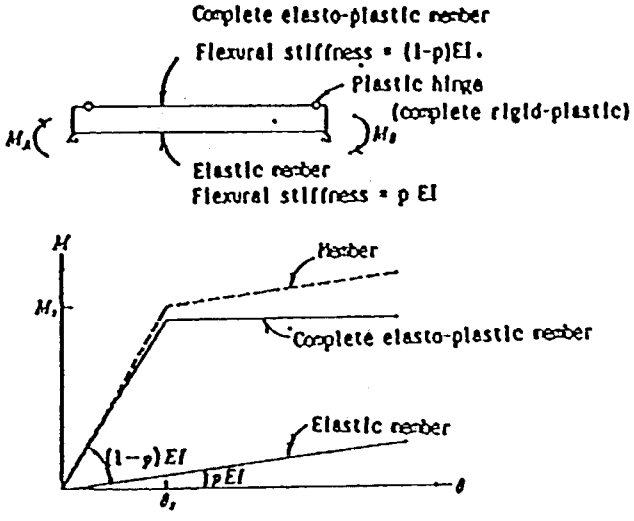


Fig. 2.1 Hybrid beam model

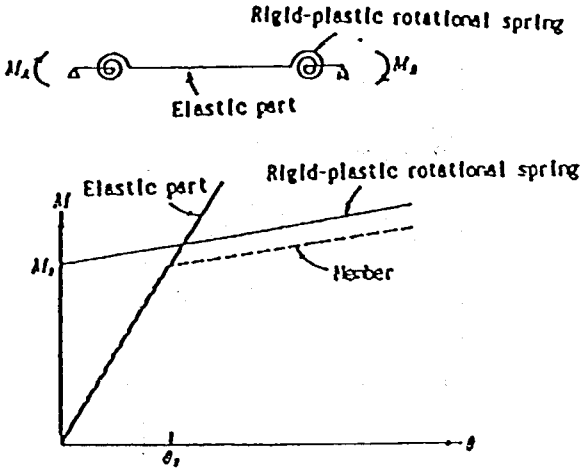


Fig. 2.2 Model where both ends are rigid-plastic rotational springs

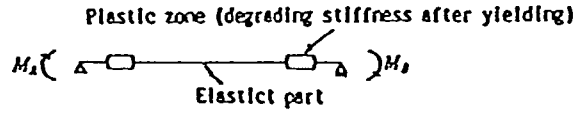


Fig. 2.3 Model where both ends are plastic hinge zones

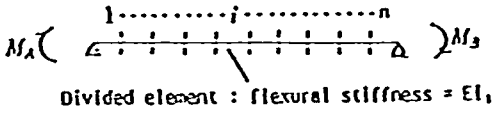


Fig. 2.4 Dividing element method

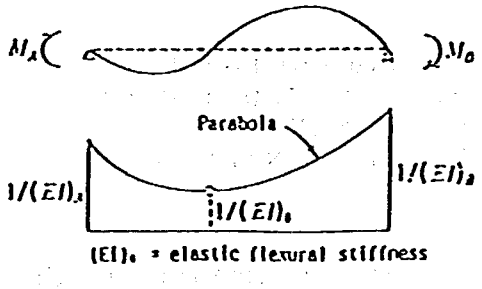


Fig. 2.5 Assumed stiffness distribution method

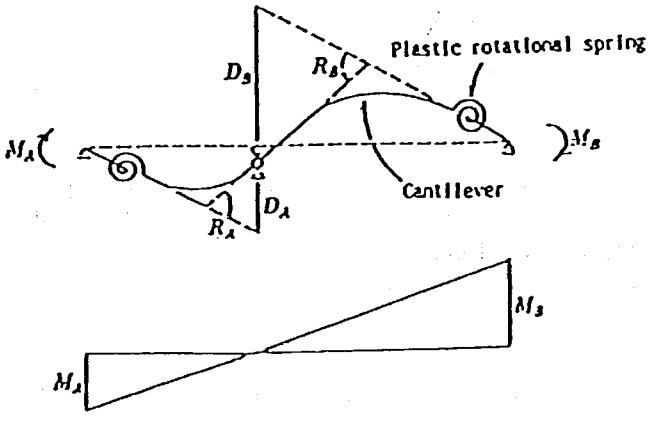


Fig. 2.6 Model where two cantilevers are connected

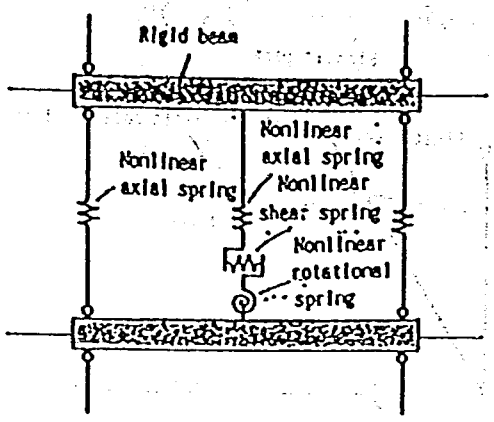


Fig. 2.7 Shear wall model

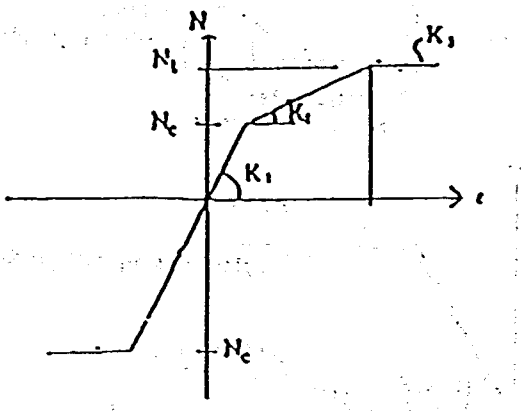
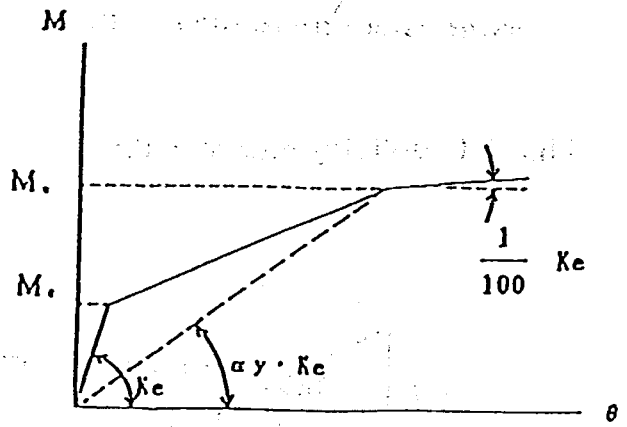
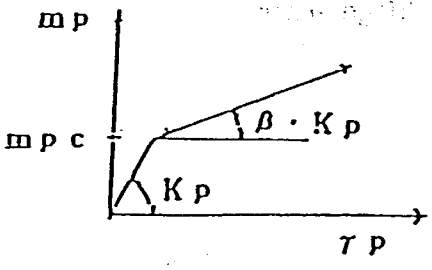


Fig. 2.10 Load-deflection relationship of the column axial force



k_e : elastic stiffness (included slab and re-ber)
α_y : reduction factor of stiffness at yielding point
M_c : flexural cracking moment
M_y : ultimate flexural force

Fig. 2.8 Load-deflection relationship of beam or column



m_p : virtual panel moment
$k_e = G_c \times {}_cV_j$ (shear stiffness)
$m_{pc} = r_c \times {}_cV_j$
${}_cV_j = j_b \times j_c \times t$ (volume)
$\beta = 0.2$

Fig. 2.9 Load-deflection relationship of beam-column connection

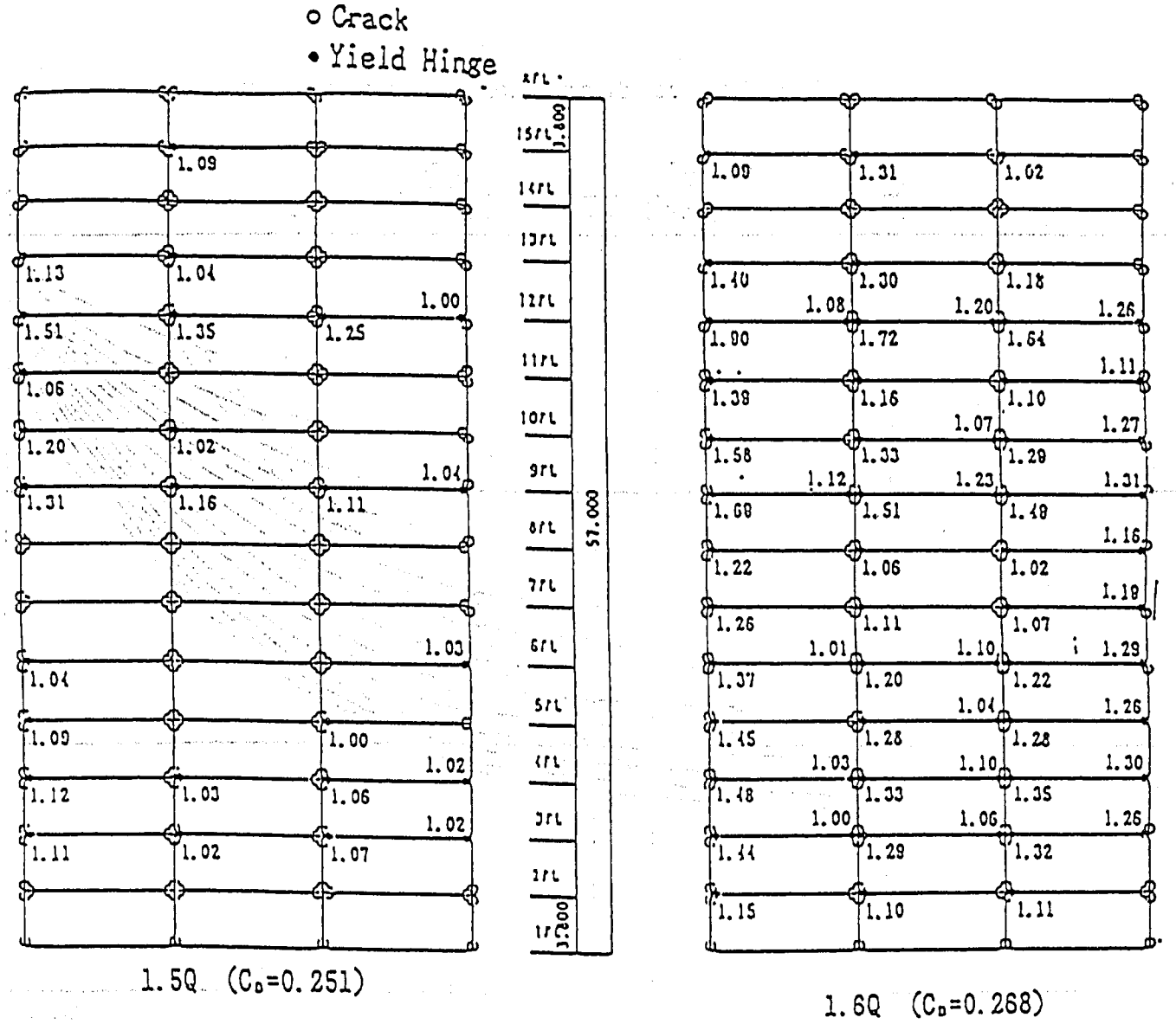


Fig. 2.11 Hinge formation and beam ductility

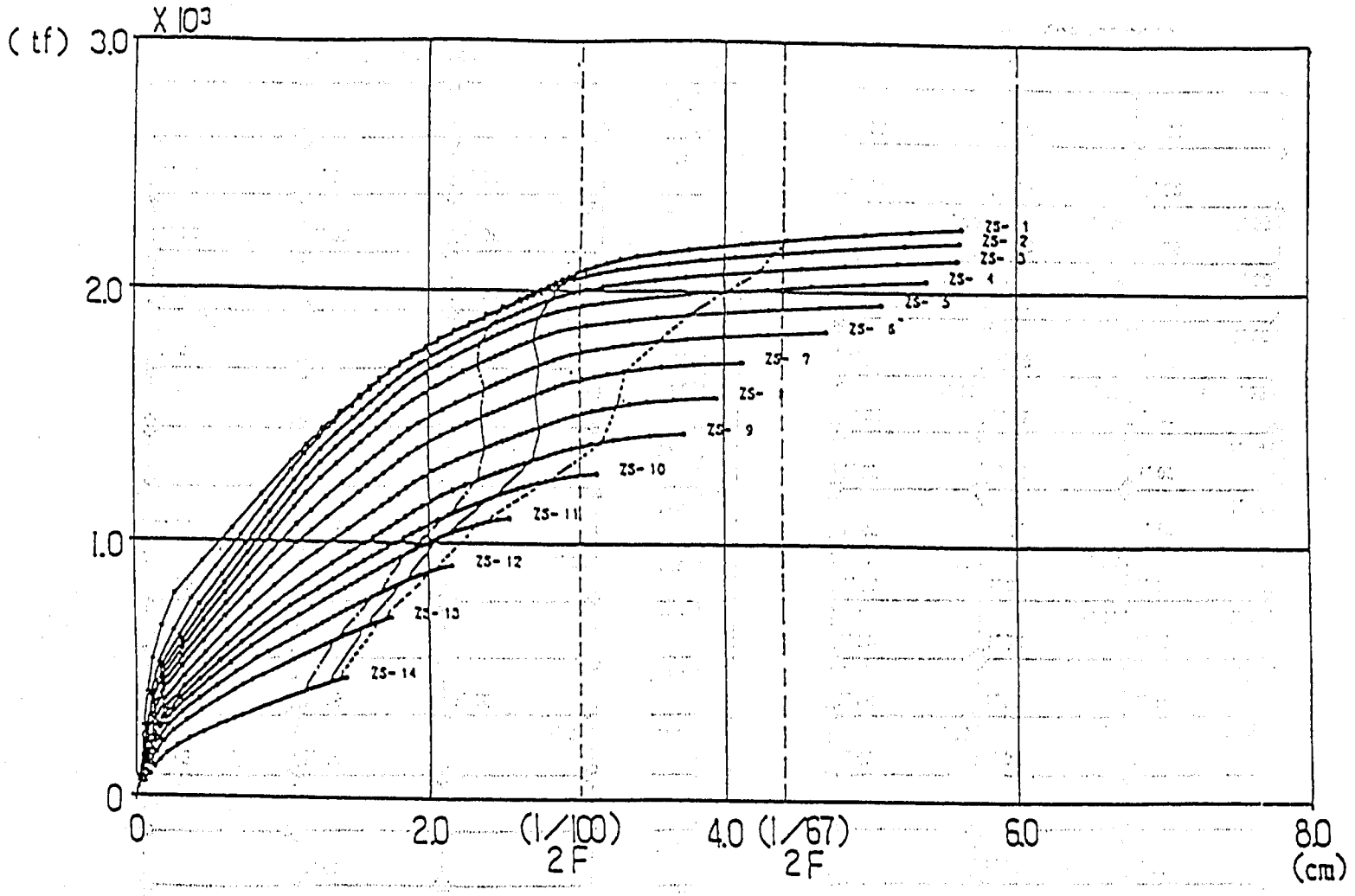


Fig.-2.12 Load-deflection relationship for every story

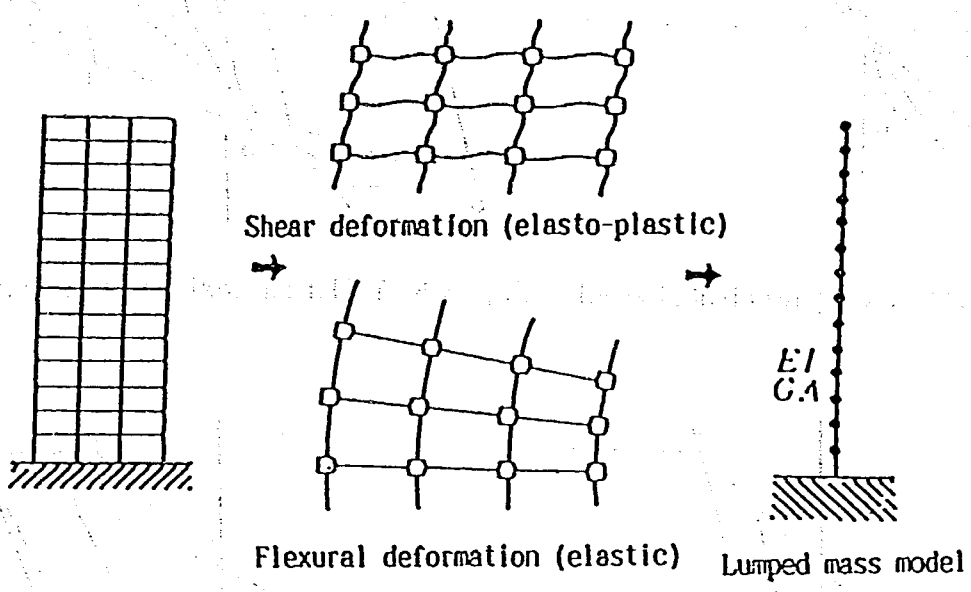


Fig. 3.1 Equivalent flexural-shear model

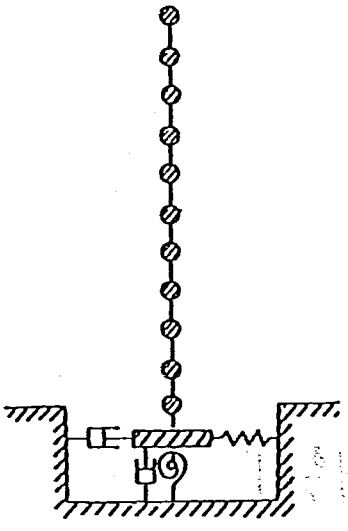


Fig. 3.2 Rocking-sway model

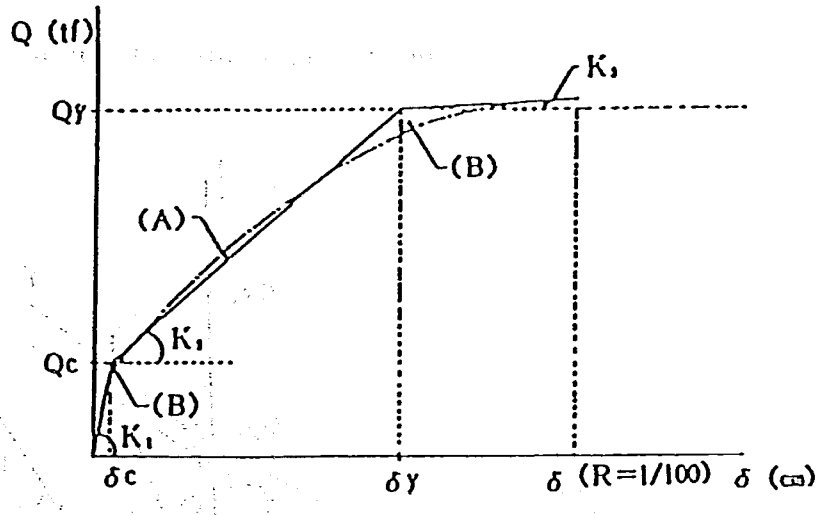


Fig. 3.3 Definition of the skeleton curve

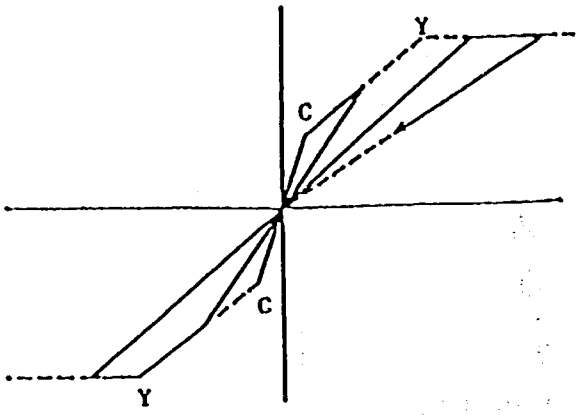


Fig. 3.4 Trilinear origin-oriented model

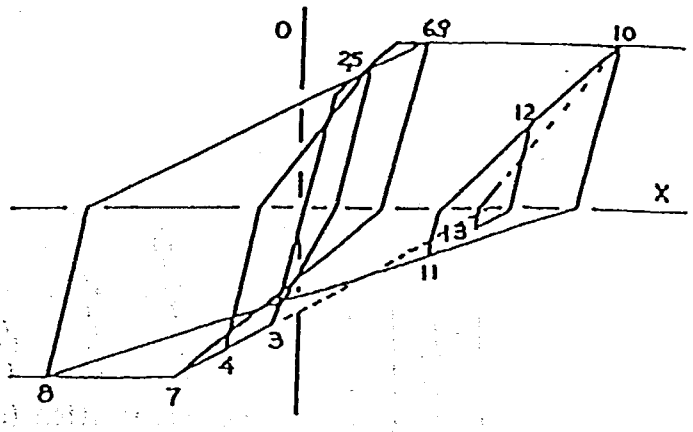
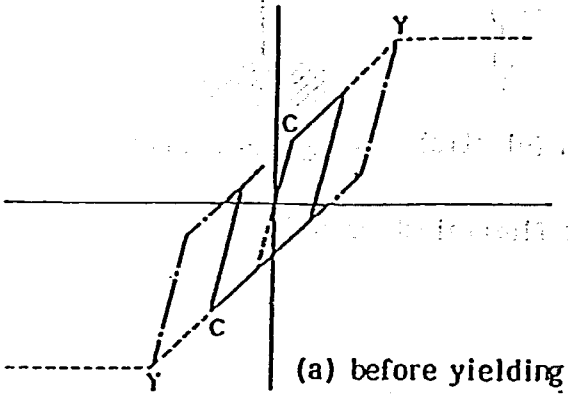
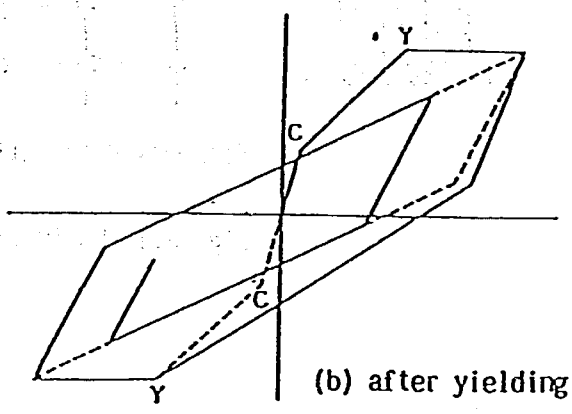


Fig. 3.5 Trilinear maximum value-oriented model



(a) before yielding



(b) after yielding

Fig. 3.6 Degrading tri-linear model

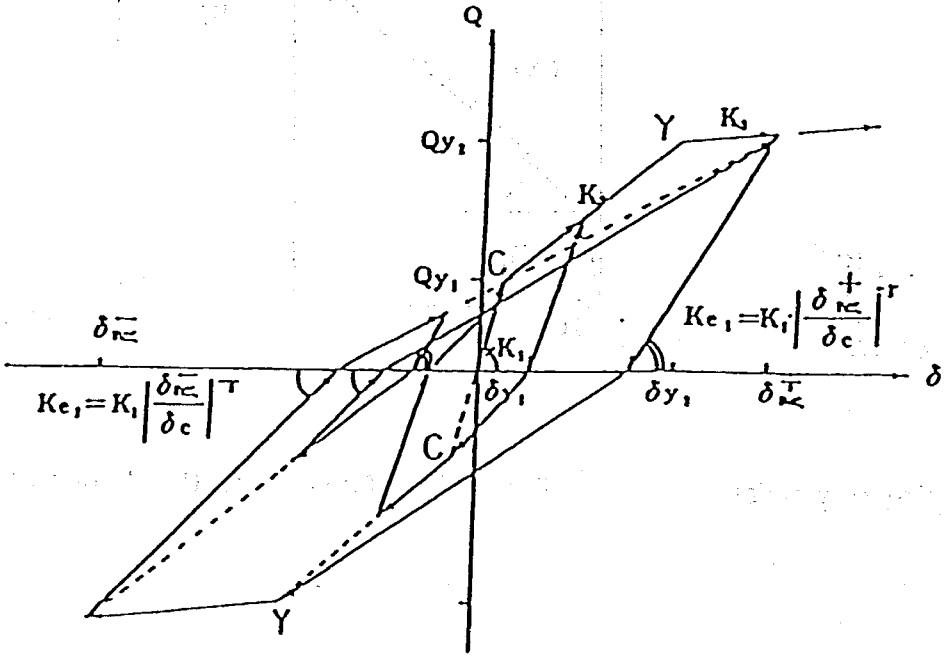


Fig. 3.7 Takeda model

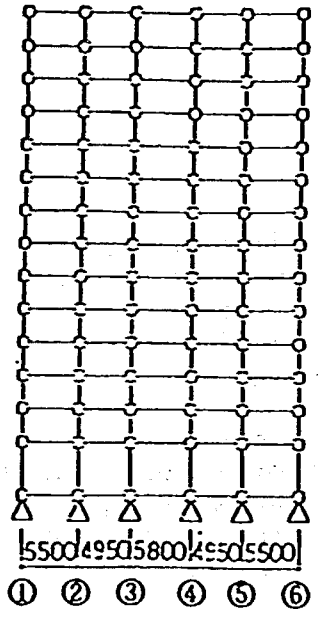


Fig. 4.1 Frame model

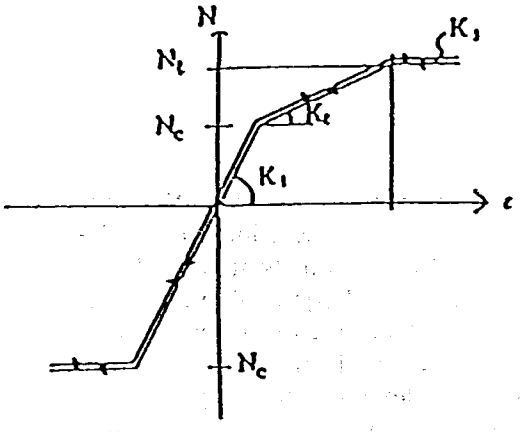


Fig. 4.2 Hysteresis rules for axial forces

U.S. Japan Cooperative Research in Precast Seismic Structural Systems (PRESSS) Program—Status Report

by

Nigel Priestly[1] and H. S. Lew[2]

ABSTRACT

The Precast Seismic Structural Systems (PRESSS) research program was initiated as the fourth topic in the United States - Japan coordinated program on large scale testing for seismic response. The first three topics were reinforced concrete, steel and masonry. The purpose of this coordinated research program is to develop recommendations for the seismic design of buildings constructed of precast concrete systems based on sound analytical and experimental research.

This report describes the scope of the joint program and the current projects being carried out by U.S. researchers.

KEYWORDS:

buildings; concrete; earthquake; precast; research; seismic; structures

1. INTRODUCTION

The fundamental objectives of the U.S.-PRESSS program are two fold:

> To develop comprehensive and rational design recommendations based on fundamental and basic research data which will emphasize the viability of precast construction in the various seismic zones.
>
> To develop new materials, concepts and technologies for precast construction in the various seismic zones.

The PRESSS program sets out to achieve these objectives by a process of examination and assimilation of existing research and design information, by development and evaluation of new PRESSS concepts in close cooperation with industry, by carefully planned analytical and experimental research, and by formulation of the results within a framework of design recommendations based on state-of-the-art design philosophy.

The program considers only the seismic behavior of building structures where precast construction forms the essence of the structural system. Current precast seismic structural systems can be divided into two categories: precast frame buildings and precast wall panel construction. Both are included in the U.S.-PRESSS program.

2. STRUCTURE OF THE U.S.-PRESSS PROGRAM

The U.S.-PRESSS program was conceived in three phases. Phase I, which is funded by the U.S. National Science Foundation for a three-year period ending in 1993, is focused on identifying and evaluating the most promising structural concepts for precast concrete building systems in seismically active regions.

Phase II involves detailed experimental studies of components and subassemblages of precast systems selected during Phase I for more detailed investigation.

Phase III involves one or more multi-story full-size "superassemblage" experiments and the finalization of the seismic design recommendations for incorporation into the model building codes.

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Phase I Program:

- Concept Development

  The project is focused on the development and evaluation of new concepts for precast concrete construction in UBC seismic zones 2, 3, and 4, particularly for low and mid-rise buildings.

- Connection Classification and Modeling

  The project involves a comprehensive review of existing test data and the development of a classification system for PRESSS connections based on type and function.

- Analytical Platform Development

  The widely used DRAIN-2D inelastic time-history analysis program for seismic response is being expanded to three dimensions. In addition, new elements are being developed to model the special characteristics of precast members including joint deformations resulting from slip and ductility.

- Preliminary Design Recommendations

  Current seismic design codes for precast concrete structures are being reviewed and preliminary prescriptive design requirements for "strong connection" precast construction are being developed. In addition, a framework for the final design recommendations, based on structural reliability theory, will be provided. These provisions will include recommendations for ductile connections design.

- Coordination

  This project provides the umbrella under which the four above-mentioned projected are interrelated, and also provides coordination with the Japan-PRESSS research team and the research program at the National Institute of Standards and technology.

Phase II Program:

- Evaluation of Ductile connections for Frame PRESSS

  This project involves the evaluation of the behavior of connecting elements, beam-column subassemblages, and multi-bay subassemblages.

- Evaluation of Wall Panel Building for Moderate Seismic Zones

  This project includes the evaluation of optimal range of connection strength and ductility for a six story building, strength and deformation characteristics required for connection elements, and tests on horizontal panel connections.

3. U.S.-JAPAN COORDINATION

The second meeting of the Joint Technical Coordinating Committee on Precast Seismic Structural Systems was held in Tsukuba, Japan, October 31 - November 2, 1991. The meeting was attended by 21 researchers and 9 observers from Japan, 4 researchers and 3 observers from the U.S., and one observer from New Zealand.

The need for, and importance of, continuing experimental and analytical research on PRESSS was reaffirmed.

Analysis of Japanese and U.S. frame buildings have indicated that satisfactory seismic response can be expected from precast frame buildings designed to meet recommendations of both countries.

The U.S. side will carry out dynamic analysis of the test-design building.

Both sides will continue to develop and classify new connection concepts.

The third joint meeting is scheduled to be held in San Diego, October 25 - November 2, 1992.

バリアブルダンパーを用いた道路橋の地震応答制御

川島　一彦[*1]　運上　茂樹[*2]　清水　英之[*3]

要　約

本文では、著者らが考案した橋梁用のバリアブルダンパーの特性について解析的に検討するとともに、バリアブルダンパーの小型模型を試作し、模型単体を用いた動的載荷実験によりバリアブルダンパーの基本的特性について検討した結果を報告するものである。

キーワード

地震応答制御、道路橋、バリアブルダンパー、小型模型、動的載荷実験

1．まえがき

バリアブルダンパー[1)2)]とは、基本的には粘性ダンパーであるが、粘性減衰力を構造物（橋梁）の応答性状に応じて図－1に示すように変化させようというものである。たとえば、

① 橋桁の振動の振幅が小さいうちは、粘性係数を大きくし、車両の制動荷重などに対しては、事実上固定支承と同じ機能をもたせる。速度の遅い温度変化による桁の伸縮に対しては可動である。

② 地震が発生し、桁の振動がある程度大きくなった場合には、エネルギー吸収を図り、下部構造へ伝達される地震力を適切に低減できるように減衰係数を下げる。

③ さらに、桁の振動が大きくなり過ぎた場合には、これ以上の桁の振動を抑えるために、徐々に粘性係数を大きくして緩衝機能付きの移動制限装置の役目をもたせる。これは、突然、移動制限装置が作動すると、衝撃的な応答が生じるためである[4)]。

したがって、バリアブルダンパーとは、従来のダンパーストッパー[5)-7)]、エネルギー吸収装置としてのダンパー、緩衝機能付きの移動制限装置の３つの装置の長所を合わせ持った装置ということができる。

ダンパーの減衰係数を状況に応じて変化させようという考え方自体は、機械分野、特に航空機や自動車のサスペンションの分野では早くから取り入れられてきている。乗車感覚の向上、走行時の車両の安定性の向上等を目的としたアクティブサスペンションがその一例である[8)9)]。しかし、橋梁のように地震力の分散という考え方を基本としてダンパーの減衰抵抗を変化させるという考え方は、全く新しい技術開発である。

バリアブルダンパーの構造としてはいろいろなものが考えられるが、最もシンプルなものは図－２のようになる。基本的な構造は、従来のシリンダー型粘性ダンパー・ストッパーであり、これにピストンをまたいでバイパスを設け、このバイパスを通る粘性体の量をコントロールすればよい。こうした流体の通過量のコントロールに関する技術にはいろいろなものが開発されており、実用化には大きな障害はないと考えてもよい。また、こうした通過量のコントロールに要するエネルギーは一般の制震設計に比較すると大幅に小さい[10)]。

本文では、著者らが考案した橋梁用のバリアブルダンパーの特性について解析的に検討するとともに、バリアブルダンパーの小型模型を試作し、模型単体を用いた動的載荷実験によるバリアブルダンパーの基本的特性について検討した結果を報告するものである。

2．バリアブルダンパーを有する道路橋の地震応答解析

2．1　バリアブルダンパーを有する多自由度構造系の地震応答解析法

バリアブルダンパーの特性を予め固定したプリセット方式では、線形多自由度系の橋にバリアブルダンパーを取り付けた場合の運動方程式は次式のように表される。

$$\underline{M}\,\ddot{\underline{x}} + (\underline{C} + \underline{C_u})\,\dot{\underline{x}} + \underline{K}\,\underline{x} = -\underline{M}\,\ddot{x}_0 \qquad (1)$$

ここで、$\underline{M}$、$\underline{C}$、$\underline{K}$は、それぞれ構造系の質量、減衰、及び剛性マトリックスを示す。$\underline{x}$及びx_0は、変位ベクトル、及び地動変位を示す。$\underline{C_u}$はバリアブルダンパーを表わす減衰マトリックスで、次式で与える。

$$\underline{C_u} = \sum_{k=1}^{n_v} \underline{c_{ij}^{v}} \qquad (2)$$

ここで、

$$\underline{c_{ij}^{v}} = \begin{bmatrix} V & -V \\ -V & V \end{bmatrix} \qquad (3)$$

ここで、$\underline{c_{ij}^{v}}$は、第ｉ自由度と第Ｊ自由度間に取り付けられたバリアブルダンパーの減衰マトリックスで、n_vはバ

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リアブルダンパーの個数を表わす。Vは、バリアブルダンパーの減衰係数で上下部構造間の相対変位もしくは相対速度の関数として与える。

構造系の減衰マトリックスCを定めるためには、いろいろな方法が提案されているが、ここでは、減衰行列をモードマトリックスで対角化できると仮定した場合の第k次のモード減衰定数h_kを与えることにより、以下のように減衰マトリックスCを算出する。

$$C = (\Phi)^{-1} \mathrm{diag}\,[2 h_k \omega_k]\ (\Phi^T)^{-1} \qquad (4)$$

ここで、Φは、モードマトリックスであり、diag［2 $h_k\omega_k$］は、2 $h_k\omega_k$（k＝1, 2, 3, ……, モード次数）を対角項とするマトリックスである。

C_vは時刻によって変化するため、式(1)の積分に際しては直接積分法を用いる。

2．2　解析対象橋及び解析条件

バリアブルダンパーの制震効果を解析的に検討するために、図－3に示す橋長30mの単純桁橋の橋軸方向振動を対象として地震応答解析を行った。バリアブルダンパーは、本来は多径間連続橋に対する適用性が良いが、ここでは、最も簡単な単純橋で特性を検討することとした。上部構造は両支点において弾性支承により橋軸方向に支持されているものとし、バリアブルダンパーはこの支点位置にそれぞれ設置する。

桁重量に相当する質量が弾性支承と橋脚の曲げ剛性に相当するバネによって支持されていると見なせば、対象橋の振動特性は近似的に1自由度振動系でモデル化ができることから、1自由度振動系の粘性減衰定数とのアナロジーから、次式のように式(3)のバリアブルダンパーの減衰係数Vを定める。

$$V = \sqrt{m_d\ k_p}\ h_v \qquad (5)$$

ここで、m_dは桁の質量、k_pは弾性支承を有する橋脚2基分の曲げ剛性、h_vはバリアブルダンパーによって橋に与えたい減衰定数である。なお、桁の重量は241.5tf、弾性支承のバネ係数は640tf/mとし、橋脚の剛性は両端ピン支承とした場合に1次固有周期が0.5秒となるように定めた。このようにすると、弾性支承を設置した場合の橋の1次固有周期は1秒となる。

式(5)によりバリアブルダンパーに与える減衰定数h_vは、上下部構造間の相対変位もしくは相対速度、あるいは、両者によって変化させる[4]。ここでは、相対変位により粘性係数を変化させた例を示す。

図－4は、解析ケースを示したものである。ここで、ケース1はバリアブルダンパーでコントロールしないケース、ケース2はバリアブルダンパーの減衰係数h_1が橋の応答によらず一定と仮定した場合である。一定とする減衰定数h_1をパラメータとして変化させた。ケース3及び4は、緩衝機能付き移動制限装置としての機能を検討するためのケースであり、変位がある程度大きくなった場合に減衰係数をh_1からh_3まで大きくなるように設定した場合である。ケース3では緩衝機能付きの移動制限装置を想定してh_3＝3とし、ケース4ではほぼ剛に近いストッパーを想定してh_3＝500とした。減衰定数がh_1からh_3に増加し始める変位は、ケース2で減衰定数を0.5とした場合にバリアブル・ダンパーに生じる最大相対変位（これをバリアブルダンパーの基本設計変位d_uと呼ぶ。後述するようにd_u＝7.7cmである）の2/3とした。ケース5は、桁の相対変位が小さい領域で交通荷重等による桁の振動を低減するためにストッパー機能を設けた場合であり、橋脚天端に対する橋桁の応答変位が$-1/6\,d_u$～$1/6\,d_u$の範囲において減衰定数をh_1からh_2まで大きくなるように設定した。ケース6は、ケース3、4、5を組み合わせた場合である。

