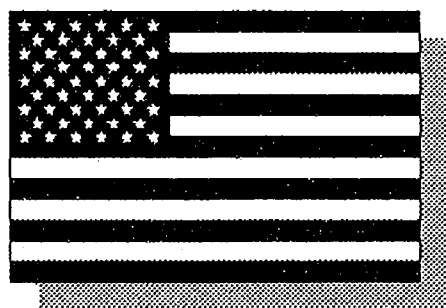
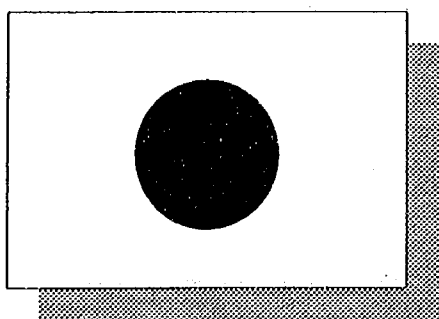


**PROCEEDINGS OF  
THE FIFTH U.S.-JAPAN WORKSHOP  
ON GEOTECHNICAL EARTHQUAKE  
ENGINEERING**

**SOIL DYNAMICS STUDIES BY USE OF CENTRIFUGE**



**September 28-29, 1998  
Tsukuba, Japan**

**Public Works Research Institute**

**Ministry of Construction**

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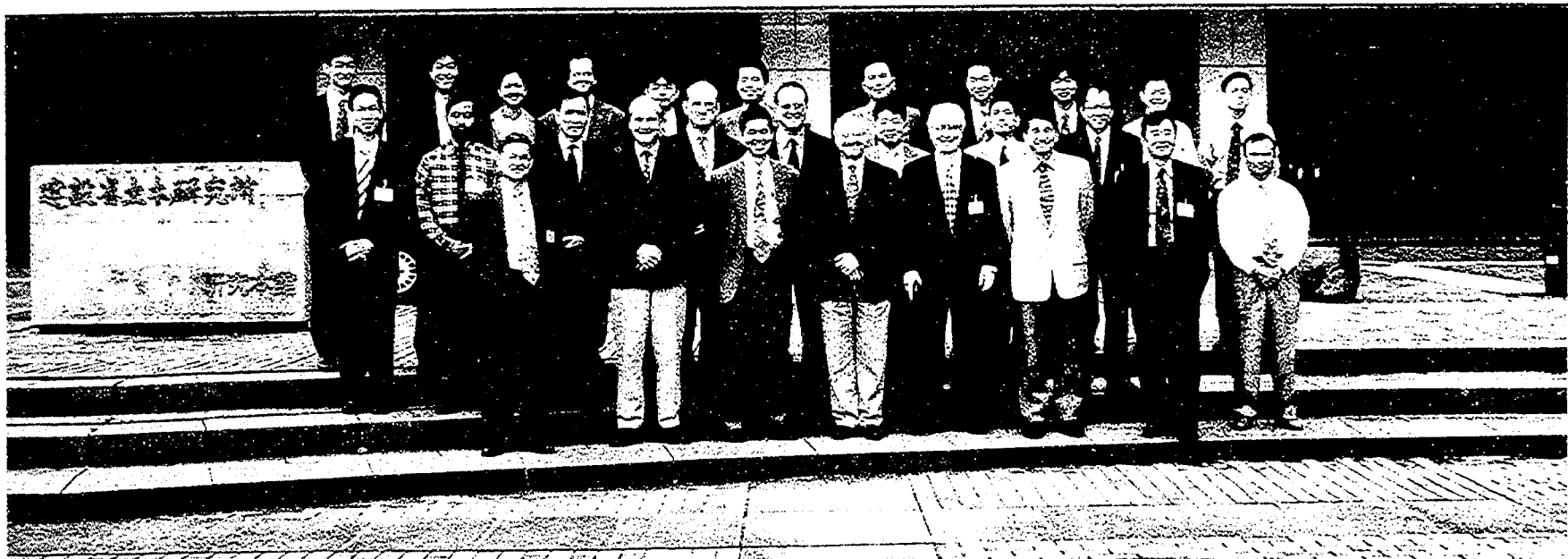
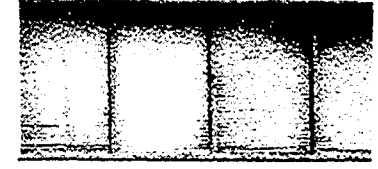
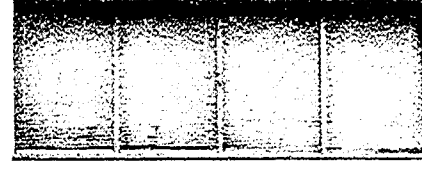
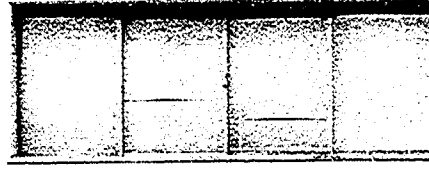
**Task Committee(H)  
On Soil Behavior and Stability During Earthquakes  
U.S.-Japan Panel on Wind and Seismic Effects  
U.S.-Japan Cooperative Program in Natural Resources**

**September 28-29, 1998  
Public Works Research Institute  
Tsukuba City, Japan**

**Synopsis :**

This proceedings include the results of the Fifth U.S.-Japan Workshop on Geotechnical Earthquake Engineering. The Workshop was held at the Public Works Research Institute on September 28 and 29, 1998

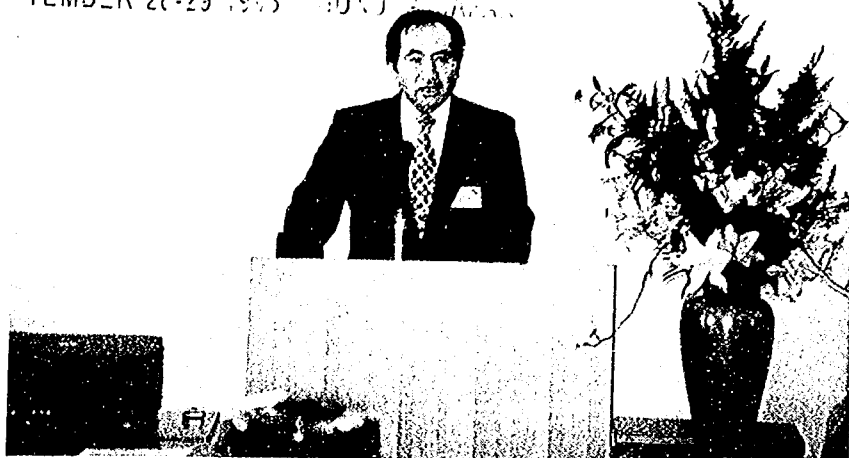
**Key words :** Soil Dynamics, Dynamic Centrifuge Test, Seismic Design



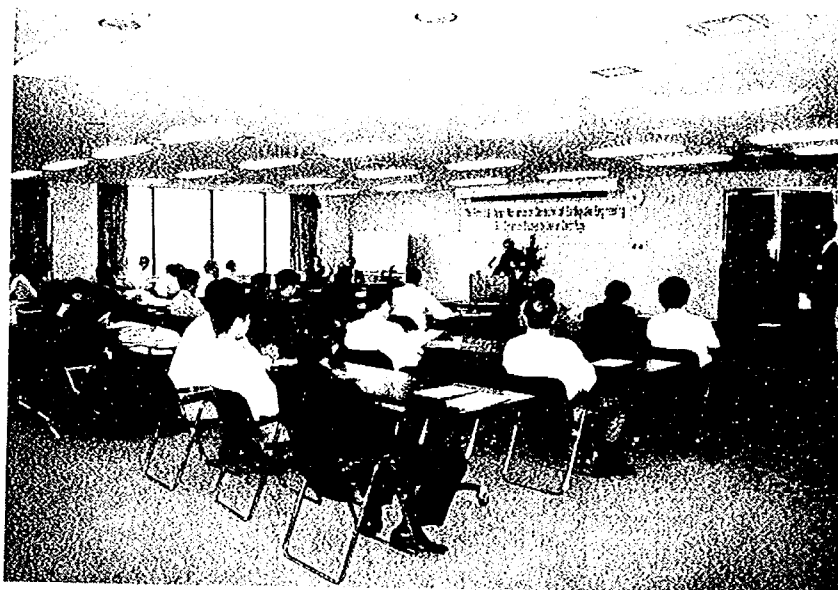
Workshop Participations in front of main building of PWRI

# lamies Studies by Use of Centrifuge

PTEMBER 28-29 1993 TOKYO, JAPAN



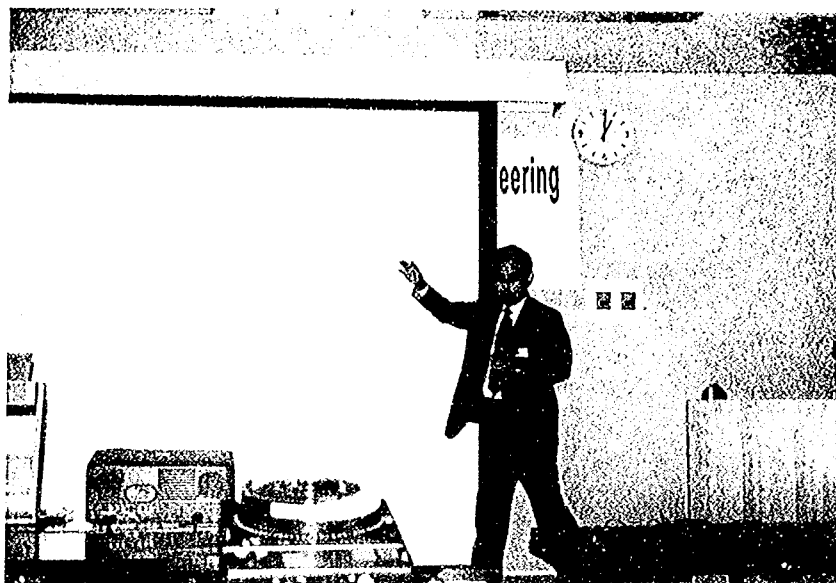
Opening Remarks by Dr. Yamamoto,  
Deputy Director - General, PWRI



Opening Ceremony



Presentation by Dr. Steedman



Presentation by Prof. Towhata



Discussion



Short Trip  
(at centrifuge facility)



Short Trip  
(at large scale 3D shaking table facility)



Reception



Closing Remarks by Dr. Ledbetter

## FOREWORD

The proceedings include the results of the fifth U.S.-Japan Workshop on Geotechnical Earthquake Engineering, subtitled as Soil Dynamics Studies by Use of Centrifuge. The workshop was held at the Public Works Research Institute on September 28 and 29, 1998, under the auspices of Task Committee H, Soil Behavior and Stability During Earthquakes, UJNR Panel on Wind and Seismic Effects. The workshop was jointly coordinated by both Japan and US side chair person of Task Committee H, Mr. O. Matsuo, Head of Soil Dynamics Division of Public Works Research Institute and Dr M. Hynes, Chief of Earthquake Engineering and Geoscience Division of USAE Waterways Experiment Station. This workshop was co-sponsored by the Public Works Research Institute and a grant-in-aid of the Tsukuba EXPO'85 Memorial Foundation.

A total of forty five researchers, nine from U.S. side and thirty six from Japan side, participated and 17 papers were presented during the workshop. The following topics were addressed;

- (a) Liquefaction of ground
- (b) Damage of structures due to ground liquefaction and remedial treatment and
- (c) Dynamic behavior of earth structures.

The workshop provided fruitful opportunity to exchange information and experiences on the geotechnical dynamic centrifuge modeling. One of the most heatedly discussed topic during sessions was the liquefaction induced lateral spreading and interaction between liquefied soil and structure. The application of centrifuge modeling to new research field including the environmental geotechnical engineering and fluid mechanics, were also thoroughly discussed.

I wish to express my profound gratitude and sincere appreciation to all participants in the workshop for supporting the workshop with active presentations and discussions.

