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RESEARCH ON EARTHQUAKE-RESISTANT DESIGN OF BRIDGES

橋梁の耐震設計に関する研究

by Toshio Iwasaki

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A Summary of "Literature Survey on Seismic Effects on Bridges"
prepared at the University of California, Berkeley

カリフォルニア大学での「橋梁に対する地震の影響に関する文献調査」の概要

January 1972

昭和47年1月

PUBLIC WORKS RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION

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構造橋梁部振動研究室

要 旨

この資料は、筆者が建設省より派遣されて「橋梁の耐震設計に関する研究」という課題のもとに、昭和46年7月1日から同年10月9日までの間、米合衆国カリフォルニア大学（バークレー分校）の地震工学研究センターにおいて実施した調査の結果を、要約して報告するものである。筆者は上記の期間、天然資源の開発利用に関する日米会議（UJNR）の耐風耐震構造専門部会の活動の一環として日米間の共同研究に従事した。

昭和47年1月

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主任研究員 岩崎敏男

Synopsis:

This Technical Memorandum is a report of the author's mission at the University of California, Berkeley, California, U.S.A. during the period of July 1st through October 9th, 1971. The author was participating in a research project "Seismic Effects on Bridges" at the University, as an activity of the Panel on Wind and Seismic Effects on Structures of the U.S.-Japan Co-operative Program in Natural Resources (UJNR).

January, 1972

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1. 概 説

本報告は、昭和46年7月1日から10月9日までの間、カリフォルニア大学（パークレー分校）地震工学研究センター（所長は Professor Joseph Penzien ）において、実施した研究成果につき、その概要のみを記述したものである。研究の課題は「橋梁の耐震設計に関する研究」であったが、具体的には、橋梁に対する地震の影響に関する文献調査を主要内容とし、主に我国における橋梁の震害例、橋梁の耐震性に関する調査研究の成果ならびに耐震設計の現行等につき一貫した報告書を作成することであった。

上記の期間に実施した調査結果の全体は、地震工学研究センター所長である Professor Joseph Penzien に、報告書として原稿のまま提出した。同教授は、この原稿に、我国以外の事例につき加筆を施し、筆者との共著の形で、「橋梁に対する地震の影響に関する文献調査（仮題）（Literature Survey on Seismic Effects on Bridges）」と題するレポートを、同大学地震工学研究センターの研究報告書として、近く公刊する予定である。従ってここでは調査結果の全文の掲載はやめ、その概要を簡単に示すにとどめる。同時に、成果の一部として、日本道路協会が昭和46年1月に制定した道路橋耐震設計指針（本文）の全文を英文に仮訳したものを、付録に掲示する。

筆者は、カリフォルニア大学（パークレー分校）の地震工学研究センターが米合衆国運輸省連邦道路庁（Federal Highway Administration, U. S. Department of Transportation）からの委託で実施している研究プロジェクト「地震力に対し十分な抵抗性を供与するための、道路橋の現行設計法の有効性に関する研究（An Investigation of the Effectiveness of Existing Highway Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances）」の研究スタッフ（土木工学科の研究専門官）として、地震工学研究センターでの研究業務に従事した。この業務は、同時に天然資源の開発利用に関する日米会議（UJNR）の耐風耐震構造専門部会の活動の一環とみなされ日米間の協力が得られた。このため、筆者は、滞米中、上記部会の臨時委員に任命され、多方面の便宜供与をうけた。カリフォルニア大学での筆者の主要業務は、前述したように橋梁の耐震設計に関連する文献（主に我が国のもの）を収集し、内容を把握した上で、系統的なとりまとめを行うこと、さらには、この文献調査を通じて、今後米国において橋梁の耐震設計を改良していく上で必要となると思われる研究課題を究明していくことであった。

実施した調査の概要ならびに道路橋耐震設計指針（英文）を以下に示すことにする。その他の事項については、今後必要に応じて、順次紹介していく予定である。

2. 「橋梁に対する地震の影響に関する文献調査」の概要

「橋梁に対する地震の影響に関する文献調査」として、表-1の目次に示す内容をもつ報告書（原稿）を作成した。この報告書の各章の内容につき、以下にその概要を記述する。

〔1〕第1章 序 論

ここでは、この調査研究の目的と必要性、調査の方法、本報告書の構成、さらに調査の概要を示している。

〔2〕第2章 橋梁構造の震害

ここでは、1923年の関東地震以降に、我国で発生した9回の地震を対象とし、橋梁構造の震害の様相を個別に記述している。

対象とした地震、そのマグニチュード、被害を記述した橋梁数は以下の通りであり、9節にわたって詳述されている。

2.1	1923年関東地震（ $M=7.9$ ）	10橋
2.2	1946年南海地震（ $M=8.1$ ）	2橋
2.3	1948年福井地震（ $M=7.3$ ）	7橋
2.4	1949年今市地震（ $M=6.7$ ）	概況
2.5	1952年十勝沖地震（ $M=8.1$ ）	概況
2.6	1962年宮城県北部地震（ $M=6.5$ ）	概況
2.7	1964年新潟地震（ $M=7.5$ ）	7橋
2.8	1968年えびの地震（ $M=6.1$ ）	3橋
2.9	1968年十勝沖地震（ $M=7.9$ ）	概況

また、本章のまとめとして、最後に

2.10 橋梁の震害の一般的特性

という節を設け、上記の多数の地震で起こった事例を要約し、橋梁の震害の一般的特性につき総括した。

〔3〕第3章 橋梁構造に対する地震の影響に関する研究活動

ここでは、我国で従来より実施されてきた、橋梁の耐震性に関連する各種の調査研究につき、その成果を系統的に紹介している。とりあげた課題は以下の節名に示す通りである。

3.1 日本における地震活動

- 3.2 地震動の特性(応答スペクトル)
- 3.3 地盤震害および地盤支持力に関する考慮
- 3.4 地震時に橋台に作用する土圧
- 3.5 地震時に下部構造に作用する動水圧
- 3.6 橋梁の動的特性(実橋に対する振動実験結果)
- 3.7 橋梁における強震観測
- 3.8 橋梁の地震応答解析
- 3.9 橋梁の動的挙動に関する模型実験

〔4〕第4章 日本の道路橋の耐震設計基準

ここでは、我国における、現行の耐震設計基準を紹介している。まず

4.1 耐震設計基準の変遷

で、我国における道路橋の耐震設計基準の変遷の歴史を記述すると共に、日米を含む多数の国における設計震度の大きさを比較することによって、各国でとられている耐震設計の現状の比較を行っている。つぎに

4.2 一般道路橋の耐震設計指針

では、我国の一般道路橋の耐震設計基準として、日本道路協会が昭和46年1月に制定した道路橋耐震設計指針の要約を示し、その特色を明らかにした。また、

4.3 道路関係の4公団における道路橋の耐震設計基準

では、日本道路公団、首都高速道路公団、阪神高速道路公団および本州四国連絡橋公団における橋梁の耐震設計基準を概説し、その特色を述べた。

〔5〕第5章 要約および結論

ここでは、上述した各種の調査から得られた成果を簡単に要約するとともに、本調査の結論を記述した。まず

5.1 要 約

では、1章～4章に記述した事項を要約した。次に

5.2 結 論

においては、今後橋梁の耐震設計法を改良していく上に、重要視される研究課題は何であるかを指摘することに重点がおかれ、本調査の結論づけが行われた。

今後必要とされる研究課題としては、以下のものが指摘された(ここでは課題のみを示す)。

- (1) 地震活動度に関する工学的評価方法の確立

- (2) 地盤の地震時挙動と橋梁構造に与える影響の究明
- (3) 橋梁下部構造の耐震設計法の確立
- (4) 橋梁上部構造および支承部の設計細目の考案
- (5) 実橋における強震観測の実施と振動特性の把握
- (6) 地震応答解析の推進
- (7) 各種耐震実験の推進
- (8) ある設計基準で設計された橋梁につき、地震に対する極限強度の定量的な評価方法の研究