入力地震動としては図－5の波形を用いた。これは道路橋示方書V耐震設計編に示される鉄筋コンクリート橋脚の地震時保有水平耐力の照査に用いる設計水平震度に相当する加速度応答スペクトル（III種地盤）に昭和58年日本海中部地震により津軽大橋周辺地盤上で得られた強震記録を周波数領域でスペクトルフィルティングさせた動的解析用の標準波形である。最大加速度は443cm/sec^{2}である。

また、上記の解析ケースでは、いずれの場合も式(4)による橋固有の減衰係数は、$c_k = 2 h_k \omega_k$としてh_k＝0.02を考慮した。

2．3　バリアブルダンパーの減衰係数と地震応答の低減効果

図－6は、バリアブルダンパーの減衰係数Vを一定とした場合（ケース2）の橋及びダンパーの最大応答値とダンパーの減衰定数h_1の関係を示したものである。ダンパーの減衰定数h_1の増加とともに、桁の最大応答変位は単調に減少し、減衰定数h_1が2以上となるとほぼ一定値に収れんする。桁の最大応答加速度は、減衰定数h_1が約0.5で最小値をとり、その後は漸増する。これは、エネルギー吸収装置としての機能よりもダンパーストッパーとしての機能に変わってくるためである。橋脚基部の断面力については、せん断力及び曲げモーメントともに、桁の最大加速度とよく似た特性を示し、減衰定数h_1が0.5程度の場合に最小になる。一方、ダンパーについて見てみると、減衰定数h_1が大きくなると最大ダンパー力は増加するが、最大ストローク

は逆に減少する。吸収エネルギーは、減衰定数h_1が約0.2で最大値をとりその後漸減する。このように、ダンパーの減衰定数h_1を桁との相対変位に対して一定とした場合には、減衰定数h_1を大きくすることにより桁の変位及びダンパーのストロークは単調に減少するが、桁の加速度及び橋の耐震設計上重要な橋脚基部の断面力は、減衰定数h_1が0.5程度の値で最小値をとり、この時にエネルギー吸収装置として最も有効であるということができる。減衰定数h_1を0.5にした場合には、ダンパーを設置しない場合に比較して、桁の最大応答変位で31%に、橋脚基部の最大曲げモーメントで37%に減少する。この時のダンパーに必要とされる最大力が34.7tf×2＝69.4tf、最大ストロークは7.7cmである。

次に、ケース2に緩衝機能付きの移動制限装置としての機能を加えた場合（ケース3）を見てみよう。バリアブルダンパーがダンパーとして機能する場合の減衰定数h_1は0.5とし、移動制限装置として機能する場合の減衰定数h_3は最大で3（ケース3）及び500（ケース4）とした。図－7は、ケース2～ケース4について、桁の応答変位及び応答加速度、バリアブルダンパーの減衰力の時刻歴を比較したものである。緩やかに移動制限機能が働くケース3に比較して、急速に移動制限機能が働くケース4の場合には、桁の加速度応答及びダンパーの減衰力に衝撃的な力が作用する。図－8は、桁に生じる最大応答とこれに要するリアブルダンパーの減衰力を比較したものである。まず、移動制限装置の機能を設けた場合と設けない場合を比較してみると、緩やかに移動制限機能が働く場合（ケース3）には、移動制限装置を設けても桁の最大応答変位や応答加速度はあまり変わらない。橋脚基部に生じる最大断面力も7%程度増加するだけである。ダンパーの最大減衰力は1.73倍になるが、最大ストロークは約10%減少する。ストロークの減少は移動制限装置が有効に機能していることを示している。

一方、急激に移動制限装置を作動させた場合（ケース4）には、緩やかに移動制限機能を働かせた場合よりも当然ながら応答値は大きくなり、桁の応答加速度及び橋脚基部の断面力は約2.0倍になる。さらに、ダンパーのストロークは移動制限装置を作動させた変位まで減少するが、ダンパーの最大減衰力は約4.3倍に大きくなる。

図－9は、桁の変位が小さいところでストッパーとしての機能を与えた場合の影響を示したものである。バリアブルダンパーの減衰定数としては、$h_1=0.5$とし、h_2を変化させている。バリアブルダンパーの減衰定数h_2が8程度までは、桁の最大応答変位は単調に減少するが、その後は増加に転じる。これは、減衰定数h_2を大きくし過ぎると、エネルギー吸収よりもダンパーストッパーとしての機能が顕著になるためである。桁の最大応答加速度は減衰定数h_2が1の付近で最小となるが、その後単調に増加する。橋脚基部の断面力は桁の最大応答加速度と同じ特性を示す。一方、減衰定数h_2を大きくするほど、バリアブルダンパーに必要とされる減衰力は単調に増加し、ストローク及び吸収エネルギーは単調に減少する。したがって、桁の応答変位が小さい領域で自動車荷重等に対する桁の振動を抑えるためにここで減衰係数h_2を大きくすることは、橋脚の地震時断面力を減少させるという目的とは反することになる。これについては、常時の使用性から要求される特性を見極めて総合的に検討して行くことが必要とされる。

表－1は、ケース1～ケース6の最大応答値を比較したものである。粘性ダンパーストッパー、エネルギー吸収装置、緩衝機能付き移動制限装置の機能をバリアブルダンパーに与えた場合には、非制御の場合（ケース1）に比較して、桁の最大応答変位および最大応答加速度は、それぞれ、25%、43%に、また、橋脚基部に生じる最大曲げモーメントは46%に減少する。この時にバリアブルダンパーに必要とされる最大減衰力は62.6tf×2＝125tf、最大ストロークは5.78cmである。これは現状で十分に設計可能なダンパー装置ということができる。

3．バリアブルダンパーの小型模型の開発

3．1　バリアブルダンパーの小型模型の概要

図－10は、バリアブルダンパーの小型模型の製作図を示したものである。模型の製作に際しては、バリアブルダンパーの減衰力の可変範囲を20kgf～200kgf、最大ストロークは±2.5cmとし、制御は構造物の変位、速度、あるいは、これらの組合せによってパーソナルコンピュータを用いて行うことを基本条件とした。

シリンダーは、ロッドを含めた全長が394mm、中間部にピストンを有するもので、ピストンの受圧面積は12.56cm^2、最大ストロークは±2.5cmである。ピストンをまたいでシリンダーから鋼管によりバイパスを設け、これを2つのサーボバルブを介してオイルタンクに接続する。サーボバルブは、ダイレクト電磁比例型切換弁と呼ばれるもので、図－11に示すように直流比例ソレノイドで発生する磁力とバルブに取り付けられたスプリングとの釣合により開度を調節するものである。また、サーボバルブには差動トランスが取り付けられており、これに発生する起電力によりバルブの位置を検出してフィードバックし、精度よい制御を行うことが可能となっている。サーボアンプからサーボバルブへの入力電圧は0～5Vとし、5Vでバルブ全開、0Vでバルブ全閉の状態となる。なお、シリンダー内に封入する粘性体としては、油圧ジャッキ等に通常用いられるオイルを用いた。

図－12は、バリアブルダンパー内部における粘性体の流

れを示したものである。ダンパーのピストンは、橋の振動に応じて左右に動く。例えば、ピストンが左方向に動いた場合には、粘性体はシリンダーからサーボバルブ1を介してタンクに流れ、一方向にのみ粘性体を流すことが可能なチェックバルブ2を介してタンクからシリンダーへ粘性体を流す。この間、サーボバルブ2は全閉の状態となる。ピストンが逆に動く場合には、前述と全く逆の状態となり、サーボバルブ1は全閉となり、チェックバルブ1とサーボバルブ2を介してシリンダー～タンク間を粘性体が流れる。したがって、減衰抵抗力は、ピストンが左に動く場合にはサーボバルブ1により、ピストンが右に動く場合にはサーボバルブ2により調整される構造となっている。

3．2　バリアブルダンパーの小型模型の動的載荷実験

バリアブルダンパーは、前述のように構造物の変位や速度に応じてその減衰特性を変化させる装置であり、必要とされる減衰抵抗を適切に設定するためには、バリアブルダンパーの基本的な特性を把握しておく必要がある。バリアブルダンパーの減衰特性は、バルブの開度でコントロールされ、バルブの開度はバルブへの入力電圧のみで制御される。このため、サーボバルブへの入力電圧とバリアブルダンパーの減衰特性の関係を求めるものとした。

図－13及び写真－1は、実験状況を示したものである。一定電圧をサーボアンプに入力してバルブの開度を一定、すなわち、バリアブルダンパーの減衰特性を一定に保った状態で、ロードセルを介して固定した加振負荷装置により強制変位を与えた。サーボアンプへの入力電圧、ダンパーの反力、ダンパーのストローク量を計測した。加振負荷装置による強制変位は正弦波とし、これを10波を基本として載荷した。

実験ケースは、表－2に示す通りである。ケース1はダンパーピストンが有する摩擦力を測定するためのケースであり、静的載荷（加振振動数は0.05Hz、加振振幅は20㎜）を行った。この時のバリアブルダンパーへの入力電圧は、バルブを全開の状態として5Vとした。ケース2は、サーボバルブへの入力電圧とダンパーの減衰力の関係を求めるためのケースであり、加振振動数、加振振幅を変化させると同時に、各加振において入力電圧を変化させた。

3．3　バリアブルダンパーの小型模型の特性

（1）ダンパーピストンの摩擦力

図－14は、ダンパーピストンの摩擦力を測定するために、加振振動数0.05Hz、加振振幅20㎜で載荷した場合の荷重～変位の履歴曲線を示したものである。荷重～変位関係はほぼ矩形状となり、摩擦力が作用する場合の典型的な関係を示している。なお、載荷の回数による履歴曲線に相違はほとんどなく再現性はよい。摩擦力の平均は約7kgfである。

（2）荷重～変位の履歴曲線

図－15は、バリアブルダンパーの荷重～変位の履歴曲線の一例を示したものである。ここには、加振振動数が1Hzの場合について、加振振幅を2㎜、6㎜及び10㎜とした場合について示している。

加振振幅が2㎜の場合には、入力電圧が1.5Vよりも小さくなると、徐々に荷重が大きくなるものの荷重がピークをとる位置は速度最大（変位零）の位置よりも遅れが生じている。これは、変位量が2㎜と小さいため、これに伴う粘性体の流量が小さくなり、粘性体の圧縮性、配管と粘性体の摩擦などが影響しているためと考えられる。したがって、これを解決するためには、少ない変位量でもある程度必要な粘性体の流量を得るためにピストンの受圧面積を大きくする必要がある。加振振幅が6㎜及び10㎜の場合には、荷重～変位の履歴曲線はほぼ楕円形を示し、理想的な粘性減衰タイプの荷重～変位関係となっていることがわかる。

（3）入力電圧と減衰力の関係

図－16は、載荷速度と減衰力の関係がサーボバルブへの入力電圧に応じてどのように変化するかを示したものである。ここには、後述する最小自乗法により求めた関数式による曲線も示している。これによれば、減衰力と載荷速度の関係は、通常のシリンダー型の粘性ダンパーと同様にべき乗関数形を有している。このため、ここでは次のように関数形を設定して実験式を導いた。

$$F = F_0 + C(V) \cdot v^{n(V)} \qquad (6)$$

ここで、F：減衰力（kgf）、F_0：ダンパーピストンの摩擦力（F_0＝7kgf）、C(V)：減衰係数（入力電圧Vの関数）、v：速度、n(V)：速度項のべき乗係数（入力電圧Vの関数）である。式(6)では、減衰係数C(V)と速度項のべき乗係数n(V)が未知数となり、これらを実験データをもとに最小自乗近似して求めた。このようにして求められた減衰係数C(V)は、入力電圧によらずほぼ一定値となり、その平均値はC(V)＝0.8となる。速度項のべき乗係数と入力電圧の関係は図－17に示す通りとなる。速度項のべき乗係数は入力電圧が小さくなると大きくなる特性を有する。入力電圧が2.5V以上ではほぼ一定で1.4程度であるが、2.5V以下で急激に大きくなる。べき乗係数と入力電圧の関係を直線近似すると図－17に示す通りとなる。

5．結　論

橋梁用の耐震装置として著者らが提案するバリアブルダンパーの特性について解析的に検討するとともに、バリアブルダンパーの小型模型を試作し、その基本的特性につい

て実験的に検討した。本検討結果の結論は以下のとおりである。

1）本解析で対象とした橋では、バリアブルダンパーに粘性ダンパーストッパー、エネルギー吸収装置、緩衝機能付き移動制限装置の３つの機能を与える場合には、$h_1=0.5$、$h_2=h_3=3$が適切である。

2）バリアブルダンパーに1）に示す特性を与えた場合には、非制御の場合に比較して、桁の最大応答変位は25%に、最大応答加速度は43%に、また、橋脚基部の最大曲げモーメントは46%に、それぞれ、減少する。これに要するバリアブルダンパーの最大減衰力は62.6tf×２＝125tf、最大ストロークは5.78cmであり、これは現状で十分に設計可能な範囲ということができる。

3）試作したバリアブルダンパーの正弦波載荷実験によれば、荷重～変位の履歴曲線はほぼ楕円形を描き、粘性減衰を有する理想的な荷重～変位関係が得られ、実際への応用の実現性が確認された。また、現在中型のバリアブルダンパーを開発中であり、その結果については今後紹介していきたい。

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Table 1 Peak Response of the Bridge and the Variable Damper

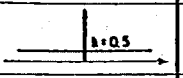
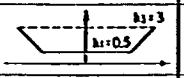
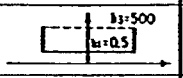
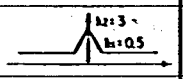
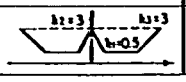
	Analytical Cases	Deck			Pier		Variable Damper			
		Displacement (cm)	Velocity (cm/S)	Acceleration (cm/S²)	Shear Force (tf)	Bending Moment (tf-m)	Damping Force (tf)	Relative Displacement (cm)	Relative Velocity (cm/S)	Total Energy (tf-m)
1	Ks Control	33.04	189.0	1300	165.9	3297	——	——	——	——
2	Constant Damping 	10.28	60.2	478	63.4	1223	34.7	7.67	44.9	93.9
3	Displacement Dependent Damping with Weak Stopper for Excessive Deck Response Displacement 	9.94	54.2	538	72.0	1395	60.2	6.90	68.1	94.4
4	Displacement Dependent Damping with Strong Stopper for Excessive Deck Response Displacement 	10.90	76.4	965	123.9	2382	151.3	5.54	112.3	94.9
5	Displacement Dependent Damping with Stopper for Small Deck Response Displacement 	8.50	51.3	567	80.1	1540	64.7	6.34	55.8	74.5
6	Displacement Dependent Damping (Combination of Case 3 and Case 4) 	8.38	47.5	555	81.2	1528	62.6	5.78	60.7	77.4

Table 2 Experiment for Small-Size Model of Variable Damper

Case	Objective	Loading Frequency (Hz)	Loading Displacement (mm)	Input Voltage (V)
1	Measurement of Friction Force of Damper Piston	0.05	20	5
2	Relation between Damping Force and Input Voltage	1, 2, 3, 4	2, 6, 10, 15	0~5

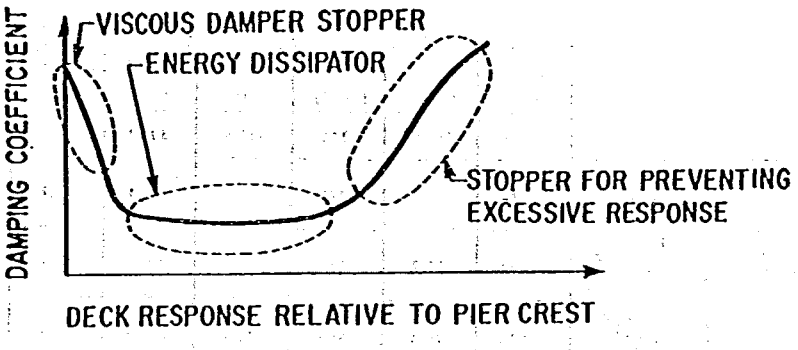


Fig.1 Basic Concept of Variable Dampers

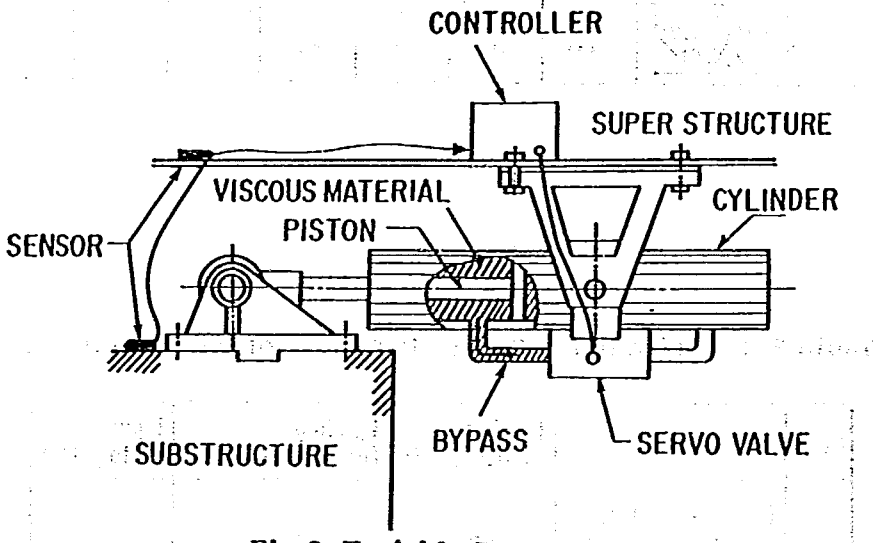


Fig.2 Variable Damper

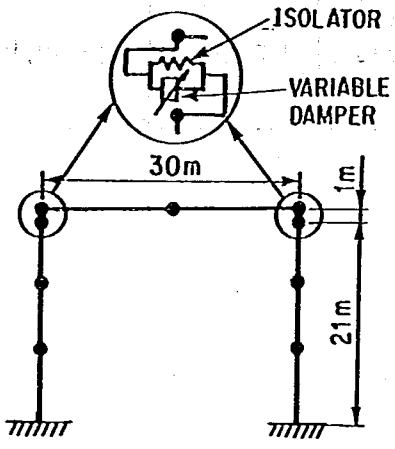
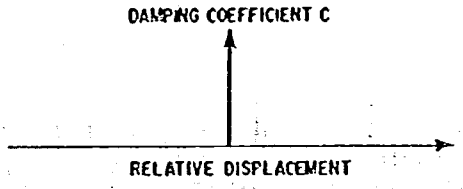
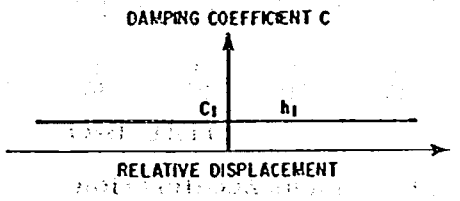


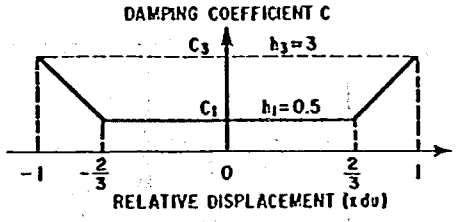
Fig.3 Highway Bridge Analyzed and Analytical Idealization



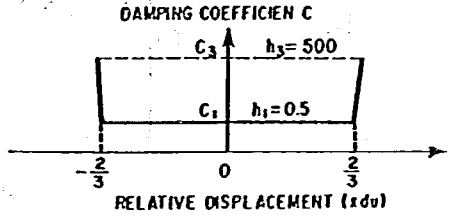
(a) Case 1 : No Control



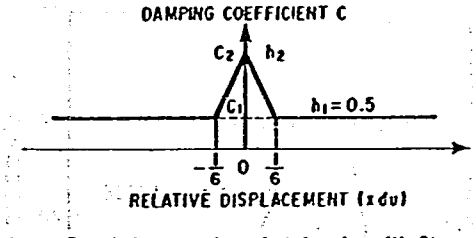
(b) Case 2 : Constant Damping



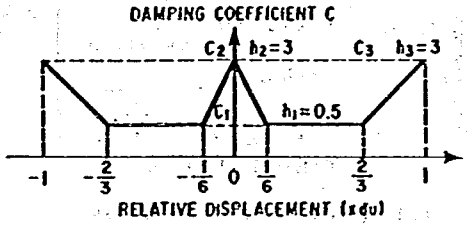
(c) Case 3 : Displacement Dependent Damping with Weak Stopper for Excessive Deck Response Displacement (du=7.67cm)



(d) Case 4 : Displacement Dependent Damping with Strong Stopper for Excessive Deck Response Displacement (du=7.67cm)



(e) Case 5 : Displacement Dependent Damping with Stopper for Small Deck Response Displacement (du=7.67cm)



(f) Case 6 : Displacement Dependent Damping (Combination of Case 3 and Case 5) (du=6.34cm)

Fig.4 Analytical Cases

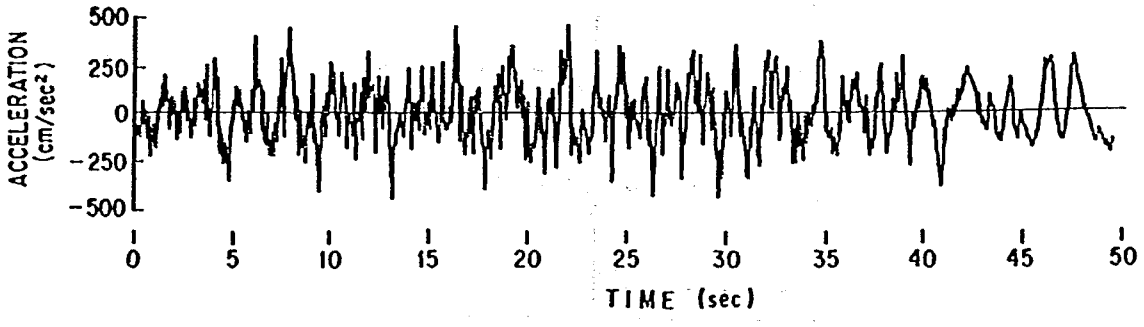
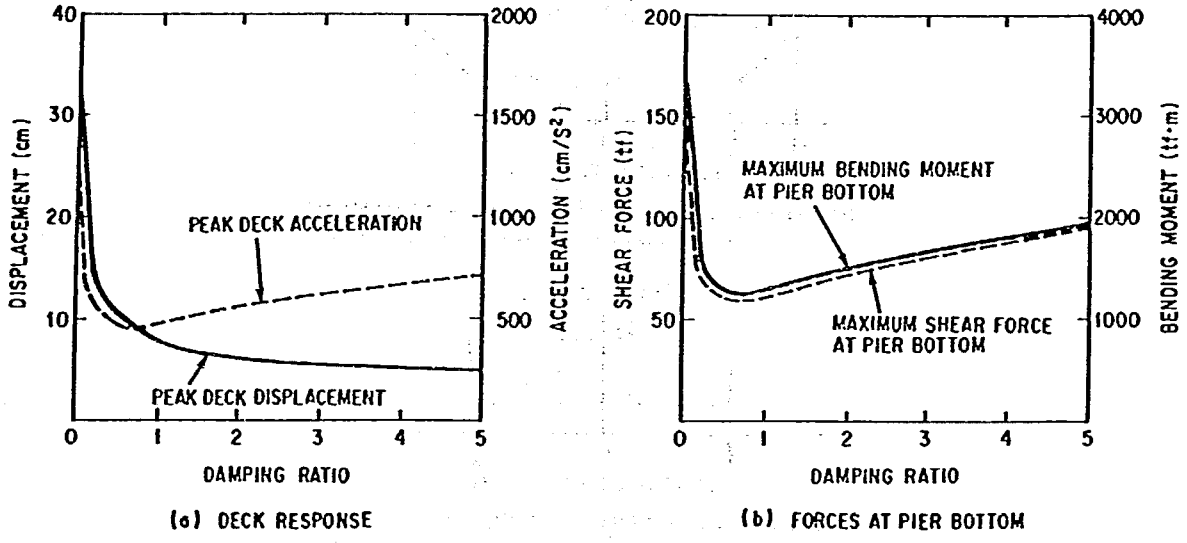


Fig.5 Input Acceleration



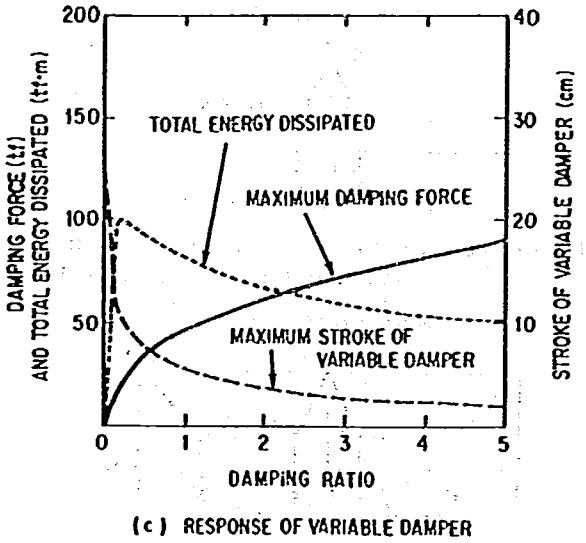


Fig.6 Peak Response of the Bridge and the Variable Damper (Case 2)

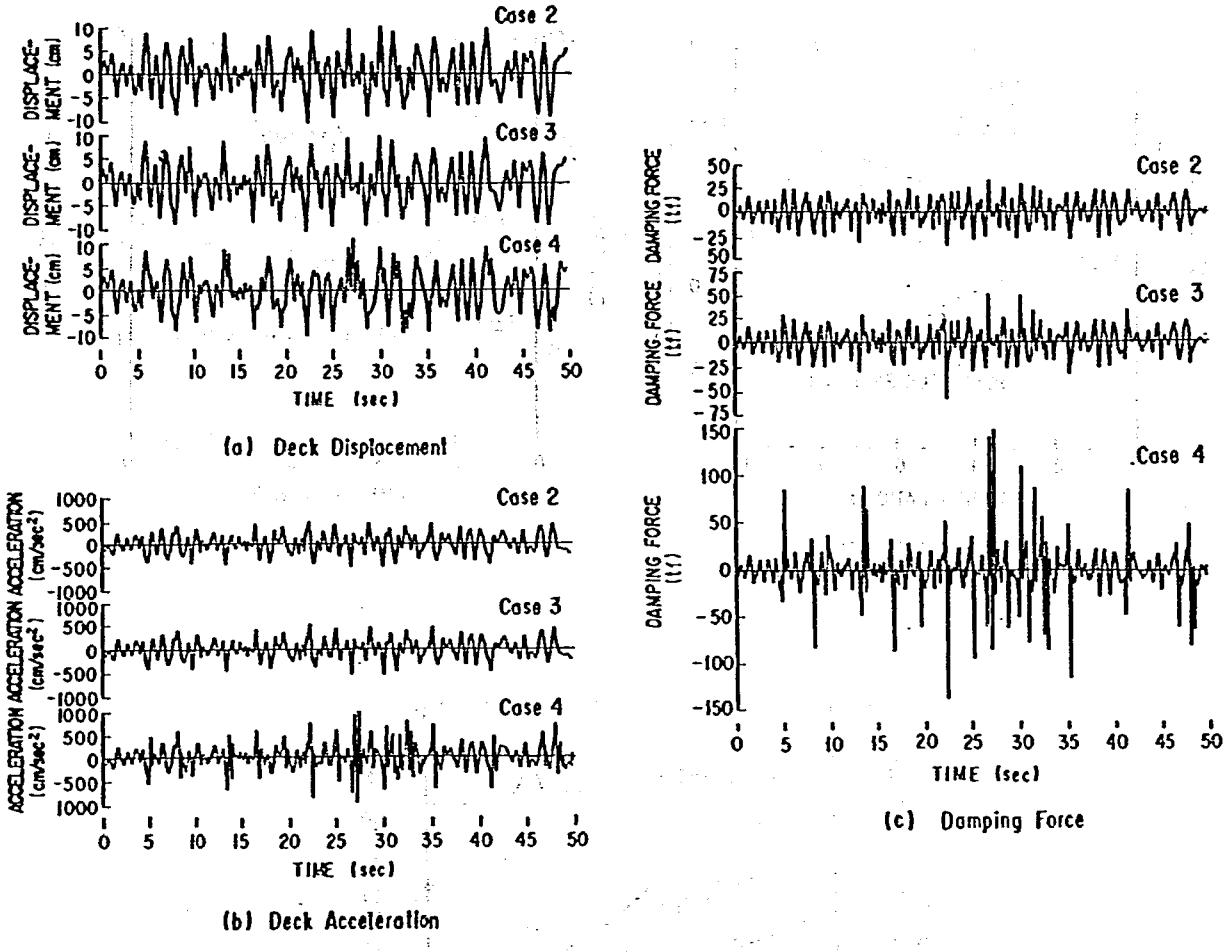


Fig.7 Computed Response of the Bridge and Variable Damper

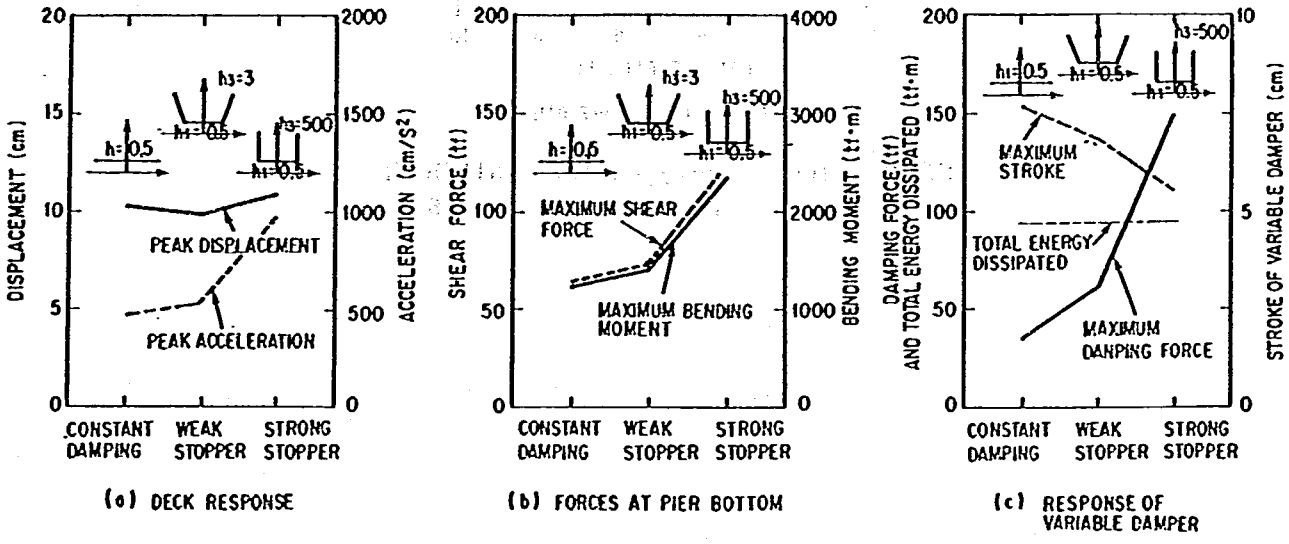
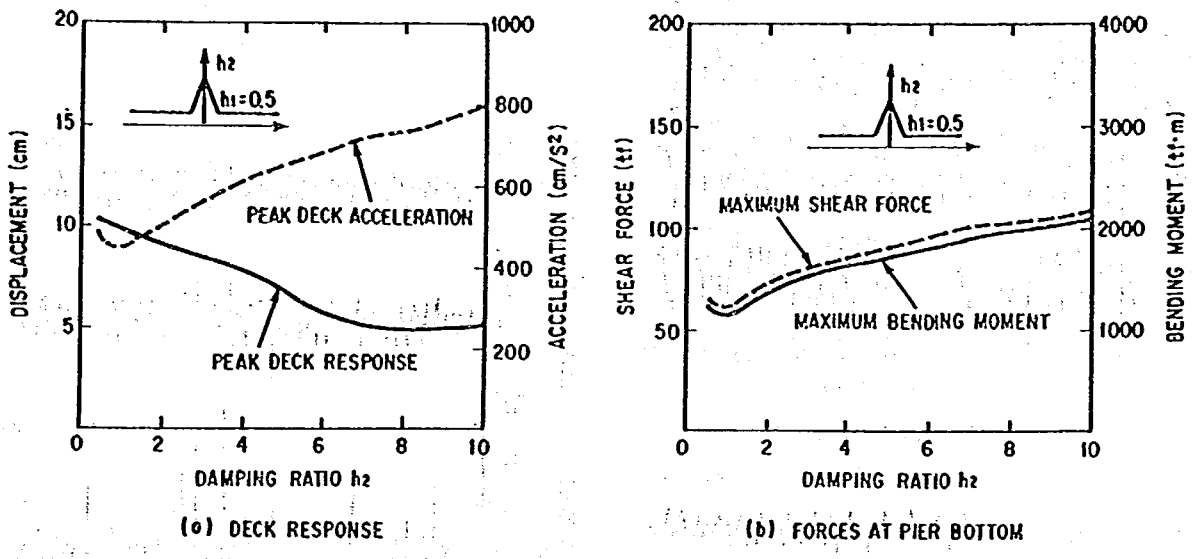


Fig.8 Effect of Weak Stopper (Case 3) and Strong Stopper (Case 4) for Excessive Deck Response Displacement



(a) DECK RESPONSE

(b) FORCES AT PIER BOTTOM

(c) RESPONSE OF VARIABLE DAMPER

Fig.9 Effect of Stopper for Small Deck Response Displacement (Case 5)

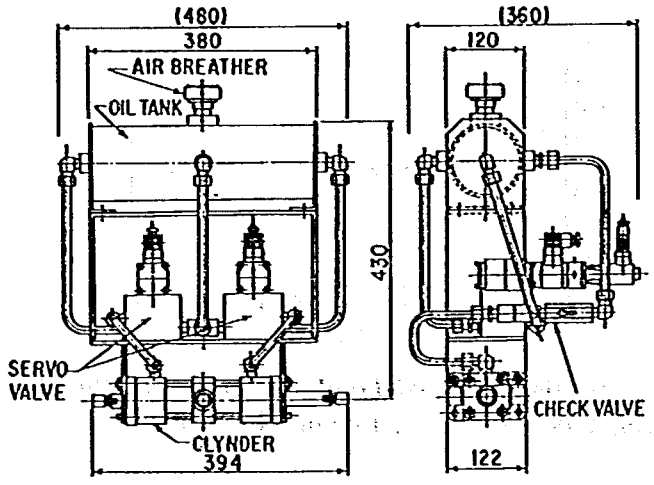


Fig.10 Small-Size Model of Variable Damper

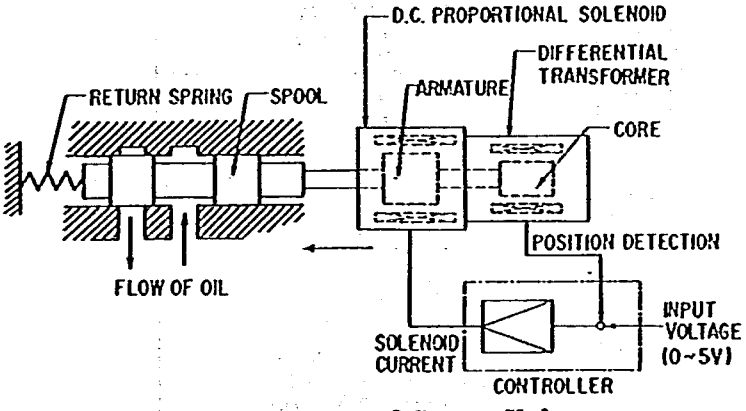


Fig.11 Structure of Servo Valve

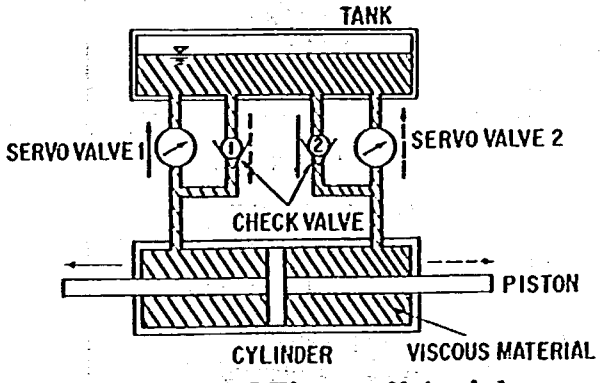


Fig.12 Flow of Viscous Material

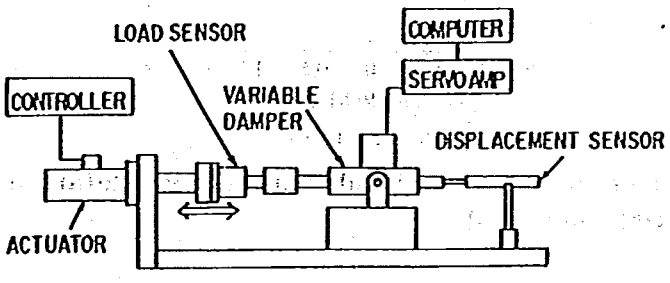


Fig.13 Set Up of Loading Test

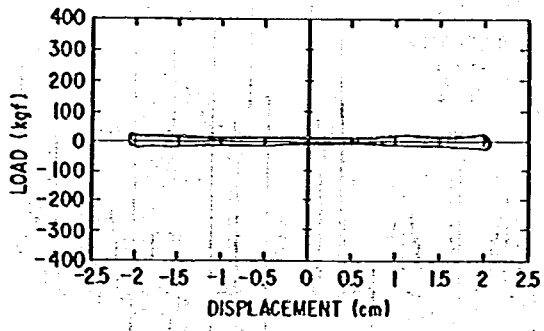
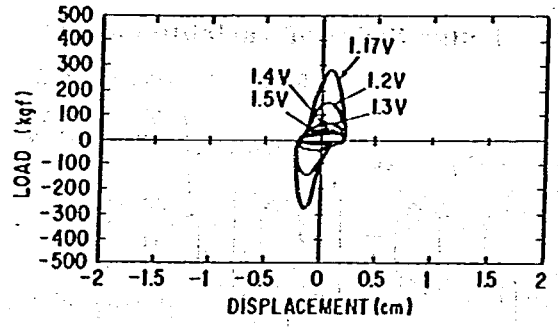
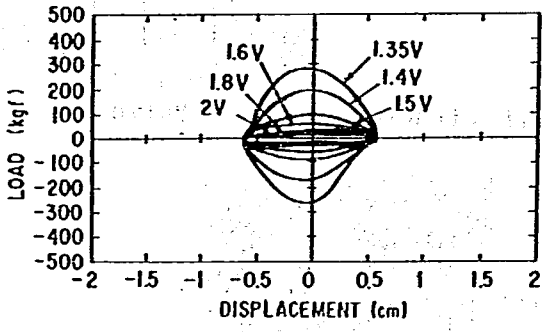