Osamu MATSUO  
Head, Soil Dynamics Division  
Earthquake Disaster Prevention Research Center  
Public Works Research Institute  
Ministry of Construction

October, 1998

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# SESSION 1

---

- *R.Dobry*: Soil Liquefaction, Lateral Spreading and Effects on Foundations
- *M.Hamada*: A Consideration on Mechanism of Large Ground Deformation Caused by Liquefaction.
- *R. Phillips*: Modeling Slides Due to Flow Liquefaction

## ABSTRACT

### SOIL LIQUEFACTION, LATERAL SPREADING AND EFFECTS ON FOUNDATIONS

R. Dobry, T. Abdoun, V. Taboada, L. Liu,  
M. Okamura, M. Sharp, and R. Ramos

The presentation focuses on seismically-induced liquefaction effects modeled in the last few years in the 100 g-ton RPI centrifuge located in Troy, NY, USA. All centrifuge tests involved in-flight shaking with various input acceleration time histories, and the lateral spreading tests used the RPI laminar box inclined a few degrees to the horizontal. In these lateral spreading tests, in addition to accelerometers and pore pressure transducers embedded in the soil and vertical LVDTs at the ground surface, lateral LVDTs on the flexible walls of the laminar box allowed measurement of both cyclic and permanent lateral displacements of the soil mass. Fine, uniform Nevada sand, placed at densities ranging from loose to dense, and saturated either with water or with a viscous fluid, was used in all tests.

Centrifuge results and associated engineering interpretations discussed cover the following systems:

- (i) response of a shallow footing on liquefiable and improved ground;
- (ii) lateral spreading of sloping clean loose sand for various ground slopes, soil relative densities/permeabilities/degrees of overconsolidation/degrees of preshaking, and a range of input accelerations and frequencies including the effect of weaker shaking following the main shock;
- (iii) centrifuge modeling of the Cone Penetration Test (CPT) through three combinations of cone diameter and g-level, and its use to characterize the soil at various relative densities and degrees of overconsolidation and preshaking, as well as to predict lateral spreading effects; and
- (iv) effect of lateral spreading of 2- and 3-layer profiles on bending moments of pile foundations, with the piles instrumented with strain gages to measure bending moments, and with the tests covering single piles, pile groups, countermeasures, and constraining effect of bridge deck longitudinal stiffness.

# MECHANISM OF LIQUEFACTION-INDUCED LARGE GROUND DISPLACEMENT

M. Hamada  
Waseda University,  
K. Wakamatsu  
The University of Tokyo, Japan

**ABSTRACT:** The authors investigated into a mechanism of liquefaction-induced large displacement of sloped ground based on case studies on the occurrences during past earthquakes such as the 1964 Niigata earthquake and others, shaking table tests of the flow of model grounds referring to a similitude law as viscous liquid and theoretical analyses. The followings were obtained as key characteristics of the flow of liquefied soil.

(1) From the case studies it was concluded that the magnitude of the liquefaction-induced ground displacement in the horizontal direction was mainly governed by gradient of ground surface and thickness of liquefiable soil layer.

(2) From a test on external forces on a model pile in flowing liquefied ground, it was found that the liquefied soil behaved as a liquid during the ground flow.

(3) A similitude law determined by Reynolds' number can be applied between model ground and actual ground. In this regard, the velocity of ground flow and the coefficient of apparent viscosity of liquefied soil are proportional to the square root of the liquefied soil thickness, and the square root of the thickness cubed, respectively.

Finally, based on the case studies the model tests and the similitude law, authors proposed a semi-empirical and semi-theoretical formula for estimation of horizontal displacement of sloped ground, where gradient of ground surface, the thickness of liquefied soil and SPT N-values were taken into consideration as governmental factors to the magnitude of the ground displacement.

## Modelling Slides Due To Flow Liquefaction

R. Phillips

*C-CORE, St. John's, NF, Canada*

Flow liquefaction can occur in saturated loose sands and is characterised by a large strength or stiffness loss resulting in substantial deformation or a flowslide. Such gravitational flowslides were induced in a series of centrifuge model tests by rapid monotonic load increments as part of CANLEX. A finite difference numerical model, calibrated against laboratory element test data, captured the essence of the observed response of the centrifuge tests in terms of pore pressure rise and deformation pattern. Both modelling techniques were used to successfully predict the response of a similar subsequent field event.

The Canadian geotechnical engineering community is continuing a major sand liquefaction study entitled the Canadian Liquefaction Experiment (CANLEX) through the collaborative efforts of industry, engineering consultants and university participants and with support from the Natural Sciences and Engineering Research Council of Canada (NSERC). The study examines the characterisation of sand for dynamic and flow liquefaction. CANLEX has comprised four phases since 1993. Phases 1, 2 and 4 were concerned primarily with characterisation of loose sand to determine its insitu state and elemental stress/strain response. The purpose was specifically to evaluate triggering resistance, residual strength and liquefaction induced displacement under static and earthquake loading conditions. These field characterisations were of hydraulically deposited oil sand tailings at the Syncrude Canada Limited site, Alberta; naturally occurring sands in the Fraser River Delta, British Columbia; and hard rock tailings from the Highland Valley Copper Mine, British Columbia respectively. In Phase 3 a controlled liquefaction field event was attempted at a Syncrude tailings disposal site with the purpose of statically triggering a liquefaction flow slide by rapidly loading a loose saturated sand deposit. Numerical modellers predicted the response of this full-scale event and compared the numerical response with the field measurements.

The scope of the CANLEX project included centrifuge model testing of flow liquefaction in Phases 1 and 2. These tests aided in the design of the Phase 3 field event and allowed for the calibration of the numerical models. They would also have served as a substitute for the field event in the scenario that a field event was not possible or not conducted.

The Phase 1 tests provided a flowslide of a 16° submerged slope of oil sand tailings at a relative density of about 21% to show the feasibility of creating a flow liquefaction by centrifuge model testing. Phase 2 comprised five further centrifuge tests. The first 4 tests were of a potential liquefaction event scenario, which comprised a partially saturated 5m high embankment overlying a 10m thick saturated sand layer. Rapid heightening of the embankment was expected to cause flow liquefaction in the underlying sands. Model tests II-1 to II-3 were homogenous and tested at 29%, 40% and 50% relative density respectively. Model test II-4 was a similar geometry but the embankment was at about 49% relative density and the sand layer was at about 25%. Rapid heightening of the embankment was simulated by sequentially lowering two 60 kPa steel surcharges onto the embankment crest. Flowslides were observed in 2 of these 4 tests, that is in model tests II-1 and II-4, with deep-seated lateral movements occurring throughout the models.

A fifth test simulated the actual liquefaction event of a compacted clay tailings impoundment dyke on a saturated sand layer at about 27% relative density. The sand used in all these tests was moist fine sub-angular to sub-rounded oil sand tailings supplied by Syncrude Canada Limited. The pore fluid was Canola vegetable oil with a dynamic viscosity of 57 centipoise. Three different numerical analyses were conducted of these centrifuge model tests. The finite difference analysis, calibrated against laboratory element test data, captured the essence of the observed undrained response of the centrifuge tests in terms of pore pressure rise and deformation pattern.

# SESSION 2

---

- *L. Finn:*
- *A. Yashima:* Prediction of Seismic Behavior of Embankment Founded on Liquefiable Soil

## Prediction of Seismic Behavior of Embankment Founded on Liquefiable Soil

A.Yashima (Gifu University)

O.Matsuo and T.Shimazu (P.W.R.I., Ministry of Construction)

R.Uzuoka, M.Mihara and K.Nishi (Hazama Corporation)

- Topics
- Analytical method
- Simulations for laboratory tests
- Simulations for dynamic centrifugal tests
- Simulations for damaged dikes
- Conclusions

## Analytical method

- LIQCA ( Oka et al. 1994)
- Coupled method
  - Two phase mixture theory
  - u - p formulation  
(displacement of the solid phase - pore water pressure)
- Discretization method
  - Spatial discretization
    - ◆ FEM for the equilibrium equation
    - ◆ FDM for the continuity equation
  - Time discretization
    - ◆ Newmark's  $\beta$ -method

## Topics

- Verification of the analysis method
  - Laboratory tests
  - Dynamic centrifugal model tests
  - Damaged dikes
- Dynamic behavior of an embankment and a foundation ground
  - Stress and strain state under the embankment
  - Stress distribution in the embankment
  - Settlement of an embankment with relation to ;
    - ◆ Type of a seismic motion
    - ◆ Soil profile

## Analytical method

- Constitutive model for sand  
(Oka et al. 1992, Tateishi et al. 1995)
  - The elasto-plastic theory
  - The non-associated flow rule
  - The concept of the overconsolidated boundary surface
  - The non-linear kinematic hardening rule
  - The generalized flow rule
  - Plastic strain dependence of the plastic and elastic shear moduli

## Analytical method

### ■ Constitutive model for clay (Oka 1992)

- The elasto-viscoplastic theory
- The overstress type flow rule
- The concept of the overconsolidated boundary surface
- The non-linear kinematic hardening rule

## Analytical cases

### ■ Laboratory tests

- Cyclic torsional shear tests for sand under different boundary conditions
- Triaxial tests for clay at different loading rates

### ■ Dynamic centrifugal model tests (Koseki et al. 1994)

- Soil embankment founded on liquefiable sandy ground
- Various Input motions

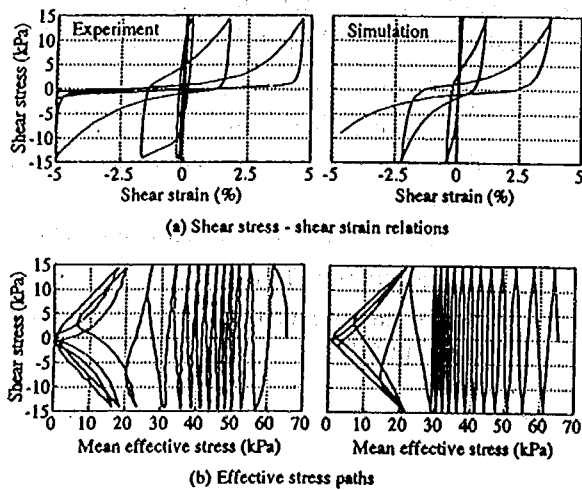
### ■ Case records

- Damaged dikes at Shiribeshi-toshibetu River during the 1993 Hokkaido Nansei-oki Earthquake
- A seriously damaged site and a non damaged site

## Performance of the model

### ■ Cyclic torsional shear tests for sand ( $K_0=0.5$ )

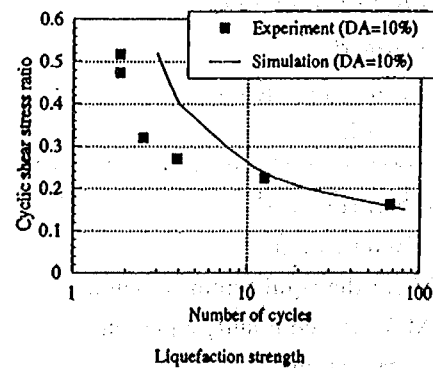
The vertical strain is constrained.