〔6〕 付 録

1) 付録 A 参考文献

ここでは上述した 1 章～ 5 章の本文中に引用された参考文献があげられている。

2) 付録 B 橋梁に対する地震の影響に関する日本の文献リスト

ここでは、橋梁の耐震性に関連する我国の各種の文献を調査し、内容別に分類し、200 論文を上まわる文献の目録を作成した。分類項目は下記の如くである。

- A. 橋梁の耐震設計基準
- B. 橋梁構造の地震被害
- C. 実橋における強震観測記録とその評価
- D. 実 施 設 計
- E. 研 究 活 動
 - Ea. 地 震 活 動
 - Eb. 地震応答スペクトル
 - Ec. 橋梁の動的特性
 - Ed. 橋梁下部構造に作用する動水圧
 - Ee. 土と基礎の問題
 - Ef. 橋梁の耐震性に関する模型実験
 - Eg. 橋梁の地震応答解析
 - Uh. 橋梁の耐震設計
 - Ei. 特殊橋梁（吊橋など）
 - Ej. 総括的事項
- F. 地震工学に関連するテキストブック
- G. 地震工学に関連する会議講演集

H. そ の 他

Ha. 日本における橋梁工学の一般的情報

Hb. 日英用語辞典

3) 付録 C 橋梁の耐震設計に関する示方書類の要約

ここでは、橋梁の耐震設計に関連する我国の各種の設計示方書・指針につき、その主要内容を、一定の書式にとりまとめている。ここでとりあげた示方書・指針類は下記の14種である。

- (1) 日本道路協会「道路橋耐震設計指針(1971. 1)」
 - (2) 日本道路協会「鋼道路橋設計示方書(1964. 6)」
 - (3) 日本道路協会「鉄筋コンクリート道路橋設計示方書(1964. 6)」
 - (4) 日本道路協会「プレストレストコンクリート道路橋示方書(1968. 3)」
 - (5) 日本道路協会「道路橋下部構造設計指針, くい基礎の設計篇(1964. 3)」
 - (6) 日本道路協会「道路橋下部構造設計指針, 調査および設計一般篇(1966. 11)」
 - (7) 日本道路協会「道路橋下部構造設計指針, 橋台, 橋脚の設計篇(1968. 3)」
 - (8) 日本道路協会「道路橋下部構造設計指針, 直接基礎の設計篇(1968. 3)」
 - (9) 日本道路協会「道路橋下部構造設計指針, ケーソン基礎の設計篇」
 - (10) 日本道路公団「設計要領(1970. 1)」
 - (11) 首都高速道路公団「耐震設計基準(1967. 7)」
 - (12) 阪神高速道路公団「耐震設計基準(1968. 12)」
 - (13) 日本土木学会「本州四国連絡橋耐震設計指針(1967. 5)」
 - (14) 日本土木学会「鉄道橋下部構造の耐震設計に関する報告(1968)」
- 4) 付録 D 日本道路協会「道路橋耐震設計指針(1971. 1)」の完訳

これは、日本道路協会が昭和46年1月に制定した「道路橋耐震設計指針」の全文を英文に翻訳したものである。この指針は、我国における最新の指針であり、近年における各種の研究成果をおりこんだ画期的な耐震設計指針といえるものである。このため、特に、その全文を完訳して示している。

なお、この指針は、米国のみでなく諸外国においても重要な参考資料になると考えられるため、今回の報告書においても、付録として掲載することにした。

表1 「橋梁に対する地震の影響に関する文献調査」の目次

	LITERATURE SURVEY ON
	SEISMIC EFFECTS ON BRIDGES
	TABLE OF CONTENTS (Tentative)
	ABSTRACT
	PREFACE
	TABLE OF CONTENTS
	LIST OF PICTURES
	LIST OF FIGURES
	LIST OF TABLES
CHAPTER 1	INTRODUCTION
CHAPTER 2	SEISMIC DAMAGE TO BRIDGE STRUCTURES
2.1	Kanto Earthquake of 1923
2.2	Nankai Earthquake of 1946
2.3	Fukui Earthquake of 1948
2.4	Imaichi Earthquake of 1949
2.5	Tokachi-oki Earthquake of 1952
2.6	Northern Miyagi Earthquake of 1962
2.7	Niigata Earthquake of 1964
2.8	Ebino Earthquake of 1968
2.9	Tokachi-oki Earthquake of 1968
2.10	General Features of Seismic Damage to Bridge Structures
CHAPTER 3	RESEARCH ACTIVITIES ON SEISMIC EFFECTS ON BRIDGE STRUCTURES
3.1	Seismic Activities in Japan
3.2	Characteristics of Strong Ground Motion (Response Spectra)
3.3	Consideration on Ground Disasters and Bearing Capacities of Ground Soils
3.4	Earth Pressures on Abutments during Earthquakes
3.5	Hydrodynamic Pressures on Substructures during Earthquakes
3.6	Dynamic Properties of Bridges (Results of Vibration Test on Actual Bridges)
3.7	Measurements of Strong Motion Earthquakes at Bridges
3.8	Dynamic Analyses on Earthquake Responses of Bridges
3.9	Model Tests on Dynamic Behaviors of Bridges

CHAPTER 4 EARTHQUAKE-RESISTANT DESIGN CRITERIA FOR HIGHWAY
BRIDGES IN JAPAN

- 4.1 Brief History of Earthquake-Resistant Design Criteria
- 4.2 Specifications for Earthquake-Resistant Design of General Highway Bridges
- 4.3 Criteria for Earthquake Resistant Design of Highway Bridges in four Highway-Public Corporations

CHAPTER 5 SUMMARY AND CONCLUSIONS

- 5.1 Summary
- 5.2 Conclusions

APPENDIX A Reference

APPENDIX B A List of Japanese Bibliography on Seismic Effects on Bridges

APPENDIX C Summaries of Specifications Relating to Earthquake-Resistant Design of Bridges

APPENDIX D A Complete Translation of "Specifications for Earthquake-Resistant Design of Highway Bridges," Japan Road Association, January, 1971

3. 謝 辞

本報告書に記述した日米間の共同研究プロジェクトの推進に際しては、建設省事務次官、建設技監をはじめとする建設省幹部ならびに関係部局の方々の積極的な協力が得られた。このプロジェクトの実現は上記の皆様の御尽力によるところが大きい。ここに記して深甚の感謝の意を表する次第である。

また、かねてより日米両政府間において、活発な協力活動を実施している天然資源の開発利用に関する日米会議（U J N R）の耐風耐震構造専門部会の果たした役割もみのがせない。すなわち、昭和46年5月東京にて挙行された同部会の第3回日米合同会議において、研究専門家の交換計画を含む両国間の積極的な協力関係の樹立を確認したことである。さらに、その後の両国部会長、日本側は建設省土木研究所長の伊吹山四郎博士、米国側は国立標準局の住宅技術計画部の部長である E. O. Pfrang 博士、間の交信により、上記の研究プロジェクトを効果的に推進していくためには、研究員の派遣を含む日米間の緊密な協力が必要であること、滞米中の筆者を上記部会の臨時委員に任命し、U J N R の各種活動の一環としてこのプロジェクトに対し日米間で協力をおしまないことが確認された。

これらのU J N R の協力は、米国の各種機関との交信、各種資料の収集、さらには必要に応じて実施した研究機関の視察等を容易に可能とし、滞米中における研究の推進に大きな貢献をなした。U J N R の耐風耐震構造部会の部会長である伊吹山博士およびPfrang博士をはじめとして、両国の委員の皆様に深く感謝する次第である。

最後にまた、筆者の滞米中を通じて、公私ともども協力を惜しまなかった地震工学研究センター所長 Professor Joseph Penzien, 同所長の秘書である Mrs. Elenora D. Utter, 更に良き同僚となり、筆者の拙文の修正に尽力してくれた Mr. William Wagy に対して心から感謝の意を表するものである。

JAPAN ROAD ASSOCIATION'S
SPECIFICATIONS FOR EARTHQUAKE RESISTANT DESIGN
OF HIGHWAY BRIDGES (JANUARY 1971)

Tentative English Version

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About the Tentative English Version

This tentative English version of "Specifications for Earthquake Resistant Design of Highway Bridges" published by the Japan Road Association in January, 1971 is printed by the Public Works Research Institute (PWRI), Ministry of Construction. The version will be the Appendix of a Technical Memorandum of PWRI, presented by Toshio Iwasaki, Senior Research Engineer of Earthquake Engineering Section as a formal report of his mission at the University of California, Berkeley, California, U. S. A. during the period of July 1st through October 9th, 1971.