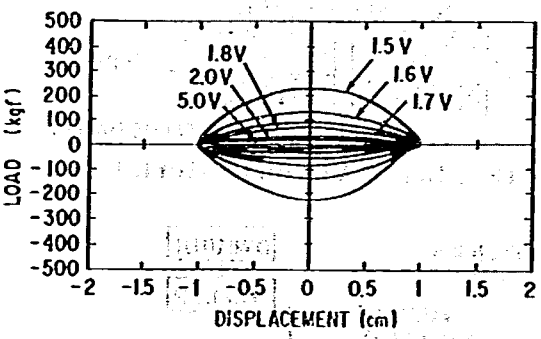
Fig.14 Hysteresis Loops of Load-Displacement Relation (Displacement=20mm, Frequency=0.05Hz)



(a) Displacement 2mm



(b) Displacement 6mm



(c) Displacement 10mm

Fig.15 Hysteresis Loops of Load-Displacement Relation (Frequency=1Hz)

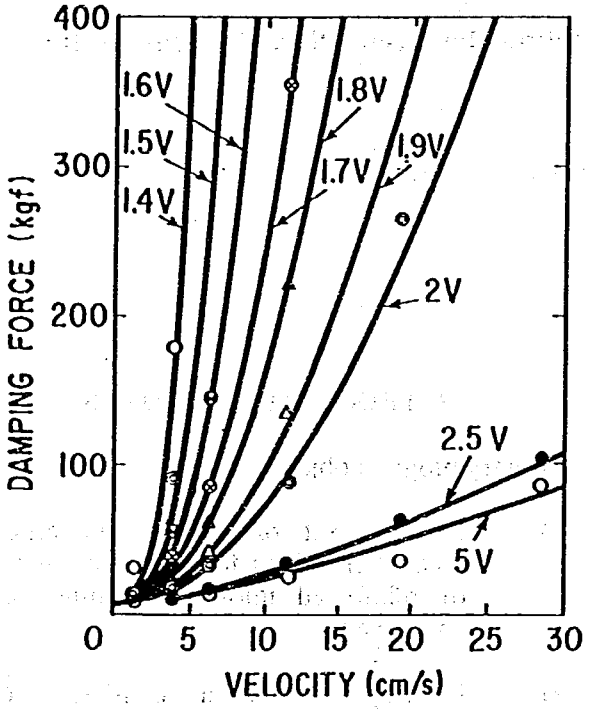


Fig.16 Damping Force vs. Peak Velocity Relation

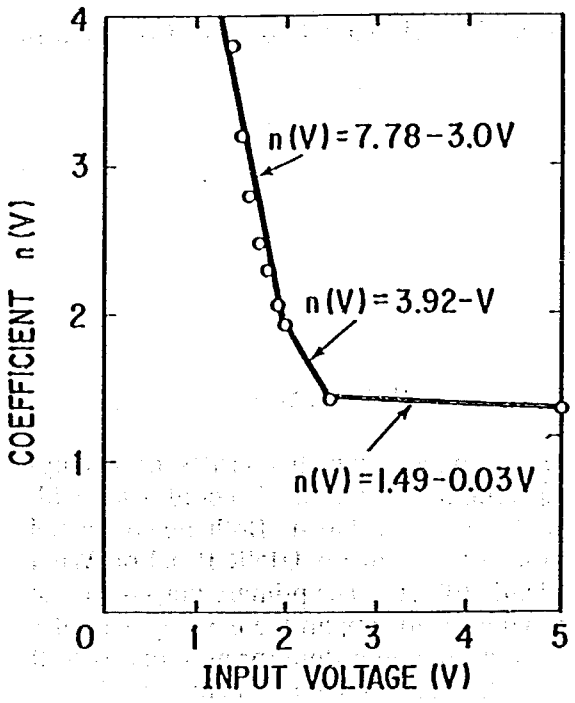


Fig.17 Power Coefficient n(V) vs. Input Voltage Relation

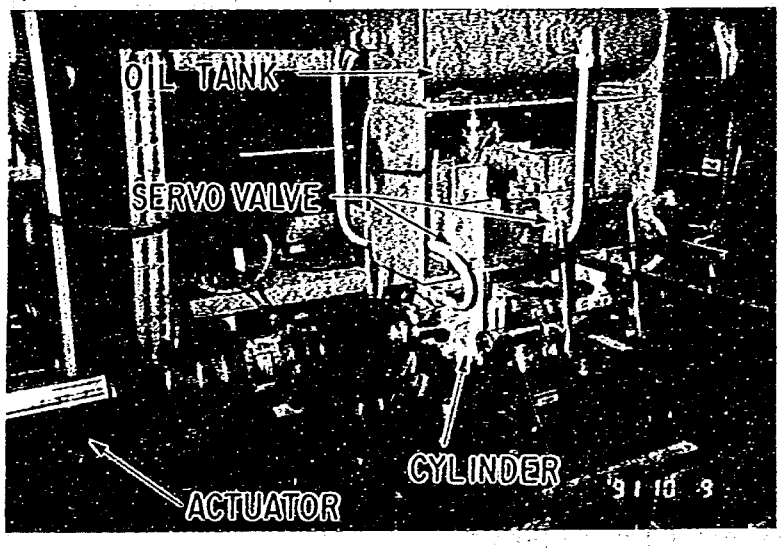


Photo 1 Experimental Set Up

U.S. Coordinated Program for Masonry Building Research – Seventh Year Status

by

James L. Noland[1]

ABSTRACT

The U.S. program, which consists of twenty-eight specific research tasks, is coordinated with a parallel program in Japan. Both are conducted under the auspices of the UJNR Panel on Wind and Seismic Effects. The primary purpose of the U.S. program is to support the development of a limit state design procedure for masonry as well as provide experimental data, analytical methods, etc. Several technical accomplishments have been realized in both modeling and experimental areas. The technology transfer plan includes dissemination of research reports, topical reports and a summary report on the specific research areas as well as by presentations and papers. Design and criteria recommendations will be an extremely important mode of technology transfer and are expected to support limit state design standard development. Current status of the program is presented. Completion is expected by July 1993.

KEYWORDS: masonry; progress; structures; technology transfer; UJNR

1. INTRODUCTION

The U.S. Coordinated Program for Masonry Building research is an integrated set of 28 specific research tasks being conducted by the U.S. Technical Coordinating Committee for Masonry Research (TCCMAR/US). The U.S. program is coordinated with a parallel program in Japan to exchange information and concepts for the mutual benefit of both countries. Both programs are conducted under the auspices of the United States - Japan Natural Resources Panel on Wind and Seismic Effects.

2. PROGRAM OBJECTIVES

Primary program objectives are:

1) Development of design and criteria recommendations for limit state design of reinforced masonry buildings and components.

2) Development of a consistent experimental database on the behavior of masonry materials, components and systems.

3) Development of analytical non-linear models for research and design office use for detailed analysis, system analysis, and dynamic loads determination.

4) Improved material and subassemblage experimental procedures for obtaining masonry properties.

5) Improved masonry fabrication procedures and standards.

6) Developing an increased awareness among engineers, architects, code bodies and the public of the capabilities of reinforced masonry in all seismic zones.

7) Interfacing with standards development groups to support development of a consensus limit state standard for masonry.

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3. APPROACH

With NSF support, the Technical Coordinating Committee for Masonry Research/U.S. (TCCMAR/U.S.) was formed in February 1984. TCCMAR/U.S. was and is comprised of researchers from academic and industrial organizations who have strong backgrounds in research into the properties and characteristics of reinforced masonry materials, structural components and systems, analytical techniques, structural dynamics, building codes, and earthquake engineering. TCCMAR researchers are listed in Table 1. TCCMAR participants defined the research program conducted, the research and will analyze and interpret the results.

The U.S. research plan is a phased step-by-step program of separate, but coordinated research tasks. Emphasis is being placed upon intra-task information exchange and consultations so the experimental and analytical efforts are mutually supportive. The research tasks comprising the coordinated program are listed in Table 2.

The program includes both experimental and analytical research tasks. The experimental and analytical efforts are conducted in parallel with each becoming more complex and sophisticated as knowledge and experience accumulate. The U.S. plan is thus a "building block" approach. The program began in October 1985 and will be completed by July 1993.

4. CURRENT STATUS

The program is approximately 90% complete. Remaining work consists of completing experimental efforts on reinforcing bar bond and splicing, experimental and analytical studies of the behavior of a five story research building, development of limit state design and criteria recommendations, and final technology documentation. Research task reports which have been published are listed in Table 3.

Construction of the five story research building has begun. It is expected that it will be completed, instrumented, and checked by June 1992. Testing should be done through the summer.

Funding for the full scale research has been provided by the National Science Foundation, the Department of Energy, and the U.S. masonry industry (organizations such as the Council for Masonry Research, the Masonry Institute of America, the California-Nevada Concrete Masonry Association, and thirty regional masonry associations).

Work on Technology Documentation and Design and Criteria Recommendations will begin formally with a meeting on 10 April 1992. Plans and assignments will be made at that time. TCCMAR Category 2 researchers have done preliminary work to support design and criteria recommendations; this, and a draft of a limit-state design standard previously prepared, will be the starting point.

The Design and Criteria Recommendations project will basically consist of refining the draft limit state design standard, validation of the design standard by trial designs of buildings of different types in different seismic zones, and refinement of the standard as required.

Technology Documentation will involve the preparation of a series of monographs on key topics to clarify and summarize research findings. The documents will be published by the Federal Emergency Management Agency.

Funding for Technology Documentation and Design and Criteria Recommendations has been provided by the National Science Foundation, the U.S. Army Corps of Engineers, and the Federal Emergency Management Agency.

TABLE 1

TCCMAR RESEARCHERS

Daniel Abrams
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Atkinson-Noland & Associates
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Russell Brown
Clemson University
Clemson, SC

Robert Englekirk
Englekirk,Hart & Sabol
Los Angeles, CA

Robert Ewing
Ewing & Associates
Rancho Palos Verdes, CA

Ahmad Hamid
Drexel University
Philadelphia, PA

Mike Hammons
US Army Corps of Engineers
Vicksburg, MS

Gary Hart
Englekirk,Hart & Sabol/UCLA
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Gilbert Hegemier
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Univ. of CA-San Diego
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P.B. Shing
University of Colorado
Boulder, CO

Leonard Tulin
University of Colorado
Boulder, CO

TABLE 2

RESEARCH TASKS

SUBJECT	RESEARCHER
Grouted Hollow-Unit Clay & Concrete Masonry in Compression	R. Atkinson
Grouted Hollow-Unit Concrete Masonry Under Combined Compression & Flexure	A. Hamid
Grouted Hollow-Unit Clay Masonry Under Combined Compression & Flexure	R. Brown
Masonry Test Methods for Limit State Design[2]	R. Atkinson
Nonlinear Force-Displacement Models of Reinforced Masonry Components & Systems	R. Englekirk
Nonlinear Finite Element Models for Analysis of Reinforced Masonry	R. Ewing
Nonlinear Lumped Parameter Models for Dynamic Response Analysis of Reinforced Masonry Systems	J. Kariotis
Nonlinear Models of Floor Diaphragms	M. Porter
Nonlinear Models of Reinforced Masonry Walls Acting Out-of-Plane	R. Mayes
Experimental Study of One-Story Reinforced Masonry Shear Walls	B. Shing
Experimental Study of Two-Story Reinforced Masonry Shear Walls	R. Klingner
Experimental Study of Three-Story Reinforced Masonry Shear Walls and Development of Automated Feed-Back Test Method for Stiff Structures	G. Hegemier F. Seible
Experimental Study, Static & Dynamic, of Out-of-Plane Behavior of Reinforced Concrete Masonry Walls	A. Hamid S. Adham

SUBJECT	RESEARCHER
Experimental Study, Static & Dynamic, of Out-of-Plane Behavior of Reinforced Clay Masonry Walls	R. Mayes
Experimental Study, Static & Dynamic, of Flanged Reinforced Masonry Walls	N. Priestley
Analytical Study Based on Prior Tests, of Wall-Floor Intersections[2]	G. Hegemier
Experimental Study (Preliminary) of Reinforcing Bar Bond & Splices in Grouted Masonry	L. Tulin
Experimental Study (continuation) of Reinforcing Bar Bond and Splices in Grouted Masonry[2]	R. Atkinson M. Hammons
Experimental Study, Static & Dynamic Behavior of Scale Model Three-Story Reinforced Masonry Buildings	D. Abrams
Development of Reliability-Based Limit-State Design Methodology for Masonry Buildings	G. Hart
Development of Reliability Factors for Masonry Materials, Components, & Systems	G. Hart
Design of Five-Story Reinforced Masonry Research Building	J. Kariotis
Design and Construction of Large Specimen Research Building[2]	G. Hegemier F. Seible
Experimental Study of Nonlinear Behavior of a Five-Story Building Under Cyclic Lateral Loads[2]	G. Hegemier F. Seible
Design & Criteria Recommendation for Limit State Design and Documentation and Technology Documentation[2]	J. Noland
Overall Program Coordination[2]	J. Noland

[2]Research Tasks to be completed.

TASK REPORTS[3]

Task No.	Author/s - Title
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1.2(a)-1:	Hamid A.A., Assis, G.F., Harris, H.G., Material Models for Grouted Block Masonry, August 1988. 67 pgs.
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1.2(b)-1:	Young, J.M., Brown, R.H., Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient, May 1988. 170 pgs.
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2.1-4:	Hart, G., Sajjad, N., and Basharkhah, Inelastic Column Analysis Computer Program (INCAP, Version 1.01), March 1988.
2.1-5:	Hong, W.K., Hart, G.C., Englekirk, R.E.,Force-Deflection Evaluation and Models for University of Colorado Flexural Walls, December 1989.
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2.1-8	Hart, G., Englekirk, R., Srinivasan, M., Huang, S.C., Drag, D.J,., Seismic Performance Study, DPC Gymnasium, Elastic Time History Analysis Using SAP90, February, 1992.
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2.1-12	Hart, G., Englekirk, R., Jaw, J.W., Huang, S.C., Drag, D.J., Seismic Performance Study, 2-Story Masonry Wall-Frame Building Designed by Tentative Limit States Design Standard, February, 1992.

[3]Task Reports are available through the Earthquake Engineering Research Library, Berkley, CA, and The Earthquake Engineering National Center, SUNY, Buffalo, NY.

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7.1-1 Paulson, T., Abrams, D., Measured Inelastic Response of Reinforced Masonry Building Structures to Earthquake Motions, October, 1990. 294 pgs.

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9.1-1: Kariotis, J.C., Johnson, A.W., Design of Reinforced Masonry Research Building, September, 1987. 42 pgs.

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9.2-1: Seible, F., Report on Large Structures Testing Facilities in Japan, September 1985. 120 pgs.

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11.1-2: Status Report: U.S. Coordinated Program for Masonry Building Research, November 1988. 170 pgs.

課題Ⅳ　日米共同研究　討論

（３９）Development of Precast Seismic Structural System-Inter Report
S. Nakata, M. Teshigawara and M. Shiobara (Yamanouchi M.*)

Ｌｅｗ　（Ｑ）　柱・橋接合部でのスリップの影響をモデル上でどう取り扱っているか？

山　内（Ａ）　今回の解析では通常の取扱いを行っている。モデル化については検討中である。

住　吉（Ｑ）　Pressの研究方向は、何を最終目的としているのか？

山　内（Ａ）　接合部についてデータベースを構築する。これをもとに設計最低基準を作る予定ある。低層建物については告示として、高層（45m以上）建物については建築基準法３８条のもとでの評価・評定で使われる。

Ｌｅｗ（Ａ）　強震帯での単体構造と同等の耐震性を有することを目標としている。鉄筋フレームの施工工程との関係がある。方針としては、建築上のメリット、安全性の検討が研究の対象となる。データベースを作り、それをもとに実大モデル建物を建設し、その安全性等を実証し、最低基準の韓国として設計グループに伝える。

（４０）U.S. Japan Cooperative Research in Precast Seismic Structure Systems (PRESS) Program-Status Report
N. Priestly and M.S. Lew*

大　谷（Ｑ）　フェーズⅢの予算化はされたか？

Ｌｅｗ　（Ａ）　されている。

大　谷（Ｑ）　上下荷重をどう考えているか？

Ｌｅｗ　（Ａ）　検討中である。現在、接合部のせん断耐力については上下荷重のみ考えている。

Abrol　（Ａ）　ボンドについてはどう考えているのか？

Ｌｅｗ　（Ａ）　本当は必要ないと考えている。

山　内（Ｑ）　地震地帯の建物を対象としている？

Ｌｅｗ　（Ａ）　Zone2.4を対象としている。

Abrol　（Q）　ぱーきんぐビルに対してコストが高いが、何かコメントがあるのか？

Ｌｅｗ　（Ａ）　地震に関して問題はない。Singleスパンで出来る。梁から柱へのモーメント伝達機構を考えている。などの内容による。

大　谷（Q）　遮音性、耐火性についてどう考えているか？

Ｌｅｗ　（Ａ）　研究スコープに入っている。今後必要と思う。

大　谷（Q）　プレキャストアパートで配管系のフラッシュ等の音が高い。ぜひ考えて欲しい。

（４１）Seismic Response Control of Highway Bridge by Variable Damper
K.Kawashima*, S.Unjo and H.Shimizu

Cooper　（Q）　減衰効果を示しているが、これは計算値か実験値か?

川　島（Ａ）　計算値である。振動台実験は今年予定しており、その後実験結果のシュミレーション解析を行う。

Celebi　（Q）　振動数範囲は？

川　島（Ａ）　振動数により相対変位が重要である。３０ｃｍ／ｓのコントロールまで可能で、それ以上についてはサーボバルブの改良が必要である。

Katona　（Q）　ダンピングのない場合と比較しているが、また可変ダンパーの効果はどうか？

川　島（Ａ）　Table 2からわかるように、非制御の場合、構造物減衰として２％を仮定している。脱散減衰があるため実際はもう少し大きくなる。このため、応答値としては小さくなり、制御のある場合とない場合との差は小さくなる。

課題Ⅴ．ＩＤＮＤＲ

U.S. Federal Agency Role in the International Decade for Natural Disaster Reduction and UJST Workshops for Natural Disaster Reduction

by

Paul Kilho Park*

ABSTRACT

Under auspices of the UJST (US-Japan Science and Technology) Agreement of 1988, annual UJST workshops for natural disaster reduction have been going on since March 1990 as part of the International Decade for Natural Disaster Reduction (IDNDR) activities of the U.S. and Japan. Two workshops held have resulted in formulation of pilot projects and identification of future thrusts, such as joint database establishment and improvement of hazard mapping via Geographic Information System (GIS). The third workshop is planned at Mt. Hood, 7-11 September 1992. The first UJST panel was formed in 1991; it is the UJST Panel on Volcanic Disaster Prevention.

KEYWORDS: earthquake; GIS (Geographic Information System); hazard mapping; human health; IDNDR (International Decade for Natural Disaster Reduction); landslides; severe storms; socioeconomics; UJST (US-Japan Science and Technology Agreement); volcano.

1. INTRODUCTION

Broad interdisciplinary cooperative agreement between the two heads of the state in 1988 initiated the U.S.-Japan Science and Technology Agreement (UJST). The seven main areas of cooperation agreed on are as follows: life science, including biotechnology; information science and technology; manufacturing technology; automation and process control; global geoscience and environment; joint database development; and advanced materials, including superconductors.

Almost at the same time the 42nd UN General Assembly in December 1987 unanimously adopted a resolution designating the 1990s as the "International Decade for Natural Disaster Reduction (IDNDR)." The 44th UN General Assembly in December 1989 unanimously adopted a resolution establishing the international framework for action by the UN and proclaimed the IDNDR beginning on 1 January 1990 ending 31 December 1999.

Within the United States, in 1988, the Federal Government's IDNDR effort was vested on the Federal Coordinating Council for Science, Engineering and Technology (FCCSET), headed by the President's Science Advisor. Within the FCCSET, the Committee on Earth and Environment Sciences (CEES) established the Subcommittee for Natural Disaster Reduction (SNDR) to directly deal with and contribute to the IDNDR activities (Bergner, 1991). Concurrently, non-governmental National Committee was established, and Professor Walter R. Lynn, Dean of the University Faculty at Cornell University, is its current chairman. The FCCSET, CEES, and SNDR are headed by Drs. D. Allan Bromley, Frederick Bernthal and William H. Hooke, respectively.

In Japan, Japanese Government Headquarters for the IDNDR, headed by the Prime Minister with the Disaster Prevention Bureau of the National Land Agency, Prime Minister's Office, acting as secretariat, was established by a decision of the Cabinet in May 1989. In August 1990, Japanese National Committee for the IDNDR was organized by the private sector, which includes the academia, industry, the Japanese Red Cross Society. Professor Jiro Kondo is the Chairman of Japan National Committee for the IDNDR. Both the U.S. and Japan have had similar organizational evolution heretofore.

Vigorous national and international activities to enhance the IDNDR effort have commenced,

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especially in Japan and the U.S. The IDNDR International Conference 1990 Japan was held in Yokohama and Kagoshima, 27 September-3 October 1990. About 1300 participants from 43 countries and 16 international organizations attended the 1990 meeting. Similar turnout was repeated at the IDNDR Summit Conference on Earthquake and Natural Disaster Countermeasures 1991 Japan, Tokyo, 8 - 11 October 1991.

On 9 January 1992, Prime Minister Miyazawa and President Bush jointly announced the Global Partnership Plan of Action in Tokyo, in which the two heads of the state agreed to "cooperate to reduce natural hazards, in accordance with the International Decade for Natural Disaster Reduction and the U.S-Japan Science and Technology Agreement, by assisting developing counties to prepare disaster reduction plans, including hazard mapping, risk assessment and the establishment of joint databases;..." Much staff work is going on to follow up this accord, especially to get ready for Prime Minister Miyazawa's planned visit to the U.S. of summer, 1992.

Between the U.S. and Japan two UJST workshops for natural disaster reduction have been held. The first workshop was at Hilo and Punalu'u, Hawaii, 26 - 30 March 1990, and the second at Karuizawa, Japan, 23 - 27 September 1991; the third workshop is planned to be held at Mt. Hood Timberline Lodge, 7 - 11 September 1992. This report describes the details of the Karuizawa workshop. Both the first and second workshops have been chaired by Drs. Yukio Hagiwara, Director-General, National Research Institute for Earth Science and Disaster Prevention (NIED), Science and Technology Agency (STA), and William H. Hooke, Chairman, Subcommittee for Natural Disaster Reduction (SNDR). In Japan, Science and Technology Agency belongs to the Prime Minister's Office, as so does the National Land Agency.

Both Japan and the U.S. dispatch disaster relief teams. When a 635-year-dormant volcano, Pinatubo, exploded in 1991, both Japan and the U.S. dispatched their teams to help the Philippines cope with the devastation. Their onsight activities were highlighted at the Karuizawa workshop to understand the casual relationship on the disaster impact upon humanity. The U.S. effort was greatly enhanced by the Office of Foreign Disaster Assistance (OFDA) in the Department of State, and the Japanese effort by Japan International Cooperation Agency (JICA).

The JICA's fiscal year (FY) 1990 budget was 133 billion yen (about one billion dollars), some of which was earmarked for emergency disaster relief. The JICA is in charge of dispatching Japan Disaster Relief Team (JDR) and providing equipment and materials needed for disaster relief and restoration activities. The JDR carries out search and rescue, emergency medical care (as of August 1989, a total of 350 doctors and nurses are registered), and emergency measurers (prevention of secondary disasters) and restoration. Stock-piled relief goods are located at Narita (Japan), Singapore, Italy and Mexico. Japanese medical and rescue teams have been dispatched to Iran (earthquake) on 21 June 1990 and to the Philippines (earthquake) on 16 July 1990. The JICA also sent an engineering team for reconstruction to the Philippines (earthquake) on 16 July 1990 and to Saudi Arabia (petroleum over Gulf) on 3 March 1991.

Although the UJST workshop for natural disaster reduction continues to focus mainly on the cutting edge of understanding of the physical processes leading to natural hazards, resulting in improved prediction, especially during the IDNDR period, 1 January 1990 - 31 December 1999, we are to cooperate with all the concurrent IDNDR activities so that we contribute not only at the scientific frontier but also to facilitate other bilateral and multilateral activities. A prominent example is to cooperate with various U.S.-Japan Natural Resources (UJNR) activities especially with the UJNR Panel on Wind and Seismic Effects. Another example is that Japan-U.S. Symposium on Snow Avalanche, Landslide, Debris Flow Prediction and Control (JUSSLDPC) was held at Tsukuba, Japan, 30 September - 5 October 1991. This effort has been ongoing since 1980. The third UJST Workshop for Natural Disaster Reduction, Mt. Hood, Oregon, 7-11 September 1992, will enhance the JUSSLDPC effort by working together with them. Similarly the non-governmental U.S. National Committee for IDNDR is planning to have a U.S.-Japan earthquake symposium to formulate pilot projects between the two countries in the near future, to be carried out by non-governmental

resources. It is a commendable effort, and when there occurs much mutually enhancing confluence between the non-governmental organization (NGO) effort and government-to-government bilateral pilot project formulation (such as via UJNR and UJST), the true spirit of the IDNDR cooperation blossoms.

I especially wish to single out that there are very active ongoing bilateral disaster-mitigation activities under the umbrella of U.S. - Japan Natural Resources (UJNR) accord for over 20 years. The original agreement was signed off by Japanese Foreign Minister and U.S. Secretary of State. Two significant panels within UJNR for disaster mitigation are that of earthquake and severe storm mitigation technology as well as forest fire. Furthermore, at working level the two countries' Geological Surveys continue to have a functioning sister relationship; there also is much cooperation between Japan Land Agency, Ministry of Construction and the U.S. Federal Emergency Management Agency. All of these ongoing activities will be enhanced by the new thrust the UJST pursues in consort with IDNDR in the 1990s.

2. THE FIRST UJST WORKSHOP, 26-30 MARCH 1990

Under the auspices of the Earth Science and Environmental Task Group of the UJST Agreement, the first workshop had a running start from initial planning in January 1990 to its execution in March 1990. The workshop was effective and successful due to the high priority attached to the work by both sides and the desire to make a substantive start. Unsung heros include Chris Newhall, Will Prescott, and Dick Janda of USGS and Shoichiro Katayama, Yukio Hagiwara and Masayuki Watanabe of Japan. Sixteen Japanese and Nineteen Americans participated in the workshop. This workshop provided an initial opportunity to define cooperative activities in disaster reduction with emphasis on knowledge, and described specific U.S.-Japan projects that complement the ongoing activities. Three broad goals thus generated are as follows:

1. Better understanding of the physical processes leading to natural hazards, resulting in improved prediction.
2. Translation of scientific advance into improvement in the human condition, through better application of scientific knowledge to disaster reduction.
3. Better scientific understanding of the effect of global change on the frequency and severity of natural hazards.

The workshop developed specific recommendations for cooperative projects in the following four specialist groups:

Earthquake prediction
Volcanic eruption prediction
Typhoon prediction
Landslide prediction

The final report of the workshop has been reprinted by the Science and Technology Agency of Japan; I shall be delighted to supply you it upon request.

3. THE SECOND UJST WORKSHOP 23-27 SEPTEMBER 1991

Formulation of cooperative pilot projects attainable by 31 December 1999, the end of the IDNDR, was one of the major objectives for the second workshop. Also an added-on objective was the establishment of joint database for disaster reduction in cooperation with the Joint Database Development Task Group of the UJST Agreement. Thirty-six Japanese and seventeen U.S. scientists participated in the workshop at Karuizawa.

To flesh out the second goal of the general recommendations generated by the first workshop, which is "translation of scientific advance into improvement in the human condition, through better application of scientific knowledge to disaster reduction", the second workshop added two new groups of socioeconomics and human health, in addition to earthquakes, volcanoes, typhoons and landslide groups. Keynote addresses were given in socioeconomics and human health as well as on climatic change and its impact on natural disasters that requires innovative extensions to current natural disaster research. Furthermore, the typhoon group was renamed "severe weather

phenomena" group to include mecoscale storms as well as locally heavy rainfall. Four comprehensive recommendations were generated by the workshop. They are as follows:

1. Better understanding of the physical processes leading to natural hazards, resulting in improved prediction. Better understanding of the opportunities for reducing natural disasters through socioeconomic and medical sciences.
2. Translation of advances in science and technology into improvement in the human condition through exchange of information and better application of knowledge to disaster reduction.
3. Better scientific understanding of the effect of global change on the frequency and intensity of natural hazards.
4. To enhance progress toward [1]-[3] above, endeavor to develop joint databases and improve capabilities for information exchange.

During the intercessional period between April 1990 and August 1991, several prominent natural disasters have occurred (earthquakes in Iran and the Philippine in 1990; volcanic eruptions at Mt. Unzen in Japan and Mt. Pinatubo in the Philippines in 1991, and a cyclone disaster in Bangladesh in 1991). Both the U.S. and Japan dispatched medical, rescue, and engineering teams for reconstruction for the Iranian and Philippines disasters. Mt. Pinatubo volcano eruption was scientifically studied by both countries; thus resulting in having a special session to compare the scientific understanding of the volcanic eruptions at Mt. Unzen and Mt. Pinatubo at the second workshop. Though painful each natural disaster is a gigantic natural laboratory for us to understand the physical processes leading to the disaster and during the disaster, and post-disaster processes.

Also during the intercessional period, the UJST Agreement has established "volcanic panel" to specialize in volcanic hazard mitigation. The panel will start its own independent deliberation in near future; at the same time it will continue to contribute to the forum such as the joint database establishment, hazard mapping through geographical information system (GIS), and advancing its close tie with landslide and earthquake prediction groups. Furthermore, snow avalanche and landslide group too plans to establish its own UJST panel to sharpen its focus on its respective specialty; it will be a cumulative product of the group's U.S.-Japan cooperation that began in 1980.

To facilitate the second workshop we issued a 411-page preprint collection at the workshop, so that the workshop does not become mere a scientific symposium where the bulk of time was spent on oral presentations. We plan to follow similar format to facilitate the third workshop, Mt. Hood, Oregon, 7-11 September 1992. The workshop preprint collection has been reprinted by the Science and Technology Agency of Japan (Japan international Science and Technology Exchange Center, 1992).

Each individual group's reports are summarized and given in Section 5. Pilot projects recommended are listed and explained in the workshop proceedings. The workshop recommends that UJST secure necessary funds to carry out the pilot projects, so that our effort does not become a mere paper exercise. For example, two projects have given the magnitude of the fund needed to carry out the pilot projects; they are a cultural and cross-hazard study for earthquake and typhoon (hurricane) in Japan and the U.S., and the Tsunami Inundation Modeling Exchange (TIME) project at Tohoku University. As Goethe said: "Until one is committed, there is hesitancy, the chance to draw back, always ineffectiveness, ... boldness has genius, power and magic in it." Political commitment has been made by the two heads of the state of Japan and the U.S. How we would implement the pilot projects the workshop generated by our governments can learn a lesson from the recent successful 7-year Kuroshio Projects Japan and China have executed under the leadership of Science and Technology Agency with timely funding.

4. THE THIRD UJST WORKSHOP 7-11 SEPTEMBER 1992

Mt. Hood Timberline Lodge in Oregon is the venue of the third workshop for natural disaster reduction; it provides a natural laboratory to study volcanic activities, snow avalanche and landslide countermeasurers. Much intercessional, preparatory activities are going on to carry out the

workshop effectively. A core of the executive committee has been established to prepare for the workshop. Drs. Richard Janda of USGS Cascades Volcano Laboratory (established in 1980 following the eruption of Mt. St. Helens) and Fred Swanson of the U.S. Forest Service in Oregon are the local co-chairmen. Professor Dennis Mileti of Colorado State University and I comprise the rest of the executive committee.

The second workshop has recommended sharing of hazard mapping techniques used and planned to be implemented as a common third workshop theme among individual groups. Geographical Information System (GIS) applied within disaster reduction and elsewhere will be scrutinized; a plan is made to invite an expert to give a keynote speech on this regard. Interestingly, the IDNDR International Conference 1990 Japan, 27 September - 30 October 1990, too pointed out that some major work must be done "to prepare hazard maps and formulate land use plan which incorporates the viewpoint of disaster prevention." On the hazard mapping an attention is being made to invite practioners to the UJST working groups.

An additional common theme to be surfaced at the third workshop is the universality of hydrologic factors that induce natural disasters. The Pinatubo volcano calamity was magnified by an onslaught of a typhoon over the volcano, which produced much volcanic ash-mud flow upon the anthrosposhere The typhoon energy was converted into damaging mud flow of phenomenal size. Snow avalanche and ice disasters also share common hydrologic factors. Receding glaciers due to global warming too can be considered an aspect of future disaster mitigation concerns.

A field trip is planned to study ecological means to combat landslides at the Andrews Experimental Forest in Oregon. It relies on soft engineering means in contract to commonly seen hard engineering practice where dams and concrete walls are built to protect human habitat. Ecological recovery at the site of Mt. St. Helens volcanic eruption too will be visited.

Since each UJST workshop for disaster reduction is a precious opportunity to advance our knowledge to mitigate natural disasters, pertinent private sector participation, especially from the academic and the user communities, is warranted so that both Japan and the U.S. go forward to make significant progress on preparedness issues.

5. PRODUCTS OF THE SECOND WORKSHOP

For the 2nd workshop two new working groups were added based on the 1st workshop recommendation (socioeconomics and human health groups). Thus the workshop spent the bulk of its time discussing progress toward the achievement of the workshop goals by the following six working groups:

Volcanic eruptions
Earthquakes
Severe weather phenomena
Landslides
Socioeconomics
Human health

In addition the workshop was charges by the UJST liaison and working levels meetings, Tokyo, 16-19 July 1991, to work on the joint database establishment between Japan and the U.S.

Three keynote addresses were given at the onset of the workshop to broaden and integrate the group deliberations. The topics covered are the casual relationship between the global change and natural hazards, socioeconomic issues in natural disaster reduction, and the role of epidemiology in natural disaster reduction.

Data management initiatives were developed by each of the working groups. In particular, participants identified the need for improved GIS capabilities.

5.1 Volcano group

The volcano group proposes 10 projects that are current, practical to start within the next few years. They are (1) volcanic hazard and risk mapping; (2) low-frequency volcanic earthquakes and volcanic tremor; (3) processes leading to volcanic eruptions; (4) quantitative characterization of eruptions in progress; (5) dynamics of dome building and related pyroclastic flows; (6) data acquisition system for volcano-genetic vibration; (7) observation and monitoring of submarine volcanoes; (8) volcanically induced melting of snow

and ice; (9) joint observation of selected Japanese and U.S. volcanoes; and (10) coordinated assistance to developing countries.

The volcano group is to become the first UJST scientific panel entitled Volcanic Disaster Prevention Panel; the new panel's charge includes dissemination and application of volcanic hazard information in addition to basic investigations of processes, both volcanic and volcano-hydrologic, contributing to volcanic disasters. The volcano group featured a special session to discuss the findings of the recent Mt. Unzen and Pinatubo eruptions. At the 3rd workshop, the volcano group will devote mainly on the substantive reports of new methods to elucidate the processes toward disaster prevention and hazard mapping by GIS.

5.2 Earthquake group

The earthquakes group covers both earthquakes and tsunamis; the group also gives recommendations in mitigation policy, for the advancement of scientific frontier must put into good use. On earthquakes, the detection of crustal movements is now greatly enhanced by space-based geodetic systems, such as Global Positioning System (GPS), Very Long Baseline Interferometry (VLBI) and Satellite Laser Ranging (SLR). Also continuous monitoring for deformation is essential to detect short-term precursors of earthquakes and volcanic eruptions, which must be coupled with evaluations of anomalies in continuous observations. Deep drilling and borehole measurements, both geophysical and geochemical, in seismically active areas is warranted to supplant the ongoing surface observations.

Tsunami is a Japanese word for destructive, large ocean waves caused by underwater earthquakes or volcanic eruption and it propagates ocean-wide. Much theoretical works, such as numerical models for forecasting fare-field tsunami as well as the Tsunami Inundation Modeling Exchange (TIME) project, which Japan has been developing, are recommended to be pursued in consort with both the field measurements of tsunami initial profile for submarine earthquakes and the development of common database and maps of tsunami disasters; the working group strongly endorses to establish a satellite communication system, both voice and data channels, to transmit seismic waveform data to strategically located ground stations in an around the Pacific basin, and to use the PEACESAT system of the U.S. for this purpose.

To mitigate the loss of life and property from natural disasters by the judicious use of scientific knowledge emerging, a comparison of current mitigation policy and programs between Japan and U.S. is recommended to be carried out in the areas of (1) education and awareness; (2) preparedness and response; (3) seismic construction provisions for new and existing buildings; and (4) land-use planning practice. The earthquake group strongly feels that both Japan and the U.S. serve the world community through this bilateral agreement by joint research and monitoring activities in the third world countries, upholding the spirit of the International Decade for Natural Disaster Reduction (IDNDR), 1990-1999, transcending national boundaries, for big earthquakes happen at many places in the world and all of these events must be scrutinized scientifically and their impacts upon humanity minimized.