## Performance of the model

### ■ Cyclic torsional shear tests for sand ( $K_0=0.5$ )

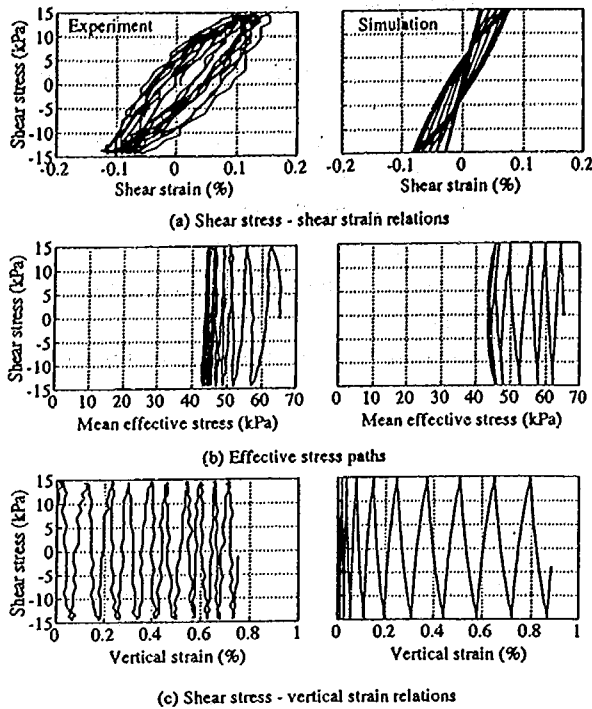
The vertical strain is constrained



## Performance of the model

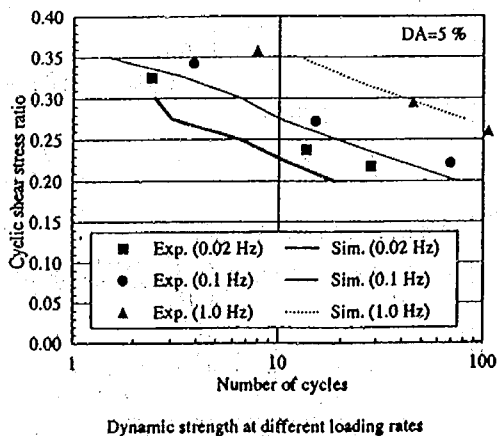
### ■ Cyclic torsional shear tests for sand ( $K_0=0.5$ )

The vertical strain is not constrained



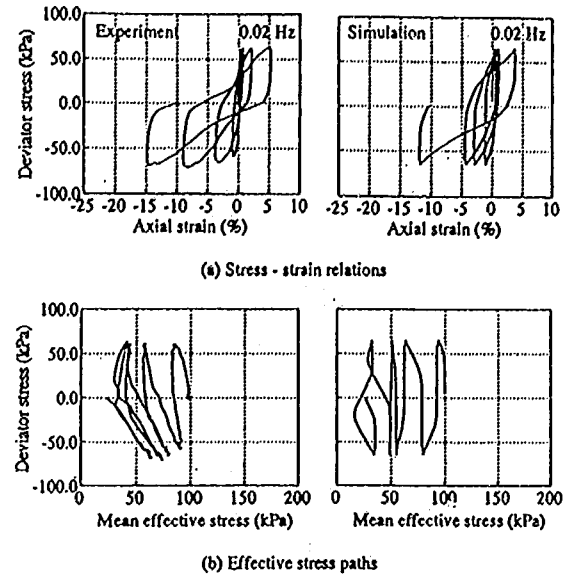
## Performance of the model

### ■ Triaxial tests for clay at different loading rate

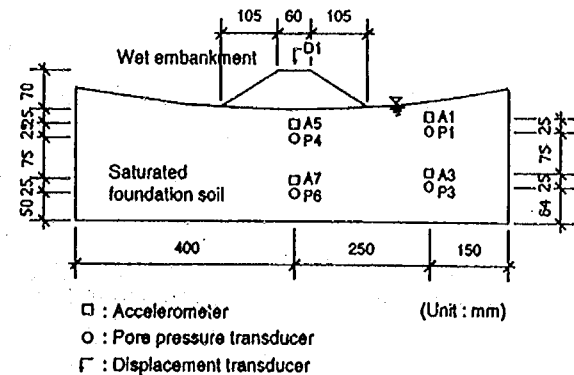


## Performance of the model

### ■ Triaxial tests for clay at different loading rate



## Dynamic centrifugal model tests



Under a centrifugal acceleration 30g

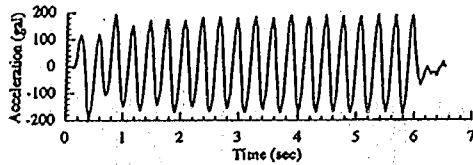
Foundation soil : Toyoura sand

Embankment soil : a mixture of Toyoura sand and a clay sand

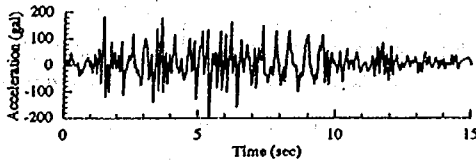
## Dynamic centrifugal model tests

- Test cases and conditions
- Input motions

| Case | Centrifuge acc. (g) | Shaking wave   | Input acc. (g) | Dr of sand layer (%) |
|------|---------------------|----------------|----------------|----------------------|
| C-1  | 30                  | Sinusoidal     | 4.0            | 57.3                 |
| C-2  | 30                  | 100 Hz         | 5.4            | 62.2                 |
| C-3  | 30                  | 20 cycles      | 8.9            | 59.6                 |
| C-4  | 30                  | Irregular wave | 6.0            | 60.2                 |
| C-5  | 30                  |                | 8.3            | 60.9                 |
| C-6  | 30                  |                | 4.3            | 62.9                 |



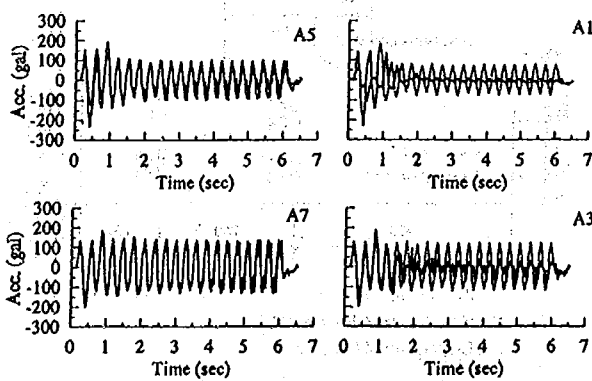
(a) Sinusoidal wave in Case C-2



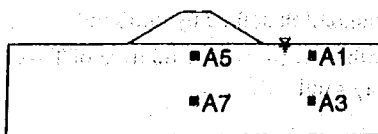
(b) Earthquake wave in Case C-4

## Analytical results

- Time history of acceleration response in Case C-2



— Experiment — Simulation



## Analytical conditions

- Model parameters

The elasto-plastic model for sand  
(the embankment, the ground)

The parameters were determined based on ;

- ◆ measurements during preparation of the test models.
- ◆ past laboratory tests on the same material as the centrifugal tests.

- Other numerical conditions

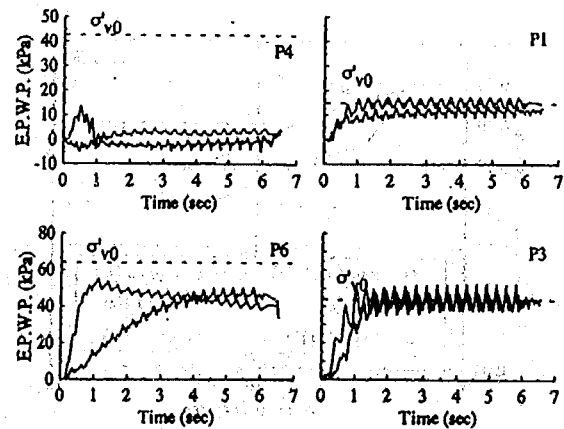
On prototype scale in the 1g field

Initial stress conditions calculated by the static elasto-plastic analysis

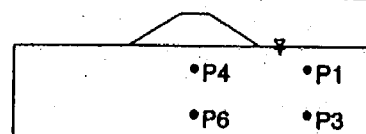
Rayleigh's damping proportional to stiffness

## Analytical results

- Time history of excess pore pressure in Case C-2

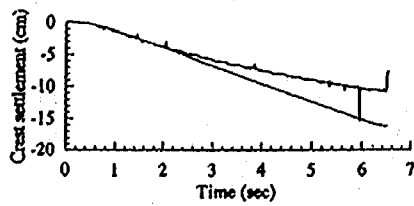


— Experiment — Simulation

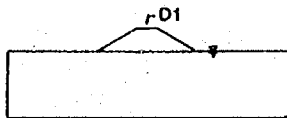


## Analytical results

### ■ Time history of crest settlement in Case C-2

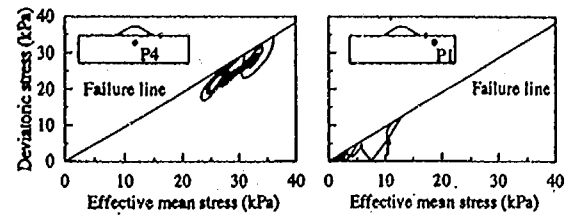


— Experiment    - - - Simulation

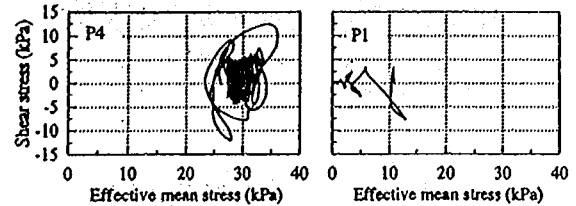


## Analytical results

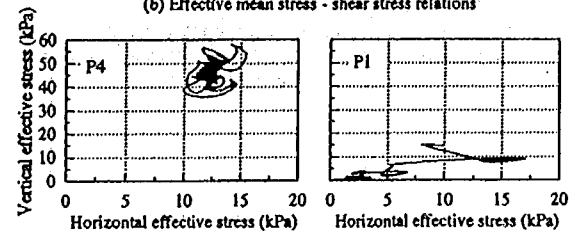
### ■ Behavior of stress and strain in Case C-2



(a) Effective mean stress - deviatoric stress relations



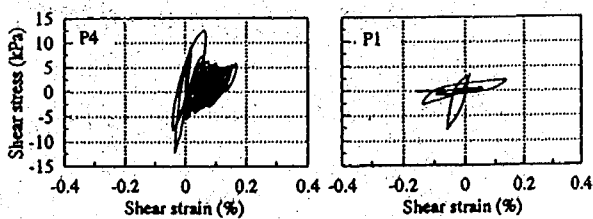
(b) Effective mean stress - shear stress relations



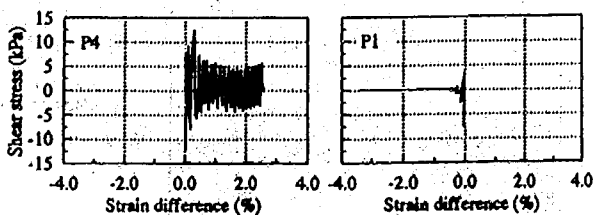
(c) Horizontal effective stress - vertical effective stress relations

## Analytical results

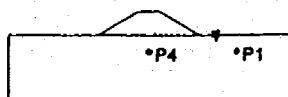
### ■ Behavior of stress and strain in Case C-2



(a) Shear strain - shear stress relations



(b) Strain difference - shear stress relations

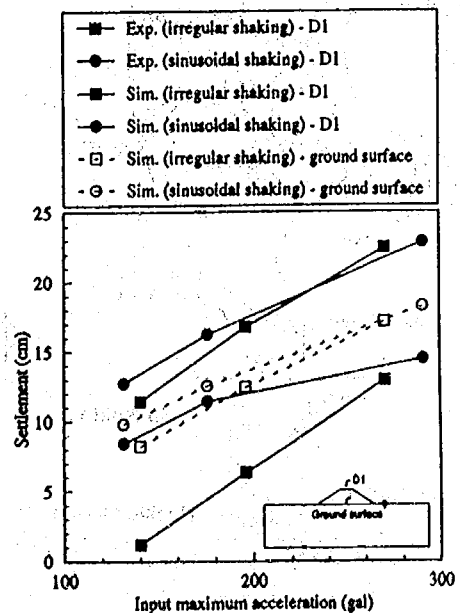


## Analytical results

### ■ Relation of input acceleration and settlement

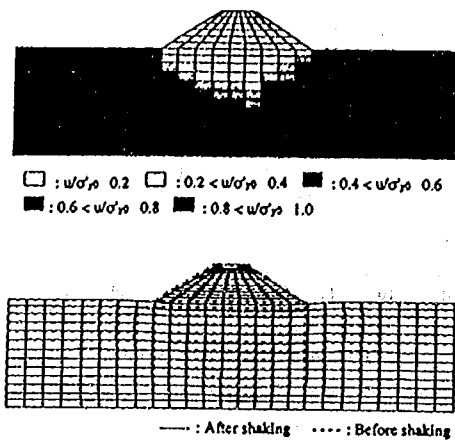
Crest settlement (D1) for experiments

Crest and ground surface for simulations



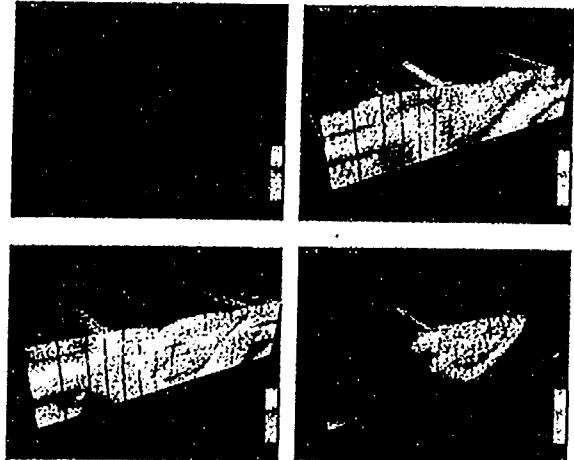
## Analytical results

- Distribution of excess pore water pressure ratio in Case C-2
- Deformation after shaking in Case C-2