The English version was originally prepared by him while he was participating in a research project "Seismic Effects on Bridges" at the University, as an activity of the Panel on Wind and Seismic Effects of the U. S. -Japan Cooperative Program in Natural Resources (UJNR). During the period he was appointed to be a temporary member of the UJNR Panel.

Professor Joseph Penzien, Director of the Earthquake Engineering Research Center at the University of California, Berkeley acts as the principal investigator of the above research project, which is sponsored by the Federal Highway Administration, U. S. Department of Transportation.

FOREWORD

The earthquake resistant design of highway bridges is currently being conducted in accordance with the provisions of Section 13 (Section 2.9 in the English version) of the Specifications for Design of Steel Highway Bridges. Since the provisions concerning earthquake resistant design are not very comprehensive as stated in the preface of the Specifications, the design is in many respects dependent on the designer's interpretation. Accordingly the results of the various designs were not always consistent. To alleviate this problem it was necessary to revise the Section 13 of the Specifications and to draw up new detailed Specifications exclusively for earthquake resistant design of highway bridges.

In response to the need for revisions, the Japan Road Association established the Earthquake Resistant Design Subcommittee under the Highway Bridge Committee in 1966. Between 1966 and 1968 a preliminary draft was prepared. The task was taken over by the Earthquake Resistant Design Branch-Committee of the Specifications Subcommittee in 1968, when the Highway Bridge Committee was reorganized.

The Subcommittee and the Branch-Committee have seriously attempted to draw up the revision through surveys of the available research work, discussions with specialists from universities and other organizations, and trial calculations and bridge designs at several stages in accordance with the interim drafts, and finally completed the revision of the Specifications in 1971.

The new Specifications are completed basically in accordance with the provisions of Section 13 of the Specifications for Design of Steel Highway Bridges, and also by extensively referring to the results of recent investigations on earthquake engineering. Therefore the design methodology of Section 13 of the current Specifications was generally traced in establishing the new Specifications.

In determining the design seismic coefficient the importance factor is newly considered in addition to the traditional factors such as zone factor and ground condition factor. Moreover, for bridges which have longer natural periods the factor on the natural periods of the structures are considered.

Although the design horizontal seismic coefficient used to be 0.1 to 0.35 in the Section 13 of the current Specifications, the value is reduced to 0.1 to 0.3 and over 0.3 only for rare occasions in the new Specifications. This is based on the results of recent investigations that it seems more essential to pay attention to soil conditions rather than to the magnitude of seismic coefficients in design of bridges against seismic forces.

Since design seismic forces in the new Specifications were determined by referring to the seismic risk map expected for 75 years presented by Dr. Kawasumi and by reflecting the results of a recent study on the meteorological data since the Kanto Earthquake in 1923, seismic forces considered in the new Specifications would cover the severest earthquakes experienced in individual regions in Japan.

The seismic forces would also correspond to those due to the greatest earthquakes in the seismological point of view.

In the new Specifications the following aspects differ from the previous method.

- (1) The design seismic coefficients of a bridge are determined systematically dependent on geographical location of the bridge site, the ground conditions at each substructure site, and the importance of the bridge.
- (2) The vertical design seismic coefficients are generally neglected.
- (3) The design seismic coefficients for structural parts, soils and water below the ground surface are neglected.
- (4) Special attention is paid to very soft soil layers and soil layers vulnerable to liquefaction during earthquakes. The bearing capacities of these layers are neglected in the design in order to assure high earthquake resistance for those structures which are constructed in these layers.
- (5) The design seismic coefficients for bridges with high-rise piers are determined in accordance with the modified seismic coefficient method considering structural response, instead of the conventional way in which the design seismic coefficients increase with the height of the piers.
- (6) Special attention is also paid to the design of structural details in view of earthquake damage previously experienced to bridge structures. To this end provisions are specified for bearing supports and devices for preventing superstructures from falling.

Although the Specifications were developed through extensive studies and discussions of research reports on earthquake engineering from throughout the world as well as Japan, some unexpected problems may arise when they are applied to actual structures. Because of this and the fact that technological improvements take place very rapidly, another revision will be required in the future. Everyone is urged to indicate any insufficient points or to make any suggestions for the present Specifications, so that they may be improved in subsequent revisions.

The present publication is limited the provisions which are considered to be sufficient for the design of bridges. Comments which describe the basis and the application methods of the provisions will be issued immediately after their completion. They will be indispensable when it is necessary to revise the present Specifications again or when any parts of the provisions are found to be uncertain in their application.

Dr. Hiroshi Kawasumi, Dr. Tsutomu Terashima, and Mr. Tsutomu Tomita participated in the discussions on special matters as special members of the Committee. Their cooperation is greatly appreciated.

January, 1971

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Chapter 1 General

1.1 Scope

The provisions in the Specifications apply to earthquake resistant design of highway bridges with spans not longer than 200 meters, to be built on expressways, national highways, prefectural highways and principal municipal highways.

In regard to matters which are not specified herein, the following Specifications shall be conformed to, in accordance with the type of the structure considered.

Specifications for Design of Steel Highway Bridges,	Japan Road Association
Specifications for Design of Welded Highway Bridges,	Japan Road Association
Specifications for Design and Construction of Composite Beams for Steel Highway Bridges,	Japan Road Association
Specifications for Design of Reinforced Concrete Highway Bridges,	Japan Road Association
Specifications for Design and Construction of Friction Type Joints in Steel Highway Bridges Using High Strength Bolts,	Japan Road Association
Specifications for Design of Substructures of Highway Bridges,	Japan Road Association
Specifications for Prestressed Concrete Highway Bridges,	Japan Road Association

1.2 Definitions of Terms

The following definitions apply only to the provisions of these Specifications.

- (1) Earthquake: A phenomenon with propagation of vibration due to a sudden naturally occurring movement at a certain portion inside the earth.
- (2) Earthquake Ground Motion: Vibration of ground during earthquakes.
- (3) Design Seismic Coefficient: A coefficient indicating the magnitude of acceleration to be considered for earthquake resistant design of structures, and expressed as a fraction of the acceleration of gravity.
- (4) Seismic Coefficient Method: A method for earthquake resistant design in which seismic forces are assumed to act on structures as static forces.
- (5) Modified Seismic Coefficient Method Considering Structural Response: A method for earthquake resistant design which is developed through modification of the seismic coefficient method by considering characteristics of earthquake ground motions, dynamic properties of structures, etc. for those structures with long fundamental periods of vibration.
- (6) Modified Design Seismic Coefficient Considering Structural Response: A design seismic coefficient used in the modified seismic coefficient method considering structural response in which the design seismic coefficient is dependent on the fundamental period of the structure.

(7) Standard Horizontal Design Seismic Coefficient: The Horizontal design seismic coefficient which applies to a bridge which is located in a zone where severe earthquakes have frequently occurred or where severe earthquakes have a high potential of occurrence, and which is constructed in soil layers with ground conditions corresponding to group 2 in Table 4.5.

(8) Seismic Zone Factor: A factor to decrease the horizontal design seismic coefficient in accordance with the geographical location of the structural site. The factor has the value of 1.0 for those zones where severe earthquakes have frequently occurred or where severe earthquakes have a high potential of occurrence.

(9) Ground Condition Factor: A factor to modify the horizontal design seismic coefficient depending upon the ground conditions of the structural site. The factor is intended to unify the margin of safety against earthquake disturbances among structures constructed in various ground conditions, in consideration of damage previously experienced.

(10) Importance Factor: A factor to modify the horizontal design seismic coefficient depending on the importance of the structure.

(11) Inertia Force: Product of the weight of the structural body and the design seismic coefficient.

(12) Seismic Force: Any forces such as inertia forces, earth pressures, etc. to which structures are subjected during earthquakes.