5.3 Severe storms group

The typhoon group was renamed "Severe Weather Phenomena Including Typhoon and Local Heavy Rains Group" (Severe storms group from now on). The group now encompasses severe local storms including heavy rainfall, floods, and hydrological aspects as well as disaster preparedness. Hydrological studies interlink volcano, landslide, and severe storms in reference to physical process studies. For instance when Mt. Pinatubo erupted a typhoon passed over, thus causing much damages to the communities nearby by volcanic ash flow. The predictive skill of the typhoon is gradually improving on the basis of the advancement in observing systems, data processing systems and numerical prediction models. Further improvement will be accelerated by cooperation between two countries through exchange of technical information in numerical modeling, ongoing and future satellite observations including new measurement missions of the TRMM (Tropical Rainfall Measurement Mission). Basic typhoon research projects proposed includes the development of diagnostics schemes for determining horizontal wind distribution in tropical cyclone intensity; understanding the mechanisms of typhoon movement in relation to asymmetric typhoon structure and environmentally large-scale

flow, and developing a more sophisticated convention parameterization scheme for moist convection. The possibility of a change in the numbers and intensities of tropical cyclones due to global climate change is of great concern to many coastal and sea-going nations. Research should be undertaken to investigate the effect of global climate change on tropical cyclone activity using a sophisticated numerical model.

Severe local storms including heavy rainfall and floods are affecting both countries continually. On account of recent technological advances in atmospheric and hydrospheric measurements, we can now link our emerging technological knowledge to assess and mitigate hazardous hydrologic disasters such as floods, flash floods, storm surges, heavy snow, freezing rain, lightening. The group recommends that joint efforts be undertaken to incorporate new measurements by satellite remote sensing, new lightening detection system, high-resolution rain-gauge networks, and polarization measurements of hydrometer particle electric fields. They greatly help improve hydrometerological model to better prediction of severe local storms and floods.

As other working groups recommend, the severe storms group considers disaster preparedness issue essential, continual and evolutionary; the public use of and reaction to severe storms warning must be timely and effective. The group as a first step in the continuing dialogue and cooperation between Japan and U.S. recommended a meeting of severe weather experts in Norman, Oklahoma, during the STORM-Fronts Experiment Systems Test (STORM-FEST), 1 February - 15 March 1992, to observe and work with the novel instrumentation and experimental systems deployed, which include NEXRAD Doppler radars, profiler network, new automated surface observation network, research aircraft and special soundings. This meeting was held during 8-10 March 1992 to plan for the 3rd UJST workshop for natural disaster reduction, Mt. Hood, Oregon, 7-11 September 1992.

5.4 Landslide group

The landslide research communities of both Japan and the U.S. have been working together since 1980. Japan is a recognized leader in research facility utilization, and the U.S has the unequalled natural laboratories. Ongoing cooperation include simulation and field observations of slope processes of snow avalanches, landslides, debris flow and stream channel processes. Physical process studies jointly undertaken are slope failures and sediment transport, rainstorms that trigger landslides and debris flow, and that associate with extensive volcanic cones.

The UJST separately supported a Japan-U.S. symposium on snow avalanche management, landslide prediction and control under the old UJST agreement at Tsukuba, Japan, 30 September -5 October 1991, where the snow avalanche and landslide specialists of the two countries exchanged their most up-to-date research efforts. Comprehensive future collaboration plans itemized in the landslide group report encompasses (1) landslide hazard assessment and information exchange assisted by experimental study of mechanism of landslide initiation and GIS (Geographic Information System) technology to overlay stability attributes; (2) landslide hazard mitigation by monitoring triggering mechanism and preparing hazard maps; (3) mechanism studies on the transformation of landslides to debris flows; (4) debris-flow hazard assessment and mitigation by better understanding of physics of flow and by preparion of hazard maps; (5) initiation of movement and mobility by snow and ice by studying its dynamics in both laboratory and field settings; (6) research on subaqueous mass movement; (7) flood hazards arising from landslide dams and sedimentation within by exchanging data on well-documented cases and educating public officials and population to long-term hazards; (8) elucidating the mechanism of earthquake-induced landslide; and (9) impact of the climatic change on landslides, including receding glaciers triggering new landslides.

5.5. Socioeconomic group

The socioeconomic group recommends the adoption of a cross-cultural approach to socioeconomic studies of natural disasters that will enhance both robustness and efficiency of our societal disaster prevention system worldwide. The group recommends a pilot project between the two countries; it is a cross-cultural and cross-hazard comparison for earthquake and typhoon disasters. Already considerable amount of data exists in both countries. Since the ultimate goals of social scientific investigations of natural disasters is to produce knowledge to enhance the adoption of effective countermeasures, such as mitigation and preparedness, by people at all levels of human aggregation, including individuals, families,

governments, and businesses, factors considered include elements of culture, experience, risk communication, conditions and perceptions that collectively form risk perception. All of these factors then are fed to the ultimate risk management for both mitigation and preparedness. The group proposes to obtain representative samples of 500 to 600 respondents via telephone interviews from four study sites, Tokyo, Nobi Plain in Aichi-ken, northern or southern California and South Carolina. A planning meeting for the pilot project in 1992 will construct the interview schedule and standardized coding procedures. The project proposed has a duration of two years.

5.6 Human health group

The human health group has had the benefit of working together in the third world countries providing medical and health reliefs following natural disasters. Japan and the U.S. have shown leadership and cooperation to improve disaster medicine preparedness and response. Sixteen recommendations prepared by the group fall under the three categories of (1) improved disaster preparedness; (2) response and operation cooperation, and; (3) disaster medicine research. Specific recommendations include the development of disaster medicine and disaster epidemiology centers at academic institutions; development of their curricula and practical textbook on disaster medicine based on the expertise and experience of the two countries; development of methods of rapid and accurate health needs assessment in disaster situations; development of standardized triage system; conducting joint workshops on focused topics, such as trauma surgery; tropical medicine, and epidemiology of disasters; cooperative research in crush syndrome and infectious diseases; joint database development; human health technology transfer to developing countries; taking a leadership role in carrying out the five priority demonstration projects recommended by the IDNDR Scientific and Technical Committee, which include improved hospital disaster planning in hurricane-prone regions and health hazard mapping. It is also important to consider the linkage between climate change and human health recognizing that climate change would alter the ecosystems of many disease-carrying vectors and agents, such as viruses, bacteria, parasites, plants, mosquitoes, snails, and increased heat-related illnesses such as heat stroke and death and injuries from coastal flooding.

6. CONCLUSIONS

Much progress has been made by the two workshops of 1990 and 1991. As the third year of IDNDR goes on now, execution, rather than planning of pilot projects; specific cooperation, rather than general; knowledge synthesis and application of emerging knowledge to prepare for and mitigate natural disasters loom and guide the two countries' joint endeavor. The third workshop at Mt. Hood, Oregon, 7-11 September 1992, will focus on hazard mapping by the use of GIS (Geographic Information System) and other means to chart various hydrologic factors, earthquakes and volcanic eruption prediction pattern to prepare for natural disasters yet to happen. Defense never rests, and the best of two countries' academic and public talent and government agencies continue to work together for the welfare of humanity; both UJNR and UJST efforts are to enhance our collective quest to reduce natural disaster damages the best way we can.

7. ACKNOWLEDGEMENTS

The bulk of my text came from the executive summary and the introduction I was asked to write by the 2nd UJST workshop to prepare its proceedings. I am responsible for interpreting and summarizing adequately the six group-deliberations the 53 workshop participants at Karuizawa laboriously developed. Mr. Keiji Doi of the Science and Technology Agency has been my constant co-worker to issue the 2nd workshop proceedings; I am most grateful.

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課題V　IDNDR

（４３）IDNDR, U.S. Federal Agency Role in IDNDR
Park, P.K.*

川　島（Q）　IDNDRのうち地震工学分野については米国は何をやろうとしているのですか？

Hays　（A）　次の３つを目標として取り組んでいます。
① 危険度評価　(Risk Assessment)
② 災害軽減及びその準備　(Mitigation & Prepardness)
③ 予知とWarningのための情報ネットワークの確立

川　島（Q）　具体的施策の記述された出版物はあるのですか？

Park　（A）　来週出版される「Reducing the Impacts of Natural Hazards, A Strategy for Nation」に記述されている。非常に沢山の試策が盛り込まれており、21世紀までかかると思われる内容となっている。

課題VI. そ　の　他

Advances in Wood Engineering and Construction Research

by

Erwin L. Schaffer[1]

ABSTRACT

The Forest Products Laboratory has served the public for nearly 80 years and will continue to serve by providing information on wood-based material properties and advice on how to apply this information to structural products. The audience is broad and our national and international cooperators are many. We welcome your requests for assistance and stand ready to provide help through our many media. The examples of research we've collaborated in over the last 10 to 15 years illustrate that we care about the efficient use of our timber resource and the world's resource, while promoting the durability, economy, and safety of wood products in construction.

KEYWORDS: construction; engineering; design stress; fire endurance; wood assembly.

1. INTRODUCTION

The Forest Service of the U.S. Department of Agriculture supports the largest, most comprehensive forestry research program in the world. The research for utilization of timber—from conversion to lumber and other forest products to recommendations for efficient use in construction—has been focused at the Forest Products Laboratory (FPL) in Madison, Wisconsin, since its establishment in 1910.

Because timber testing was recognized early as both an industrial engineering problem and an important aspect of forest management, the brand-new FPL found its fund of knowledge in engineering already well established. Researchers concentrated on the fundamental problems of determining the mechanical properties of wood and wood products. They evaluated the effects of fire, drying, and preservatives on mechanical properties, and prepared lumber grading rules. Efforts in characterizing the strength properties of structural timbers led to one of the earliest ASTM standards for grading structural timbers. Broadening of the species base in wood utilization by FPL also had a strong and favorable impact on national forest management.

Researchers in the 1920s developed the equipment for testing large structural members, expanding FPL's ability to develop design data for wooden columns used in buildings, bridges, and other such structures.

Large-scale construction research was successful in the design and economics of the laminated arch in the 1930s. Exhaustive screening of appropriate fasteners and joint configurations provided the basis for modern industry standards. The broad public need for information on the engineering properties of wood and FPL's wealth of knowledge complemented each other in the publication of the first edition of the *Wood Handbook* in 1935.

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Expanded fire-testing facilities and the development of the "stressed-skin" or sandwich panel principle, prefabricated wall units, and improved moisture barriers were also part of FPL's contribution in the 1930s. When World War II broke out, FPL engineers sought improved materials and construction for aircraft—first assessing alternate solid-wood species, then experimenting with the Lab's "sandwich" construction. Laminated product strength testing helped fill the need for ship timbers and other military structures.

After the war, renewed research by FPL in housing showed that lighter framing could conserve wood and still meet performance standards. Those principles were supported by a definitive book on construction techniques for the homebuilder. Engineers at FPL developed a series of plans for more affordable housing and studied earthquake and tornado damage to better understand a building's response to natural forces.

As homeowners of the 1970s increased their awareness of ecological concerns and the need for energy conservation, FPL responded with guidelines for thermal insulation, moisture and noise control, and renovation and remodeling techniques. The concerns of economy, strength, and speed in new construction were met in the Truss-Framed System, a prefabricated system suitable for framing houses and small commercial buildings.

In the 1980s, basic work in determining strength properties of members and systems continued, including the assessment of lumber grades and the effects of environmental conditions, loads, constructions, fasteners, and treatments. Building components are increasingly modelled and analyzed to help assess and meet performance standards. A structural test frame subjects an entire house to realistic loads that help measure the performance of materials and construction.

Computer-aided modelling of structural behavior, moisture movement,and performance of wood members under fire exposure are opening new opportunities for rational analyses of entire building systems.

2. MEETING USER'S NEEDS

The Forest Products Laboratory has evidenced the ability to meet the needs of a variety of interests. The most popular view of FPL is the traditional one for any R + D laboratory—a producer and disseminator of research results. The FPL is this but much more! We serve a multitude of "users" via many media. Publications—from highly technical reports, to summary compendiums that integrate results in major areas of research, to popular articles for the journalistic press—are a key medium, but other less obvious media are sometimes tools as well (see Appendix).

Whom do we view as "users" of our research, and how do we serve them? Let's deal with answers to these simple questions to illustrate how we relate to our "audience" and might better serve them. Our real challenge is to "package" and "facilitate" the use of our research.

We've attempted to group our national and international research-users as follows:

User group	Specific users
General public	Consumers
Industry	Companies, associations
Legislative bodies	Federal, State
Regulatory groups	Code-making agencies Federal and State agencies
Standards' writers	ASTM, ANSI, ISO, DIN
Professional/Technical groups	ASCE, CIB, RILEM
Educators	Universities
Foresters	Federal, State, Private
Research peers	

As we develop our research planning and conduct, we identify whom we expect to "serve" by conducting the work and through what media we expect to transfer the results to the user. Technology transfer, therefore, is taken seriously as a responsibility of researchers. Of the users cited, providing research results to research peers solely to build a knowledge base is currently considered the weakest use of our resources.

Based upon this variety of users, what are the typical media employed for technology transfer in addition to publications? Media are as many and varied as user groups:

Presentations: Meetings of common interest groups, which provide unique opportunities to pass the word.

Advice: Inquiries or informal meetings regarding a common concern that draw information from completed research.

Committee Participation: Incorporation of research results into appropriate standards, guides, specifications, design criteria, etc.

Cooperation: Cooperation with users who would like to adopt results but require some assistance to do so.

Translation: Interpretation or modification of research results to allow ready adoption and use by educators, consumers, legislators, code-making agencies, and regulators.

Review: Examination of proposed guidelines prepared by others for consistency with completed research.

Extrapolation/Interpolation: Evaluation and guidance on extending research results to similar applications.

Reports: An opportunity for users to interact with researchers and benefit from all stages of research—from planning to performance through review of completed work.

All of these media reflect the variety of roles that FPL researchers satisfy appropriate to the need.

The FPL is a problem-solving R + D laboratory. The above media and users reflect the intense effort to generate useful information. We see ourselves as advisors only. We have no regulator or standards-writing authority, but we will provide assistance to those who have such authority. A typical example is our advisory function to the National Forest Products Association on design procedures, criteria, and procedures for setting allowable properties for structural lumber and fastenings that lead to the National Design Specification.

3. CODE IMPACTING RESEARCH

The Forest Products Laboratory, as a member of the wood resource community, has a long history of providing guidance, solving problems, and generating new concepts and products that have had an impact on codes and standards. Some wood engineering research and technology transfer activity over the last 10 years should be of special interest to this Pacific Rim Group. Following are examples of activities that have resulted in useful products, which may not be familiar to you. We hope that briefly describing the research and results will be informative as well as useful.

3.1 Truss-Framed System

The Truss-Framed System (TFS) is an innovative new building system for residential and light commercial construction. The system offers substantial savings in both materials costs and construction time, with improvement in structural durability and strength. The idea for TFS was conceived at FPL in the late

1970s to meet the need for less costly, high quality home construction.

The system's key structural component is a unitized floor-roof truss, which is joined together by regular wall studs and spaced at 24-in. (610-mm) intervals. The system is a marriage of factory and site-built construction. Trusses are fabricated in a plant under quality conditions and delivered prebuilt to the construction site, where they are erected by a small crew or with light mechanical equipment such as a crane or forklift (Fig. 1). The entire unitized frame is constructed from nominal 2- by 4-in. (38- by 89-mm) dimension lumber rather than the more expensive nominal 2- by 8-in. (38- by 191-mm) or 2- by 10-in. (38- by 184-mm) lumber used in conventionally designed homes (Fig. 2). Because the trusses can be designed to span the width of most homes, supports are not needed in the basement and load-bearing walls are not necessary on the first floor. Once connected with sheathing, the structurally engineered trusses create a strong and durable wood frame building (Fig. 3).

Truss framing was developed after field observations of building damage from high winds or earthquakes showed that roof-to-wall or wall-to-floor joints often failed before the structural members. These observations were confirmed by FPL full-scale house tests. Since the unitized frame assures structural continuity between the critical joints, TFS buildings are often sturdier than those constructed through conventional means.

The innovative system also requires about 30 percent less structural framing lumber than the conventional stick-built home. Because less on-site effort is needed to frame a structure, the average size home can usually be erected in a few hours. As a result, builders have reported savings of about 10 percent of the overall costs by using the TFS building technique.

Since January 1982, U.S. builders have used TFS to construct over 2,000 homes, apartments, and office buildings in 31 states. The TFS has been assigned a public patent and is available to anyone who wishes to use it. A TFS Construction Manual has been published by the National Association of Home Builders Research Foundation (1983) in cooperation with the Forest Products Laboratory. The manual includes sections on the design, detailing, and construction of TFS buildings.

3.2 Calculated Fire Endurance

To improve flexibility in structural wood design, efforts continue to develop models and design procedures for determining the fire endurance ratings of wood members and assemblies. Traditionally, the fire endurance rating has been obtained by testing the assembly in an ASTM E 119 (ASTM, 1983) test furnace. These ratings are published in listings such as the Underwriters Laboratories' *Fire Resistance Directory* (1986), Gypsum Association's *Fire Resistance Design Manual* (1984), and those in the model building codes. These listed ratings are limited to the actual assembly tested. As a result, designers are limited in the type of fire rated assembly that they can use. Modifications such as adding insulation, changing member size, changing or adding interior finish, or changing the spacing between members are generally not permitted. Extrapolation or interpolation of ratings to or for similar assemblies is often questionable. In recent years, two fire endurance design procedures for wood that allow greater flexibility have gained U.S. building code acceptance. Two code-approved calculation procedures have been advanced by the National Forest Products Association to predict fire endurance. These are (1) additive calculation procedure for walls and floors, and (2) procedure for design of 1-h fire-resistive exposed wood members (nominal ≥6 in. (140 mm). In

addition, other procedures have been proposed or are being developed.

The fire endurance of a wood member or assembly depends on the performance of its protective membranes (if any), the charring of the structural wood element, and the structural capacity of the remaining uncharred portions of the structural wood elements. Models have also been proposed for fire-exposed light-frame floor assemblies, fire-exposed glued-laminated beams, and fire-resistive coatings on wood. The first two models emphasize compatibility with reliability-based design for fire exposure performance.

3.2.1 Fire-Exposed Light-Frame Floor Assemblies

Woeste and Schaffer (1981) developed a model for time-to-failure of fire-exposed unprotected wood joist floor assemblies. Replicate ASTM E 119 (ASTM, 1983) tests were run to validate appropriate values for the critical parameters. The model has been extended to floor-truss assemblies. Figures 4 and 5 show how the truss assembly is designed and how time to failure of unprotected truss assemblies theoretically varies with applied load. This model is used as part of a second-moment reliability analysis of the floor assemblies. In the future, reliability analysis will hopefully provide systematic and realistic evaluations of building fire safety.

3.2.2. Fire-Exposed Glued-Laminated Beam

A second approach to evaluating the fire endurance of a wood member is to assume the uncharred region consists of layers (Fig.6). The compressive and tensile strengths and modulus of elasticity of each layer are assumed to be fractions of the room temperature values. Using one 1.5-in. (38-mm) heated layer with reduced properties, Schaffer et al. (1986) analyzed a beam using transformed section analysis. This result was then used to calculate an equivalent zero-strength layer (δ) The δ was estimated to be 0.3 in. (7.6 mm) thick. This zero-strength layer (δ) was added to the char depth (βt) to obtain the total zero-strength layer. The rest of the member was then evaluated using room temperature property values.

Bender et al. (1985) developed a reliability-based model to predict the strength of glued-laminated beams under normal temperature conditions. Using the approach of a $\beta t + \delta$ zero-strength layer, they extended the model to include fire endurance analysis. The char rate (β) and thickness of the zero-strength layer (δ) are assumed to remain constant. The glued-laminated model uses transformed section analysis to determine the stresses within the laminates. In the fire endurance model, the char depth is increased at each time increment until the calculated stresses within the laminates exceed the corresponding tensile strength values. The critical moment permitted by lateral torsional buckling is also calculated. The results compare favorably with tests and allow both a mean time to failure and variability to be calculated. The method allows more flexibility in design for fire performance through the ability to control lumber quality to enhance fire endurance in fabrication of glued-laminated beams, and also provides a basis for reliability-based strength fire design.

3.2.3 Fire-Resistive Coatings

At this time, the fire endurance of wood members needs to be improved. The steel industry improves the fire endurance of steel members by covering them with fire-resistive materials or coatings. Fire-resistive coatings are not available for use on wood. Coatings available for wood are only designed to reduce flame spread. We evaluated the ability of fire-retardant coatings and fire-resistive coatings to improve the fire resistance of plywood

(Figs.7,8). Based on these small-scale tests (White, 1984), we developed empirical equations that can be used to determine the coating thickness needed to increase the time before a given char depth is obtained in the standard fire test. These equations can be combined with fire endurance models for a fire-exposed wood member or assembly to obtain the fire endurance rating of a fire-resistive coated wood member or assembly (White, 1986). Validation of these equations is underway.

3.3 Gypsum Wallboard Protected Wood Assemblies

The rate of heat release (HRR) is increasingly used to assess the overall contribution to fire growth of both individual materials and assemblies. Gypsum panel (0.6 in. (16 mm) thick) protected wall assemblies containing either wood members treated with fire retardants or steel members were exposed to fire using ASTM Method E 119 (1983). The purposes of the test were (1) to obtain additional information on the performance of fire-retardant-treated, wood-based structural assemblies exposed to fire and (2) to extend methods of measuring the rate of heat release in individual construction materials to full-scale structural assemblies.

The results of this study provide information about the fire performance of fire-retardant-treated wood assemblies in terms of HRR. At least 23 min elapsed from the start of the ASTM E 119 test to the time when wall assemblies containing treated wood began to contribute heat (Fig. 9). This result illustrates the effectiveness of gypsum wallboard and fire-retardant-treated studs in restricting contribution of heat from the assembly to fully developed fires within compartments. Two factors contribute to the substantial time delay in HRR: the thermal barrier effect of the gypsum wallboard and the fire-retardant treatment of the wood members. Heat contributions were in the range of up to 80 to 100 Btu/ft^2/min (15,128 to 18,910 W/m^2) near the end of the test. Average HRR values for the assemblies containing treated wood were 20 to 70 Btu/ft^2/min (3,782 to 13,237 W/m^2) over the active period of heat release.

Results of this study demonstrate that current methods used to measure HRR in individual construction materials can be extended to full-scale wall assemblies, and that protection of wood stud assemblies with gypsum wallboard can inhibit stud contribution to fully developed fires for 20 min or more.

3.4 Roof, Floor, and Wall Composite Action

Although the structural performance of a full-scale, light-frame house has not been well-understood, the low incidence of structural failures strongly indicates that the system has been at least adequate and even overbuilt. We have been developing methods to predict the structural characteristics of floor, wall, and roof systems individually. We also endeavor to describe how these components interact when assembled into a structure.

3.4.1 Floors

As a result of extensive research during the past decade, the structural performance of light-frame floors under uniform and concentrated loads is well understood. A comprehensive computer-based analysis program developed by researchers at Colorado State University (CSU) provides a model of the structural interaction of the floor joist, sheathing, and connectors. In a cooperative research project, CSU and FPL used this model to predict the performance characteristic of floors (McCutcheon et al., 1981; Vanderbilt et al., 1983). Research at FPL also showed that a different type of model can closely predict the deflection performance of floors and this model can be

easily adapted to microcomputers (McCutcheon, 1984). The modeled influence of 0.75-in. (19-mm) floor sheathing on assembly deflection as compared to 2 by 8 (38 by 191 mm) joists alone is illustrated in Fig. 10. The joists have high variability in their stiffnesses. As the sheathing bending modulus increases, the variation among joist deflections decreases. Note that much of the variability is eliminated by using a sheathing with a modulus of elasticity of 1×10^6 lb/in^2 (6.89 kPa), which is a realistic value for common floor sheathing materials. Further increases in sheathing stiffness have only a small effect. This example assumes no interaction between the joists and sheathing in the direction of the joist span. Additional work has been aimed at predicting the ultimate strength of floors (Wheat and Moody, 1984).

Most floor analysis has been aimed at predicting performance under stationary or static loads. However, some research indicates that the dynamic or impact performance of floors may often be the critical factor in consumer acceptance. Although some work in this area will soon be completed in Canada, the dynamic performance of floors needs to be better understood—particularly as new products capable of spans beyond the traditional 14- to 16-ft (4.3- to 4.9-m) lengths become more widely available.

3.4.2 Walls

Walls perform several distinct structural functions that require different types of analysis. In a racking or shear mode, walls serve to keep the structure square and prevent cascading failure of other walls ("dominoing"). Analytical methods to predict both the strength and stiffness performance of walls under shear loading take into consideration the nailing between sheathing and studs that provide shear rigidity (McCutcheon, 1985; Tuomi and McCutcheon, 1977). Recently, the significant contribution of gypsum board or "drywall" to the overall shear rigidity was quantified (Wolfe, 1983; Patton-Mallory et al., 1984). Racking loads of 0.5-in. (12.7-mm) gypsum-wallboard single-faced stud walls consistently are 1,100 lb (454 kg) at 0.5-in. (12.7-mm) deflection. Predicting and validating the combined effects of walls sheathed on both sides with dissimilar materials are proving useful to designers and code-making agencies (Patton-Mallory and McCutcheon, 1987).

One function of walls is to transmit roof and/or second-story loads to the foundation through axial loads in the studs. A second function is to resist the bending loads acting perpendicular to the surface. These two functions are satisfied by their combination in an analysis procedure developed by Oregon State University (OSU), which requires as input the properties of interior sheathings, exterior sheathings, studs, and fasteners to predict the strength and stiffness performance of light-frame wall systems. Cooperative work between OSU and FPL has yielded estimates of the effects of the major variables on wall performance (Polensek and Gromala, 1984).

Investigations of wall behavior, similar to research on floors, have primarily considered only static loads. However, dynamic loads produced by earthquake or wind gusts control wall performance in many parts of the United States (Soltis et al., 1981). Thus, new work is underway in the area of dynamic performance of walls under shear loading (e.g., Polensek, 1988).

3.4.3 Roof

The excellent performance of the residential pitched truss roof system confirms that this system adequately resists structural loads. These metal-plate-connected trusses are the most highly engineered component of the wood framing. However, design does not

take into account all the factors that influence overall structural performance of roof systems. An extensive research program at FPL has been underway to better characterize the performance of light-frame roof systems. The objective of the research is to develop a model, similar to the models used in floor and wall research, to predict the load distribution and deflection performance of roof systems (Moody, 1984).

Research on truss modeling has been updated with a modern analysis method for evaluating truss designs (Suddarth and Wolfe, 1984). Work is continuing to improve the methods of predicting the performance of trusses with metal plate connectors. Work reported by Wolfe et al. (1988) has substantially contributed to our understanding of load distribution and composite action in plywood roof sheathed metal-plate truss roof assemblies. The deflection prediction of load-carrying capacity and effect of repetitive members (trusses) on 24-in. (610-mm) centers were evaluated from five full-scale light-frame roof assemblies characteristics and compared to model predictions.

The actual deflection of the assembly is compared with that of loaded pitched trusses (3:12) alone in Fig. 11. This dramatically illustrates the benefit to stiffness of the assembly provided by the composite plywood sheathing-truss assembly compared to the trusses alone.

The distribution of load between adjacent trusses with load applied to one truss in an assembly (truss load influence matrix) is shown in Fig. 12. The National Design Specification currently sets allowed repetitive member increases for lumber joist design stress at 15 percent. This figure indicates that for load distribution from a stiff truss (truss 5) to a less stiff truss (truss 6), the repetitive load-sharing factor is 11/45 or 24 percent—substantially greater than the 15 percent now allowed for floor or roof joists! More detailed analyses and reports are in preparation. We are counting on these results to be used in the development of methodology for reliability-based design of truss roof assemblies.

3.4.4 Full Structure

To better understand the overall structural performance of the entire light-frame structure, investigators at FPL evaluated the structural performance of a conventional full-scale house during progressive stages of construction (Tuomi and McCutcheon, 1974). Wind loads and snow loads were simulated in a large structural test frame (Fig. 13). In wind-load simulation, plywood sheathing on the walls provided more than adequate racking resistance, probably far in excess of normal requirements. In fact, we were unable to really test the walls because the failures in tests first occurred in the connection between the loaded wall and the floor, and later at the connection to the foundation.

Commercially fabricated metal-plate-connected trusses on 24-in. (610-mm) centers were used for the roof system. Tests showed no evidence of distress in the roof system up to 2.5 times the design load. At this point, one member of one roof truss failed, but no collapse occurred. The trusses failed at more than three times the design load; had this been a snow load, the roof would have collapsed.

Of special interest is an assessment of three roof-to-wall plate connections in providing uplift resistance under extreme winds (Conner et al., 1987). The three nailed or nailed/lag-screw connections are shown in Fig. 14. The uplift withdrawal capacities of these connections were compared with probability distributions for occurrence likelihood of extreme winds. Figure 15 shows that 90 percent of severest wind speeds (tornadoes) fall below 157 mph, and a Type-H connection can be expected to

withstand from 110 to 180 mph. Type M appears sufficient for most windspeeds from 70 to 120 mph. Less effective is Type L. Thus, a reliable design for fasteners to withstand expected winds requires a full reliability-based analysis of fastener variability or a function of wind load. Also necessary is an assessment of duration of peak wind speeds to compensate for applying the duration of load factor to the connections.

3.5 Improved Lumber Properties and Design Procedures

Information is available for selecting construction lumber for various applications based on species and characteristics of wood. However, the assignment of strength properties covers a range of values, so selection is based on the lower end of the range. This often results in over-design for much of the material. Research has been conducted to determine the actual strength and stiffness of material as used in the field and how the duration of time loads applied affect the material.

3.5.1 Field Testing

For years, tests of small, clear wood specimens formed the basis for developing engineering stresses for lumber. However, evaluations in the 1970s indicated that this method is inadequate when lumber is used in highly engineered applications. Thus, a very extensive field testing program, called the "In-Grade Testing Program," was conducted in cooperation with representatives of the lumber industry during much of the 1980s (Green, 1983a,b). The program was completed in 1988. The major result of this extensive testing program is currently leading to new standards for assessing structural lumber properties. This will eventually lead to increased quality control and more efficient utilization of lumber when applied during the 1990s.

3.5.2 Duration of Load Adjustment Factor

Historically, a duration of load (DOL) adjustment factor has been used in design to account for a strength reduction of wood members after an elapsed time under load (Gerhards, 1977). The DOL adjustment factor (Fig. 16) is one of the primary factors used to determine allowable design stresses from clear wood strengths, and it has been obtained from bending tests on small clear specimens of only one species. The DOL adjustment factor has been extrapolated to all stress conditions, such as bending, compression, and tension, as well as to all grades, sizes, and species of structural lumber containing a wide range of strength-reducing characteristics. This method of extrapolation is being reassessed. The FPL is conducting an extensive research program in cooperation with the Forintek Corporation in Canada to establish DOL relations for lumber that include the effects of stress condition, species, grade, and size.

Results of the DOL study and of the In-Grade Testing Program will allow comprehensive redefinition of the engineering properties of lumber. The objectives of these study areas are to promote structural designs that use wood efficiently and to form a data-base for future reliability-based design codes.

3.6 Unified Behavior Model for Fasteners

Characterizing the strength of nailed and bolted joints, as previously noted, was among the early investigations conducted at FPL. T.R.C. Wilson initiated nailed joint behavior and design work in 1917. George W. Trayer was responsible for the early study on steel bolts in 1927. These and subsequent studies up to 1980, both at FPL and universities, were the basis for much of our understanding of mechanical joint behavior and for current design procedures and properties in the National Design Specifications. During the last 50 years, a

multitude of studies conducted globally have investigated one or more of the properties that affect connection behavior. Direct comparisons have been difficult because studies have usually differed in measuring more than one connection property or have been unique to the country conducting the tests. In addition, recent failures of bolted joints have raised doubts about our basic understanding of their behavior and the design criteria used.

A general "yield" theory for estimating the ultimate lateral load of nailed or bolted joint was developed by Johansen (1941) and confirmed by experiments in Europe (Aune and Patton-Mallory, 1986). The yield theory assumes the bearing capacity of a bolted or nailed connection is attained when either (1) the compressive strength of the wood beneath the bolt or nail shank is exceeded or (2) one or more yielding plastic "hinges" develop in the shank. A typical joint yield load curve as a function of length of bolt in the center member in a three-member joint is illustrated in Fig. 17. This curve is substantially more conservative for large bolts but more liberal for small bolts, as seen in Fig. 18.

The codes of several European countries (CIB 1983) have adopted allowable load-carrying capacities for bolted and nailed connections based upon this model. A series of studies have been conducted, and more are well underway, to validate yield model theory to design of joints typical in North America (e.g., Aune and Patton-Mallory, 1986; Soltis et al., 1986). Results to date continue to support the flexibility of the method and eventual endorsement of the procedure for assigning bolted and nailed joint properties and design criteria in the National Design Specification. We are grateful to European researchers for publicizing the technique and showing its attractive practical attributes.

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FIGURE CAPTIONS

Figure 1. Construction of the truss-framed system (TFS) enables a small crew of laborers to frame an average house in only a few hours.

Figure 2. Unitized truss frames are fabricated in a plant to insure maximum quality control and material efficiency.

Figure 3. A simple TFS home after completion

Figure 4. Floor-truss design subjected to test conditions of ASTM E 119. Upper chord was loaded with tanks simulating a uniform load of 55.1 lb/ft^2 (269 kg/m^2) that resulted in a combined live and dead load of 60 lb/ft^2 (293 kg/m^2). 1 ft = 0.3048 m. 1 in. = 25.4 mm.

Figure 5. Relationship of applied load to time-to-failure for floor-truss assembly of Fig. 4.

Figure 6. Illustration of charring and effective residual sections for wood beams and columns.

Figure 7. Coated plywood sheathed stud wall section for fire-resistance test.

Figure 8. Example of intumescent coating for fire-resistance enhancement before and after fire exposure.

Figure 9. Heat release rate (HRR) levels from fire-retardant-treated wood-gypsum wallboard assemblies. 1 BTU/(ft^2 · min) = 189 W/m^2.

Figure 10. Sheathing acts as a beam spanning across floor joists and reduces the variability in floor deflection caused by variation in the stiffness of the floor joists. The floor deflection under uniform load is shown for four different sheathing stiffnesses. Note that the realistic sheathing stiffness of 1 x 10^6 lb/in^2 (6.89 kPa) greatly reduces the variability.

Figure 11. Assembly influence on truss deflection for Group I 3:12 assembly. Trusses individually loaded to their design load value outside the assembly show a larger average deflection as well as greater variability in deflection than when tested as part of an assembly loaded to the same design load value.

Figure 12. Truss load influence matrix. As each truss was loaded in the assembly, loads were distributed to other truss reactions as indicated. The number in each cell is the percentage of the total applied load measured at the reactions of the truss noted on the truss reaction axis. The height of the graphic in each cell is proportional to the load carried by that truss when only the "loaded truss" is loaded directly.

Figure 13. Structural loading frame and test house with partially complete roof.

Figure 14. Joint configurations: Connection details for low, medium, and high strength joints.

Figure 15. Velocity pressure as a function of wind speed with three joint capacities superimposed. 1 mph = 0.447 m/s; 1 psf (lb/ft^2) = 0.042 kg/m^2.