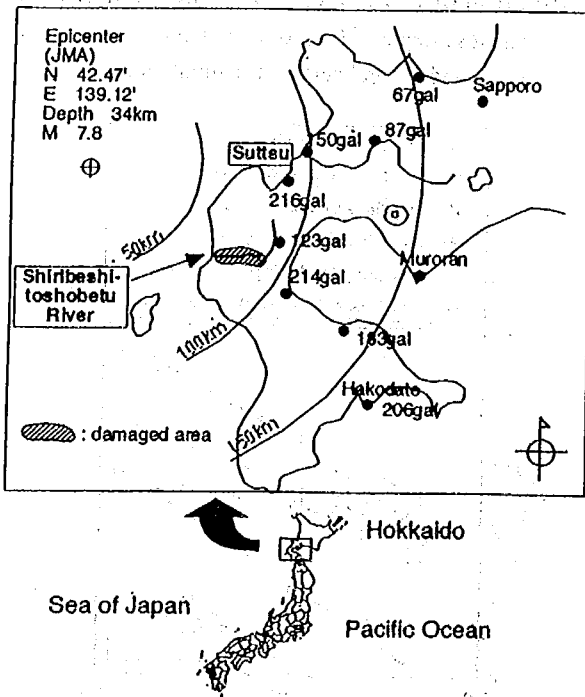
## Analytical results

- Excess pore water pressure ratio  
Case : C-2 (sinusoidal shaking)  
Output times : 0.0, 1.0, 2.0, 3.0 seconds

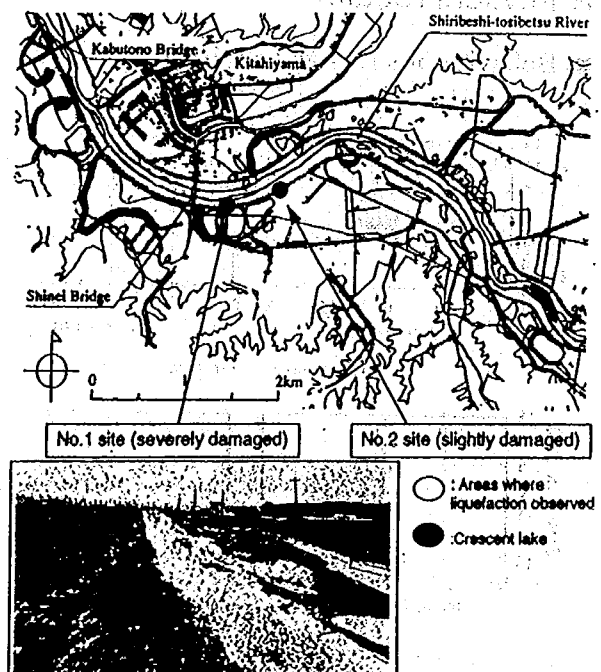


## Damaged dikes

- The 1993 Hokkaido Nansei-oki Earthquake

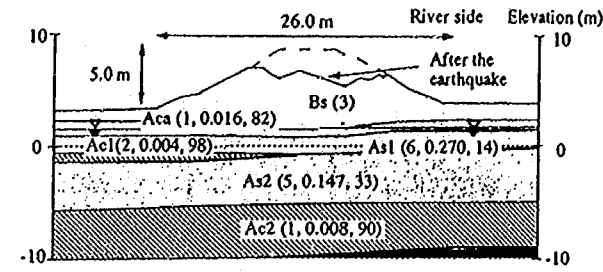


## Damaged dikes

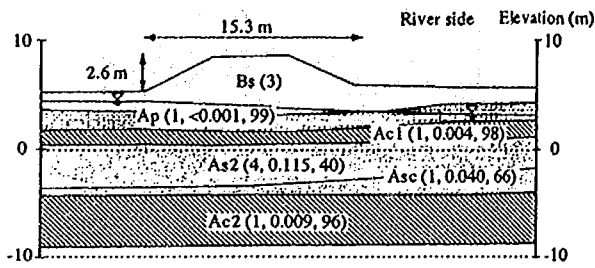


## Damaged dikes

### ■ Soil profiles



(a) No.1 site



(b) No.2 site

□ : Sandy soil layers    ▨ : Clay layers    ▩ : Peat layers

Notes (N, D<sub>50</sub>, F<sub>c</sub>)  
N : Averaged SPT N-values (blows/0.3m), D<sub>50</sub> : Averaged mean grain size (mm), F<sub>c</sub> : Averaged fines content (%)

## Analytical conditions

### ■ Model parameters

The elasto-plastic model for sandy layers and the embankment

The elasto-viscoplastic model for clay and peat layers

The parameters were determined based on ;

- ◆ In-situ tests (SPT-value, V<sub>s</sub>)
- ◆ Laboratory tests with undisturbed samples (characteristics of dynamic deformation and strength)

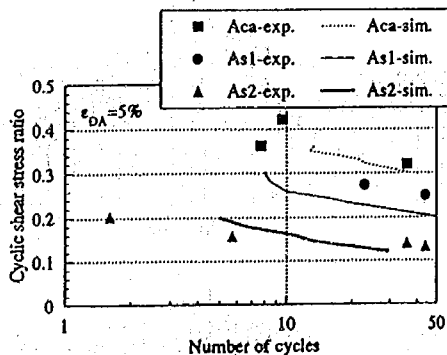
### ■ Other numerical conditions

Initial stress conditions calculated by the static elasto-plastic analysis

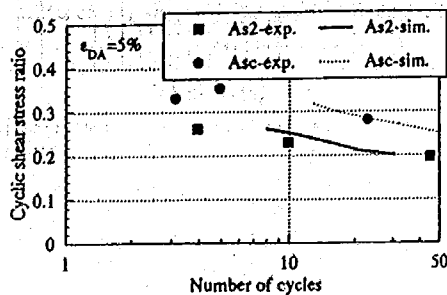
Rayleigh's damping proportional to stiffness

## Analytical conditions

### ■ Liquefaction strength



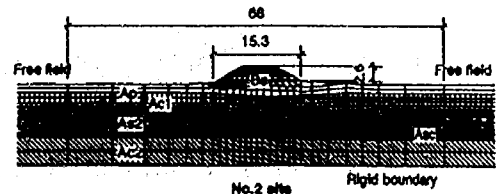
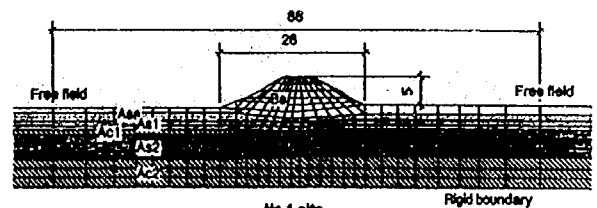
(a) No.1 site



(b) No.2 site

## Analytical conditions

### ■ FEM models



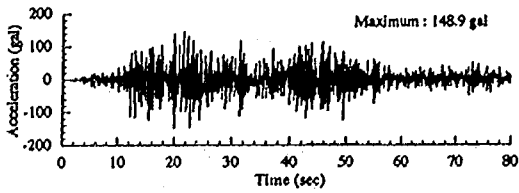
(Unit : m)

Free field : Attached wide elements which express one dimensional response

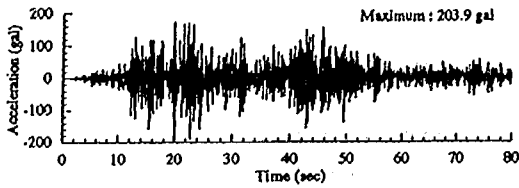
## Analytical conditions

### ■ Input motions

Deconvoluted waves from measured waves to the base layer at Suttsu



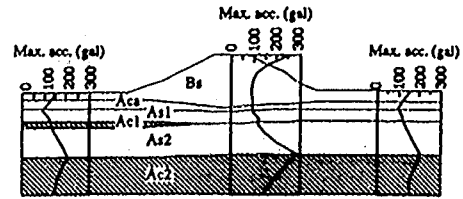
(a) at No.1 site



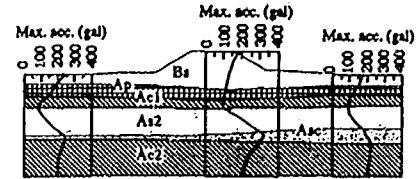
(b) at No.2 site

## Analytical results

### ■ Distribution of maximum acceleration response



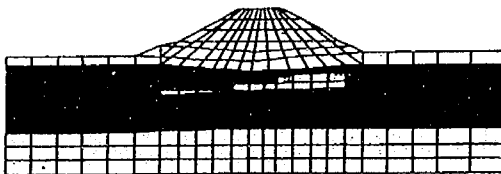
(a) at No.1 site



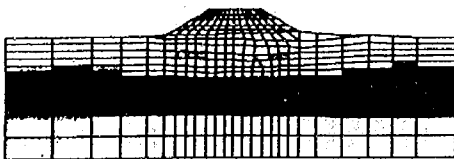
(b) at No.2 site

## Analytical results

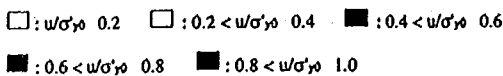
### ■ Distribution of excess pore water pressure after earthquake



(a) at No.1 site

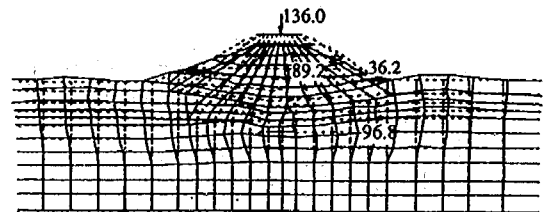


(b) at No.2 site

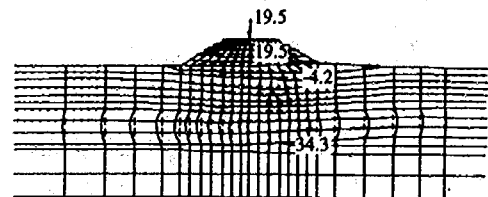


## Analytical results

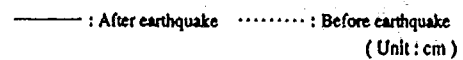
### ■ Deformation after earthquake



(a) at No.1 site

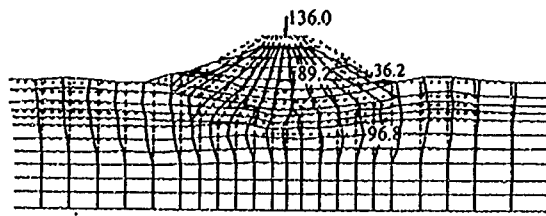


(b) at No.2 site

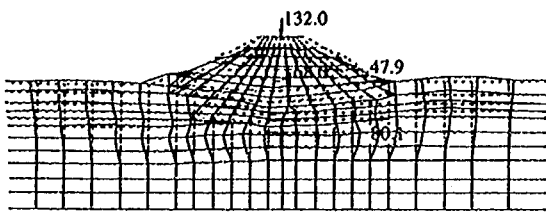


## Analytical results

### ■ Deformation after earthquake at No.1 site



(a) Hard embankment



— : After earthquake    ..... : Before earthquake  
(Unit : cm)

(b) Soft embankment

## Analytical results

### ■ Distribution of safety factor in the embankment at No.1 site with smaller liquefaction strength in Aca layer



□ :  $1.20 < F_s$     □ :  $1.15 < F_s \leq 1.20$     □ :  $1.10 < F_s \leq 1.15$   
■ :  $1.05 < F_s \leq 1.10$     ■ :  $F_s \leq 1.05$

After 20 seconds from the beginning of earthquake

## Analytical results

### ■ Distribution of safety factor in the embankment at No.1 site



(a) after 30 seconds



(b) after 40 seconds

□ :  $1.20 < F_s$     □ :  $1.15 < F_s \leq 1.20$     □ :  $1.10 < F_s \leq 1.15$   
■ :  $1.05 < F_s \leq 1.10$     ■ :  $F_s \leq 1.05$

## Conclusions

- The method provided good simulation of the soil and structure's behavior ;  
the buildup of excess pore water pressure  
the settlement of the embankment.
- There was a non-liquefied area under the embankment. The soil's behavior of this area was affected by how the initial anisotropic stress varies spatially and when the free ground liquefied.
- The existence of a cohesive soil layer near the ground surface affected the deformation of the embankment. The modeling for clay layers is as important as it for sandy layers.
- The failed configuration of the embankment was affected by the behavior of the foundation ground, especially near the surface, underneath the embankment.