(13) Coefficient of Subgrade Reaction: A coefficient given by the following formula, which is also shown in Section 3.1 of the Volume

of Spread Foundations, the Specifications for Design of Substructures of Highway Bridges.

$$K = \frac{p}{\delta}$$

where

K : coefficient of subgrade reaction in kg/cm^3

p : loading pressures in kg/cm^2

δ : displacement in cm

(14) Earthquake Response Analysis: An analysis of the dynamic behavior of structures during earthquakes.

Chapter 2 Basic Principles for Earthquake Resistant Design

- (1) The earthquake resistant design for a highway bridge shall provide sufficient stability against seismic disturbances for the structure as a unit and also for all parts thereof, including superstructures, substructures, and surrounding soils, considering topographical and geological conditions at the site.
- (2) The seismic coefficient method basically shall apply to the earthquake resistant design of relatively rigid structures.
- (3) The modified seismic coefficient method considering structural response shall apply to the earthquake resistant design of relatively flexible structures which are of long fundamental periods of vibration, such as bridges with high-rise piers.

The earthquake response analysis shall also be adopted for those structures, for which detailed investigations are required.

- (4) Particular attention shall be paid to preventing the fall of superstructures from substructures due to the movements during earthquakes.

When the design allowing for partial failures of the whole system or local failures of certain structural members is required on the basis of economical considerations, special attention to preventing the fall of superstructures shall be paid.

Chapter 3 Loads and Conditions in Earthquake Resistant Design

3.1 General

(1) The following loads shall be taken into account in earthquake resistant design. The appropriate loads shall be selected from this list on the basis of the location and the type of the structure.

1. Dead Loads
2. Earth Pressures
3. Water Pressures
4. Buoyancy or Uplifts
5. Effects of Temperature Change
6. Effects of Shrinkage due to Humidity in Concrete Structures
(Including Creep Effects)
7. Seismic Effects
8. Effects of Friction at Bearing Supports
9. Effects of Consolidation and Settlement of Ground
10. Effects of Movements of Supports
11. Other Loads

(2) Combination of Loads

Design conditions shall be determined considering the following four cases. The combination of various loads listed above shall be decided in accordance with the provisions in individual Specifications according to the type of the bridge considered.

- | | |
|--------|---|
| Case 1 | Maximum stresses will be expected in the members |
| Case 2 | Maximum reactions will be expected in the foundation soils |
| Case 3 | Maximum displacements will be expected at the supports |
| Case 4 | The structure will become critical conditions due to overturning, sliding, etc. |

3.2 Seismic Effects

The following seismic forces shall be taken into account to determine seismic effects in design of a bridge structure.

1. Inertia forces due to the dead weight of the structure: The magnitude of the inertia forces shall be the product of the weight of the structure and the design seismic coefficient. The design seismic coefficient shall be obtained in accordance with the provisions in Chapter 4 "Design Seismic Coefficient ."
2. Inertia forces due to the superimposed weight: The magnitude of the inertia forces shall be the product of the superimposed weight and the design seismic coefficient. The design seismic coefficient also shall be obtained in accordance with the provisions in Chapter 4 "Design Seismic Coefficient ."
3. Earth pressures during earthquakes.
4. Hydrodynamic pressures during earthquakes.

3.3 Inertia Forces

- (1) Both for the seismic coefficient method and the modified seismic coefficient method, bridge substructures shall be subjected to the

inertia forces of superstructures during earthquakes as follows

(refer to Fig. 3.1):

- (a) Horizontal seismic force acting on A_L (left abutment) shall be

$$H_{AL} = R_{AL} \cdot f_{AL} \quad (3.1)$$

$$\text{where } H_{AL} \leq \frac{1}{2} k_b W_A \quad (3.2)$$

In other words,

$$H_{AL} = \text{smallest of } \begin{cases} R_{AL} \cdot f_{AL} \\ \text{or} \\ \frac{1}{2} k_b W_A \end{cases}$$

- (b) Horizontal seismic force acting on P_1 (Pier 1) shall be

$$H_{AL} + H_{BL} = \text{largest of } \begin{cases} k_b W_A \text{ (} H_{BL} = 0 \text{ in this case)} \\ \text{or} \\ \frac{1}{2} k_b W_A + R_{BL} \cdot f_{BL} \end{cases} \quad (3.4)$$

In equation (3.4), $R_{BL} \cdot f_{BL}$ shall satisfy

$$R_{BL} \cdot f_{BL} \leq \frac{1}{2} k_b W_B \quad (3.5)$$

- (c) Horizontal seismic force acting on A_R (right abutment) shall be

$$H_{BR} = k_b W_B \quad (3.6)$$

where

f_{AL} : Static friction coefficient at movable support A_L ,

f_{BL} : Static friction coefficient at movable support B_L ,

H_{AL} : Horizontal seismic force (inertia force or friction force) acting on A_L due to Girder A,

H_{AR} : Horizontal seismic force (inertia force) acting on P_1 due to Girder A,

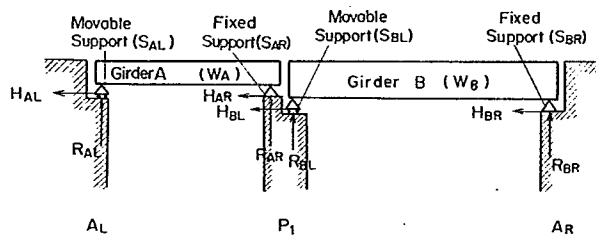


Fig. 3.1 Inertia Forces of Superstructures

- H_{BL} : Horizontal seismic force (inertia force or friction force) acting on P_1 due to Girder B ,
- H_{BR} : Horizontal seismic force (inertia force) acting on A_R due to Girder B ,
- k_b : Horizontal design seismic coefficient (in accordance with the provisions in Chapter 4 "Design Seismic Coefficient": k_b in equation (4.1) for the seismic coefficient method, or k_{bm} in equation (4.2) for the modified seismic coefficient method, shall be employed) ,
- R_{AL} : Reaction at A_L due to W_A ,
- R_{AR} : Reaction at P_1 due to W_A ,
- R_{BL} : Reaction at P_1 due to W_B ,
- R_{BR} : Reaction at A_R due to W_B ,
- W_A : Dead weight of Girder A , and
- W_B : Dead weight of Girder B .

(2) Inertia forces shall be assumed to act at the center of gravity of the structure. In designing the substructure, inertia forces exerted from superstructures may be assumed to act at the level of the base of the supports in the longitudinal direction to the bridge axis, and at the level of the center of gravity of the superstructures in the transverse direction.

In the transverse direction the level of the center of gravity of the superstructures may generally be taken as the lower level of the floor slab.

(3) Inertia forces shall be assumed to come from two horizontal directions: longitudinal and transverse to the bridge axis, or parallel and perpendicular to the principal axis of any structural elements of the bridge.

3.4 Earth Pressures during Earthquakes

Earth pressures during earthquakes shall be determined in accordance with the provisions in the Specifications for Design of Substructures of Highway Bridges (See Reference 1 at the end of the English Version). The horizontal seismic coefficient in the calculation of earth pressures shall be in accordance with the provisions in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method."

3.5 Hydrodynamic Pressures during Earthquakes

Hydrodynamic pressures during earthquakes shall be determined by the following formulas. The pressures shall be assumed to act in the same direction as that of the inertia forces given by the provisions in Section 3.3 "Inertia Forces."