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APPENDIX

Information About Publications Available from the USDA Forest Products Laboratory

The Forest Products Laboratory publishes a great number of reports and articles on various aspects of wood utilization and research. To facilitate acquisition, our reports are classified into publication lists. Periodic releases on new publications are provided during the year. Subject areas that may be of special interest are as follows:

- Biodegradation and preservation of wood
- Economics
- Energy from wood
- Finishing (exterior only)
- Fire performance of wood
- Particleboard and panel products
- Veneer and plywood
- Architects, builders, engineers, and lumber retailers

Detailed instructions for ordering publications in these areas are given in the publication list for each area. If you wish to be placed on our mailing list for one or more of the publication lists, please call or write

Current Information Office
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One Gifford Pinchot Drive
Madison, WI 53705-2398
(608) 264-5637

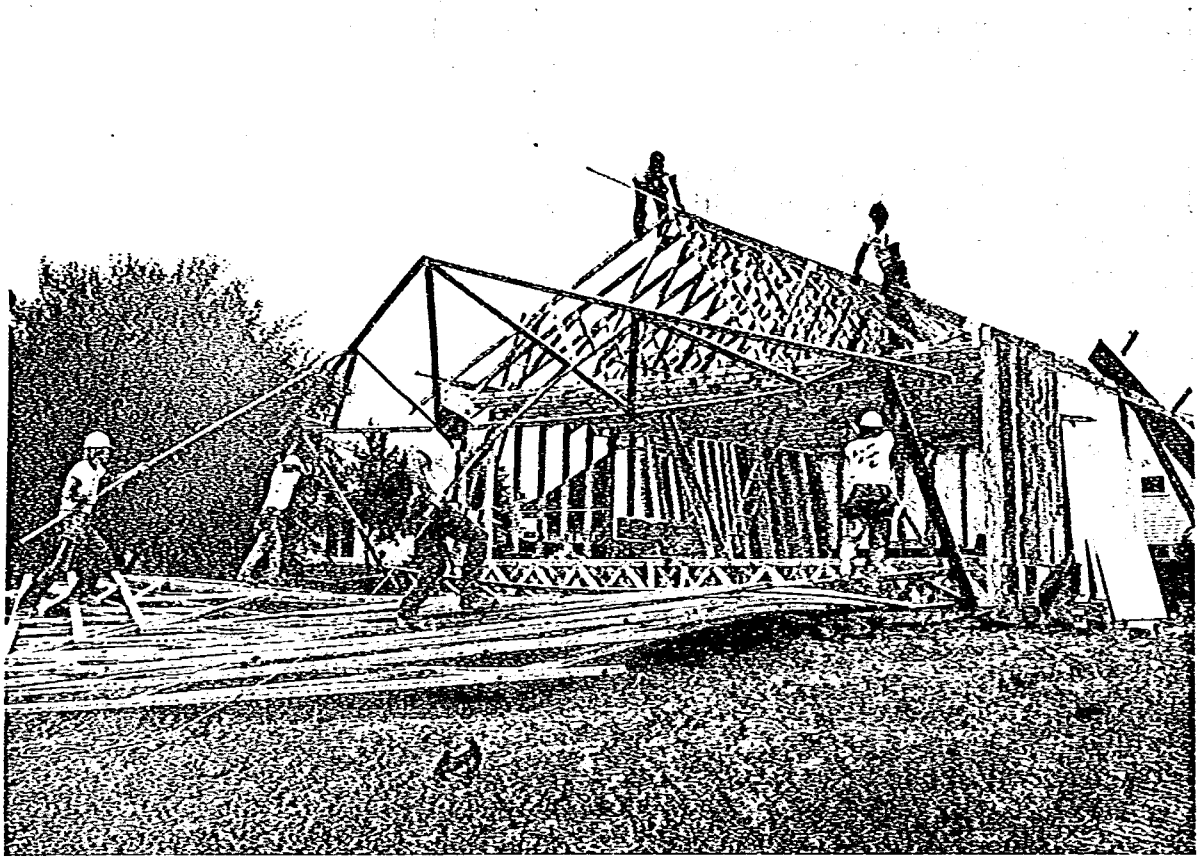


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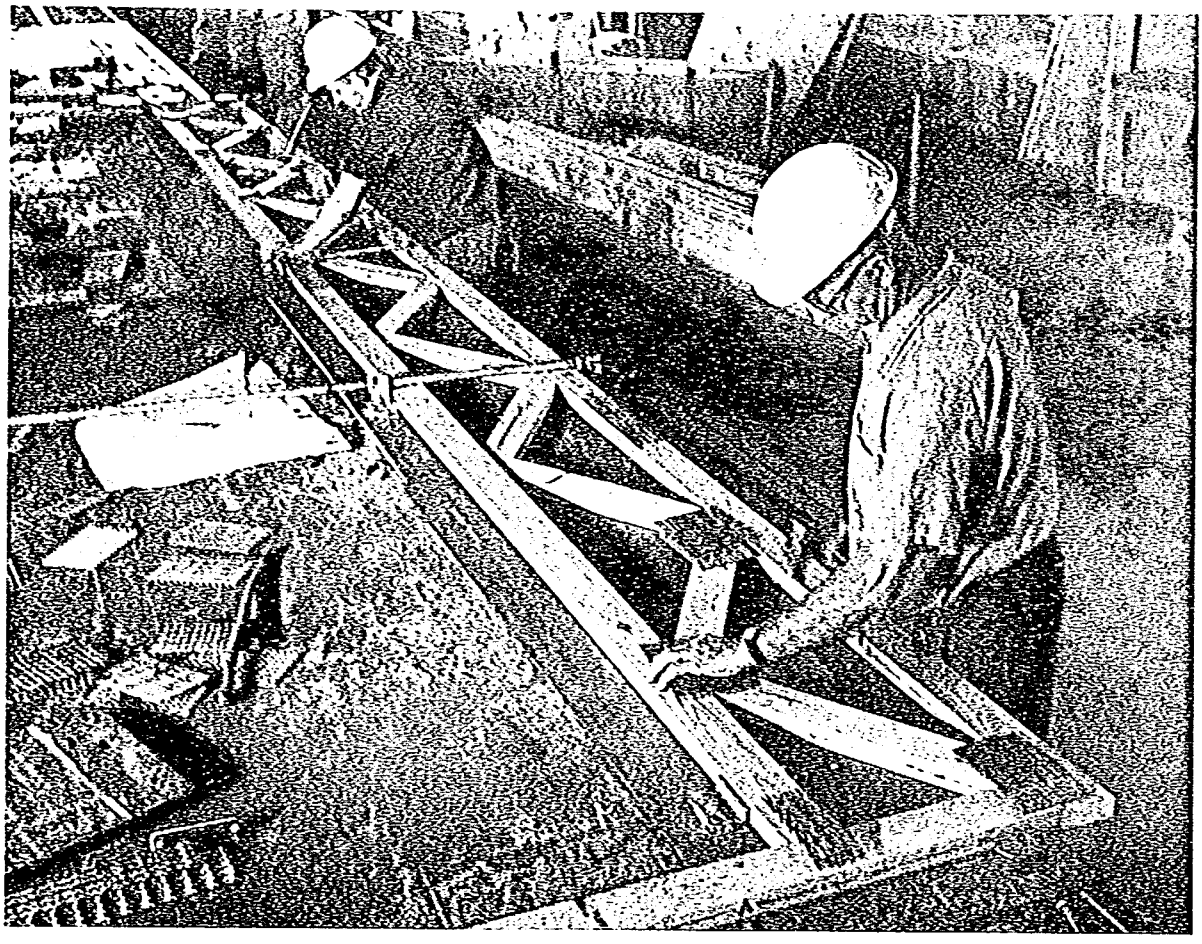


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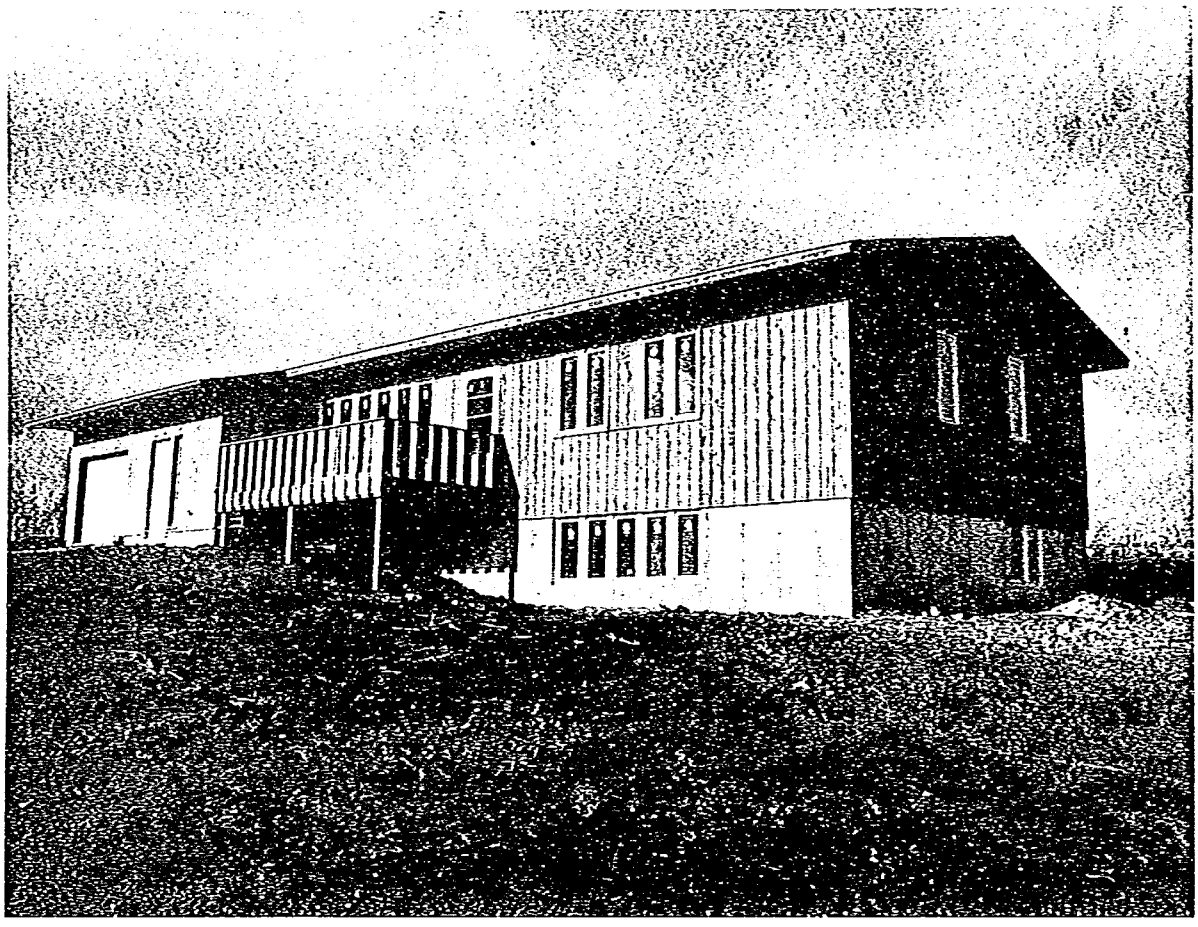


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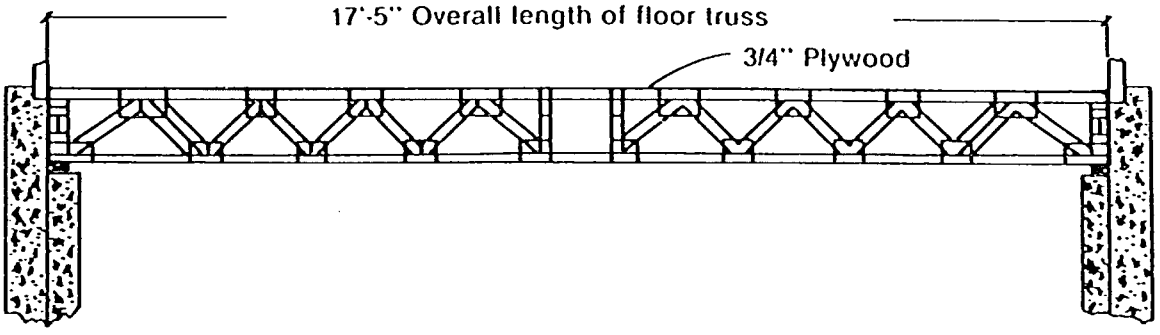
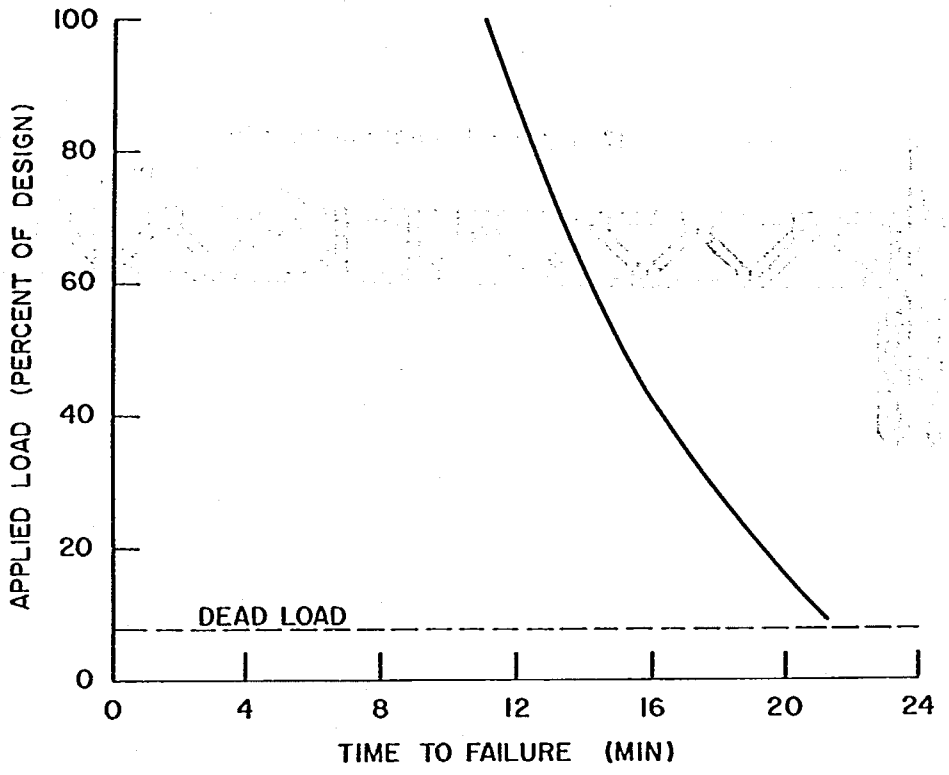


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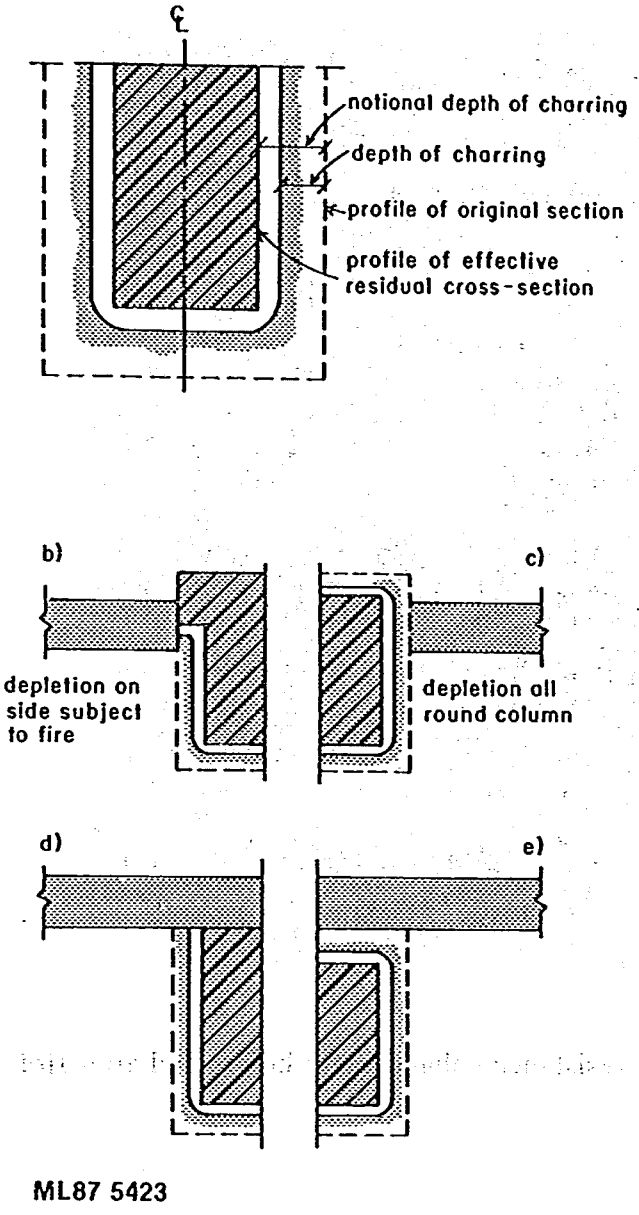


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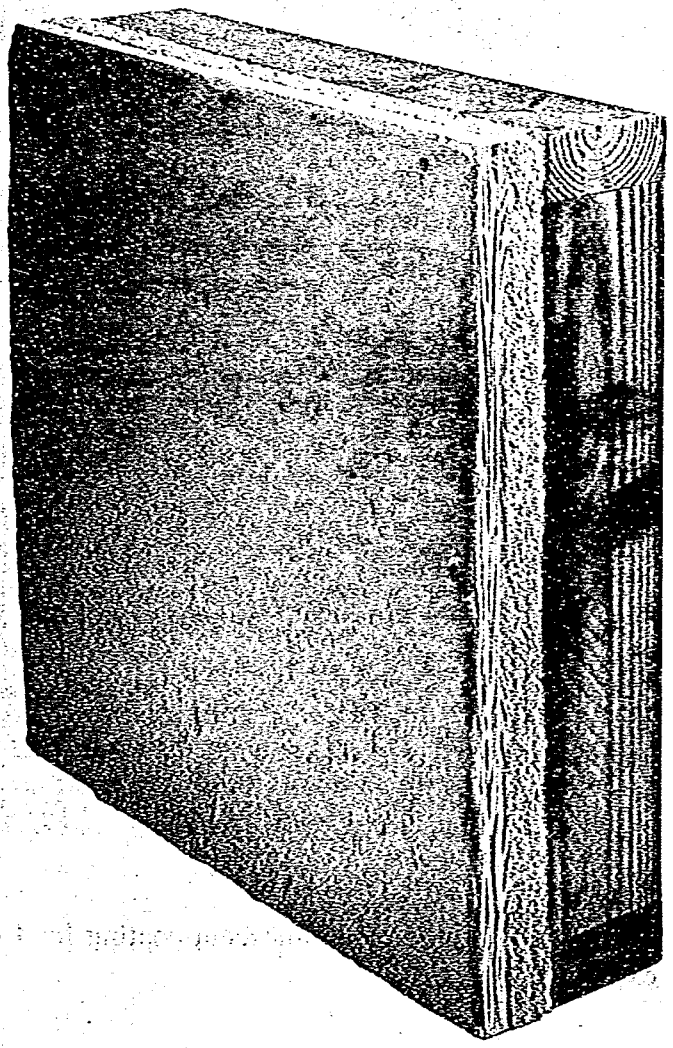


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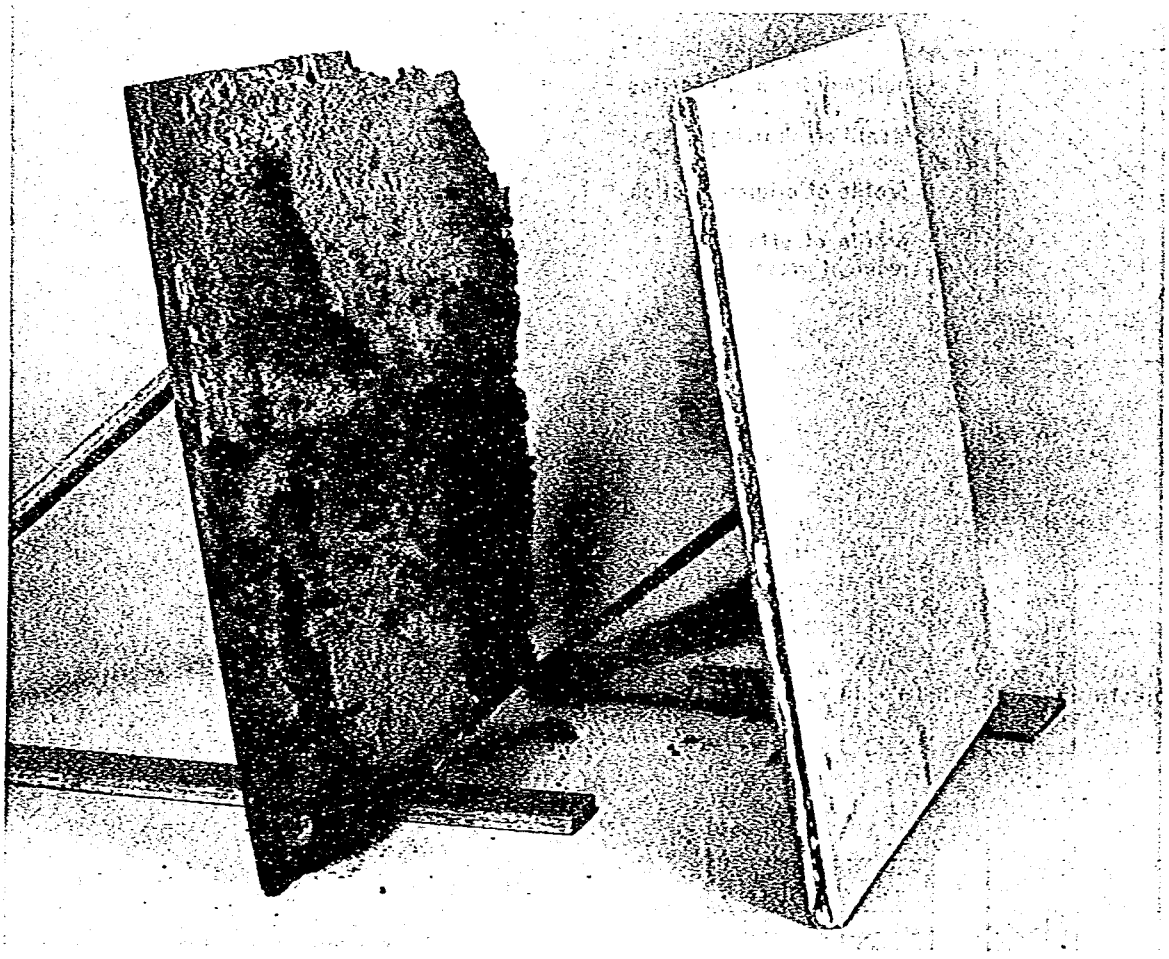


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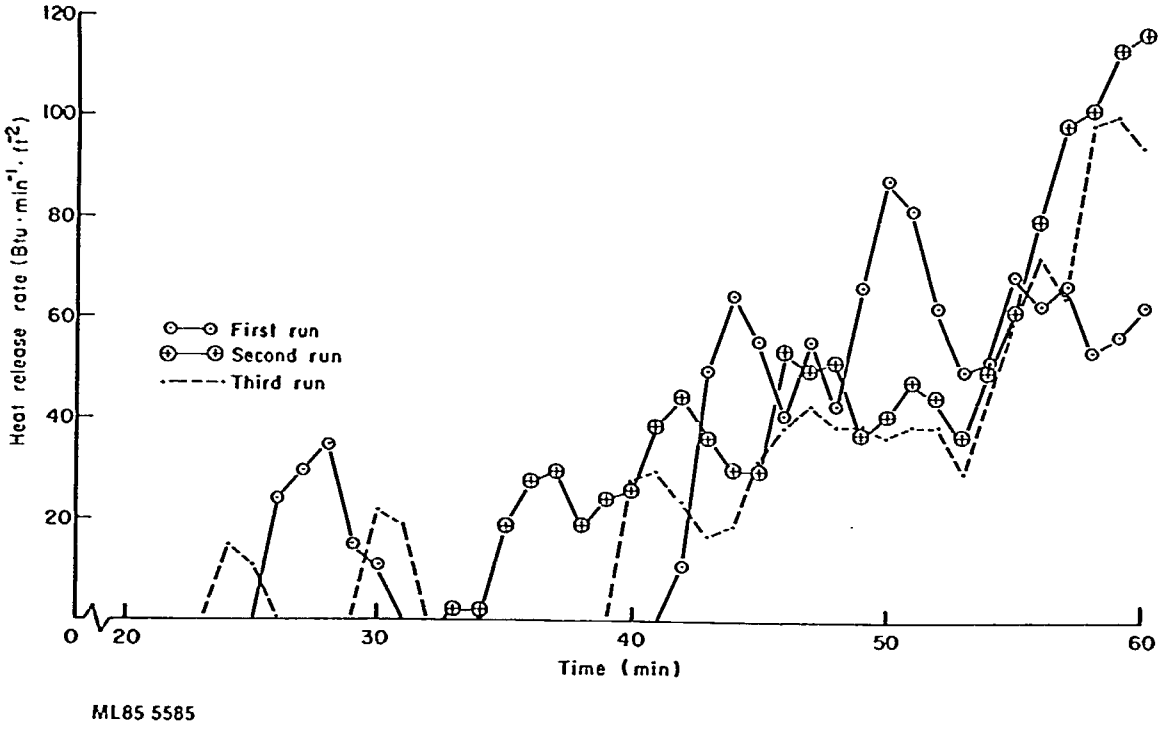


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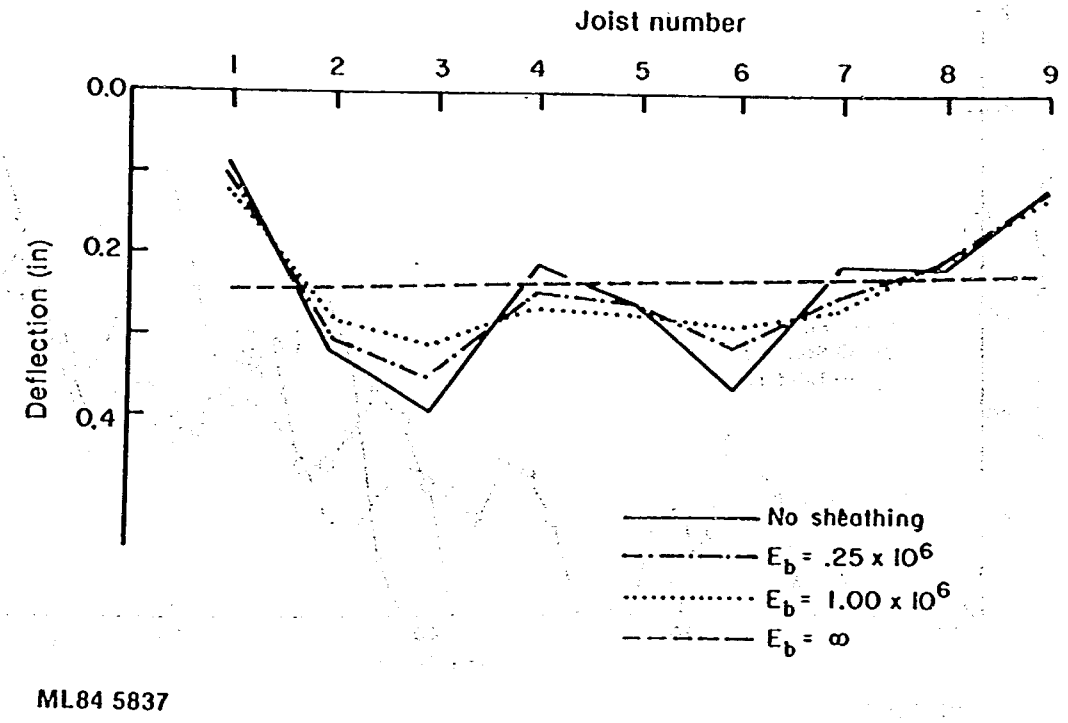


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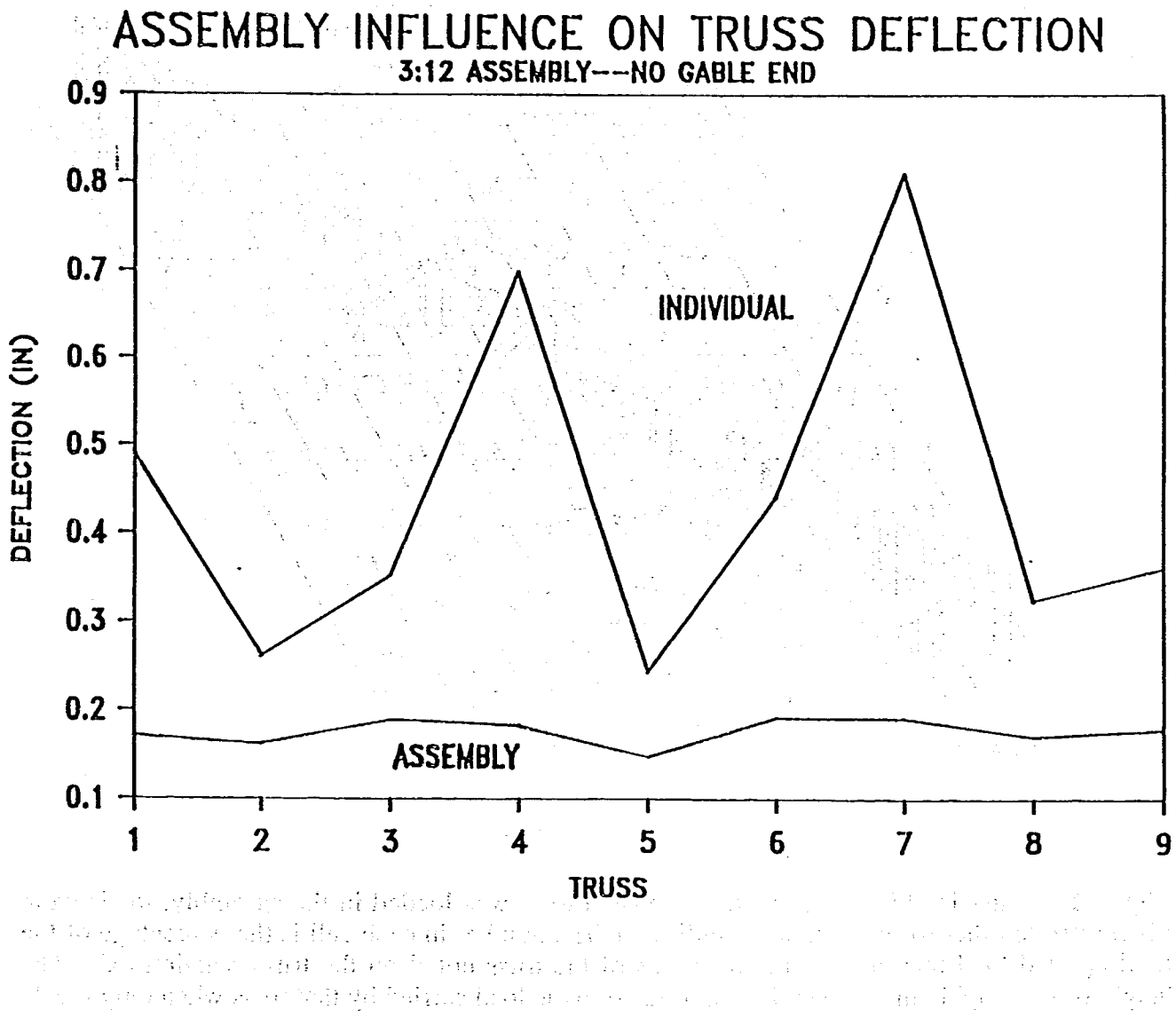


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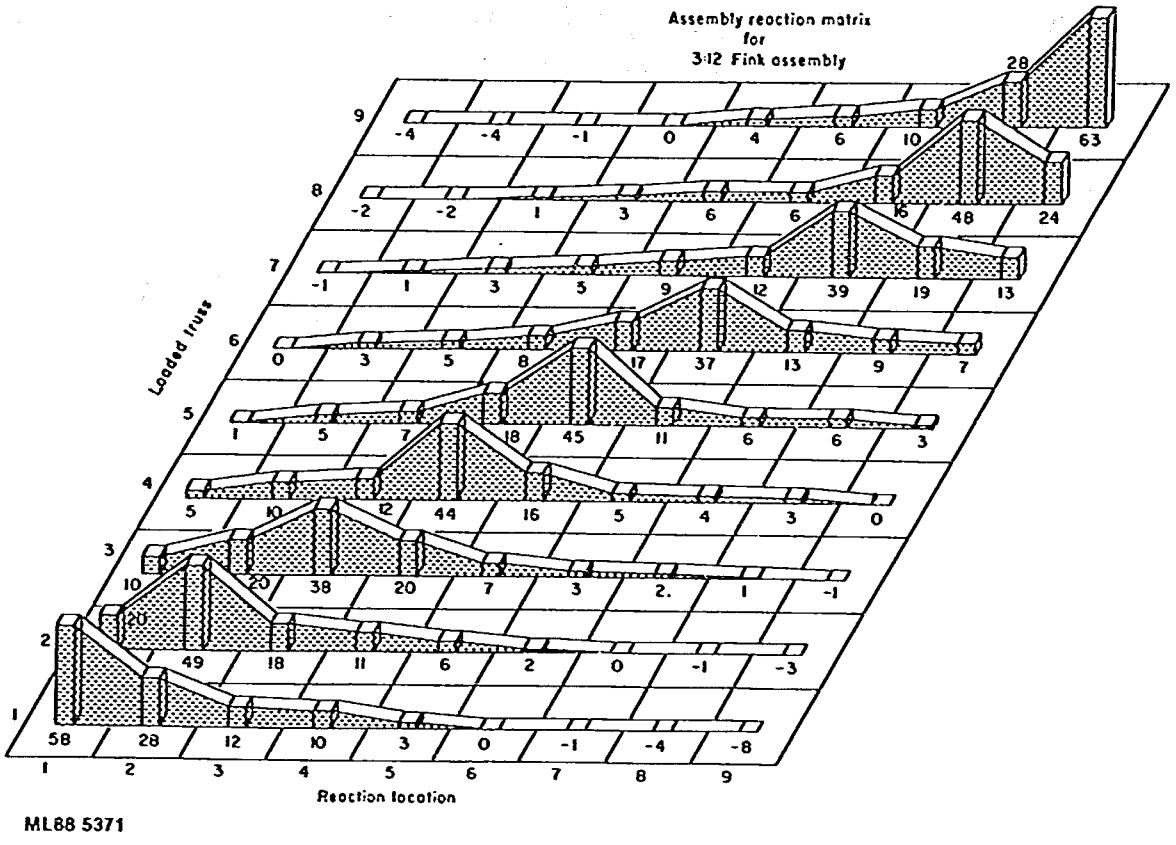


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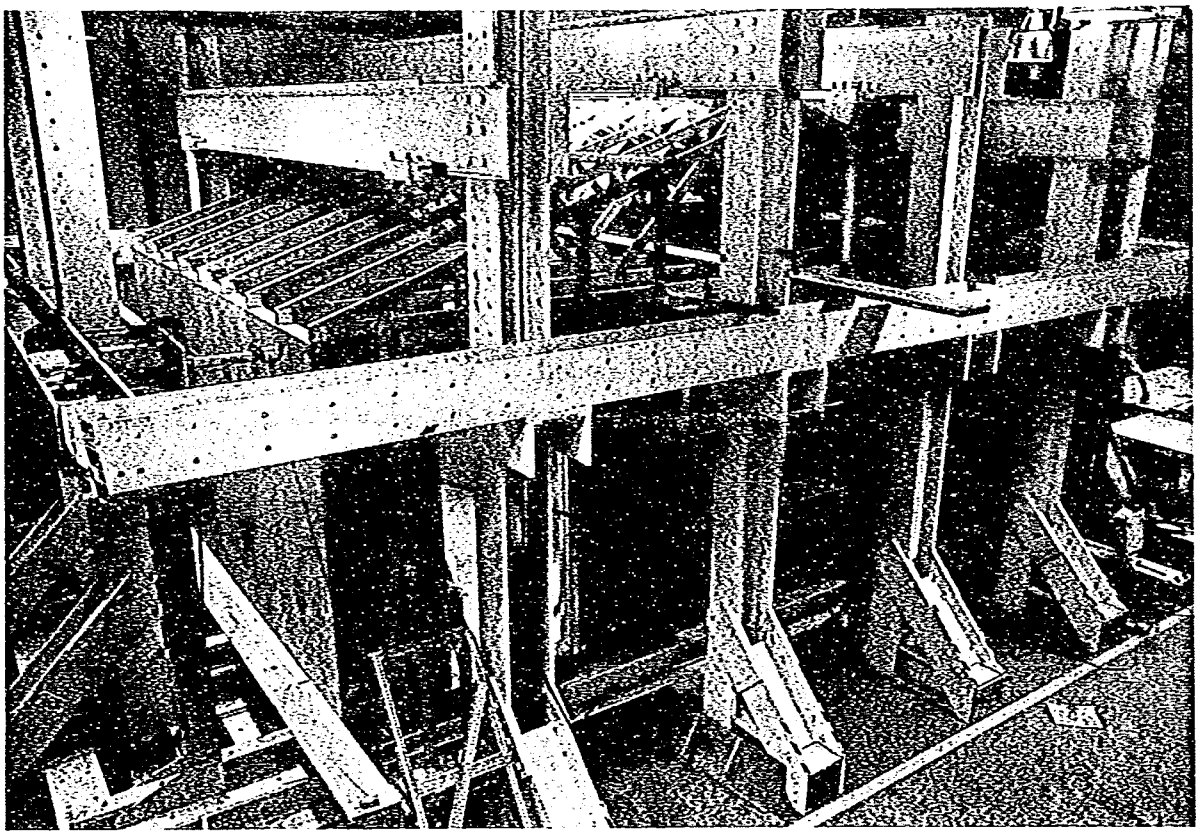


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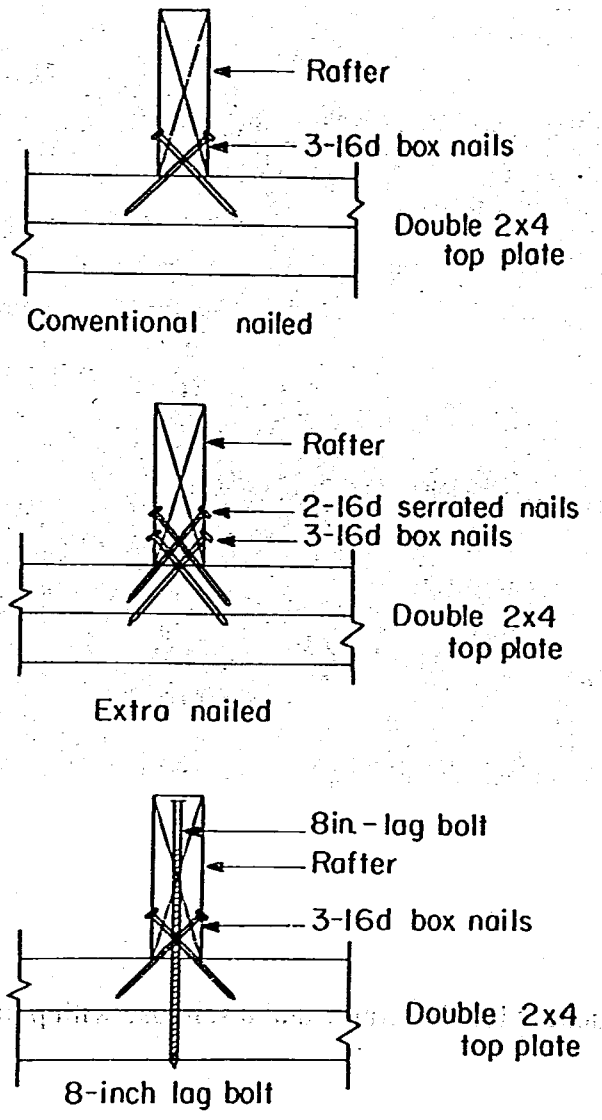