## SESSION 3

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- *H.-Y. Ko* : A study of Seismic Effects on Retaining Walls and Philosophies Behind Centrifuge Modeling
- *O. Matsuo*: Seismic Performance of Rigid Retaining Walls
- *T. Tsutsumi*: The Dynamic Geotechnical Centrifuge at PWRI

# A study of seismic effects on retaining walls and philosophies behind centrifuge modeling

Hon-Yim Ko<sup>1</sup> and Mandar M. Dewoolkar<sup>2</sup>

## Abstract

The basic principle behind centrifuge modeling testing is to create a stress field in the model that simulates prototype conditions to allow observations to be made than can otherwise only be made on full scale prototypes. With this principle, there are four different philosophies that can be adopted in using centrifuge modeling to address different problems in geotechnical engineering. The philosophies include : modeling of real or hypothetical prototypes, investigation of new phenomena or concepts, conducting parametric studies, and validation of design, analytical, and numerical methods. In the absence of prototype results, different scaled models of the same prototype can be tested at corresponding g-levels, and if the results are internally consistent, their model behavior can be extrapolated to prototype response using the scaling relations involved. This concept is known as *modeling of models*.

Seismically loaded retaining walls with liquefiable backfills is a complex dynamic-soil-fluid-structure interaction phenomenon. At the University of Colorado at Boulder, a comprehensive study was conducted to investigate seismic behavior of retaining walls with saturated backfill. All four philosophies of centrifuge modeling were addressed in the investigation.

Aluminum model walls simulated hypothetical, fixed-base, prototype reinforced concrete walls. A substitute pore fluid, metolose, was used to obtain a common time scale. Pertinent scaling relations were verified through a modeling of models study. Also, modeling of models was conducted on the wall models with water-saturated backfills.

A new concept was observed from the experimental results which was also verified through numerical simulations. It was demonstrated that a false appearance of modeling of models can be obtained with water depending on the type of the model, intensity of base input motion, and drainage boundary conditions.

Parametric studies were performed. Parameters included intensity of base input motion and flexural stiffness of retaining walls.

Finally, experimental results were used to evaluate existing design methods and to verify the finite element program DIANA-SWANDYNE II. Parametric studies were performed using the verified numerical code. Based on the experimental and numerical results, a more appropriate design method was suggested.

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2: Research Associate, University of Colorado at Boulder, U.S.A.

# **Seismic Performance of Rigid Retaining Walls**

by

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## **ABSTRACT**

Shaking tests of rigid retaining walls were performed in centrifuge. Gravity-type retaining walls and cantilever retaining walls were tested by applying base acceleration of sinusoidal waves and real earthquake waves. The relationship between magnitude of input acceleration and displacement, earth pressure acting on the walls, were investigated. Experimental results were compared with conventional seismic design with regard to earth pressure and seismic displacement.

## **INTRODUCTION**

Seismic design of rigid retaining walls in Japan has been performed by using conventional pseudo-static design procedure, so called the seismic coefficient method. In the design procedure, the magnitude of seismic earth pressure acting on walls is estimated based on the Mononobe-Okabe earth pressure formula, Okabe(1924) and Mononobe and Matsuo(1929), and the stability of retaining walls are examined against sliding, overturning and bearing capacity failures. Due to its simplicity, the method has been widely accepted in practice. However, the design procedure has the following limitations:

- (1) It cannot consider the amount of displacement induced by earthquakes. Therefore, it is difficult to be incorporated into the performance based design procedure.
- (2) Relationship between the value of seismic coefficient and the magnitude of anticipated earthquake motion has not been cleared yet.

After the 1995 Hyogoken Nanbu earthquake, seismic design criteria and specifications for various engineering structures in Japan have been reviewed so that they are capable of considering the level 2 earthquake in the design. Earth structures including embankments and retaining walls are not the exception.

In the experimental study presented here, seismic stability of the retaining walls that were designed based on the existing design manual, the Japan Road Association, 1987, were investigated.

## **DESIGN OF MODEL RETAINING WALLS**

Gravity type and cantilever type retaining wall models were designed according to the existing design manual in which active earth pressure acting on the wall is computed from the Mononobe-Okabe earth pressure formula and the stability of the wall is examined with respect to sliding, rotation and bearing failure. The manual (Japan Road Association, 1987) specifies that,

for important walls or walls exceeding 8 m in height, the safety factor against sliding should be more than 1.2, the safety factor against bearing capacity failure be more than 2, the resultant force acting at the base plate be within the double middle-third, under a standard seismic coefficient of 0.15.

A friction angle,  $\phi$ , of 41 degree was used for the backfill soil that were obtained from triaxial compression test. Interface friction angles,  $\delta$ , between the wall bottom and the foundation soil, and between the wall and the backfill soil were set to  $2/3 \phi$  and  $1/2 \phi$ , respectively, according to the manual.

Relationships between seismic coefficient and factor of safety for the designed retaining wall models are presented in Fig. 1. This figure shows the result of the cases in which dry Toyoura sand was used for backfill soil with a relative density of 80 % with no raising embankment.

## TESTING PROCEDURES

A series of the shaking tests were conducted by using the PWRI's large scale centrifuge, Matsuo et al. (1998). Configurations of the models are shown in Fig. 2 and the test conditions are summarized in Table 1. In a rigid container with an internal dimensions of 150 cm long, 50 cm deep and 30 cm wide, dry Toyoura sand was rained to form foundation soil of 10 cm deep with a relative density of approximately 80 % with an exception of Case 2 ( $D_r = 61\%$ ). A model retaining wall with a height of 30 cm was placed on the foundation soil and the sand was pluviated again to form backfill behind the wall. Relative density of the backfill was the same as that of the foundation soil. In only Cases 10 and 13, Toyoura sand with a water content of 5 % was compacted to form model ground with a relative density of approximately 60 %. Accelerometers were embedded at the predetermined positions intermittently during the model preparing process. On the bottom and side of the wall models, a total of ten or twelve load cells were built in the gravity type and cantilever type wall models, respectively, which could measure normal and shear forces.

The tests being carried out at 30 g centrifugal acceleration, the models are equivalent to prototype walls of 9 m high, except for Cases 5 and 14 (centrifugal acceleration: 60 g, prototype wall height: 18 m). The models were excited horizontally with the Kobe wave for Case 3, the Aomori wave for Case 4 and approximately sinusoidal wave with 20 uniform cycles of 60 Hz for other cases. The time axes of the two earthquake time histories were scaled so that the predominant frequency becomes 60 Hz, see Fig. 3. Shaking test was repeated several times for each model, with increased acceleration level, until the wall exhibited significant displacement.

## TEST RESULTS

In the following, test results are shown in the prototype sense otherwise mentioned.

### Modes of Deformation

Figures 4 and 5 show photographs taken after shaking tests in Cases 6 and 7, respectively. It can be clearly observed in Case 6 of gravity type retaining wall that the wall displaced and tilted forward and an active soil wedge with apparent slip surface developed in the backfill after

shaking event of 0.34 g (amplitude of the input acceleration). The angle of the slip surface to the vertical was about 46 degree which is close to 45.9 degree predicted from well-known Mononobe-Okabe approach (Okabe, 1924; Mononobe and Matsuo, 1929) with the peak input acceleration and an angle of shearing resistance of 41 degree. After the shaking event of 0.43 g, another slip surface with slightly gentler slope was formed. Also the foundation soil at the toe of the wall exhibited bearing capacity failure, resulting in tilting of the wall. The deformation modes for Case 7 of cantilever retaining wall are presented in Fig. 5. The mode is virtually similar to that of Case 6, except that two slip surfaces developed in the backfill both originating from the heel of the wall.

### **Input Acceleration vs. Displacements**

Figs. 6 and 7 plot relationship between the magnitude of input acceleration and horizontal wall displacement, for the gravity type and the cantilever type retaining walls, respectively. In the figures, the horizontal wall displacements are the incremental ones measured at the top of the walls and are normalized by the wall height.

The following can be observed in Figs. 6 and 7 by comparing the data with each other.

- (1) Amount of displacement increased when relative density of the foundation and backfill soils decreased (compare Cases 1, 2 and 10).
- (2) The walls in Cases 3 and 4 subjected to real earthquake motion experienced smaller displacements than in Case 1 subjected to sinusoidal input motion when compared at the same acceleration amplitude. This result is easily understood as that the sinusoidal wave of 20 uniform cycles had more effect on developing permanent deformation than the earthquake waves shown in Fig. 3 with the same amplitude.
- (3) The wall in Case 8 in which the height of backfill was increased to 11.4 m exhibited larger displacement than that in Case 1 with a backfill height of 9 m.
- (4) The walls of 18 m high in Cases 5 and 14 gave a smaller displacement ratio than that of 9 m high in Cases 1 and 7.

### **Seismic Earth Pressures**

#### **Earth pressure distribution**

Earth pressure acting on the wall side and soil reaction acting on the wall base were measured by using two-directional load cells; normal and shear loads. In Fig. 8 are presented distribution of normal earth pressures measured for the Case 1 wall. In the figures, (a) plots maximum earth pressures at each point during each shaking step, and (b) the residual pressures. In the figures showing earth pressures acting on the wall side are inserted active earth pressure distribution at rest that was obtained by using the Coulomb earth pressure theory.

The following are pointed out from Fig. 8(b):

- (1) At the initial condition, earth pressure acting on the wall side is close to the active theoretical earth pressure at rest at the upper part, but far less than the theoretical one at the lower part. Such a small earth pressure measurement at the lower part might be due to arching

action caused by friction at the wall face. Or it might be related to the fact that the horizontal displacement of the wall during application of centrifugal acceleration was larger at the bottom (S3), 0.09 % of the wall height, than at the mid-height (S10), 0.02 %.

(2) The earth pressure distribution acting on the wall side is getting to converge to the theoretical active earth pressure in the last two shaking steps, at which considerable displacement was developed in the wall, see Fig. 6.

(3) At the initial condition, normal earth pressure at the base plate is large near the heel and small near the toe, i.e., the center of resultant force is biased to the heel side. Due to accumulation of shaking steps, the point of resultant force is moving forward, which is qualitatively consistent with the change of earth pressure distribution on the wall side.

The following are pointed out from Fig. 8(a):

(4) Seismic incremental earth pressure acting on the wall side increases as the increase in the magnitude of input acceleration and the increment is getting larger with depth.