(1) Hydrodynamic Pressures on Walls

Hydrodynamic pressures acting on one side of a wall-type structure shall be determined as follows (refer to Fig. 3.2):

$$P = \frac{7}{12} k_h W_o b h^2 \quad (3.7)$$

$$h_g = \frac{1}{2} h \quad (3.8)$$

where

b : Width of the wall in meters in the perpendicular direction to that of the pressure ,

h : Depth of water in meters ,

h_g : Height of the total hydrodynamic pressure in meters above the bottom of the water ,

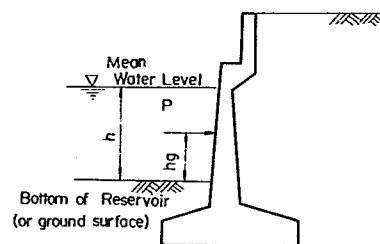


Fig. 3.2 Hydrodynamic Pressures on Wall-Type Structures

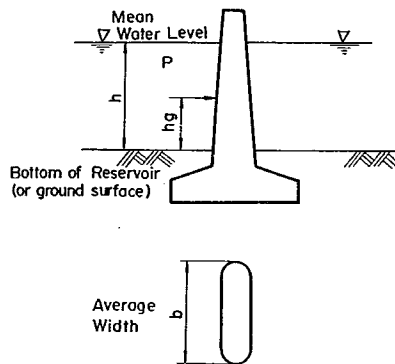


Fig. 3.3 Hydrodynamic Pressures on Column-Type Structures

k_h : Horizontal design seismic coefficient given by the provisions in Section 4.2 " Design Seismic Coefficient in the Seismic Coefficient Method , "

P : Total hydrodynamic pressure in t, and

W_o : Unit weight of water in t/m³ .

(2) Hydrodynamic Pressures on Columns

Hydrodynamic pressures acting on a column-type structure surrounding by water shall be determined as follows (refer to Fig. 3.3):

$$P = \frac{3}{4} k_h W_o b^2 h \left(1 - \frac{b}{4h}\right) \quad \text{for } \frac{b}{h} \leq 2 \quad (3.9)$$

$$P = \frac{3}{8} k_h W_o b^2 h \quad \text{for } \frac{b}{h} > 2 \quad (3.10)$$

$$h_r = \frac{1}{2} h \quad (3.11)$$

3.6 Ground Surface Assumed in Earthquake Resistant Design

In earthquake resistant design the bearing capacities of soil layers which are specified in Section 3.7 "Soil Layers Whose Bearing Capacities are Neglected in Earthquake Resistant Design" shall be neglected. Ground surface in design shall be assumed to be the level of the lower boundary of the neglected layer, if the layer extends continuously below the actual ground surface.

For this case the neglected layer shall be assumed to have properties of zero cohesion and zero angle of internal friction.

3.7 Soil Layers Whose Bearing Capacities are Neglected in Earthquake Resistant Design

(1) Sandy Soil Layers Vulnerable to Liquefaction

Saturated sandy soil layers which are within 10 meters of the actual ground surface, have a standard penetration test N-value less than 10, have a coefficient of uniformity less than 6, and also have a D_{20} -value on the grain size accumulation curve between 0.04 mm and 0.5 mm, shall have a high potential for liquefaction during earthquakes. Bearing capacities of these layers shall be neglected in design.

Saturated sandy soil layers which have a D_{20} -value between 0.004 and 0.04 mm or between 0.5 mm and 1.2 mm, may liquefy during earthquakes, and shall be given particular attention. Estimation whether or not these layers will liquefy shall be made in accordance with the available information on liquefaction problems.

When a special investigation is performed, the provisions in this item (1) in Section 3.7 may not be required to apply.

(2) Cohesive Soil Layers and Silty Soil Layers

Bearing capacities of cohesive soil layers and silty soil layers, which are within 3 meters of the actual ground surface, and are very soft such as those with the compression strength, determined by unconfined compression tests or field tests, less than 0.2 kg/cm^2 , shall be neglected in design.

(3) Weight of Soil Layers Whose Bearing Capacities are Neglected

The weight of soil layers whose bearing capacities are neglected shall have surcharge effects on the lower ground.

3.8 Buoyancy or Uplifts

Buoyancy or uplifts shall be determined in accordance with the provisions in the Specifications for Design of Substructures of Highway Bridges (See Reference 2 at the end of the English Version).

Chapter 4 Design Seismic Coefficient

4.1 General

The design seismic coefficient shall generally be determined in accordance with the provisions in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method." For those bridges, however, which have flexible piers and long fundamental periods, such as those with piers higher than 25 meters above the ground surface, the design seismic coefficient shall be determined in accordance with the provisions in Section 4.4 "Design Seismic Coefficient in the Modified Seismic Coefficient Method Considering Structural Response."

4.2 Design Seismic Coefficient in the Seismic Coefficient Method

(1) The horizontal design seismic coefficient shall be determined by the following formula:

$$k_h = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_o \quad (4.1)$$

where

k_h : Horizontal design seismic coefficient ,

k_o : The standard horizontal design seismic coefficient (= 0.2) ,

ν_1 : Seismic zone factor ,

ν_2 : Ground condition factor , and

ν_3 : Importance factor .

The value of k_h shall be rounded to two decimals. The minimum value of k_h shall be considered as 0.10. The values of factors ν_1 ,

ν_2 and ν_3 shall be obtained by the provisions in Section 4.3

"Factors for Modifying the Standard Horizontal Design Coefficient."

(2) The vertical design seismic coefficient, k_v may generally be considered as 0. Vertical seismic forces for design of bearing supports, however, shall be determined in accordance with the provisions in Section 5.3 "Vertical Seismic Forces for Design of Connections between Superstructures and Substructures."

(3) The horizontal design seismic coefficient may be considered as 0 for structural parts, soils and water below the assumed ground surface in design. The assumed ground surface in design shall be determined in accordance with the provisions in Section 3.6 "Ground Surface Assumed in Earthquake Resistant Design."

Item (3), however, shall not apply to underground structures such as culverts.

4.3 Factors for Modifying the Standard Horizontal Design Seismic Coefficient

(1) Seismic Zone Factor

Seismic zone factor shall be determined in accordance with Table 4.1, in which the zone classification shall be determined from Fig. 4.1 or Table 4.2.