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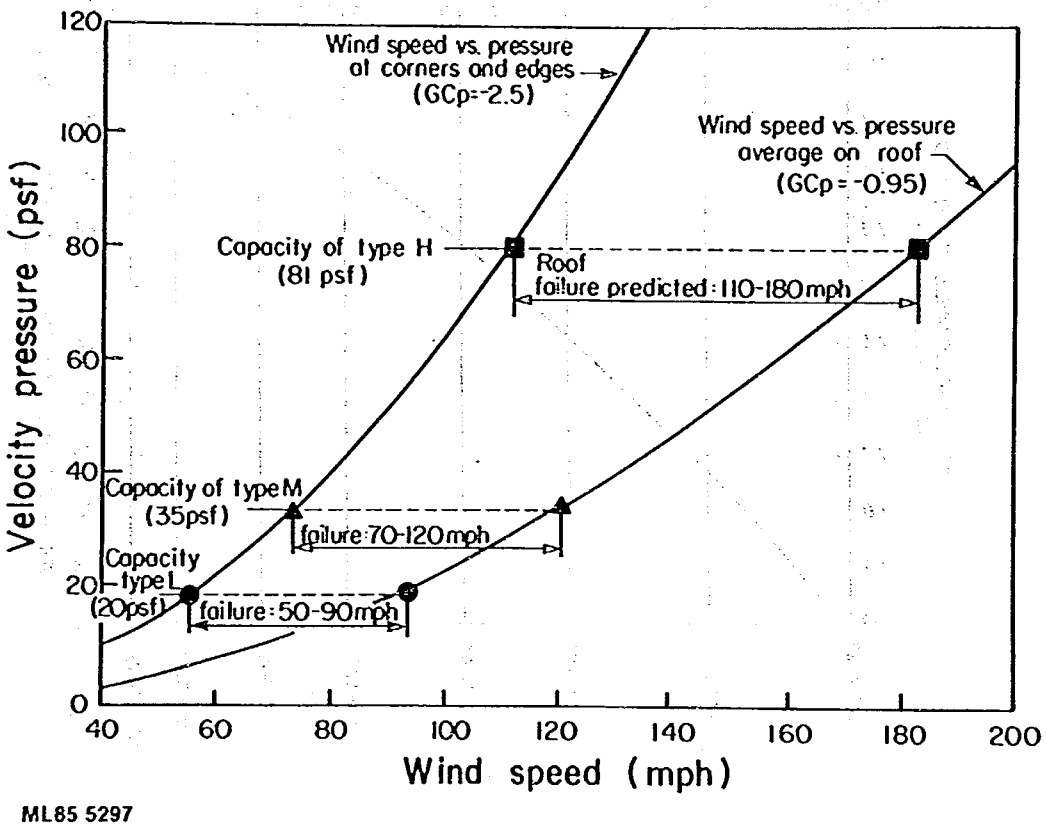


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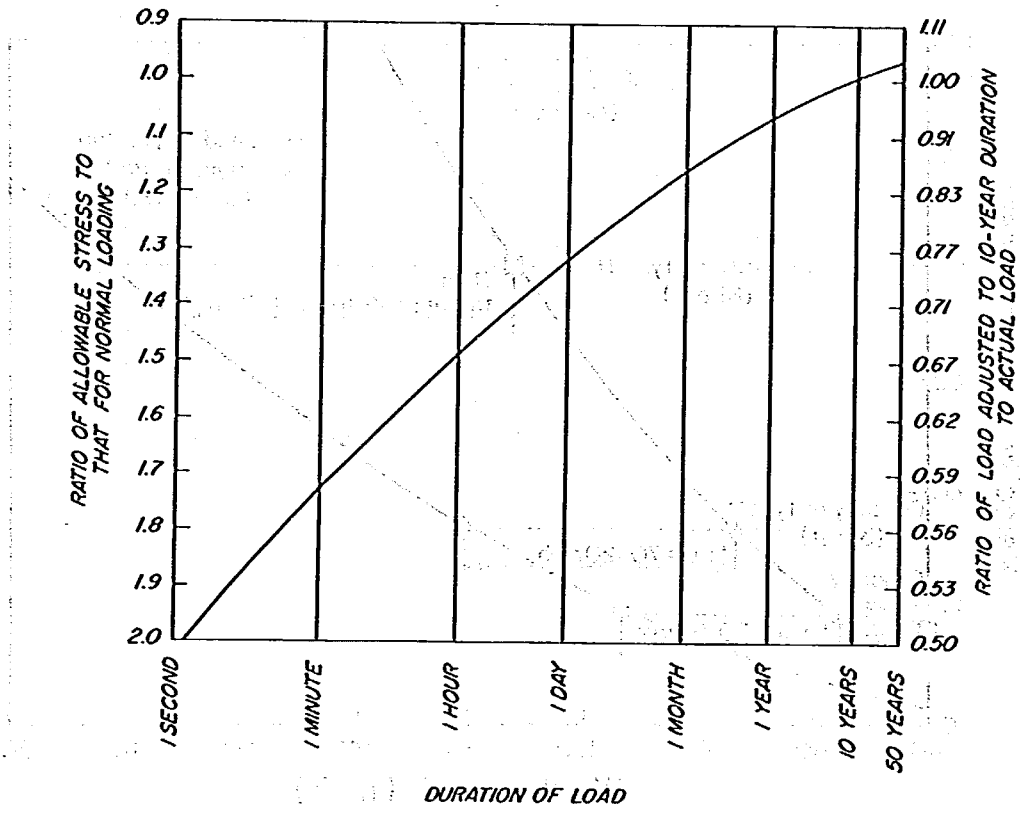


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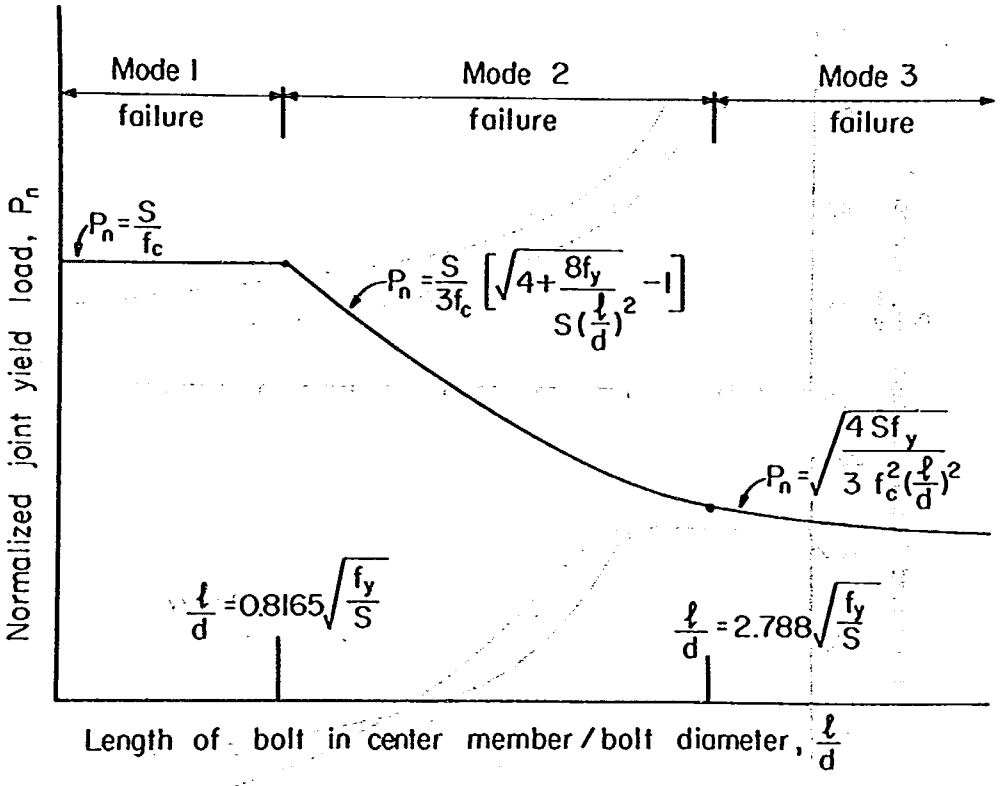


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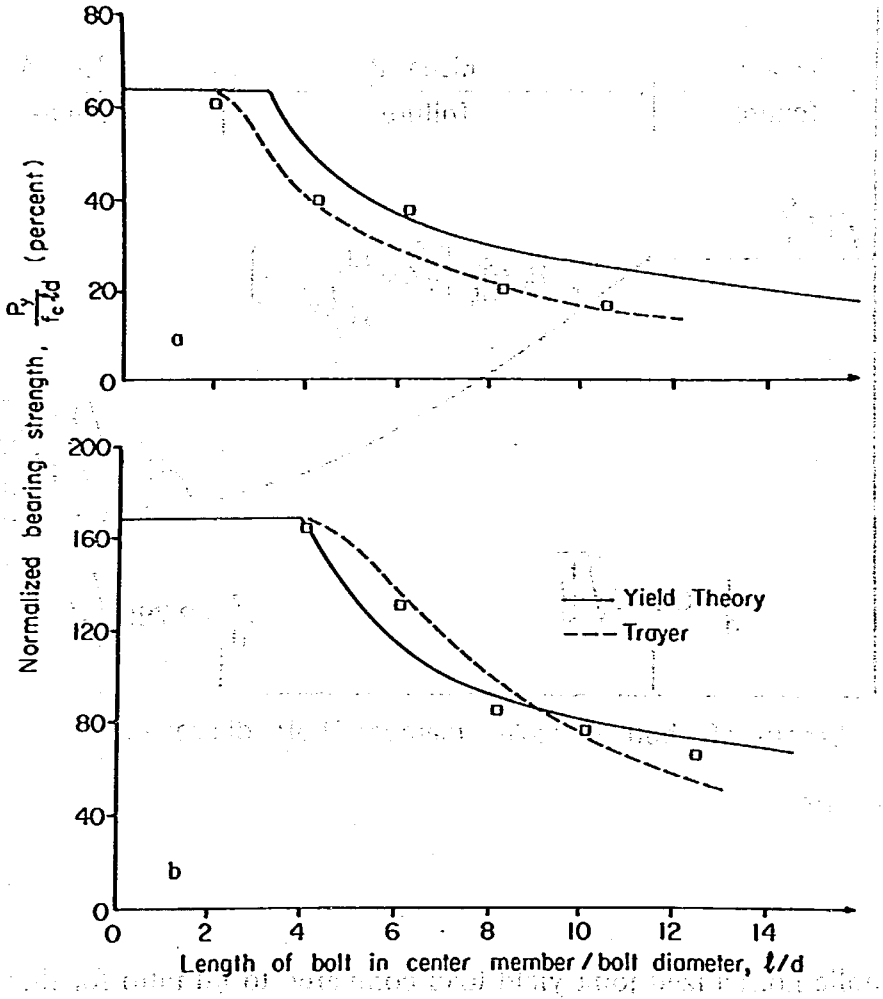


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Bridge Scour/An Element of Bridge Management

by

J. Sterling Jones[1] and Stanley R. Davis[2]

ABSTRACT

Several catastrophic bridge failures in the United States were attributed to bridge scour. These events spurred the engineering community to take a hard look at methodologies for evaluating bridges for scour. The Federal Highway Administration (FHWA) took the lead in assembling the technology and State Highway Agencies have taken various approaches for incorporating bridge scour as a major element in existing bridge management programs. A program that is being used by the state of Maryland is described in this paper as an example of how the bridge scour assessments are being handled and as a basis for projecting the magnitude of the program nationwide.

KEYWORDS: bridges; bridge management; bridge scour; hydraulics; waterways.

1. INTRODUCTION

The Federal Highway Administration (FHWA) and the State Highway Agencies in the U.S. are undertaking a major venture to assess the nation's approximately 481,000 bridges over waterways for scour. This venture was prompted in part by two catastrophic failures which resulted in several fatalities and caught the attention of safety officials, news media and legislators. One was the Schoharie Creek bridge in N.Y. which collapsed in April 1987 and caused 10 fatalities. The other was the Hatchie River bridge in Tennessee which collapsed in April 1989 and caused 8 fatalities.

These events were milestones in this nation's concept of bridge scour as a significant component of the overall bridge management program. They spurred the engineering community to take a hard look at existing methods for evaluating new and existing bridges to resist damage from scour. At that time, there was no comprehensive guidance or direction generally acceptable to the engineering community on how to address bridge scour in design and inspection.

To respond to this lack of guidance, FHWA headed up a national cooperative effort among engineers from The National Transportation Safety Board, State Highway Agencies, U.S. Geological Survey, consultant firms and universities to assemble available information and publish what later became Hydraulic Engineering Circulars 18 "Evaluating Scour at Bridges" (Richardson, et. al. 1991) and 20, "Stream Stability at Highway Structures" (LaGasse, et. al. 1991). FHWA and State Highway Agencies updated key items on the National Bridge Inventory to include more information on bridge hydraulics and scour. FHWA developed a demonstration project on underwater inspection and training courses on

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stream stability and bridge scour. FHWA then set timetables for the State Highway Agencies to assess the susceptibility of bridges to scour and to evaluate and take appropriate action for the bridges that are scour susceptible.

After some discussion with State Highway Agencies, FHWA set two target dates for completion of a bridge scour assessments. April 1992 was set as a target date for completion of a screening to determine which bridges are scour susceptible. January 1997 was set as the target date for completing the more detailed evaluations of the scour susceptible bridges to determine which ones are critical and require remedial action.

State Highway Agencies faced a major bridge management problem in implementing this program. First, scour information was not well documented in the past. Second, there were limited resources both in personnel and in funds to do the scour evaluations. Third, there was a perceived liability and accountability issue that requires a process that is consistent and supportable based on prudent and reasonable judgment. Bridge scour by its nature involves some risk, but State Highway Agencies have to balance the risk against available resources to determine which bridges are evaluated first and what detail of evaluation is appropriate.

Each State Highway Agency has developed some type of management approach that is adaptable to the information available to make prudent and reasonable judgment. This paper describes an approach that was developed by the State of Maryland and relates it to a national perspective.

2. CHARACTERIZATION OF BRIDGES

Bridges in the United States are listed in a National Bridge Inventory (NBI) which is updated by State Highway Agencies and maintained by the FHWA. Elias (1992) studied data in the NBI to establish a profile of bridges in the inventory. Based on his analysis, which reflects a December 1990 inventory, there are 577,000 bridges with approximately 481,000 (or 83%) over waterways. Of the 481,000 bridges over waterways, approximately 90,000 (19%) are actually culverts which are classified as small bridges because they span more than 20 feet.

The remaining 391,000 bridges have the following characteristics:

- Approximately 50% of the bridges in the U.S. have been built within the last 35 years
- Approximately 7% are more than 75 years old
- Approximately 40% of the bridges are simple span construction
- Approximately 40% of the bridges have Average Daily Traffic (ADT) less than 100 vehicles per day
- Approximately 28% have ADT greater than 1,000 vehicles per day.

Based on the nearly complete national screening results, approximately 25% of the bridges are being categorized as scour susceptible and will require more detailed evaluation to determine which ones are scour critical. Based on a relatively small number of the evaluations that have been completed, approximately 11% of the scour susceptible bridges are being categorized as scour critical.

The State of Maryland has 1354 Federal-aid bridges over waterways in The National Bridge Inventory. Four hundred forty of these (32%) are culverts. Two hundred fifty six (19%) of the Maryland bridges are in tidal waterways and will have deferred evaluations for three reasons: (1) there is a general lack of technology to evaluate scour in tidal areas; (2) tidal estuaries tend to dampen the velocity currents for the inland bridges, and (3) the really critical bridges on the coastal waterways are under constant surveillance anyway. Maryland considers that all of the 658 remaining bridges over waterways are scour

susceptible and require a rating. These bridges will be subjected to varying degrees of evaluations depending on the severity of the scour ratings.

3. DEVELOPMENT OF A BRIDGE SCOUR MANAGEMENT PROGRAM

The Maryland Highway Administration recognized the need to draw resources from several different offices and from consultants. A key decision in the use of consultants was that the state must retain the responsibility for assessing the risks involved in the bridge scour management program, and for deciding on the actions necessary to maintain the integrity and safety of the bridge structures.

One of the first things done was to form an interdisplinary team to provide guidance and direction to the team, and to promote coordination and cooperation among the many offices and individuals involved with the general bridge program. Responsibilities of various offices involved in the bridge scour management program were assigned as listed below:

3.1 Bridge Hydraulics Unit

- Technical expertise, guidance and direction
- Overall plan coordination
- Establish agenda for interdisciplinary team
- Prepare bridge scour evaluation reports and review scour reports prepared by others
- Prepare status reports,
- Coordinate program with FHWA, USGS, etc.
- Maintain oversight of research efforts
- Coordinate with other state DOTS, University of Maryland, etc.
- Conduct screening and prioritization studies of bridges for vulnerability to scour damage
- Rate Bridges for Item 113 of The National Bridge Inventory
- Coordinate Training Needs; Conduct In-House Training as Necessary to Meet Program Objectives

3.2 Office of Bridge Development

- Overall management decisions on program approaches and directions, coordination with FHWA and counties, etc.
- Bridge Design Division lead office for designing bridges over waterways; makes decisions on designing foundations to resist scour using guidance and recommendations from Bridge Hydraulics Unit

3.3 Bridge Inspection and Remedial Engineering

- Decisions on what actions to take and relative priorities on bridge needs
- Bridge Inspection Program
- Emergency concerns, closing of bridges, etc.
- Administering Action Plan for scour critical bridges
- Maintaining records on the National Bridge Inventory

3.4 Office of Materials and Research

- Soil borings, stream bed and bank samples, etc.
- Evaluation of foundations in rock; scourability of rock, etc.
- Other guidance on GeoTech matters

The Maryland State Highway Administation then set up two agreements to establish a data base for the program. First, and very important, was an agreement with the U.S.

Geological Survey (USGS) to have two three-man teams conduct a preliminary field survey of non-tidal bridges over waterways that were in the NBI. These crews assigned a USGS rating factor to each bridge based on the following:

- Stability of channel bed and banks
- Approach and "get away" flow conditions to bridge
- Degree of observable scour at bridge
- Susceptibility to damage from scour based on conditions observed at the site.

The USGS teams also rated NBI items 61 (channel), 71 (waterway adequacy) and special state item 321 (scour) for each bridge they surveyed. The fact that there were only two teams gave a degree of uniformity to these ratings that gives them a high weight in a ranking system.

Second, was a contract with a consultant to develop a spread sheet that could be readily managed by state engineers. The spread sheet included the following factors for each bridge:

- Traffic (ADT)
- Superstructure (continuous or simple spans)
- Foundation (spread footing, piles, etc.)
- Waterway area (Item 71)
- Channel Stability (Item 61)
- Observable Bridge Scour (Special State Item 321)
- USGS Rating Factor (from field survey)
- (SHA) Engineer Rating Factor (Based on engineer's evaluation of all available field and office data)

Numbers are assigned to each of these factors and the spread sheet priorities are updated at least monthly as additional information becomes available.

4. SCREENING AND SCOUR ASSESSMENT

The Maryland State Highway Administration uses several levels of screening and assessments to determine which bridges require a full scour evaluation. Advance screening eliminated culverts and tidal bridges for reasons cited previously. The preliminary screening process essentially assigns numbers to the spread sheet factors and ranks bridges from worst to best. The office scour assessment involves some preliminary scour computations, review of foundation information, and special circumstances for the bridge. At this stage some bridges can be categorized as scour critical and others can be categorized as low risk and can be eliminated from further assessment. The office assessment is a relatively low cost effort that requires approximately one person-day per bridge. The spread sheet rating factors, especially the (SHA) engineer rating factor, are revised after this assessment. The field and office scour assessment involves site visits by State Highway engineers, more detailed data gathering, and more precise computations. Again, the spread sheet rating factors are revised after this assessment. Some bridges then will require a full scour evaluation which typically involves taking soil borings, rerunning water surface profile models to get flow and velocity distribution, estimating expected scour depths, and determining structural integrity of bridges with partially exposed foundations. Table 1 shows the expected number of bridges and the relative costs for each process.

5. FACTORS USED TO DEVELOP THE SCOUR PRIORITY NUMBER

A central focus of Maryland's Bridge Scour Program is the spread sheet ranking system which is continuously updated after each process level and whenever new data or better technology becomes available. This system keeps the highest risk bridges at the center of

attention.

The factors used to develop a ranking are of interest and deserve some discussion. Table 2 summarized the spread sheet factors with guidelines used by Maryland for assigning numerical values that calculate a scour priority number for each bridge. Using Maryland's guidelines, the lower the scour priority number, the greater the likelihood that the bridge will be damaged by scour.

ADT reflects primarily on the value of lost time, and traffic running costs if an important bridge fails. It also is a secondary indicator of the probability of fatalities if a bridge does fail, although there are not enough data to establish any correlation between ADT and the probability of fatalities.

Superstructure type is important because continuous spans have more redundancy and less chance of catastrophic failure with little or no warning. This is also an indicator for the probability of fatalities.

Foundation type is definitely one of the most important factors. Historically, most of the scour related failures have been bridges on spread footings.

Waterway adequacy reflects the probability that pressure flow scour is likely to occur. It also may be an indicator of contraction scour. It is an available piece of data that has merit in a ranking system.

Channel stability reflects the tendency for degradation and channel migration. This factor should have a high weight because instable channels generally have more local scour and react to small channel changes.

Bridge Inspection Observations are important because they highlight the bridge inspectors identification of potential scour problem.

The USGS rating factor is especially important in Maryland because of the uniformity of that factor since the same survey crews inspected and rated all of the bridges. The same survey crews also rated the NBI items.

The SHA Engineer rating factor takes into account the accuracy of records and conservatism of engineers at various periods of time. This factor is based on an overview of all of the information in the system, and is the most dynamic factor as the screening and assessment progresses.

The "screamer" factor allows for any bridge to rise to top of the list at any stage of the assessment if anyone reports evidence that a bridge is in danger.

The scour priority number is calculated by multiplying the numerical values assigned to each of these factors. The ratings can be criticized because there is no provision for weighing factors according to importance and reliability of information. It would appear that the wide range of values for some of the less important factors, such as waterway adequacy, would make them have more weight than they are intended to have and lead to erroneous ranking. This was an initial concern with the use of the system, but subsequent testing has satisfied Maryland State Highway Administration engineers that the system, while not perfect, does a reasonable job of prioritization. This success is due to several moderating effects, including the dynamic nature of the program, the flexibility to revise numerical values as more information becomes available, and the central tendency of the NBI rating numbers.

Other State Highway Agencies have developed similar bridge scour management programs. There has been at least one attempt by Elias (1992) to consolidate the best features of the programs into one that can be promoted nationally, but that procedure is still in draft stage and has not been tested by any state.

7. SUMMARY

The Federal Highway Administration took the lead in assembling available information, developing training courses, and providing demonstration projects to deal with an accelerated program to assess the nation's bridges for scour susceptibility. The State Highway Agencies have had to integrate this program into their overall bridge management program with limited resources.

Maryland is a relatively small state with fewer bridges than the average state, but it is a sampling that reflects the magnitude of the overall scour program in the United States. If the average $4,000 per bridge cost for managing the bridge scour program is extrapolated to the total 391,000 bridges (excluding culverts) over waterways, the total cost of this program nationally will exceed $1.5 billion over the next decade.

Maryland has taken a somewhat pragmatic approach in developing its bridge scour program, placing emphasis on engineering judgment to limit the number of bridges requiring full scour evaluation. This approach, coupled with an active bridge inspection program, works for that state. The following considerations were essential to the success of their program:

- Support of top management
- Formation of an interdisciplinary team
- Knowledge of and sensitivity to concerns about scour by the engineers in the bridge management program
- Development of a plan of action to identify the tasks to be addressed
- Assistance of the FHWA and the USGS in training efforts and in developing the program
- Written directives establishing policies and rsponsibility for the scour program
- Importance of the Bridge Inspection Program in identifying and prioritizing potential scour problems
- Use of the Scour Priority Number Spreadsheet as a management tool for evaluating existing bridges

8. REFERENCES:

1. Richardson, E.V., Harrison, L.J., and Davis, S.R. (1991). FHWA Hydraulic Engineering Circular No. 18, "Evaluating Scour at Bridges," Publication No. FHWA-IP-90-017, Washington, D.C.

2. LaGasse, P.F. et. al. (1991). FHWA Hydraulic Engineering Circular No. 20, "Stream Stability at Higway Structures," Publication No. FHWA-IP-90-14, Washington, D.C.

3. Elias, Victor (1992). "Strategies for Managing Unknown Bridge Foundations," Unpublished Draft Publication No. FHWA-RD-92-030, FHWA R&D, McLean, Va. 22101.

Table 1. Maryland Bridge Scour Program

Process	Expected No. of Bridges	Estimated Cost per Bridge	Expected Cost
Preliminary Screening	658	$ 515	$ 340,000
Office Scour Assessment	658	$ 350	$ 230,000
Field & Office Scour Assessment	219 (Approx. 33%)	$ 1600	$ 350,000
Full Scour Evaluation	110 (Approx. 17%)	$ 15,000	$ 1,650,000
Grand Total			$ 2,570,000
Average Cost per Bridge			$ 4,000

Table 2. Maryland Factors used to Rate Bridges for Scour Susceptibility

Factor	Criteria and Numerical Indicators
ADT	> 30,000 veh/day = 1 5 - 30,000 veh/day = 2 < 5,000 veh/day = 3
Superstructure	simple span = 1 continuous = 2
Foundation	Spread footing, erod mat'l. = 1 Unknown = 2 Short Pile = 4 Long Pile > 20' = 6 Found on Solid Rock = 6
Waterway Adequacy NBI Item 71	1 to 9 from Inventory Data
Channel Stability NBI Item 61	1 to 9 from Inventory Data
Bridge Inspection Notes Regarding Scour Special Maryland NBI Item 321	1 to 9 from Inventory Data
USGS Rating Factor	severe = 1 moderate = 2 mild = 3
SHA Engineer Rating Factor	Preliminary = 3 Revised after Office Assessment severe = 1 moderate = 2 mild = 3
"Screamer" Factor	Overrides all others and places a bridge at the top of the list.

Ⅷ. ワークショップ報告

RESOLUTIONS OF THE FOURTH U.S.-JAPAN WORKSHOP ON EARTHQUAKE DISASTER PREVENTION FOR LIFELINE SYSTEMS

Biltmore Hotel, Los Angeles, California
August 19-21, 1991

1. The Fourth U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems was held under the sponsorship of the Public Works Research Institute of Japan and the U.S. National Science Foundation under the auspices of Task Committee F, "Disaster Prevention Methods for Lifeline Systems," UJNR Panel on Wind and Seismic Effects at the Biltmore Hotel, Los Angeles, California, United States on August 19-21, 1991.

2. The workshop provided a valuable forum to exchange information and ideas on technical and socioeconomic issues important to the lifeline earthquake engineering communities of both countries. The following topics were addressed:

   a. Soil Effects
   b. Seismic Design and Analysis: Transportation
   c. Seismic Design and Analysis: Pipelines and Tunnels
   d. Seismic Risk Analysis
   e. Post-Earthquake Response/Damage Detection
   f. Socioeconomic Impacts
   g. Future Directions for Research Application and Design.

3. The workshop identified, without priority order, the following topics relevant to lifeline systems for further cooperative research programs:

   a. Effects of Soils on Lifeline Components;

   b. Prioritization Procedures for the Seismic Inspection and Retrofit of Lifeline Structures;

   c. Use of Passive, Active, and Hybrid Control Techniques for Improving the Seismic Safety and Performance of Lifeline Structures;

   d. Dynamic Response and Design of Lifeline Structures;

   e. System Reliability Methods for Lifeline Systems;

   f. Repair and Rehabilitation of Lifeline Systems;

   g. Post-Earthquake Performance and Restoration of Lifeline Systems;

h. Post-Earthquake Damage Detection Procedures;

i. Socio-Economic and Environmental Impacts of Lifeline System Failures;

j. Emergency and Disaster Response Management of Lifeline Systems;

k. Information Dissemination on the Development of Seismic Performance Standards for Lifelines, Particularly Telecommunication Systems;

l. Use of Innovative Materials, Advanced Sensors and Intelligent Control Systems for Lifeline System Operations, Emergency Control, Damage Isolation, Replacement and Restoration;

m. Collection of Data (e.g., strong-motion response, laboratory testing and earthquake damage data) to Verify Methodologies and Models Used in Lifeline Earthquake Engineering; and

n. Lifeline Interaction Concerns, i.e., Dependency on Other Lifeline Services.

4. The workshop acknowledges current efforts in both countries to establish seismic design guidelines and standards for lifeline systems. Encouragement in the actual development of such standards should be placed on the U.S. effort. Existing UJNR channels should be fully utilized to facilitate the exchange of relevant technical information concerning standards development. Possible collaboration of joint development of lifeline system standards and guidelines should be pursued.

5. The importance of the "International Decade for Natural Disaster Reduction" (IDNDR) in its relation to lifeline earthquake engineering and disaster mitigation research under the UJNR program continues to be emphasized. Future joint activities between both countries in the area of earthquake disaster prevention for lifeline systems shall consider their contribution to the achievement of the goals of the IDNDR program. These future activities shall be defined by the Co-Chairmen of Task Committee "F" of the UJNR Panel on Wind and Seismic Effects.

6. The workshop recognizes the importance to continue exchange of research personnel, technical information, research data and use of available research facilities in both countries. The workshop further recognizes the benefit of identifying specific activities to compare the results of different design, analysis and experimental methods, and disaster mitigation strategies of both countries.

7. In view of the importance of cooperative research programs on seismic effects and earthquake disaster prevention methods on lifeline systems, it is recommended to hold the 5th Workshop in late 1992 in Japan. The location and time of the workshop will be determined through correspondence between the Co-chairmen of Task Committee "F" of the UJNR Panel on Wind and Seismic Effects.

Resolutions of the First U.S.-Japan Workshop on Hazard/Risk Assessment and Design Earthquake Loading (DRAFT)

Public Works Research Institute
Tsukuba Science City, Japan
May 28 and 29, 1991

1. The First U.S.-Japan Workshop on Hazard/Risk Assessment and Design Earthquake was held under the sponsorship of the Task Committee E, "Ground Motion and Design Force", UJNR Panel on Wind and Seismic Effects at the Public Works Research Institute, Tsukuba Science City, Japan on May 28 and 29, 1991. The cooperation provided by the University of Tokyo, Kyoto University, Toyohashi Institute Technology is also noted.

2. The workshop provided a valuable opportunity to exchange technical information beneficial to the hazard/risk assessment and design earthquake loading communities in both countries. The following topics were presented and discusses ;
   a) strong motion measurement ;
   b) attenuation of ground motion ;
   c) analysis of dense array data ;
   d) design earthquake loading ;
   e) seismotectonics for earthquake hazard mitigation ;
   f) earthquake hazard and risk (loss) assessment ;
   g) thematic mapping for hazards ;

3. The workshop identified without priority the following technical topics for further cooperative research programs.
   1) Seismic Hazard/Risk Assessment
   2) Application of Seismotectonics to Zoning
   3) Vulnerability of Structures
   4) Description of Ground Motion
   5) Application of Hazard Analysis to Critical Structures
   6) Cooperation on Projects Related to the International for Natural Disaster Redaction
4. The workshop recognizes the importance to continue exchange of personnel, technical information, research data and use of available research facilities in both countries. The workshop further recognizes that more specific plans for increased exchange of research personnel

should be developed.

5. The importance of the United Nations Program "International Decade for Natural Disaster Reduction" is recognized and its relation with hazard/risk assessment and design earthquake loading research under the UJNR program is noted. Considerations shall be made in the planning, development and execution of future cooperative efforts to complement and support the goals of the International Decade program.

6. It is anticipated that the interaction among the U.S. and Japanese participants during the discussions and technical study tour will lead to and enhance exchange of relevant information and data, and will also lead to the establishment of cooperative research efforts on hazard and risk assessment technology and on design earthquake loading.

7. In view of the importance of cooperative research programs on hazard and risk assessment technology and earthquake loading, it is recommended to hold the 2nd workshop in May 1992 in conjunction with the 24th Joint Meeting of the Panel on Wind and Seismic Effect, UJNR in the United States. The location and time of the workshop will be determined through correspondence between the Co-chairmen of Task Committee "E" of the UJNR Panel on Wind and Seismic Effects.

RESOLUTION OF THE EIGHTH U.S.-JAPAN BRIDGE ENGINEERING WORKSHOP ANALYSIS, DESIGN, PERFORMANCE AND STRENGTHENING

Chicago, Illinois
May 11 and 12, 1992

The following resolutions are proposed by the organizers of the 8th U.S.-Japan Bridge Engineering Workshop to the Task Committee J of UJNR:

1. Whereas; the eighth U.S.-Japan Bridge Engineering Workshop provided valuable exchange of technical information which was beneficial to bridge engineering communities of both countries. Forty seven participants representing government, academia, industry and private practice presented 44 papers in eight technical sessions. Main themes included: (1) Long-Span Bridges; (2) Wind Effects and Codes; (3) Design; (4) Bridge Bearings; (5) Seismic Issues; (6) Evaluation, Rating, and Management; (7) Foundations; and (8) Experimental Issues.

2. Whereas; the Workshop recognizes the importance of continued exchange of personnel, technical information, research data and bridge response data from significant earthquakes and strong winds; the use of available unique research facilities in both countries; and encourages cooperative efforts in mutually agreeable areas of bridge research.

3. Whereas; the primary purpose of the Workshop is to exchange technical information on a timely basis that is usable by the bridge engineering profession. The specific objectives of this Workshop are:

   - To ensure a first-hand exchange of information resulting from the unique problems each country is faced with; for example, the development and implementation of limit state or load and resistant factor design methods.
   - To stimulate collaborative research work in the important area of bridge engineering for better design construction, performance, rehabilitation and strengthening of bridge structures in both countries.
   - To facilitate the exchange of technical information and ideas and to promote the broadening of viewpoints for those involved.

4. Whereas; the Workshop identifies the following technical topics for continued cooperative research programs:

   - Dynamic control and seismic isolation of bridge structures including: active, passive and hybrid control, damping augmentation, experimental studies and application to long-span cable-supported bridges.
   - System identification techniques for wind and seismic engineering including: data interpretation studies, development of mathematical models, identification of

nonlinear systems, reduced-order models, computational algorithms, and optimum sensor locations.

- Nondestructive evaluation techniques including damage detection by sensor monitoring, expert systems, inspection techniques, structural monitoring, condition assessment for safety, performance evaluation, and repair procedures.
- Experimental and analytical studies on the effects of dynamic soil characteristics, including bridge-soil interactions, on the behavior of bridge structures.
- Retrofitting and strengthening methodologies for bridge structures.
- Development of design criteria based on limit-state concepts.

5. Whereas; the Workshop identifies the following technical topics for future cooperative technical programs:

- Jointless bridge criteria, design, and performance.
- Use and performance of new and enhanced structural materials.
- Development of standard test procedures for base isolation devices.

6. Whereas; the interaction among the delegates from the Untied States and Japan during the round-table discussions and the technical study tour has lead to an enhanced exchange of data and innovative ideas, and will also lead to the establishment of cooperative research efforts, both theoretical and experimental, on bridge engineering.

7. Be it resolved that; in view of the importance of cooperative programs on the subject of wind and seismic effects on bridges and on performance, strengthening, structural monitoring, safety, nondestructive evaluation and design innovation of these transportation structures, the continuation of technical cooperation and interaction is considered essential. Further, it is recommended to conduct the 9th bridge engineering workshop in Tsukuba just prior to the 25th UJNR joint meeting in Japan, in May 1993. A study-tour will be planned to visit research laboratories and bridge-construction sites.

Resolution of the First U.S.-Japan Workshop on Earthquake Protective System for Bridges

RESOLUTIONS

Background

The first U.S.-Japan Workshop on Earthquake Protective Systems for Bridges was held in Buffalo, New York on September 4 and 5, 1991. Held under the auspices of Task Committee G on Passive, Active and Hybrid Controls of the UJNR Panel on Wind and Seismic Effects, the workshop was attended by 35 Japanese and 28 U.S. participants. In addition, four observers from Taiwan and People's Republic of China were in attendance. U.S. sponsorship for the meeting was provided by the National Center for Earthquake Engineering Research (NCEER) which is headquartered on the campus of the State University of New York at Buffalo. The organizer for the Japan-side was the Public Works Research Institute (PWRI) of the Ministry of Construction.

This comprehensive workshop comprised the presentation and discussion of 29 papers which covered a wide range of topics including seismic isolation hardware, the testing of isolated bridges and components, active control and hybrid systems, design issues and applications, long span bridges, seismic retrofit and new construction. Fruitful discussion on many issues of common concern took place. Of particular interest was the difference in philosophy between menshin design in Japan and seismic isolation as practiced in the United States. The need to lengthen the period of vibration is not so important (in Japan) as the requirement to keep joint clearances small. Other issues included joint design, displacement restraint devices, and durability of isolation hardware in the field.

In conjunction with the workshop, a Study Tour of base isolated bridges in the United States was undertaken by the Japanese participants. The purpose was to obtain first-hand knowledge about U.S. practice concerning the use of base isolation for seismic retrofit and the construction of new bridges.

Resolutions

1. There are many topics of common interest related to the development and implementation of these systems. It is therefore resolved that continued cooperation between researchers and bridge engineers in both countries be actively encouraged. The free exchange of research and design experience, in both the laboratory and field, will facilitate the advancement of this technology and its practical implementation in both countries.

2. In view of the success of this first workshop, the importance of this technology and the rapidly advancing state-of-the-art, it is resolved that a second U.S.-Japan Workshop on Earthquake Protective Systems for Bridges be held in Japan late in 1992.

3. It is further resolved that the second workshop be oriented towards research and application rather than state-of-the-art reviews. Topics to be covered should include: (1) innovative protective systems; (2) design methods for base isolated/menshin bridges; (3) design of utilities carried by isolated bridges so as to accommodate movement; and (4) full-scale verification of performance.

Another activity for the second workshop should be the comparison of U.S. and Japanese isolation/menshin designs for the same bridge. This project requires the design exercises to be completed before the next meeting, so that the results are available for presentation and discussion. For such an exercise to be useful, practitioners and designers should be active participants in the Workshop.