(5) Fluctuation of soil reaction stress on the wall base due to shaking, which can be read by comparing figures (a) and (b), is much larger near the toe than near the heel. This suggests that the center of rocking motion exists at the heel side.

The shearing stresses on the wall side and bottom, though not presented here, tended to increase with accumulation of shaking steps.

### Comparison with the Mononobe-Okabe earth pressure

Fig. 9 plots the relationship between horizontal earth pressure coefficient measured at the mid-height (load cell No. S10, see Fig. 2) and the bottom (S3) of the wall side and the wall displacement for Case 1. Also marked by the dotted lines are the active earth pressures based on the Mononobe-Okabe theory, where experimentally obtained shearing soil friction angle,  $\phi = 41$  degree, and friction angle,  $\delta = 0$  degree, between concrete face and soil, assumed, were used. The following are observed in Fig. 9.

(1) The peak earth pressure does not agree with the theoretical one.

(2) The peak earth pressure tends to converge to the theoretical one with the increase in the wall displacement.

Fig. 10 shows the relationships between the amplitude of input acceleration and total seismic earth pressure coefficient, for sinusoidal shaking (Cases 1 and 6, Fig.(a)) and real earthquake shaking (Case 3, Fig.(b); Case 4, Fig.(c)). The total seismic earth pressure was obtained as the maximum value of the total earth pressure time history for each shaking step. Also plotted in the figure are the theoretical seismic earth pressures that were computed by using  $\phi_p = 41$  degree (peak shear strength),  $\phi_r = 35$  degree (residual shear strength) and  $\phi_d = 30$  degree (conventionally used design value), and  $\delta = 0$  degree, and also a seismic coefficient equated with the peak input acceleration divided by the centrifugal acceleration. The following may be pointed out in the figure:

(1) The experimental data are roughly close to the theoretical ones corresponding to  $\phi_p$  or  $\phi_r$ .

(2) In case of sinusoidal shaking, Fig.(a), and the Aomori record shaking, Fig.(c), the

experimental data seem to get closer to the theoretical value for  $\phi_r$  rather than that for  $\phi_p$ , when the input acceleration increases. This might coincide with the finding by Koseki et al., 1997. However, in case of the Kobe record shaking, Fig. (b), it is not the case. The exceptional trend of the earth pressure increase with the increase in input acceleration for Case 3 is considered to be due to the fact that the response acceleration amplification at over 0.5 g shaking was less than the other cases.

It should be noted regarding Figs. 9 and 10 that the peak seismic earth pressure may not be occurring when the backfill soil is in an active failure state or when the wall is displacing forward, and the base input acceleration not but the response acceleration of the backfill soil was used in computing the theoretical active earth pressure. Thus further analyses are left to be tackled for these points.

## DISCUSSION ON THE CONVENTIONAL PSEUDO-STATIC DESIGN

It is of great interest for most engineers to know how well retaining walls perform during strong earthquakes that is designed based on the conventional design method. In the following, this point is discussed by using the test results of gravity retaining walls.

According to the result of stability analysis as already presented in Fig. 1, the sliding mode is most critical and the critical seismic coefficient,  $k_{cr}$ , that yields a safety factor of unity is 0.22, while seismic coefficient that yields a safety factor of 1.2 is 0.16. Minimum safety factor of 1.2 against sliding has been specified in most of the seismic design codes in Japan. Therefore, one can state that the model retaining wall was designed against a design seismic coefficient,  $k_a$ , of 0.16.

It can be seen in Fig. 6 that the retaining wall with  $k_a = 0.16$  can sustain against earthquakes with a peak acceleration of at least 0.3 g, with negligible deformation. According to the concept of performance based design philosophy, structures are generally assigned a certain amount of allowable displacement or deformation depending on factors including the structures' function, importance, level of design earthquake and so on. In Fig. 6, for example, if the retaining wall is allowed a horizontal displacement equal to 2 % of the wall height (18 cm), then the wall can sustain up to 0.35 g PGA for sinusoidal motion (Case 1), up to 0.5 g PGA for the Aomori record (Case 4), and up to 0.55 g PGA for the Kobe record (Case 3), respectively.

As discussed in the above, the relationship between the design seismic coefficient and the PGA depends on the allowable displacement. Fig. 11 shows the relationship between the ratio of design seismic coefficient to PGA and allowable displacement. The figure was reproduced from Fig. 6 and design seismic coefficient of  $k_a = 0.16$  was used, except  $k_a = 0.12$  for Case 2. This figure suggests that the smaller the allowable displacement, the larger the design seismic coefficient to be adopted, and vice versa. Also the ratio of design seismic coefficient to PGA divided by gravitational acceleration is smaller for real earthquake acceleration than for sinusoidal acceleration. Further, the ratio for the Kobe record is smaller than that for the Aomori record. The difference in the ratio is strongly related to the input acceleration characteristics. In Fig. 3, the number of major pulses appearing in the time histories is getting smaller in the following

order: sinusoidal wave, Aomori wave and Kobe wave. This coincides well with difference in induced displacement.

Lastly, an example problem is given in order to visualize the relationship between the design seismic coefficient and the PGA. In Fig. 11, if the Kobe record with a PGA of 0.8 g were adopted as a design earthquake and the allowable displacement of the retaining wall were set as 3 % of the wall height, then the ratio is given as:

$$k_a / (\text{PGA}/g) = 0.25.$$

This gives the design seismic coefficient as:

$$\begin{aligned} k_a &= 0.25 \cdot 0.8 \\ &= 0.2. \end{aligned}$$

## CONCLUSIONS

The paper presented a part of the experimental results on seismic performance of rigid retaining walls. Although the data has not been fully examined and analyzed, the following conclusions are derived.

1. Magnitude of earthquake-induced displacement depends not only on the peak ground acceleration but also on the time history characteristics of input earthquake motions.
2. Retaining walls that are designed by the conventional pseudo-static approach with a specific seismic coefficient,  $k_a$ , can sustain against earthquakes with PGA much larger than  $k_a$  times gravitational acceleration,  $g$ . For example, gravity retaining walls that are designed so as to satisfy a minimum safety factor of 1.2 with  $k_a = 0.2$  may only experience as much displacement as 3 % of the wall height when excited by the Kobe accelerogram with a PGA of 0.8 g.
3. Total seismic earth pressure acting on the wall from the backfill soil was close to the active earth pressure based on the Mononobe-Okabe theory. However, the distribution along the wall height were different from the theoretical one.

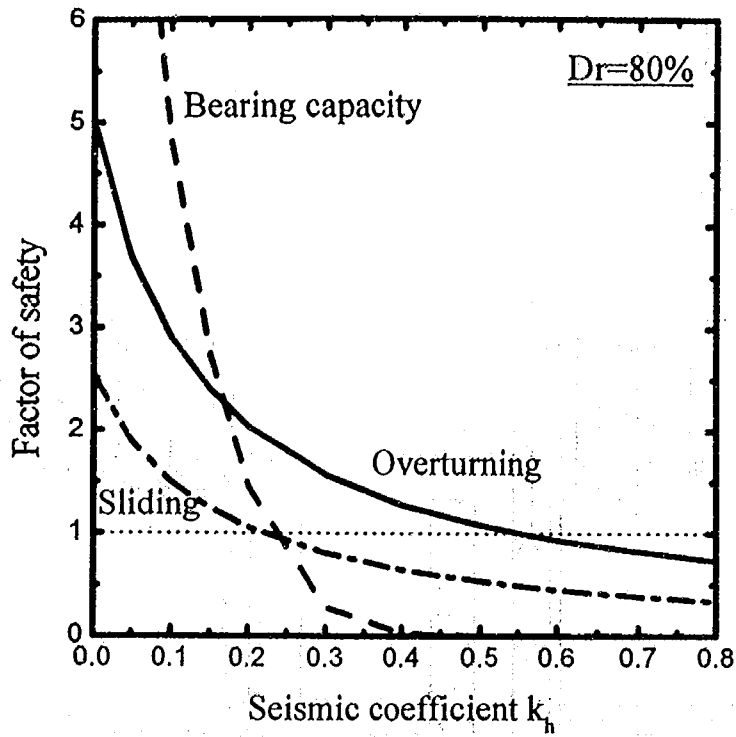
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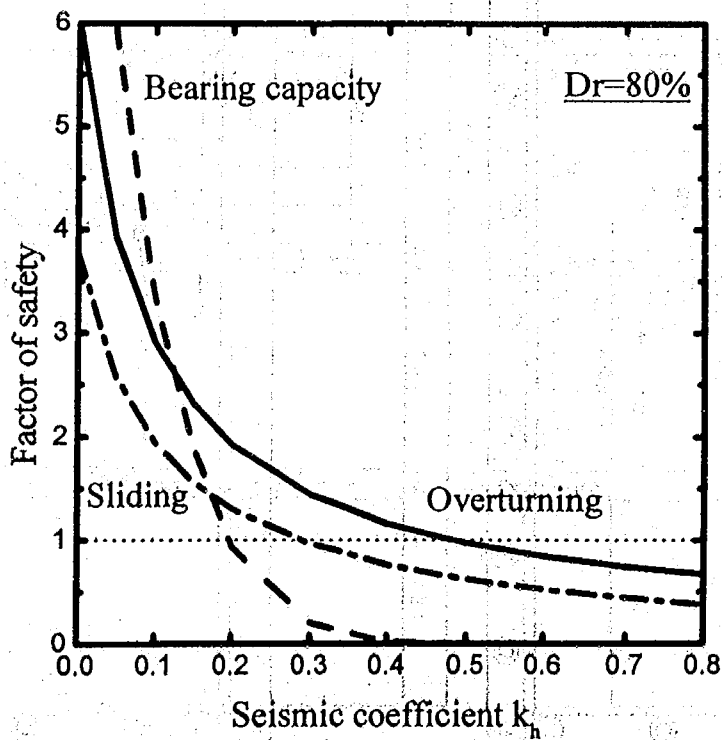
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- Fig. 2 Configurations of the models
- Fig. 3 Time histories of input accelerations
- Fig. 4 Deformation of the model (Case 6)
- Fig. 5 Deformation of the model (Case 7)
- Fig. 6 Input acceleration vs. horizontal displacement for the gravity retaining walls
- Fig. 7 Input acceleration vs. horizontal displacement for the cantilever retaining walls
- Fig. 8 Distribution of earth pressures acting on the side and bottom of the gravity retaining wall for Case 1
- Fig. 9 Relationships between horizontal earth pressure coefficient acting on the wall side and the wall displacement for Case 1
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- Fig. 11 Relationship between the ratio of design seismic coefficient to PGA and allowable displacement