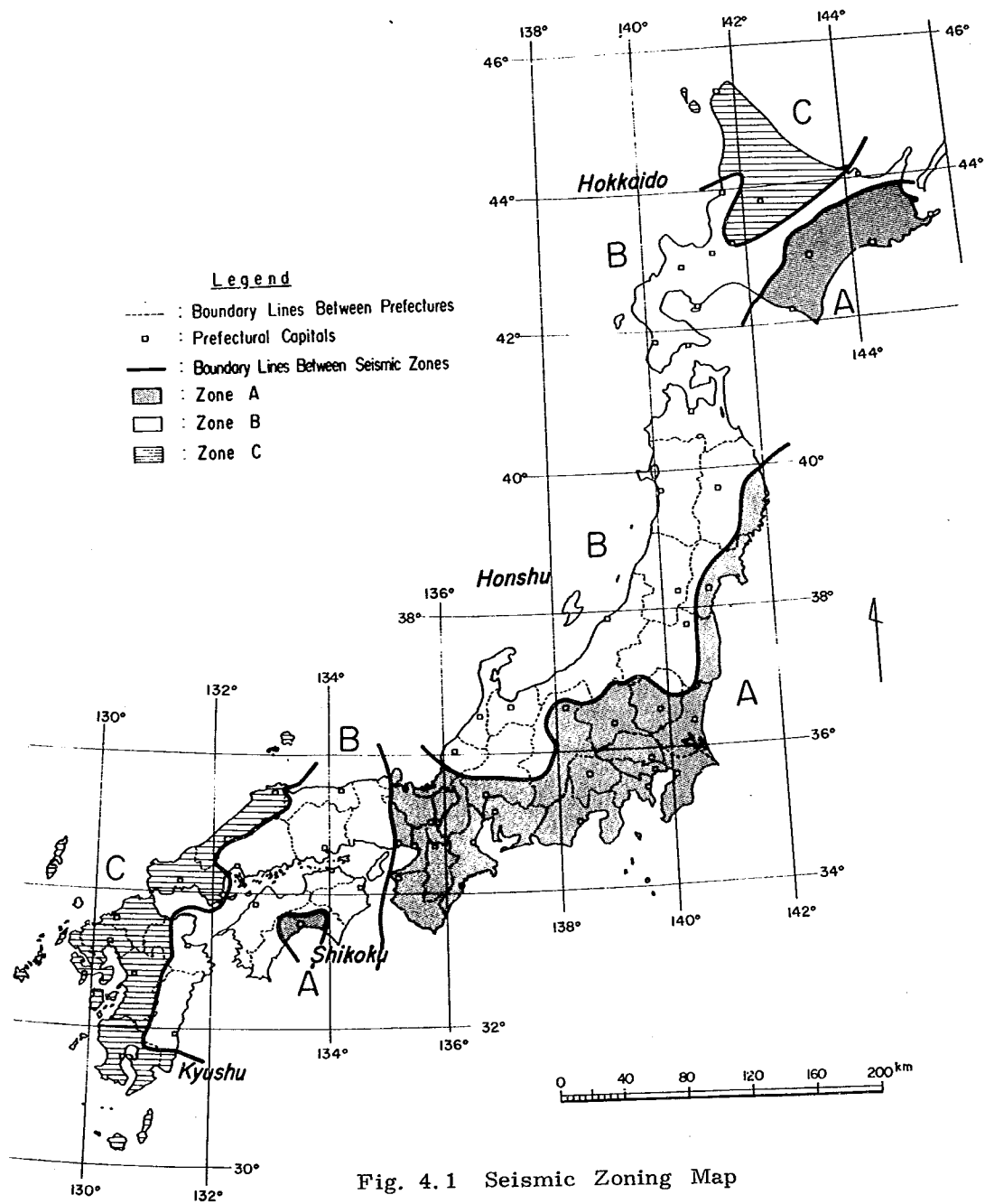


Fig. 4.1 Seismic Zoning Map

Table 4.1 Seismic Zone Factor ν_1

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

(Note)

Table 4.2, which indicates in detail the seismic zone through the use of place names, is omitted in the English version.

(2) Ground Condition Factor

Ground condition factor shall be determined in accordance with Table 4.3.

Table 4.3 Ground Condition Factor ν_2

Group	Definitions ¹⁾	Value of ν_2
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer ²⁾ with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer ²⁾ with depth greater than 10 meters above bedrock (2) Alluvial layer ³⁾ with depth less than 10 meters above bedrock	1.0
3	Alluvial layer ³⁾ with depth less than 25 meters, which has soft layer ⁴⁾ with depth less than 5 meters	1.1
4	Other than the above	1.2

- (Notes) 1) Since these definitions are not very comprehensive, the classification of ground conditions shall be made with adequate consideration of the bridge site.
- Depth of layer indicated here shall be measured from the actual ground surface.
- 2) Diluvial layer implies a dense alluvial layer such as a dense sandy layer, gravel layer, or cobble layer.
- 3) Alluvial layer implies a new sedimentary layer made by a landslide.
- 4) Soft layer is defined in Section 3.7 "Soil Layer Whose Bearing Capacities are Neglected in Earthquake Resistant Design."

(3) Importance Factor

Importance factor shall be determined in accordance with Table 4.4 .

Table 4.4 Importance Factor ν_3

Group	Definitions	Value of ν_3
1	Bridges on expressways (limited-access highways), general national highways and principal prefectural highways. Important Bridges on general prefectural highways and municipal highways.	1.0
2	Other than the above	0.8

Note: The value of ν_3 may be increased up to 1.25 for special cases in Group 1.

4.4 Design Seismic Coefficient in the Modified Seismic Coefficient

Method Considering Structural Response

The design seismic coefficient specified in this Section shall apply to the design of superstructures and substructures of those bridges which have flexible piers and relatively long fundamental periods, such as ones in which the height of the substructures is 25 meters or more above the ground surface in design.

The above-mentioned ground surface in design shall be specified in Section 3.6 "Ground Surface Assumed in Earthquake Resistant Design."

The Design seismic coefficients in this Section shall be determined by modifying the horizontal design seismic coefficient in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method," on the basis of characteristics of strong earthquake ground motions recorded, and dynamic properties of bridge structures, such as natural periods, mode shapes and damping capacities of bridge substructures.

After obtaining the design seismic coefficient, the method for applying seismic loads for the design shall be the same as specified by the provisions in Chapter 3 "Loads and Conditions in Earthquake Resistant Design."

The provisions in this Section shall not apply to the design of superstructures of those bridges in which superstructures are flexible and have longer periods, such as suspension bridges.

4.4.1 Design Seismic Coefficient

(1) The horizontal design seismic coefficient shall be determined by the following formula:

$$k_{hm} = \beta k_h \quad (4.2)$$

where

k_{hm} : Horizontal design seismic coefficient in the modified seismic coefficient method considering structural response ,

k_h : Horizontal design seismic coefficient given by eq. (4.1) , and

β : A factor dependent on the fundamental period of the bridge, and obtained by Table 4.5 or Fig. 4.2.

For structures whose fundament periods are shorter than 0.5 sec., β may be considered as 1.0.

The value of k_h shall be rounded to two decimals. The minimum value of k_h shall be considered as 0.05.

(2) The vertical design seismic coefficient shall be provided in accordance with the provisions in Item (2) of Section 4.2.

(3) The horizontal design seismic coefficient for the portions below the assumed ground surface in design, shall be provided in accordance with the provisions in Item (3) of Section 4.2.

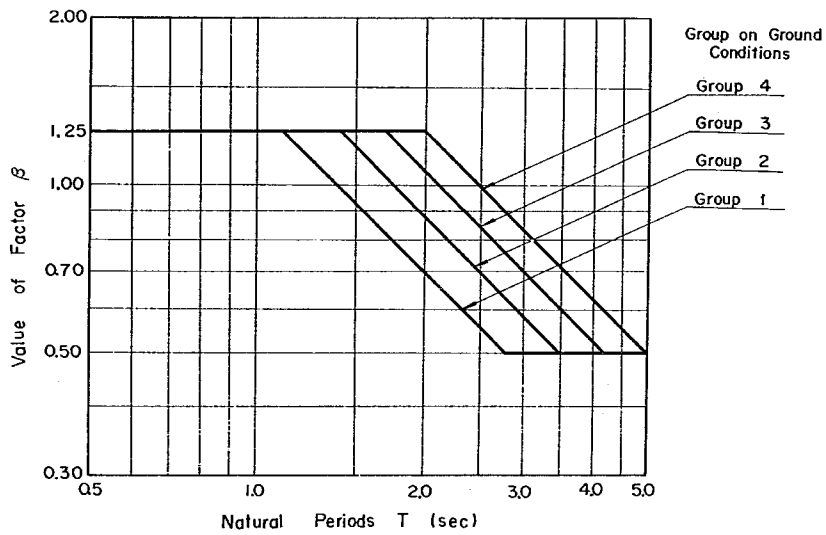


Fig. 4.2 Factor β (For reference of Table 4.5)

Table 4.5 Value of β

Group on Ground Conditions	Value of β for Fundamental Period T (sec.)		
1	$\beta = 1.25$ for $0.5 \leq T \leq 1.1$	$\beta = 1.40/T$ for $1.1 \leq T \leq 2.8$	$\beta = 0.50$ for $T \geq 2.8$
2	$\beta = 1.25$ for $0.5 \leq T \leq 1.4$	$\beta = 1.75/T$ for $1.4 \leq T \leq 3.5$	$\beta = 0.50$ for $T \geq 3.5$
3	$\beta = 1.25$ for $0.5 \leq T \leq 1.7$	$\beta = 2.10/T$ for $1.7 \leq T \leq 4.2$	$\beta = 0.50$ for $T \geq 4.2$
4	$\beta = 1.25$ for $0.5 \leq T \leq 2.0$	$\beta = 2.50/T$ for $2.0 \leq T \leq 5.0$	$\beta = 0.50$ for $T \geq 5.0$

(Notes) 1) Refer to Fig. 4.2.

2) Refer to Table 4.3 regarding groups on ground conditions.

4.4.2 Method for Obtaining Fundamental Periods

Fundamental natural periods of a bridge shall be determined for the individual system consisting of each substructure and the part of superstructures supported by it rather than for the structural system as a whole.

(1) Bridges Supported by Spread Foundations or Pile Foundations

For those bridges which are supported by spread foundations or pile foundations, the fundamental periods may be obtained from Table 4.6.

Any formulas in Table 4.6 shall apply to bridges in which the level of the base of the footing is lower than that of the assumed ground surface in design and the deformation of the substructure is mainly caused by the elastic flexural deformation of the pier which is the upper part of the substructures above the top of the footing. Therefore, they shall not apply to bridges in which the level of the base of the footing is higher than that of the assumed ground surface in design.