IX. 閉 会 式 挨 拶

IX　閉会式挨拶

日本側部会長　住吉　幸彦

第２４回合同部会の閉会にあたり、日本部会を代表して一言、御挨拶を申し上げます。この第２４回合同部会では、耐風耐震技術の最新の情報が交換され、また今後の活動計画が議論され、非常に有意義な会議でありました。ライト部会長、ローファスト事務局長を始めとする米国側関係者の皆様のご協力に対して、改めて感謝を申し上げる次第です。

先程最終結論が採択されましたが、これに従いまして、来年５月にいたる１年間にさらに活発な活動ができるように、日米間の協力を一層密にしていきたいと念じております。

特に、来年はこの耐風耐震構造専門部会にとって記念すべき２５周年－１／４世紀－に当たります。２５周年記念事業として防災セミナーの開催や部会の成果を取りまとめた出版物の刊行などが考えられますが、今後日米の事務局でよく相談していただき立派なものにしてもらいたいと考えております。筑波で開催される第２５回合同部会には、米国から一人でも多くのメンバーの方々が参加されることを期待いたしております。

最後に、会議期間中に示された米国部会の皆様方の友情に感謝しつつ、視察旅行に旅立ちたいと思います。今回の視察旅行では興味ある研究機関、実務の現場を訪問し、更に技術発表も予定されておりまして、すばらしいプログラムになっております。米国側事務局の努力に感謝いたします。

皆様方の前には、日本の風土で熟成した豊じょうの液体がおいてございます。私どもの心からのお礼のしるしとしてお持ち帰り下さい。

最後に、会議の期間中、日米両国部会委員の良きパイプ役を果たしていただいた通訳のお二人のご苦労に対して、お礼を申し上げます。

ありがとうございました。

米国側部会長　Richard N. Wright

住吉議長、日本側出席者、アメリカ側出席者のみなさん、これまで、地震、強風による被害を軽減について、非常に活発な、そして実りある議論をしてきました。出席されました方々の各機関の研究所、作業部会、そして専門部会での発表や討論の成果を誇りに思います。

地震防災の研究、実状とも日米両国において大いに進展したことが発表、討論から判りました。しかし、（地震の）脅威は厳しいものがあり、研究、特に、既存の建物、ライフラインで危険なものを見つけ、補修することに関する研究は、鋭意、進めていかなければなりません。また、特にアメリカにおいてですが、風に対する防災の研究、実状はあまり進んでおりません。実際に風でひどい被害をうけるまで風に対する防災の研究には支持を得られないのでしょうか。

専門部会全体として、特に作業部会において、重要でやりがいのある共同研究を計画してきました。第２４回合同部会では、耐風・耐震構造専門部会が大いに進展を見せ、日米両国、広くは全世界において地震、強風による被害を軽減するための重要な目標を掲げています。

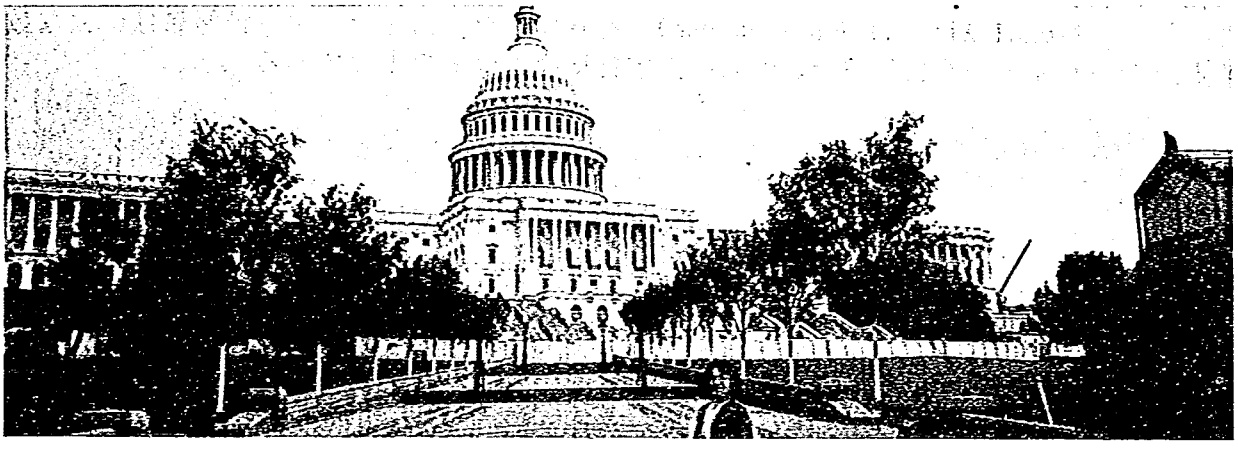
この会議を開催するのに御尽力頂いた事務局の方々に感謝いたします。交流を滑らかにするのに一役買っていただいた通訳の方々に感謝いたします。研究を公表して頂いた著者の方々、重要な点について討論して頂いた方々に感謝いたします。特に、御尽力頂き、はるばる日本から来て研究について話して頂いた日本からの出席者に感謝いたします。このような認識のもと、この自動製作研究所（Automated Manufacturing Research Facility）が準備いたしました記念品は先進技術での協力のシンボルであります。どうもありがとうございました。

X. 見　学　記

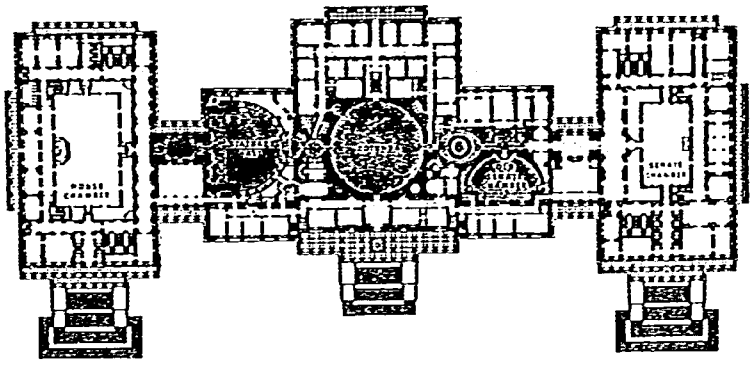
X　見学記

5月18日（月）　ワシントン市内・・・・・・・・・・・・・・・・・・・・・・・藤原　稔

明日からの合同部会を前に、日本側出席者全員は米側のローファスト氏とクーパー氏の案内でワシントン市内を視察した。午前中は国会議事堂（Capitol）である。国会議事堂がフィラデルフィアからワシントンの現在の地に移ったのは1800年のことであり、当時はロタンダ（ Rotunda ）と呼ばれる中央のドームとその左右に配置された当時の上下院議場（Senate Chamber, House Chamber）を含む建物のみであった。その後合衆国に加盟する州の増加に伴って、1850年代に現在の上下院議場が入っている左右の建物が追加された（写真－1、図－1）。また上下院議員会館は国会議事堂から約100mのところに1900年代はじめに建設され、国会議事堂とはそれぞれ専用地下鉄で結ばれている。ちなみに現在上院（Senate）の定員は各州2名ずつ計100名であり、下院（House of Representative）の定員は435名、各州の定員はその人口比による。



写真－1　国会議事堂

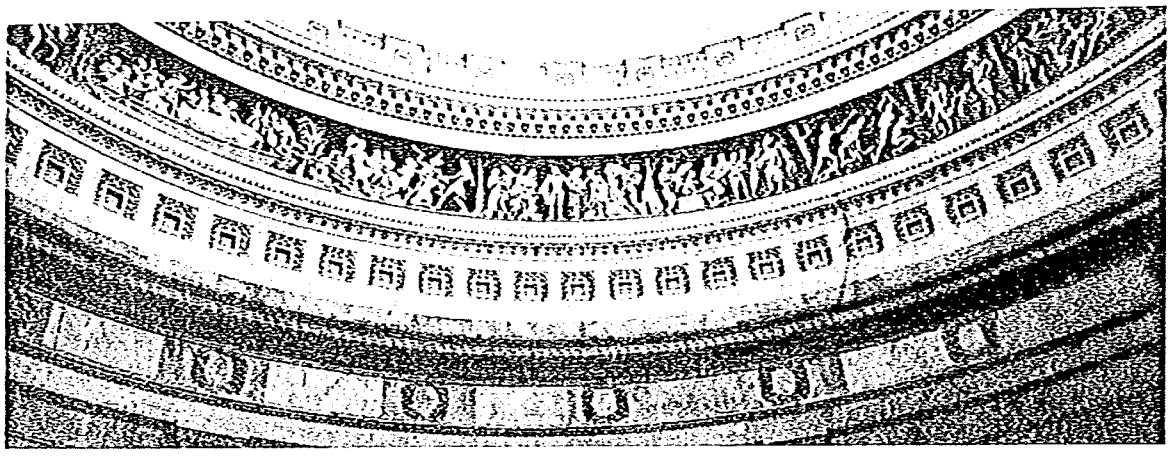


図－1　国会議事堂の平面図

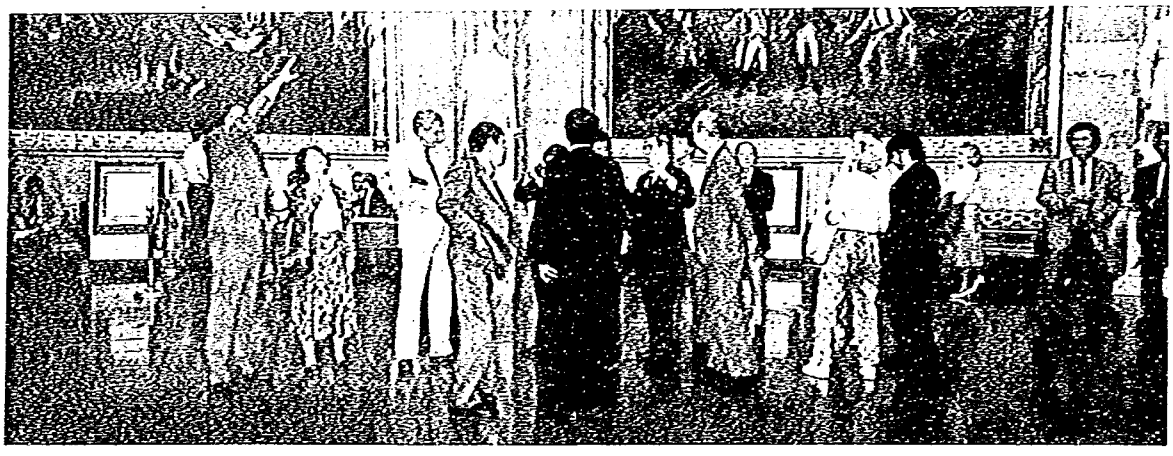
一般の見学者はロタンダと旧上下院議場のみの見学が許されるが、今回は米側の配慮により、Mr. William ENSIGN氏 (Assistant Architect, Office of the Architect of the Capitol)を表敬訪問し、Mr.James E. ELLISON氏(Assistant of the Assistant Architect)により、これらに加えて、現在の上下院議場も見学でき、国会議事堂建設の歴史や建築学的説明を聞くことができた。ロタンダは国会議事堂の中心であり、歴代大統領の葬儀などの儀式が行われる所である。そのドームの上には自由の像が立ち、内部は 180ftの高さがある。壁面には独立を描いた巨大な４つの絵画が下がり、その上方には米国の歴史を彫ったレリーフが施されている（写真－２，３，４）。旧上院議場は現在各州の著名な市民の像が立ち並ぶ銅像の間となっている。現在の上下院議場も今回は見学することができ、日頃マスコミに登場する政治の現場を実感することができた。ただしカメラの持ち込みは禁止であった。昼は専用地下鉄に乗って上院議員会館に行き（写真-５）、Mr.William ENSIGN氏主催の昼食会でご馳走になった。

昼食後、住吉土研所長、藤原、横山の３名はクーパー氏と共に地下鉄でＦＨＷＡの本部に行き、ラーソン長官以下のスタッフと道路新技術に関する日本の建設省とＦＨＷＡとの技術協力協定に関する事前打ち合わせを行った。 その後クーパー氏の案内で Turner-Fairbank研究所の構造関係の施設（風胴、橋脚の水理模型、鋼桁の疲労試験、新素材、大型車両の輪荷重など）を見学した。

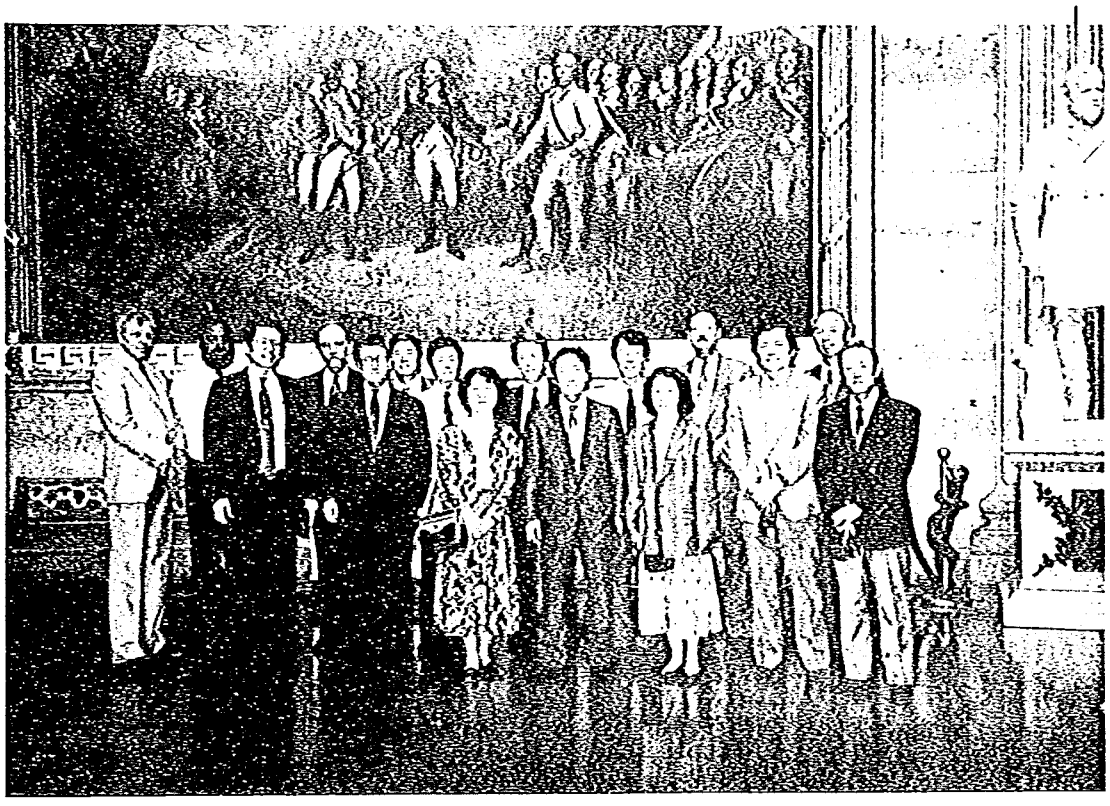
他の参加者は、昼食後スミソニアン博物館（Smithonian Museums）の中の国立航空宇宙博物館（National Air and Space Museum） を見学した。スミソニアン博物館は広大な敷地と施設を有する総合博物館で、国立航空宇宙博物館はその１つである。



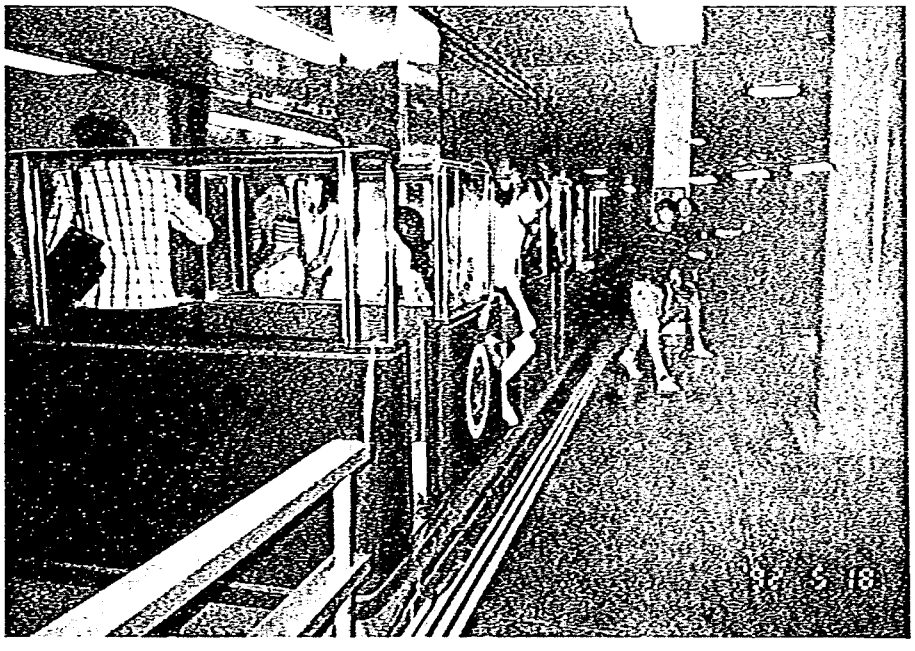
写真－２　ロタンダ内部上方に彫られた米国の歴史のレリーフ



写真－３　ロタンダ内部にてELLISON氏より説明をうけるＵＪＮＲ一行



写真－４　ロタンダ内部での記念撮影（撮影：藤原）

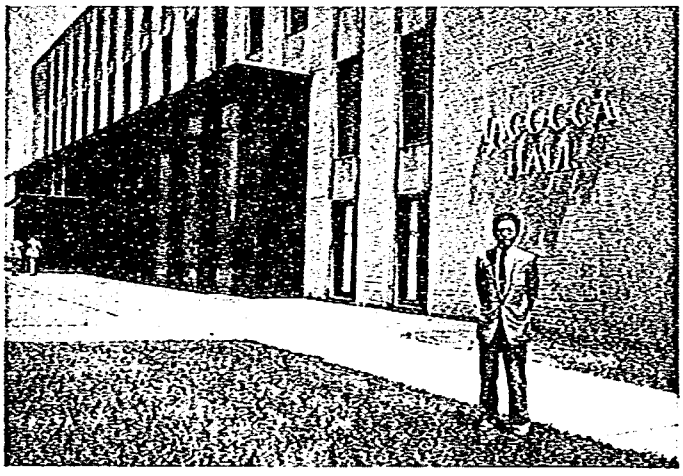


写真－５　国会議事堂と議員会館を結ぶ専用地下鉄

5月23日(土曜日) リーハイ大学 ・・・・・・・・・・・・・・・・・・・・・大谷 圭一

前日までの合同部会での技術的会合を終了した我々は，日本から参加した藤原，中野の両氏と別れて，スタディツァーに出発することとした。スタディツァーの第一日目は，午前6時ホテルロビーに集合，6時20分出発という厳しいスケジュールでのスタートとなった。米側のWight部会長，Raufaste事務局長，Brady委員と共に，借り上げたバンに乗り込み、Bethlehem，Pennsylvaniaにあるリーハイ大学まで田園風景の続く中をひたすら走り続けた。早朝の出発であったが，Raufaste氏の秘書の手配による弁当を走行中の車内で食べながらの，ちょっとしたドライブ気分での移動であった。途中I－81がSusquehanna川に差し掛かった時，1979年に事故を起こしたTree Mile Island原子力発電所の巨大な排気筒が達望され，TMI事故のもたらした影響に思いをはしらせたものである。

約3時間半のドライブの後，リーハイ大学 IACOCCAホール（写真－6）に到着した我々を，土曜日にも係わらず，大型構造システム先端技術センター(Center for Advanced Technology for Large structural Systems(ATLSS))のProf. J. W. Fisher所長，Dr. J. E. Bower所長を始めとして、多数の研究者が出迎えてくれた（写真－7，8，9）。コーヒーとドーナツで喉を潤した後、Prof. Fisher からＡＴＬＳＳの概要説明があった。このＡＴＬＳＳは，米国科学財団（ＮＳＦ）が建設している工学研究センターの一つとして，1989年から供用開始されたものである（第20回合同部会の際のスタディツアーで訪問した国立地震工学研究センター（National Center for Earthquake Engineering Research, SUNY at Buffalo)は同種のセンター第一号である)。ＡＴＬＳＳでは，構造物の接合部と鉄骨造構造物の疲労問題について集中的に研究を実施しており、既に多くの成果を得ているとの説明があった。なお，ＡＴＬＳＳの予算は年間約1千万ドルで，25.7%がＮＳＦから，25%が海軍関係から，15.4%が民間等から，残りがリーハイ大学からの資金で運営されているとのことである。つづいて，研究者のMr. John Abruzzoがプレキャストコンクリート造の鉄板とスタッドボルトによる接合部の実験結果について，講演を行った。



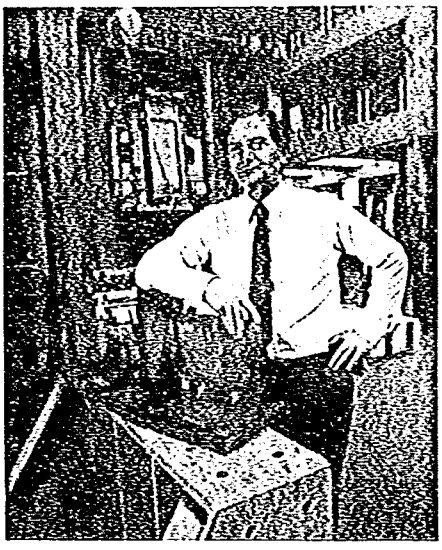
写真－6 ＩＡＣＯＣＣＡ ＨＡＬＬ



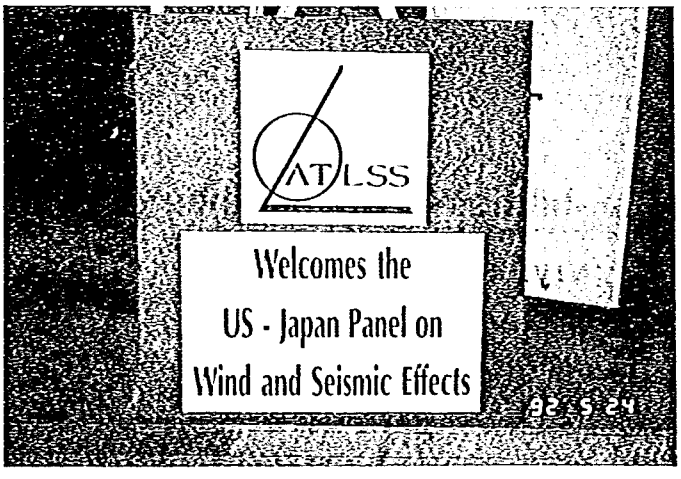
写真－7 ロビーで歓談中の
ＵＪＮＲメンバーと
リーハイ大学関係者

この講演の後，我々は所長以下の説明を受けながら，ＡＴＬＳＳの見学を行った。表－１に示すような仕様で，高さの異なる反力壁（写真－１０）を有するユニークな実験施設となっている。ここでは，複数の架台にオイルジャッキを取り付けて，各種の疲労試験（写真－１１）を精力的に実施していた。ここで，我々は古い友人，Prof. Le-Wu LU と再会した。同氏は日米協同大型実験研究に計画段階から参画されており，日本に多くの知人を持つ，鉄骨造のエキスパートである。実験施設見学の後，再びIacoccaホールに戻り，展望塔で昼食をとった。

午後は，北川，川島両委員が日本に於ける免震・制震構造の現状について，大谷が日本の大型実験施設の現状について報告をし，討論が行われた。引き続いて，Ｆｒｉｔｚ実験施設を見学した。ここには５百万ポンドの万能試験機（写真－１２）があり，鉄の町（リーハイ大学のあるベツレヘムは，古くからの製鉄の町である）にふさわしい装置で研究が行われている。見学の時間がひ常に限られていたが，リーハイ大学の構造関係の研究の一端を見学した我々は急いでベツレヘム・アレントン空港へ移動した。この空港からワシントン・ダレス空港までは，定員１３人の小型飛行機での移動であった。ワシントン 空港でブリディ，北川委員と別れて，我々はニューオルリンズへ夕方到着し，長かったスタディツァー第一日目もやっと終了の時を迎えた。



写真－８　ＡＴＬＳＳのフィシャー所長



写真－９　我々を歓迎してくれたボード

表－1　ＡＴＬＳＳとＦＲＩＴＺ実験施設性能

Testing Capabilities—ATLSS Multidirectional Experimental Laboratory

SIZE

- 102' x 40' floor space
- Reaction wall heights up to 50'
- Anchor points on 5' grid on floor and wall

HYDRAULIC ACTUATORS

- Computer control with digital servovalves
- Individual static loads to 460K with strokes to 38 inches
- Individual fatigue loads to 340K or strokes to 24 inches
- Fatigue testing frequencies to 5 Hz
- Actuators may be ganged for higher loads

CAPACITIES

- Individual anchor points, 300K axial and 500K shear
- Moment at wall base, 2500 to 4500K ft/ft width varying with wall height

LOADINGS

- Static and pseudodynamic
- Fatigue
- Impact
- Multidirectional

HYDRAULIC SYSTEM

- Three-dimensional loading
- 3000-psi capability
- Independent systems for multiple tests
- Independent high volume and low volume supply lines
- Four independent computer controls
- 120 to 360 gallons per minute
- Full access to any part of test bed and wall

ADDITIONAL CAPABILITIES

- Charpy V-Notch testing
- 60,000 lb capacity Universal Testing Machine

Testing Capabilities—Fritz Engineering Laboratory

UNIVERSAL TESTING MACHINES

Capacity (lb.)	Hydraulic Stroke (in.)	Testing Height (ft.)	Clear Width (in.)
5,000,000	36	40	120
800,000	Mechanical	25	36
300,000	12	5	30
120,000	Mechanical	4	28
60,000	6	4.5	30

FATIGUE TESTING MACHINES

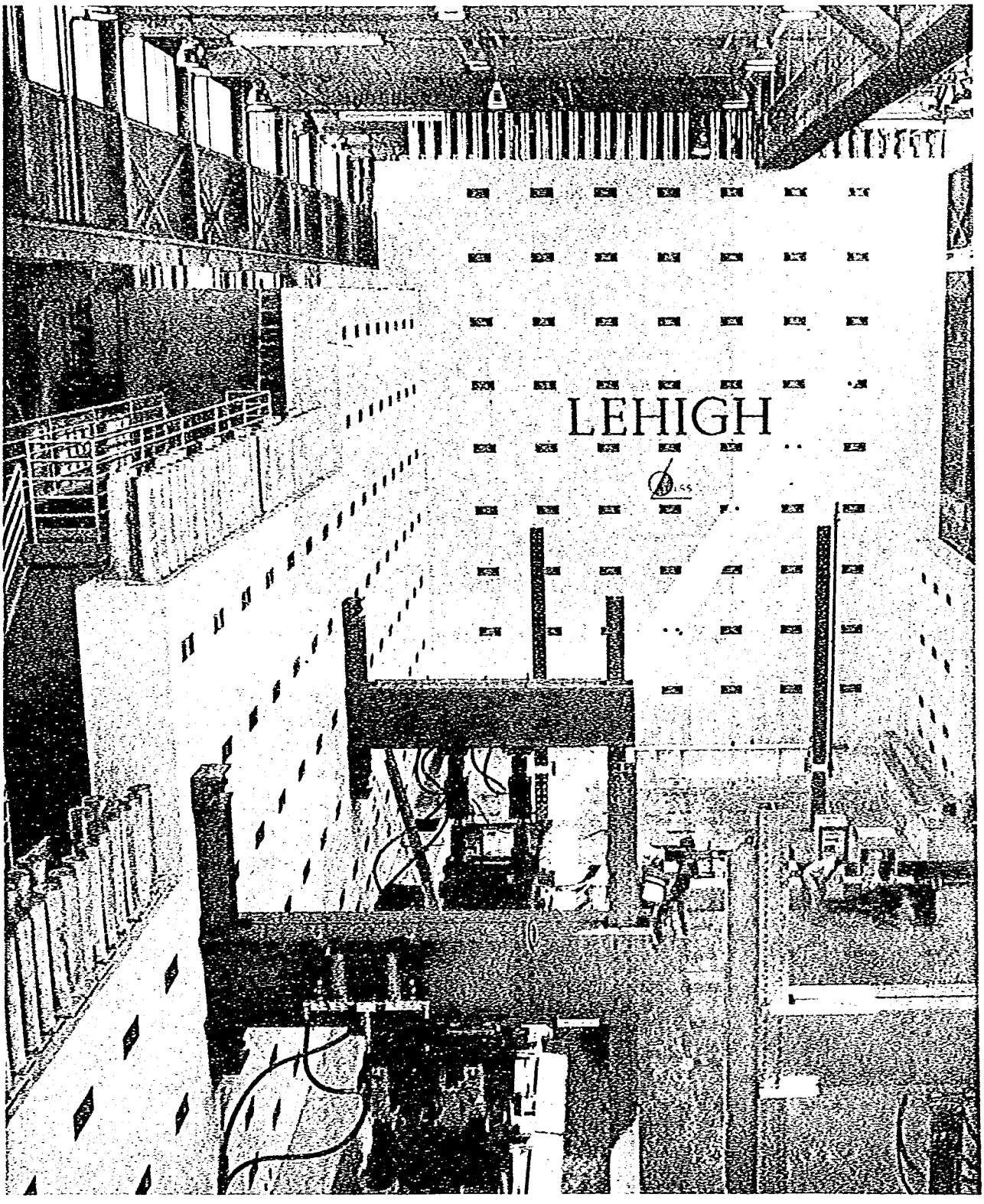
Type	Capacity (lb.)	Testing Space (ft.)	Clear Width (in.)	Cycling Rate (Hz)
Alternating Stress	220,000	10	18.6	0 to 1.33, or 4.17, or 8.33
Alternating Stress	22,000	1.5	15	58.3 to 300

DYNAMIC TEST BED HYDRAULIC SYSTEMS

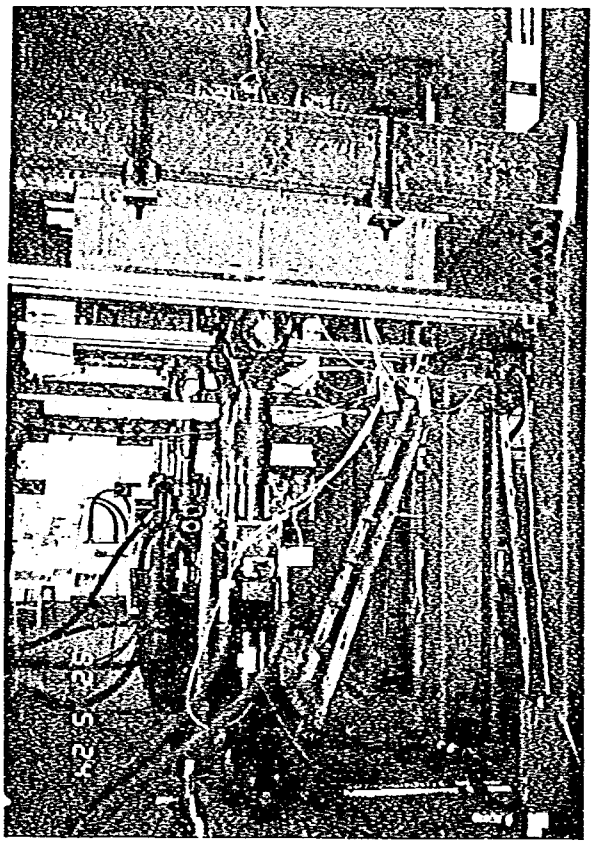
- 3000-psi capability
- 40 to 80 gallons per minute
- Individual static-load jack capacities from 10.5 to 440K
- Individual fatigue-load jack capacities from 5.5 to 220K or strokes to 6 in.
- Fatigue testing frequencies to 8.33 Hz

ADDITIONAL CAPABILITIES

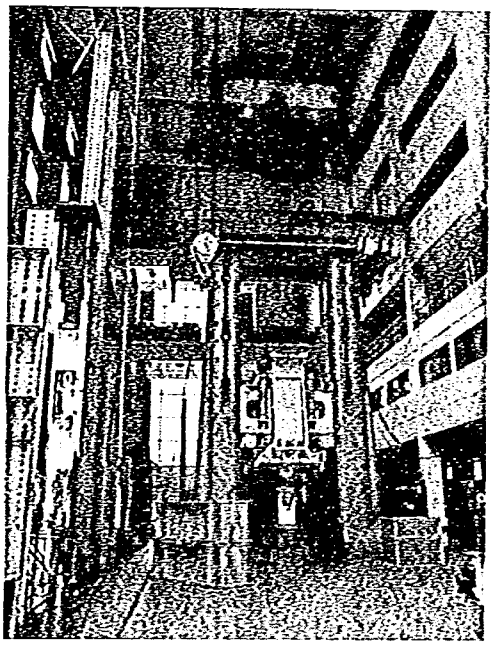
- Drop-weight tester, 8,000 ft.-lb. capacity
- Torsion testing machine, 24,000 in.-lb. capacity
- Calibrations



写真－１０　ＡＴＬＳＳの反力壁



写真－１１　鉄骨部材の疲労試験の状況



写真－１２　ＦＲＩＴＺ実験施設の５百万ポンド万能試験機

5月25日（月曜日）　Ｓｔｅｎｎｉｓ宇宙センター・・・・・・・・・・・・・朝倉　肇

Ｓｔｅｎｎｉｓ宇宙センターはニュウーオリンズの東方４５マイルにあり、その面積は13,480エーカーで125,325エーカーのバッファーゾーンに囲まれている広大な施設である。

当センターの第一の役割はＮＡＳＡ最大のロケットのテスト施設であり、スペースシャトルの全ての主エンジンの地上テストが行われている。最近はスペースシャトルの打ち上げ頻度が落ちてはいるが、３基の巨大なテスト台では週１回程度のテストが行われているそうで休日にもかかわらず作業員の姿が見受けられた。

第二の役割は、センターの科学技術研究所が実施しているリモートセンシング、センサー及びデータシステム生命及び環境科学の分野の研究でありその成果はセンター自身の事業や民間需要に活用されている。

第三の役割は13,480エーカーという広大な施設の管理とセンター内にある連邦や州に所属する１９の研究所等に、所要のサービスを行うことである。

５月２５日は米国の祝日であるメモリアルデーのため、ドライブ方々センターを訪問。

ビジターセンターで種々の展示物を見たあと、広大なセンター内をバスで見学。休日にもかかわらずセンターの女性職員が案内をしてくれたがかなりなまりがあるうえ、機関銃のような早口で、英語の達者な面々にも殆ど聞き取れなかったようである。

見学は午前中で終わり、帰途ドライブインで食事、元気で気のきくウェイトレスと記念撮影。ニューオリンズへはポンチャンジ湖にかかる全長２４マイルの一直線の橋を通って帰る。その後は三々五々ニューオリンズの町を散策。

5月26日　CORPS New Orleans地区本部、New Orleans港・・・・・・・・・・野田節男

朝８時３０分にホテルを出発した一行は、車窓より全天候式球場スーパードームの巨大な屋根を眺めつつRoper氏の案内で約２０分後にミシシッピー河畔にある米国工兵隊ニューオリンズ地区本部に到着した。

ウォータフロント施設に関する意見の交換が行われ、まず最初に地区本部のV.Landry氏より、この地区の主要業務はミシシッピー河の航路の維持浚渫および洪水対策であり,30000平方マイルの範囲、２８００マイルの航路に対して1400名の職員、年間予算3.5億ドルで対応しているとのことであった。ミシシッピー河流域はこれまでに洪水による甚大な被害を被っており、１９２７年の大洪水を契機に Mississippi River and Tributaries (MR&T) Projectと呼ばれる米国初の総合的な洪水調節．航路維持計画が発足した。ここでは計画洪水量や防護堤防に関する概要が紹介された。その他、１９６５年のハリケーン被害に対応したプロジェクト等の説明があった。

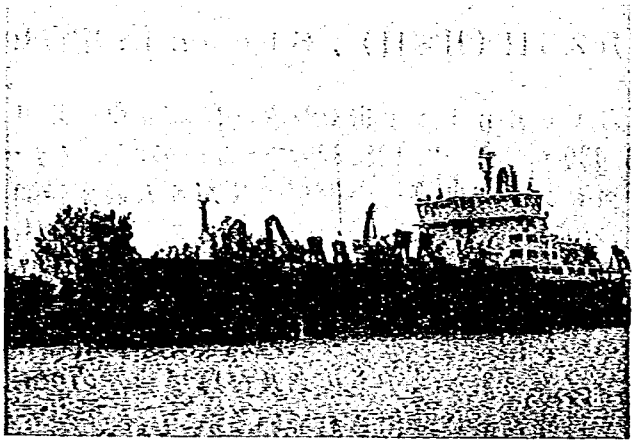
続いて、水路試験所(WES)/B.Ebersole氏から Beach Profile Response to Storm Surge and Waves と題して、海岸の断面変形に関する数値解析モデル（ＳＢＥＡＣＨ）の説明があった。この研究の目的は、ストームから水際線の構造物を守るために海岸付近の海底地盤の侵食、堆積のメカニズムを表現するプログラムの開発である。Oregon州立大学との共同研究による大型水槽実験結果や、SUPERTANKﾌﾟﾛｼﾞｪｸﾄと呼ばれる実海域での海岸変形データとの整合性もかなり良好であった。港湾技術研究所でも同様の研究を行っており、今後の情報交換の必要性を感じた。

最後に、小生が日本における防波構造物の開発の現状を紹介した。石積み堤が主流の欧米に対して、わが国では直立ケーソン式混成堤が一般的である理由を説明し、近年開発が進められているＰＣの２重円筒ケーソン構造、発電ケーソン、ＲＣと鋼管杭の複合構造など種々の新形式防波堤を紹介した。日本では、研究所が開発したこれらの新しい構造物が