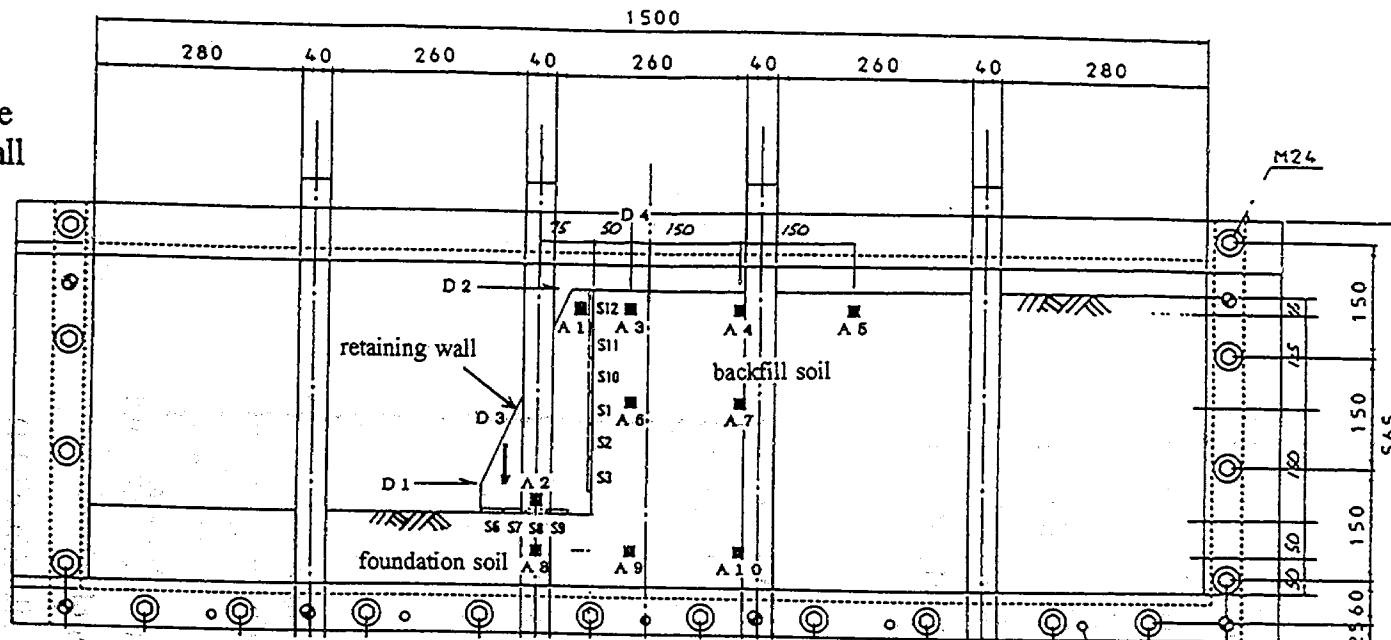
(a) Gravity retaining walls (Case 1, 3, 4, 5, 6)



(b) Cantilever retaining walls (Case 7, 14)

Fig.1 Seismic stability analysis results for the model retaining walls based on the pseudo-static stability analysis

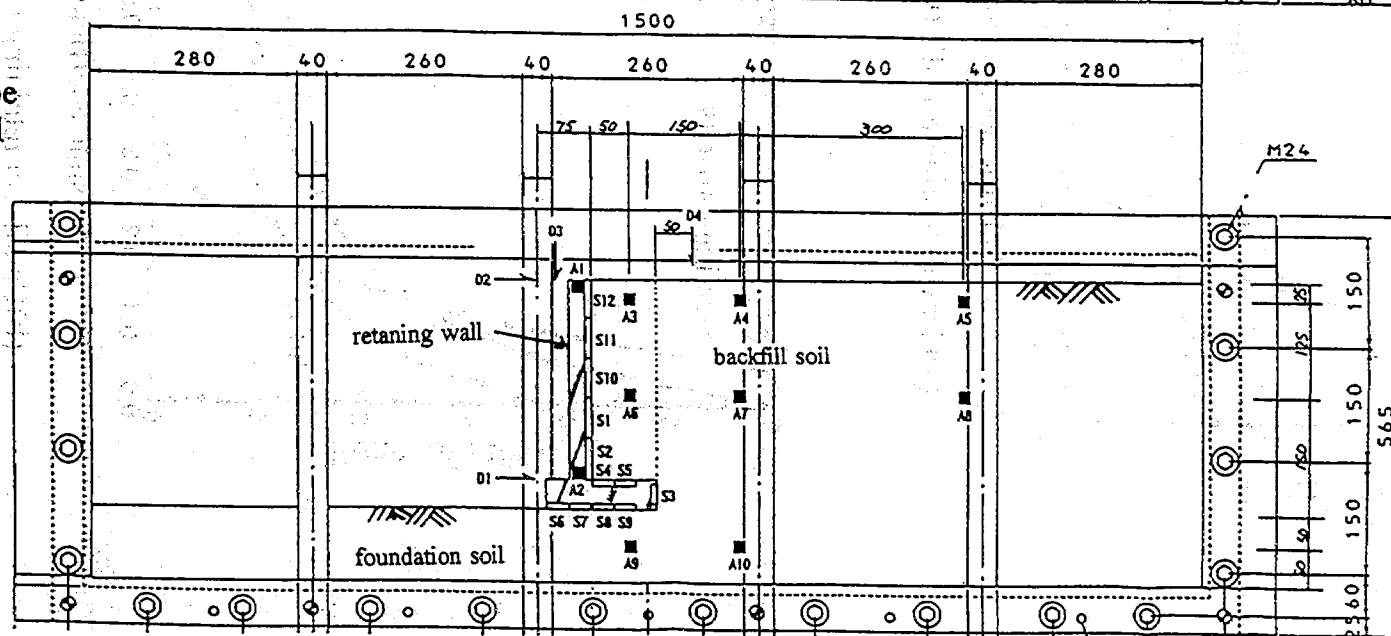
Gravity-type  
retaining wall



Legend

- A Accelerometer
- D Displacement gauge
- S load cell

Cantilever-type  
retaining wall



(unit:mm)

Fig.2 Configurations of the models

Table 1 Test conditions

| Case No. | Wall type  | Wall height* (m) | Embankment height* (m) | Relative density of backfill (%) | Water content (%) | Input waveform*          | Maximum input acceleration* (gal)          | Centrifugal acceleration (G) |
|----------|------------|------------------|------------------------|----------------------------------|-------------------|--------------------------|--------------------------------------------|------------------------------|
| 1        | Gravity    | 9                | 9                      | 82                               | 0                 | Sinusoidal 2Hz, 30cycles | 68, 96, 185, 299, 309, 372, 422            | 30                           |
| 2        | Gravity    | 9                | 9                      | 61                               | 0                 | Sinusoidal 2Hz, 30cycles | 78, 206, 310, 448                          | 30                           |
| 3        | Gravity    | 9                | 9                      | 82                               | 0                 | Kobe wave fp=2Hz**       | 93, 167, 242, 252, 350, 480, 542, 676, 702 | 30                           |
| 4        | Gravity    | 9                | 9                      | 83                               | 0                 | Aomori wave fp=2Hz**     | 92, 232, 261, 359, 408, 497, 536           | 30                           |
| 5        | Gravity    | 18               | 18                     | 84                               | 0                 | Sinusoidal 2Hz, 30cycles | 87, 167, 271, 296, 349, 429                | 60                           |
| 6        | Gravity    | 9                | 9                      | 83                               | 0                 | Sinusoidal 2Hz, 30cycles | 345, 431                                   | 30                           |
| 8        | Gravity    | 9                | 11.4                   | 77                               | 0                 | Sinusoidal 2Hz, 30cycles | 233, 333, 433                              | 30                           |
| 10       | Gravity    | 9                | 9                      | 57                               | 5                 | Sinusoidal 2Hz, 30cycles | 210, 350, 433                              | 30                           |
| 7        | Cantilever | 9                | 9                      | 82                               | 0                 | Sinusoidal 2Hz, 30cycles | 109, 187, 294, 415                         | 30                           |
| 12       | Cantilever | 9                | 11.4                   | 80                               | 0                 | Sinusoidal 2Hz, 30cycles | 217, 333, 417                              | 30                           |
| 13       | Cantilever | 9                | 9                      | 54                               | 5                 | Sinusoidal 2Hz, 30cycles | 217, 333                                   | 30                           |
| 14       | Cantilever | 18               | 18                     | 81                               | 0                 | Sinusoidal 2Hz, 30cycles | 192, 192, 333, 417, 450                    | 60                           |

\*in prototype scale

\*\*predominant frequency

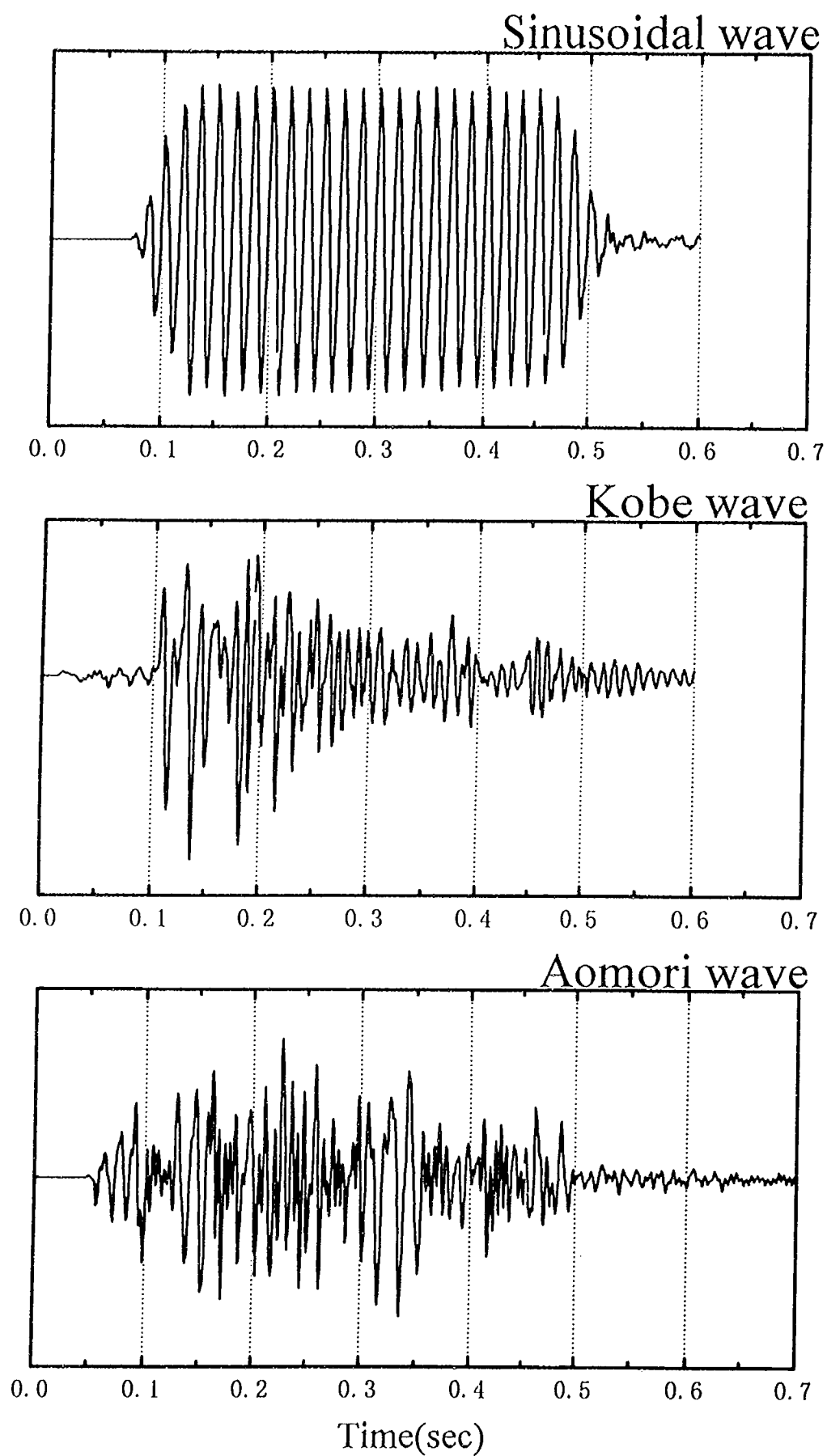
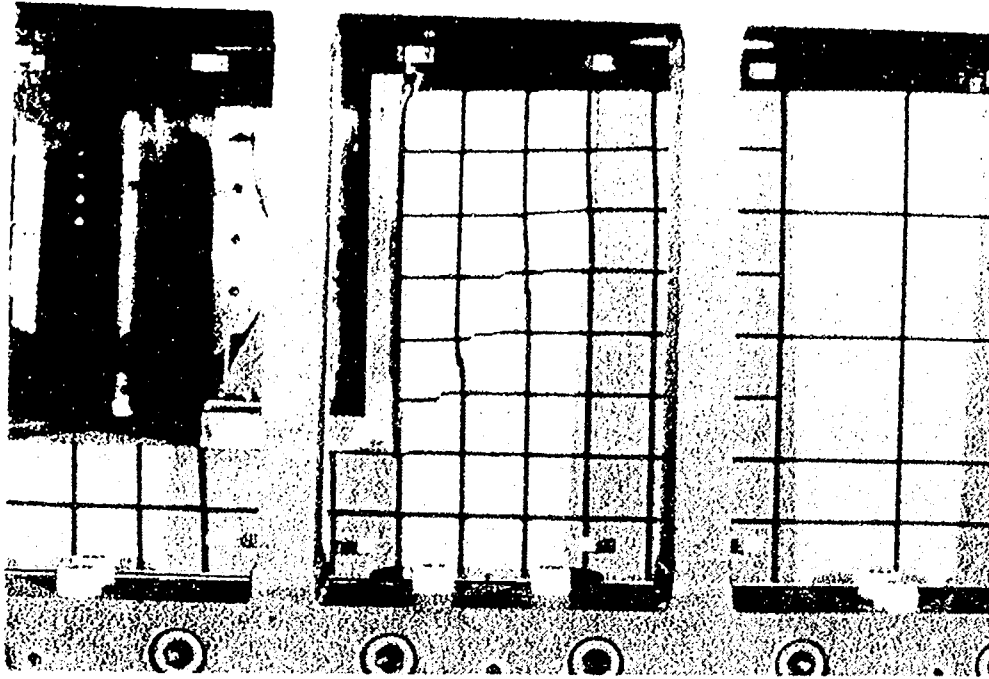
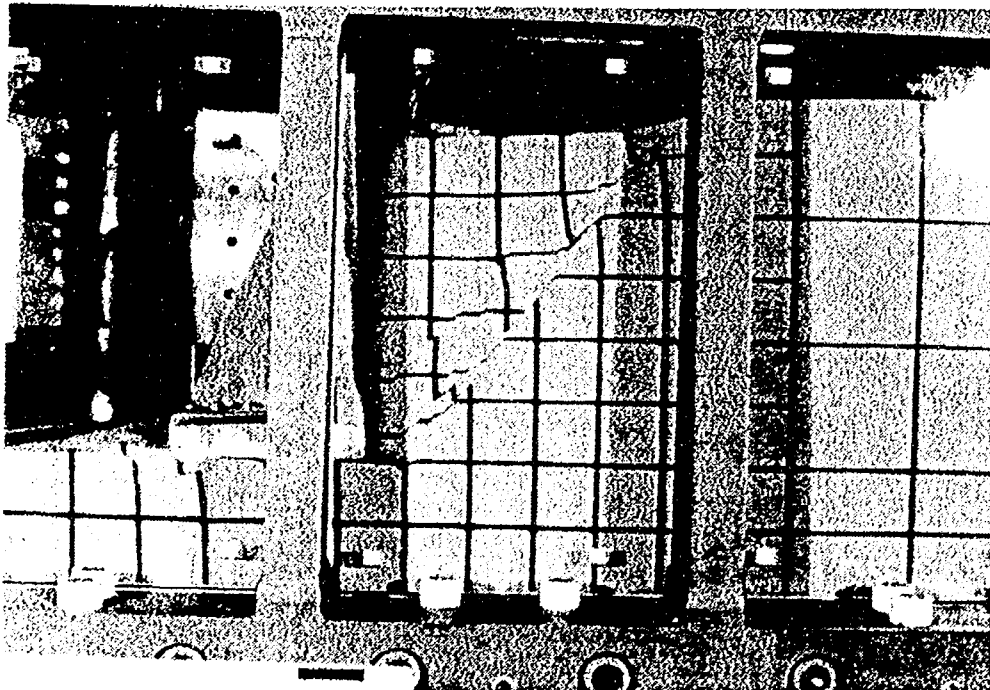