Table 4.6 Fundamental Periods of Bridges Supported by Spread Foundations or Pile Foundations

Type of Structural System	Direction	Formulas for Fundamental Periods	
		Material of Pier	
		Reinforced Concrete	Steel
1 A bridge where most superstructures are continuous, have fixed supports (or movable supports specified in Article 5.2.1) on most substructures, and also have rigid abutments, to one of which the extreme end of the superstructures is connected with a fixed support (See Fig. 4.3)	Transverse	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{3EIg}} h^3$ (4.3)	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{4.5EIg}} h^3$ (4.4)
	Longitudinal	$T = \frac{\pi}{8} \sqrt{\frac{W_p}{EIg}} h^3$ (4.5)	
2 Other than the above: For example, a bridge with simple supports	Longitudinal or Transverse	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{3EIg}} h^3$ (4.6)	

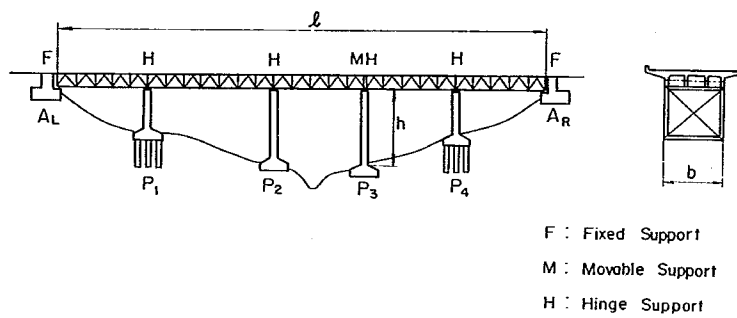


Fig. 4.3 An Example of Type 1 of Structural System in Table 4.6

where

T : Fundamental period in second of the system consisting of a substructure and the section of the superstructures which it supports,

W_p : The weight of the pier in t ,

W_s : The weight of the section of superstructures in t supported by the substructure being considered ,

E : Young's modulus of the pier in t/m^2 ,

I : Moment of inertia of the pier in m^4 in the direction considered.
For piers with varying section with the height, I may be an average value ,

h : The height of the pier in m , and

g : Acceleration of gravity (= 9.8 m/sec^2) .

(Note)

Eq. (4.4) shall apply to those bridges which have a ratio of the length between supports on both abutments to the width between outside girders, less than approximately 50 (refer to l/b in Fig. 4.3).

(2) Bridges Supported by Caisson Foundations

For those bridges which are supported by caisson foundations, the fundamental periods may be obtained from Table 4.7.

Table 4.7 Fundamental Periods of Bridges Supported
by Caisson Foundations

Type of Structural System		Direction	Formulas for Fundamental Periods
1	Type 1 in Table 4.6	Transverse	one of eqs. (4.3) (or (4.4)) and (4.7), which gives the largest value of β
		Longitudinal	eq. (4.5)
2	Type 2 in Table 4.6	Transverse	one of eqs. (4.6) and (4.7), which gives the largest value of β
		Longitudinal	

(Note)

$$T = 2\pi \sqrt{\frac{(h + \frac{2}{3}l)^2 W_u + (\frac{1}{3}h^2 + \frac{2}{3}hl + \frac{4}{9}l^2) W_p + \frac{l^2}{9} W_c}{g \{ K_H \frac{bl^3}{36} + K_S \frac{Al^2}{9} + K_V lB \}}} \quad (4.7)$$

where

T: Fundamental period in second of the system consisting of a substructure and the section of the superstructures which it supports,

W_p : The weight of the pier in t,

W_u : The weight of the part of superstructures in t supported by the substructure being considered,

W_c : The weight of the caisson foundation in t,

h: The height of pier in m,

A: Cross-sectional area in m^2 at the base of the caisson foundation,

I_B : Moment of inertia in m^4 at the base of the caisson foundation in the direction considered,

K_H : Horizontal coefficient of subgrade reaction in t/m^3 at the level of the base of the caisson foundation,

- K_v : Vertical coefficient of subgrade reaction in t/m^3 at the base of the caisson foundation ,
- K_s : Horizontal coefficient of subgrade reaction in t/m^3 for shear deformation at the base of the caisson foundation , and
- g : Acceleration of gravity (= 9.8 m/sec^2).

Chapter 5 General Provisions for Design of Structural Details

5.1 General

Every bridge structure or every portion thereof shall be designed and constructed to resist seismic forces as provided in Chapter 1 through Chapter 4 and to meet the provisions for design of structural details specified in this Chapter.

Moreover, attention shall be paid to the following respects.

(1) For those abutments which are constructed in soft ground layers, the failure of the ground layer during earthquakes shall be checked.

(2) For those bridges in which any adjacent substructures have different ground conditions, different type of structural systems, or different structural dimensions, special attention shall be paid to the design of structural details, considering that those two substructures may respond differently during earthquakes.

(3) For those portions such as joints between superstructures and substructures, connections between piers and foundations, or connections between footings and piles in pile foundations, where the seismic forces may not be transmitted smoothly and seismic failure have been observed often in the past, particular attention shall be paid to the design of structural details, considering accuracy of evaluation of ground conditions, existence of construction joints, accuracy of construction, etc.

5.2 Devices for Preventing Superstructure from Falling

Movable supports shall have stoppers (special devices for resisting large movements of superstructures during earthquakes) to prevent the superstructures from falling from the substructures, caused by the dislocation of the upper shoes of the supports from the lower shoes during strong earthquakes (refer to 5.2.1).

For the girder ends one of the following methods shall be employed as well as the above-mentioned consideration.

- (1) A method extending the length between the end of the support and the edge of the substructure (or widening the width of the crest of the substructure in the longitudinal direction to the bridge axis) in order to prevent the superstructures from falling from the substructure (refer to 5.2.2 or 5.2.3).
- (2) A method connecting adjacent girders on the substructure to prevent the superstructures from falling from the substructure even if they become dislodged from the substructure (refer to 5.2.4).

5.2.1 Stoppers at Movable Supports

The allowable movable length in the design of stoppers at movable supports shall be assumed as the sum of the movement due to temperature change, the movement due to the deflection of the girder when subjected to live loads, a margin for covering construction errors, and 20 mm.

The above-mentioned provision need not apply to those stoppers which are not installed near the supports.

The horizontal design seismic coefficients for designing stoppers shall be determined by increasing the horizontal design seismic coefficient given by eq. (4.1) or (4.2) by 50% or more.

5.2.2 Method of Extending the Length between the End of the Support and the Edge of the Substructure

For those substructures which support the ends of girder, the length S (in cm) between the end of the support and the edge of the substructure, shall be equal to or more than the value given by the following formulas:

$$S = 20 + 0.5 \quad l \quad \text{for } l \leq 100 \text{ m}$$

$$S = 30 + 0.4 \quad l \quad \text{for } l > 100 \text{ m}$$

where

S : Length between the end of the support and the edge of the substructure in cm, and

l : Span length in meters.

For particularly important bridges constructed in soft ground layers (Group 4 in Table 4.3), the value of S shall be equal to 35 cm or more.

5.2.3 Suspended Joints

For suspended joints the length between the ends of girders shall be equal to 60 cm or more, as shown in Fig. 5.1. The length for those bridges constructed in soft ground layer (Group 4 in Table 4.3) shall be equal to 70 cm or more.

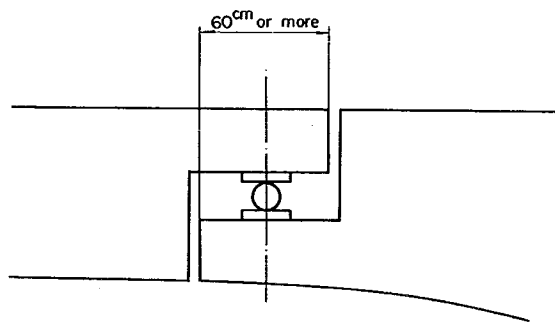


Fig. 5.1 Length between the Ends of both Girders

5.2.4 Method of Connecting Adjacent Girders

Devices for connecting adjacent girders on substructures shall have the movable length specified in Section 5.2.1, for cases in which at least one of two supports is a movable one.