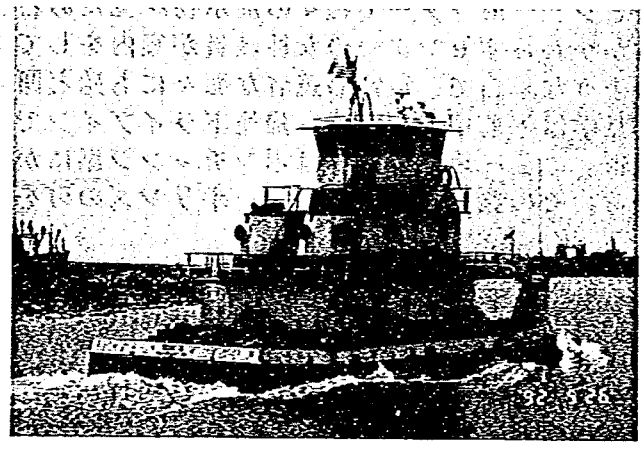
比較的スムースに実際の海域で実証試験されており、米側の現状と異なっていることが話題となった。

ウォータフロント に関する意見交換の後、地区本部前の桟橋からボートでNew Orleans港の視察に出発した。 航路浚渫が主要業務ということで、本部前の桟橋には写真－１３のような自前の浚渫船が係留されていた。当日視察したInner Portにはコンテナクレーン等の近代的な施設は見あたらなかったが、小規模な係留施設、倉庫群が両河畔に長く連なっている状態から見ると取扱貨物もかなりの量なると推測された。港内で目を引いたのは写真－１４のタグボートと多数の箱型バージである。ミッシシッピー河上流の穀倉地帯から小麦等の穀物を運搬しているとのことであった。ミシシッピー河と近くの Pontchartrain 湖とは 運河でつながっており、水位差のある両者の間をLock（閘門）を設けて船は航行している。また、防護護岸および多数の排水機場が整備されており、地盤高の低いこの地域における洪水防護施設の重要性が実感された。写真－１５は観光用の外輪船の発着岸壁に無造作に取り付けられている丸太の防舷材である。日本では立派なゴム製のものが一般的であるが、当地では大型の岸壁でもほとんどの場合丸太が用いられている。水位差が大きいこと、木材が豊富なためであるが、国により適合する技術は様々であることの一例であろう。

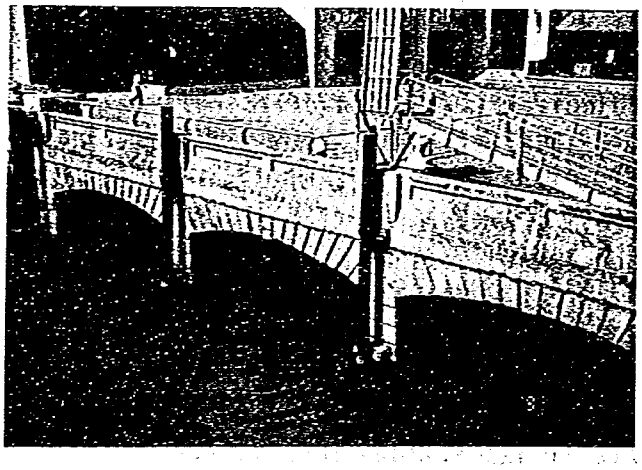
Landry氏、Roper氏の説明で一行は約３時間のクルージングを楽しみ、午後３時頃無事ホテルに戻った。



写真－１３　浚渫船



写真－１４　タグボート



写真－１５　丸太の防舷材

5月27日（水曜日）　・・・・・・・・・・・・・・・・・・・・・・・・・・・・横山 功一

この日ニューオリンズでの最終日であり、住吉部会長、川島委員は前夜一足早くサンフランシスコへ向かったため、日本側メンバーは5名となった。最年長で、ＵＪＮＲ経験の長い野田委員に部会長代行をお願いした。

８時にホテルを出発し、米側メンバーの運転する乗用車に分乗して、ＭＭＳ（内務省鉱物管理部）のニューオリンズ地方事務所へ向かった。事務所は数年前にニューオリンズ市の中心部から移転し、北へ約３０分ほどの場所にある。

先ず、地方事務所の会議室で所長のRod　Pearcy氏、技師のDonald　C. Howard氏からこの地方事務所の活動、役割の概要紹介を受け、その後質疑応答が行われた。

ＭＭＳの役割は大別すると、①鉱区のリース、②鉱物資源開発の規準化であり、メキシコ湾では石油を中心に、ガス、砂、骨材などの開発が行われており、合計約４００基の海洋構造物が有る。ＭＭＳは６３人の監督員により、１２機のヘリコプターを所有し、これらの海洋開発事業を監督している。用いている構造規準は、ＡＰＩ（American Petroleum Institute）規準に準拠しており、仕事上は、ＥＰＡ、ＤＯＴパイプライン、沿岸警備隊、陸軍工兵隊その他と関係する。

業務は１９５４年からリースを開始し、１９８３年から民間の要望(nomination）により、リースする方式に切り替えられている。ちなみに、リース代は＄３／エイカー。

その後、事務所の近くのヘリポートからメキシコ湾上１００マイルにある海上掘削基地に向かった。ヘリコプターの定員が１１名であったため、ローパー（Roper）夫人が同行することが出来ず、一人別行動となってしまったのは本当に気の毒であった。

海上掘削基地への往復のヘリコプターからは、前日船で視察したミシシッピー河と河川構造物が違った角度から視察することが出来て、ミシシッピー河の雄大さ、ミシシッピー河が形作るデルタの大きさ、その中で植物での様子、土地利用の様子、工兵隊による洪水管理と航路維持の重要さが良く理解できた。

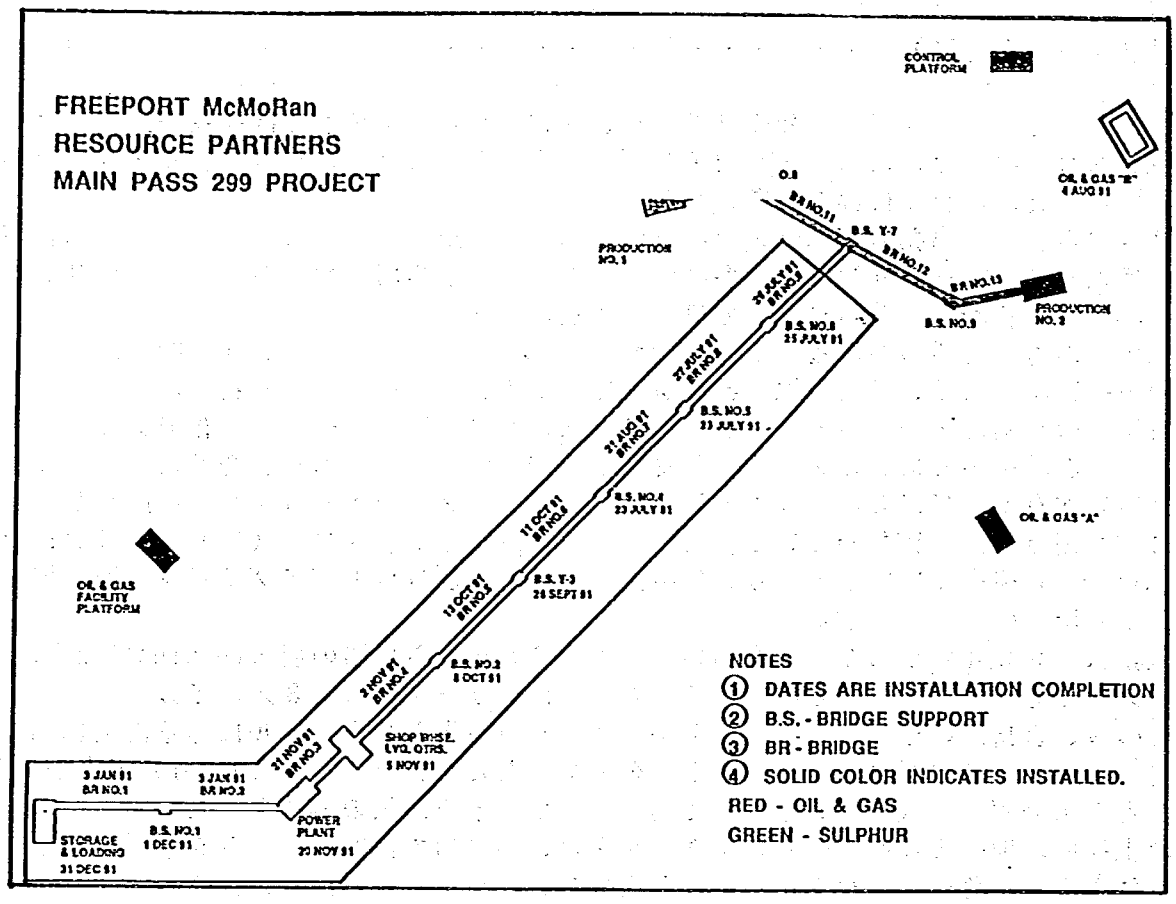
海上基地は、Freeport-McMoran　Grobal　Resource　Companiesにより硫黄の掘削を行うために建設されたものであり、メキシコ湾の海上基地構造物としては１，２を争う大きさである。この海上基地は図－２の平面図に示されているように、長さ約１．１マイルのジャケット基礎を橋梁で結んだ構造で、硫黄の掘削、貯蔵、積み出し、それらの作業の制御など作業スペースと、１５０名の作業員の居住スペースなどから構成されている。水深は２５０ft、構造物基礎深さは３５０ftで、全鋼重は100,00トンとのことであった。１９８８年からＦ．Ｓを始め、１９９２年４月から生産を開始している。

硫黄の掘削方法は、３５０°Ｆに加熱した空気、水を地下に送り込むと直径約２マイルほどのソルトドームが形成され（図－３）、２４０°Ｆという高温の硫黄を回収できる。このオペレーションの人数は約１５０人、勤務体制は１日２交代で１週間単位であり、掘削の操作は中央制御で行われている。

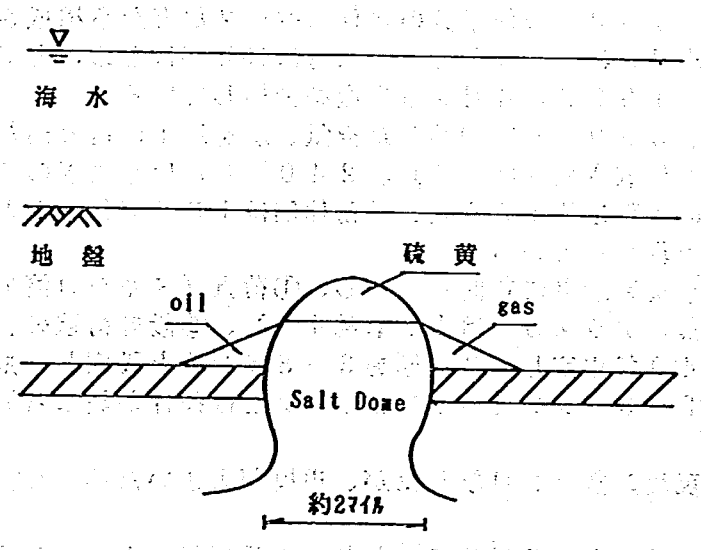
日本側メンバーから構造設計に質問が集中し、①荷重（メキシコ湾では風・波浪、カリフォルニア沖では地震、アラスカでは氷が卓越する）、②設計荷重の再現期間、耐用期間、③腐食（稼働期間を４０年想定し、部材厚を３－８インチと設定し、飛沫帯は塗装している）、④生物付着（２－３インチでカリフォリニアとは条件が異なる）等、興味ある質疑が行われた。

説明を受けた後、現場の隅々を見学したが、規模が大きいため、４，５人乗りの作業車で移動した。

この日の説明は、スライド、ＯＨＰで行われ、入手資料は１枚の海上基地平面図だけであった。（構造などの詳しい事は国際会議などに発表されているとのことであった）。



図－2　海上掘削基地平面図



図－3　硫黄掘削の概念図

5月28日（木曜日）　・・・・・・・・・・・・・・・・・・・・・・・・・・・川島　一彦

8時30分にホテルを出発し、カリフォルニア州交通局の橋梁の耐震補強現場を訪問した。同行したのは、カリフォルニア州交通局構造部のJ.Roberts部長、J.Gates耐震設計担当課長、M.Sultan主任技師である。M.Sultan氏は、平成3年4月から平成4年4月までの1年間土木研究所耐震研究室に科学技術庁の招へい研究員として滞在し、米国に帰ったばかりである。

ロマプリエータ地震で被害を受けたサンフランシスコ・オークランド・ベイ・ブリッジを通って、サンフランシスコ市内に入り、まず、州道280号線の耐震補強現場事務所で概要説明を受けた。説明は、カリフォルニア州交通局構造部のJames　Mason氏が行った。設計図面を基に280号線沿いのサザン・フリーウェイの2層式高架橋の耐震補強の現況に対する詳細な説明を受けた。

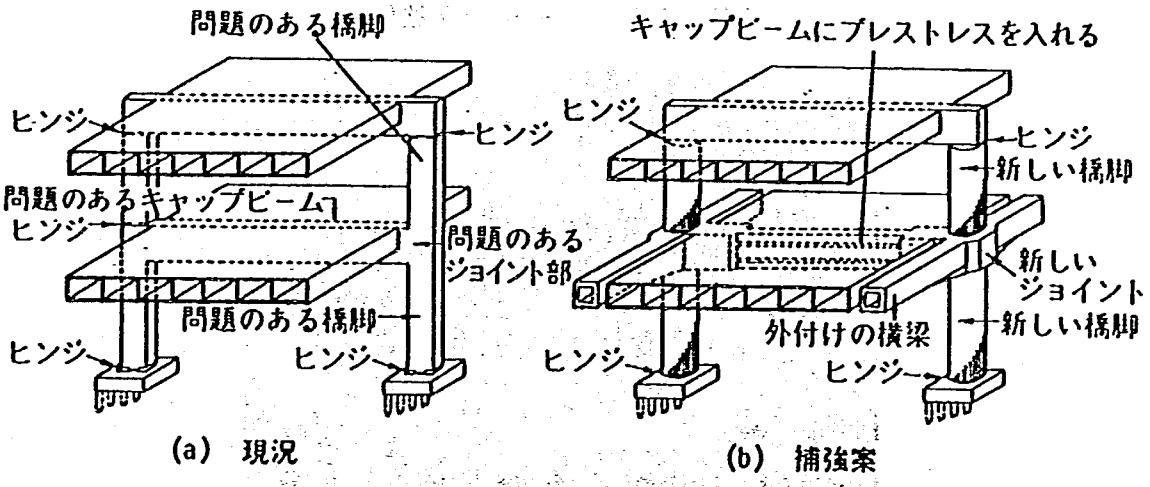
基本的な発想は、橋脚を造りなおした上で、図－4に示すようにエッヂビームと呼ばれる橋軸方向の梁を新たに加え、構造物の剛性を増そうというものである。キャップビームと呼ばれる桁を橋軸直角方向に支持する梁と新設する柱の結合をどうするかが問題で、日本側からもこの点について色々質問が出された。当初の設計では地震の影響をほとんど考慮していないために、横梁と柱の結合部はほとんどヒンジ結合となっている、耐震補強するならば、なぜ、ヒンジを撤去しないのかという質問も出された。このあたりが、日本側からみた関心であったように思われる。

また、エッジビームによる補強と並んで、フーチングの拡幅と増杭による下部の耐力向上も行われていた。写真－16，17，18は、こうした状況を示したものである。

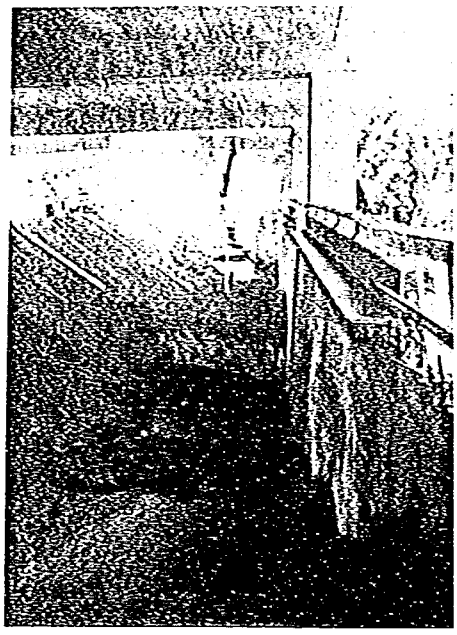
また、杭の上下方向の耐力、特に引き抜きに関する現場載荷実験が計画されており、その概要の説明も行われた。水平方向の載荷も考慮しているようである。実験を計画しているのは、被害のあったサザン・フリーウェイ沿いのオープンスペースである。

橋梁の耐震補強現場を見学後、カリフォルニア大学バークレイ校のキャンパス内にあるファカルティークラブでローレンス・バークレイ実験施設主催の昼食会が行われた。昨年の第23回合同部会に参加された元ローレンス・バークレイ実験施設のD.Eagling氏が歓迎の挨拶をされた後、昼食会が始まった。

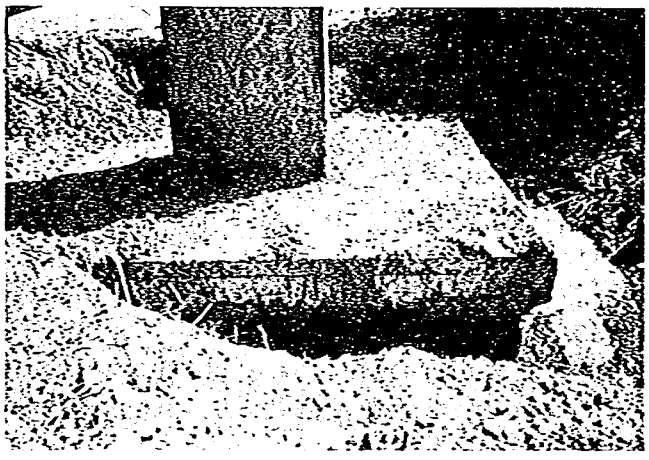
昼食後、同じ部屋で2つの発表が行われた。最初は、D.Eagling氏でローレンス・バークレイ実験施設の耐震補強について、2人めはUSGSのG. Brady氏でせん断振動系と曲げ振動系の違いを模型を用いて説明された。



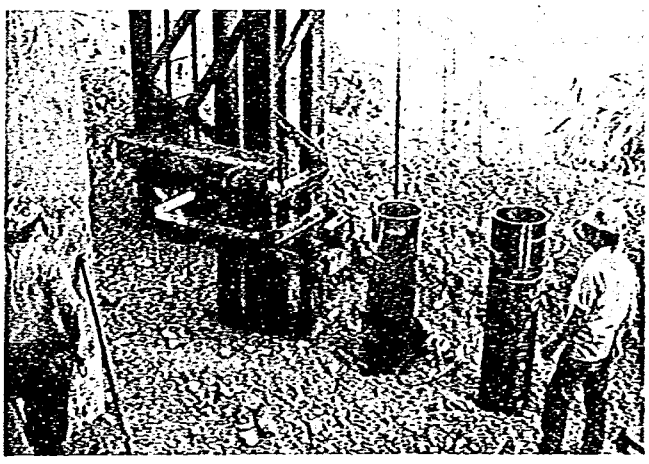
図－4　2層式高架橋に対する横梁形式の耐震補強



写真－１６　高架橋耐震補強作業状況



写真－１７　フーチング拡幅



写真－１８　増杭作業

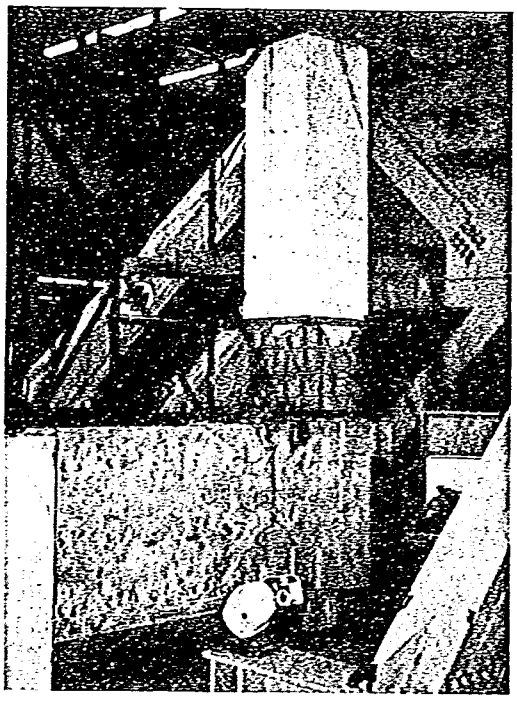
昼食と２編の発表後、D.Eagling氏とFrank McClure氏の案内で、ローレンス・バークレイ実験施設の耐震補強の状況を見学した。ＬＢＬのサイクロトロンや電子計算機センキセンターの入った建物等の見学も含まれていた。特に、後者の建物は、筆者が７年ほど前にＵＣＢに留学時に毎日通った建物であり、大変印象深かった。耐震補強現場の見学後、バークレイヒルからベイエリアを展望したが、眼下にＵＣＢのキャンパスやサンフランシスコ市、ゴーレデンゲート橋等が広がり、大変すばらしいものであった。

その後、予定よりもかなり遅れていたが、ＵＣＢのDavis Hall内の構造物実験施設でちょうど実験中の２層式高架橋の部分模型実験を見学した。これは、カリフォルニア州交通局の委託でＵＣＢが実験しているものである。写真－１９に示すように、柱とエッヂビームの結合部をモデル化したものである。ＵＣサンディエゴ校で実施された実験との違いは、エッディビームが桁と一体となっている点で、J.Moehle教授によればこの方が耐力が高いという。

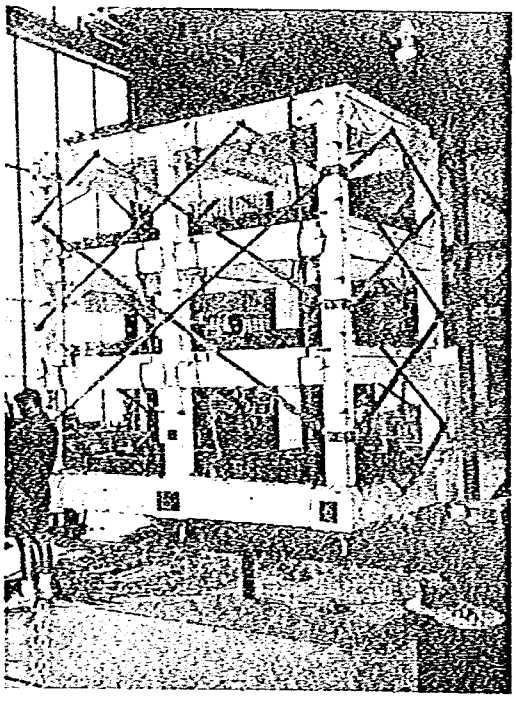
３時半からＵＣＢのUniversity Hallの耐震補強を見学した。これは、斜めブレースを用いて構造体の耐力を高めたもので、ブレースをいれたところと入れないところを設ける等、景観面の配慮もしている。説明は、この設計を担当したDegenkolb社のJ.D.Pino氏が行った。

予定より約１時間遅れで、ＵＣＢのRichmonnd Field Stationにある地震工学研究センター（ＥＥＲＣ）を見学した。説明は、現在センター所長のＪ．Moehle教授が行った。最初の会議室で概要説明を受けた後、図書室を見学した。全世界から集められた多数の地震工学に関する図書が納められていた。この中には、土木研究所や、防災科学技術研究所、港湾技術研究所から送られた強震記録の数値化資料も見ることができた。たまたま、土木研究所から送られた筆者らの報告書がワゴンに載っていた。手に取ってみていたら、サインをするように指示されたため、サインをして残してきた。

図書室見学後、振動実験施設を見学した。ここには、写真－２０に示すように水平２方向、上下方向の加振ができる振動台がある。ユニークなのは、振動台がＰＣであることである。重量が重くて非効率的ではないかとも思われるが、現在までに多数の振動実験に利



写真－１９　２層式高架橋の部分模型実験



写真－２０　振動実験施設

用されてきている。

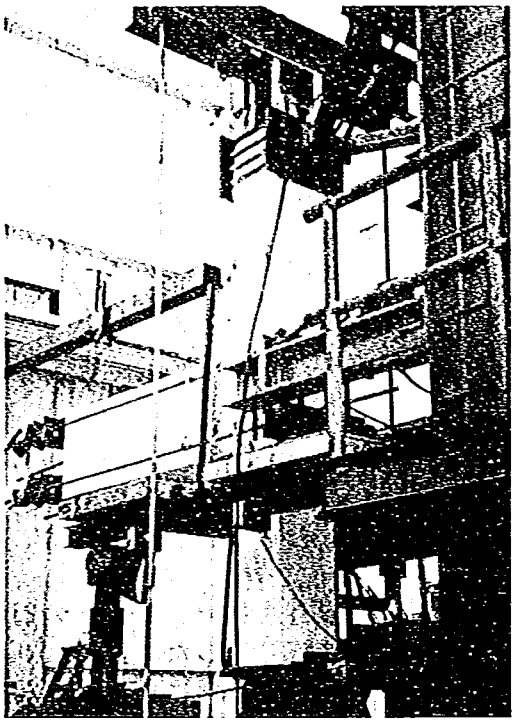
ちょうど、振動台には清水建設との共同研究として実験中の建物の模型が載せられていた。ただ、率直な印象としては、ＵＣＢが行う研究としてはレベルが低いのではないかと思われた。実験室には、免震支承が多数おかれており、Kelly 教授を中心とする免震に関する研究が中心となっていることが伺われた。

振動実験施設の後、すぐ隣にある構造物実験施設を見学した。ここでは、写真－２１に示すように、カリフォルニア州交通局の委託で実施中の２層式高架橋の張り出し橋脚(Outriggerと呼ばれる）の載荷実験が行われていた。ポイントは橋脚と梁の結合部の動的耐力にあり、現状の帯鉄筋を用いた場合が現在実験中で、将来、これを改良したモデルについても実験を行う予定とのことである。

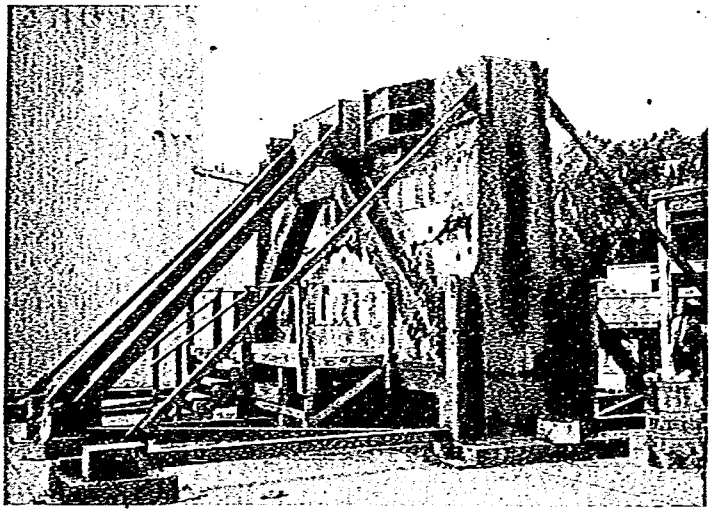
実験室の外側では、写真－２２に示すように、壁のせん断実験に用いた供試体や破断実験に用いた免震支承の模型がおかれてあった。

以上で、非常に多岐に渡る本日の予定を終え、５時４５分ごろホテルに帰った。

夜は、日本側メンバーの招待で、サンフランシスコのフィシャーマンズ・ワーフのCastanolaで夕食会が開催された。日本側を代表して、野田委員が挨拶と乾杯をして米側メンバーとの打ち解けた夕食を楽しんだ。最後は、日本側メンバーを代表して大谷委員がお礼の言葉で、夕食会を締めくくった。



写真－２１　２層式高架橋の
張り出し橋脚の
載荷実験



写真－２２　実験室の外の
使用済み供試体

5月２９日（金曜日）建築物補修及びCorps訪問・・・・・・・・・・・・・・・山内泰之

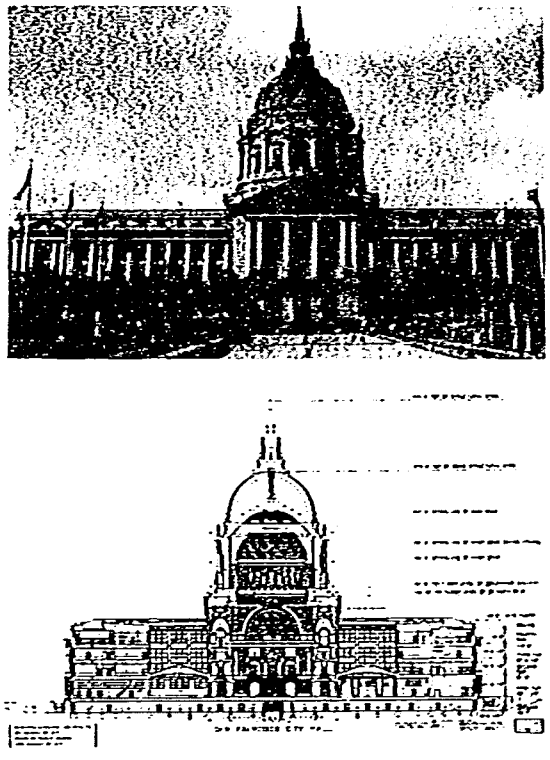
９：００Ａ．Ｍサンフランシスコ市内のForell/Elsesser事務所はカリフォルニア有数の構造及び地震工学に関するコンサルティング及び設計を行っている建築事務所である。当日は、当事務所の顧問格のRonald　Sharp氏の司会・進行で、１９８９年のロマ・プリータ地震で被害を受けた歴史的建築物で当事務所が行った補修設計について担当者からプレゼンテーションを受けた・

（１）サンフランシスコ・シティー・ホール

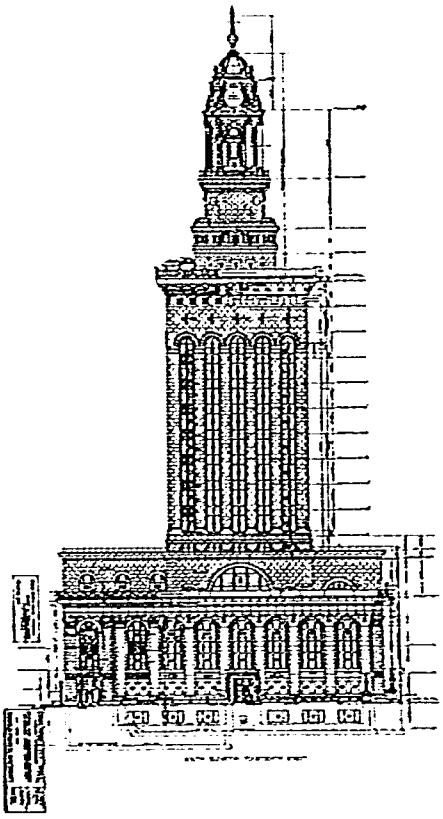
この建築物はサンフランシスコのランドマーク・ドームとして１９１３年に設計された歴史的建築物（図－５）であるが、ロマ・プリータ地震により、レンガ造り間仕切壁や無補強ブロック造りの外壁、床に集中的な被害を受けた。建築物の主体構造は鉄筋ヲフレームとトラスであるが、水平力に対しては主として無補強ブロック外壁及び煉瓦の間仕切壁が抵抗する構成となっている。被害の補修・補強はいくつかの案の中から、コストが低いこと、地震時形状が最も良いことからBase　Isolationを用いた方法が採用されている。

（２）オークランド・シティー・ホール

本建築物は、１９１４年に建設された１８階建（324フィート）の時計台を有する市庁舎である（図－６）。主体骨組は鉄骨造で、鉄筋コンクリート床、無筋コンクリート及び無筋組積造の内外壁を有する非常に複雑な構造となっている。被害は主として内外壁の脱落、クラックであった。また、多数の鉄筋リベット接合部も破損していた。補修方法は、各階の耐震性が１５％から５５％も低下していることを考慮して、Base　Isolationを採用するとともに上部構造も下層階を鉄筋コンクリート壁、上層階を鉄骨ブレースで補強する方針とした。補修工事は現在進行中である。



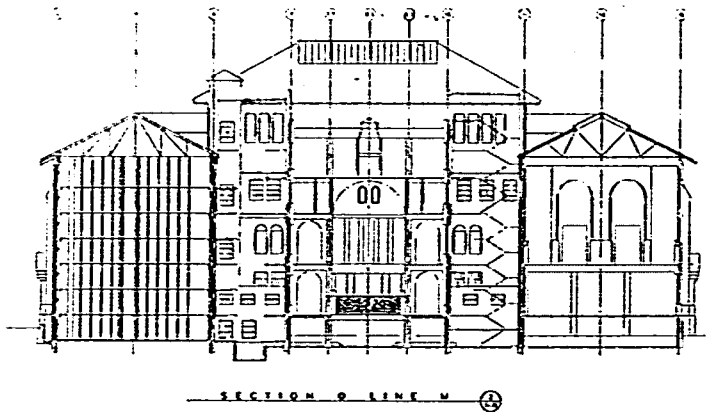
図－５　ｻﾝﾌﾗﾝｼｽｺ・ｼﾃｨｰ・ﾎｰﾙ



図－６　ｵｰｸﾗﾝﾄﾞ・ｼﾃｨｰ・ﾎｰﾙ

（３）スタンフォード・グリーン・ライブラリー・ウェスト

１９１７年設計の本建築物は１９８０年までスタンフォード大学の中央図書館であり、現在も人文、経済学関係の主要な図書が収蔵されている（図－７）。構造は４階建ての鉄骨造で外柱はＳＲＣである。内外壁は無筋組積造及びレンガ造りである。被害は内外壁の大破であった。補修補強方法は主として破壊した無筋コンクリート壁に換え鉄筋コンクリート壁を入れること、鉄骨ブレースの新設、残った無筋コンクリート壁を補強することである。



図－７　スタンフォード・グリーン・ライブラリー・ウェスト

これらのプレゼンテーションの間の休憩時間には事務所内を見せてもらった。設計図面、オフィス空間や仕事風景など米国の設計事務所を初めて訪れた人にとって興味深いものだったものと思われる。

昼はサンフランシスコ・ベイ対岸ソーサリートの有名なレストラン「スピネッカー」で事務所の人も交え気軽な昼食会を行った。



写真－２３　ｻﾝﾌﾗﾝｼｽｺ湾・ﾃﾞﾙﾀ・干満水理ﾓﾃﾞﾙ

昼食後、ソーサリートにあるUS Army Corps of Engineers（工兵隊）所管の「サンフランシスコ湾・デルタ・干満水理モデル」を視察した。本モデルは太平洋、サンフランシスコ湾、サンパブロ湾、サンジョアクイン・デルタ域、サクラメント川などを縮尺模型で構成し、水位の変化、水の流れ、塩分の分布などを再現し、サンフランシスコ湾を中心とする複雑な河口水域システムに関する情報を得て様々な水利計画に役立てることを目的に、１９５６－５７年に作られたものである（写真－２３）。モデルは約１エーカーの面積を占める精密なものであり、深さなどのディメンションは水理学的相似則によりにより定められており、均等な縮尺モデルとはなっていない。サンフランシスコ湾はサクラメント川などいくつかの川上での開発等により土砂、汚水などが流入しており、一方で外洋に通ずるのはゴールデンゲートのある海峡だけで湾は汚れが増えている。湾内の水が自然の干満により全部入れ替わるには２０００年かかるそうである。また、今ではコンピューター解析で替えられるのでこのモデルが実際に使われるのもあと数年とのこと。

この後はサンフランシスコ市内を見学する人と直接ホテルに帰る人とに分かれて行動した。夜はオークランドのイタリア系の歌声レストラン（？）で最後の夕食会を開いた。

XI. 耐風・耐震構造専門部会委員名簿

日本部会委員名簿

（アルファベット順）

部会長	住吉幸彦	（建設省）土木研究所	所長
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代理委員

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Dr. Mehmet K. Celebi	内務省　地質調査所地震研究部
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XII. 作業部会委員名簿

作業部会委員名簿

作業部会	日本側委員	米国側委員
(A)強震計配置計画と強震記録	*野田　節男	*A.G. Brady
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	清野　政明	M.K. Celebi
	常田　賢一	A.G. Franklin
	山内　泰之	M.B. Joyner
		R.B. MacDonald
		F.G. McLean
		H. Meyers
		J.R. Hill
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	山崎　裕	J. Beavers
	横山　功一	S. Sweeney
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	藤原　稔	P.A. Brady
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		D. Eagling
		S. Woodson
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	藤谷徳之助	S.A. Asar
Evaluation of Structural Performance	藤原　稔	M.K. Celebi
	室田　達郎	G.R. Fuller
	岡原美知夫	E.L. Schaffer
	大塚　伸	R.L. Hall
	山内　泰之	J.R. Hill
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(H)地震時における地盤の挙動と安定 Soil Behavior and Stability during Earthquakes	*常田　賢一 藤沢　侃彦 古賀　泰之 水野二十一 野田　節男 岡原美知夫 佐々木　康	*A.G. Franklin C.J. Astill R.D. Borcherdt C.Y. Chen F.G. McLean C.E. Smith
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(J)交通システムの耐風耐震技術	*藤原　稔	*J.D. Cooper
	藤谷徳之助	A.G. Franklin
Wind and Earthquake Engineering for Transportation Systems	川島　一彦	J.H. Gates
	西川　和広	J.B. Scalzi
	岡原美知夫	H.S. Lew
	佐々木　康	
	常田　賢一	
	横山　功一	
(K)海洋施設の耐風・耐震技術	*上部　達生	*C.E. Smith
	藤谷徳之助	C.S. Barrientos
Wind and Earthquake Engineering for Marine Facilities	室田　達郎	W.E. Roper
	野田　節男	J.E. Sabadell
	岡原美知夫	M.C. Miller
		M.K. Celebi

＊印：　作業部会長

UJNR
耐風・耐震構造専門部会
第24回合同部会会議録

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要 旨 1992年5月にアメリカ・メリーランド州のNISTにおいて開催された耐風・耐震構造専門部会第24回合同部会の会議録をまとめたものである。主な課題は耐風工学，耐震工学，高潮・津波，日米共同研究，IDNDRについてである。

キーワード：耐風工学，耐震工学，高潮・津波

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