The devices shall be designed to rotate freely to allow the rotation of the girder subjected to live loads, for cases in which both two supports are fixed on one pier.

5.3 Vertical Seismic Forces for Design of Connections between Superstructures and Substructures

The vertical design seismic coefficient for the design of connections between superstructures and substructures shall be assumed as 0.10. When the vertical design seismic coefficient applies upward, only seismic forces shall be considered, neglecting the effects of the dead loads.

The same value of the vertical design seismic coefficient shall be employed for the design of any connections similar to the above.

5.4 Methods for Transmitting Seismic Forces at Connections between Superstructures and Substructures

The method for transmitting seismic forces at connection between superstructures and substructures shall be as follows:

- (1) For cast-in-place reinforced concrete bridges, the means of transmission of seismic forces shall be due to the bearing pressure between the swelling at the base of the lower shoe and the concrete at the crest of the substructure. The concrete portion near the

base of the lower shoe shall resist seismic forces as one body together with the pier of the substructure. In the above-mentioned cases, the means of transmission of seismic forces between the upper shoe and the girder shall be due to the anchors fixed on the upper shoe.

For a margin of safety, anchor bolts between the lower shoe and the substructure shall be designed to resist seismic forces alone, in consideration of cases where no resistance between the swelling at the base of the lower shoe and the concrete at the crest of the substructure can be expected.

The above method is recommended not only for cast-in-place concrete bridges, but, if possible, also for prefabricated concrete bridges or steel bridges.

(2) In cases where no bearing resistance of concrete can be expected, anchor bolts shall be employed to transmit the seismic forces. In these cases one of the following two methods shall be considered.

(a) A method in which a steel plate with anchor bolts is fixed firmly on the crest of a substructure while concrete is being placed, and then the lower shoe of the support is welded to the steel plate after the erection of the girders

(b) A method in which a hole is prepared when concrete is placed, a support is set up near the hole, and then anchor bolts are fixed by placing cement mortar into the hole, or a method in which anchor bolts are set up while concrete is being placed, and then the lower shoe is fixed on the anchor bolts.

(3) Anchor bolts used shall be 25 mm or more in diameter, and the depth of the anchor bolts fixed in the concrete shall be 10 times the diameter or more.

5.5 Devices Expected for Decreasing Seismic Forces

When any devices which are expected to decrease seismic force are employed for bridge structures, sufficient investigations shall be conducted on their effectiveness, and special attention shall be paid to preventing the superstructures from falling.

Chapter 6. Miscellaneous Provisions

When sufficient reasons exist, these Specifications need not apply to the design of bridges.

Reference

References are provided exclusively in the English version.

(Reference 1)

Specifications for Design of Substructures of Highway Bridges

Volume for General Survey and Design

Part 3 Design, Chapter 2 Loads.

Section 2.5 Earth Pressures

Earth pressures acting on a wall shall be the distributed loads given by the following formulas:

(1) Normal Earth Pressures

(a) Earth pressures acting on a movable wall during normal time shall be determined by the Coulomb's theory as follows:

i) For Sandy Soils

$$P_A = \gamma \cdot K_A x + K_A q$$

$$P_P = \gamma \cdot K_P x + K_P q$$

ii) For Cohesive Soils

$$P_A = \gamma \cdot K_A x - 2C\sqrt{K_A} + K_A q$$

$$P_P = \gamma \cdot K_P x + 2C\sqrt{K_P} + K_P q$$

(b) Earth pressures on a fixed wall during normal time shall be determined by

$$P_A = \gamma \cdot K_s \cdot x + K_s q$$

(2) Earth pressures during earthquakes shall be determined by the Mononobe-Okabe method.

$$P_A = (1 - k_v) \gamma \cdot x \cdot K_{EA}$$

$$P_P = (1 - k_v) \gamma \cdot x \cdot K_{EP}$$

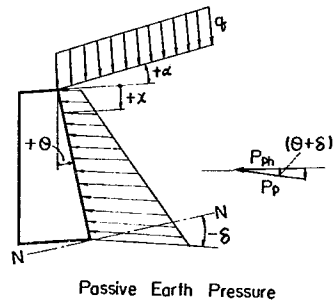
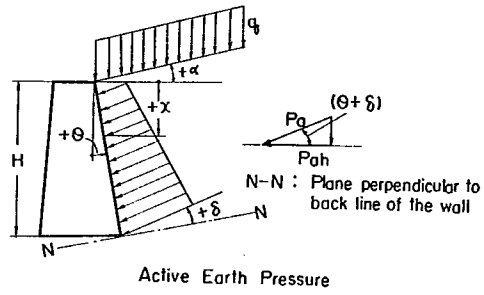


Fig. 1 Earth Pressure

where

C : Cohesion of the soil in t/m^2 ,

K_A : Active earth pressure coefficient for Coulomb's theory ,

K_P : Passive earth pressure coefficient for Coulomb's theory ,

K_{EA} : Active earth pressure coefficient during earthquakes ,

K_{EP} : Passive earth pressure coefficient during earthquakes ,

K_s : Earth pressure coefficient at rest ,

k_v : Vertical seismic coefficient ,

P_A : Active earth pressure in t/m^2 at depth of x meters ,

P_P : Passive earth pressure in t/m^2 at depth of x meters ,

q : Surcharge in t/m^2 on ground surface ,

x : Arbitrary depth in meters ,

α : Angle between the ground surface line and the horizontal line ,

γ : Unit weight of the soil in t/m^3 ,

δ : Angle of friction between the wall and the soil ,

ϕ : Angle of internal friction of the soil , and

θ : Angle between the back line of the wall and the vertical line .

(Comments)

K_A , K_P , K_{EA} and K_{EP} are expressed as follows:

$$K_A = \frac{\cos^2 (\phi - \theta)}{\cos^2 \theta \cdot \cos (\theta + \delta) \left(1 + \sqrt{\frac{\sin (\phi + \delta) \cdot \sin (\phi - \alpha)}{\cos (\theta + \delta) \cdot \cos (\theta - \alpha)}} \right)^2}$$

$$K_P = \frac{\cos^2 (\phi + \theta)}{\cos^2 \theta \cdot \cos (\theta + \delta) \left(1 - \sqrt{\frac{\sin (\phi - \delta) \cdot \sin (\phi + \alpha)}{\cos (\theta + \delta) \cdot \cos (\theta - \alpha)}} \right)^2}$$

$$K_{EA} = \frac{\cos^2(\phi - \theta_0 - \theta)}{\cos \theta_0 \cdot \cos^2 \theta \cdot \cos(\theta + \theta_0) \left[1 + \sqrt{\frac{\sin \phi \cdot \sin(\phi - \alpha - \theta_0)}{\cos(\theta - \theta_0) \cdot \cos(\theta - \alpha)}} \right]^2}$$

$$K_{EP} = \frac{\cos^2(\phi - \theta_0 + \theta)}{\cos \theta_0 \cdot \cos^2 \theta \cdot \cos(\theta - \theta_0) \left[1 - \sqrt{\frac{\sin \phi \cdot \sin(\phi + \alpha - \theta_0)}{\cos(\theta - \theta_0) \cdot \cos(\theta - \alpha)}} \right]^2}$$

where $\sin(\phi \pm \alpha - \theta_0) = 0$ when $\phi \pm \alpha - \theta_0 < 0$

and δ is assumed to be zero during earthquakes.

where

$$\theta_0 = \tan^{-1} \frac{k_h}{1 - k_v}$$

k_h : Horizontal seismic coefficient

k_v : Vertical seismic coefficient

[Reference 2]

Specifications for Design of Substructures of Highway Bridges

Volume for General Survey and Design

Part 3 Design, Chapter 2 Loads

Section 2.7 Buoyancy or Uplifts

When it is apparent that buoyancy forces or uplifts act on structures, they shall be taken into account in the design.

(Comments)

When it is unknown whether they act or not, both cases shall be taken into account in the design.

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