Fig.3 Time histories of input accelerations

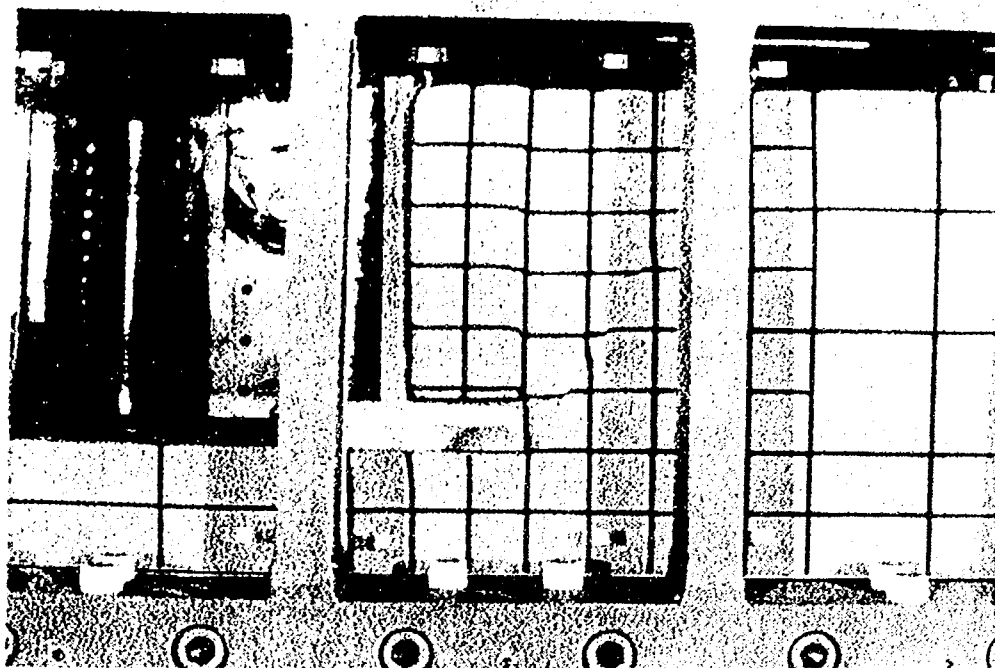


(a) after 1st shaking (345gal)

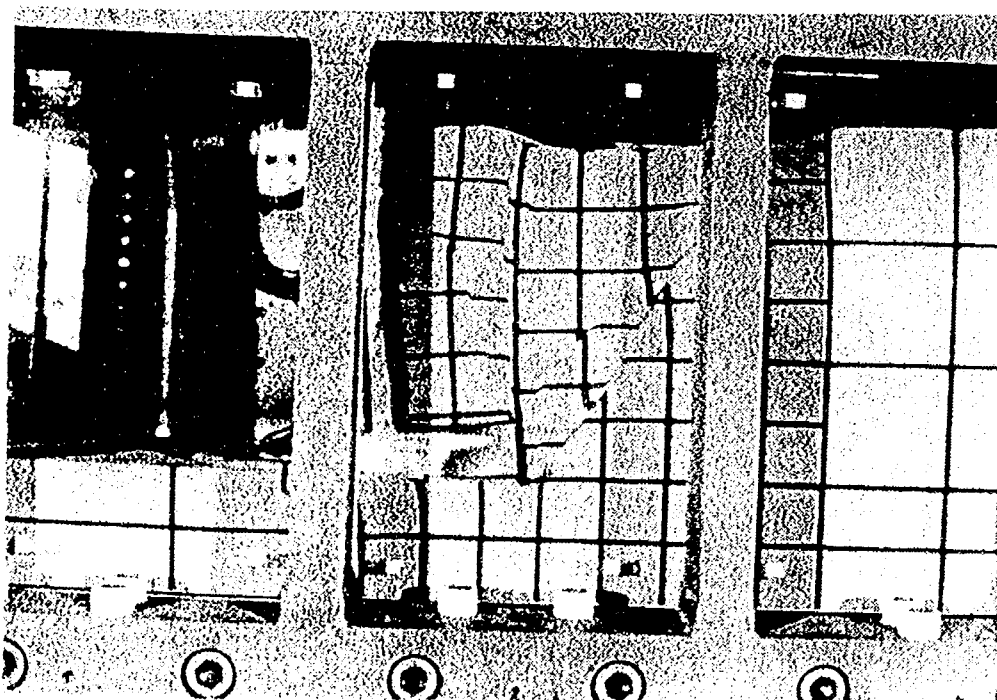


(b) after 2nd shaking (431gal)

Fig.4 Deformation of the model (Case 5)



(a) after 3rd shaking (294gal)



(b) after 4th shaking (415gal)

Fig.5 Deformation of the model (Case 7)

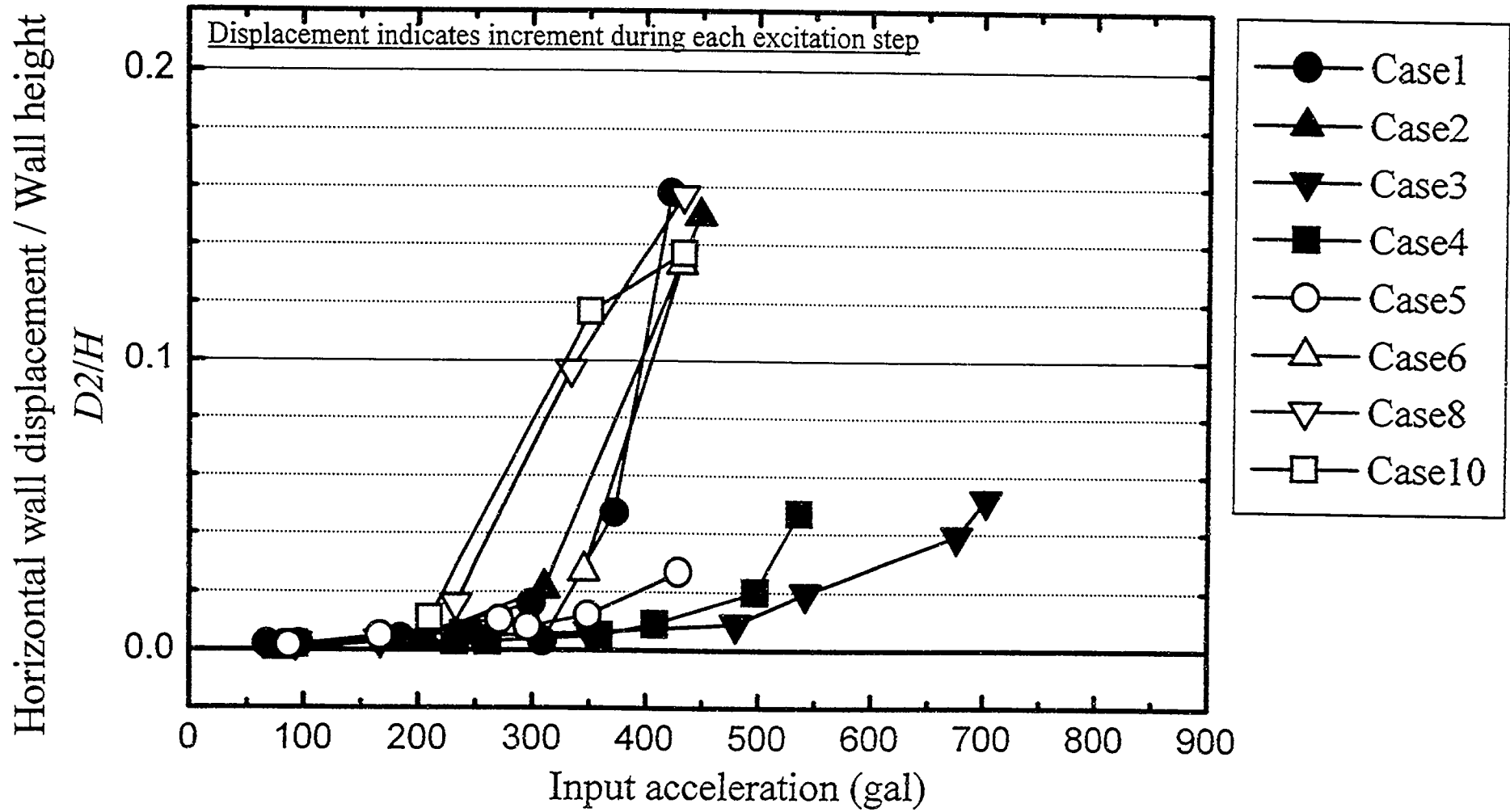


Fig.6 Input acceleration vs. horizontal displacement for the gravity retaining walls

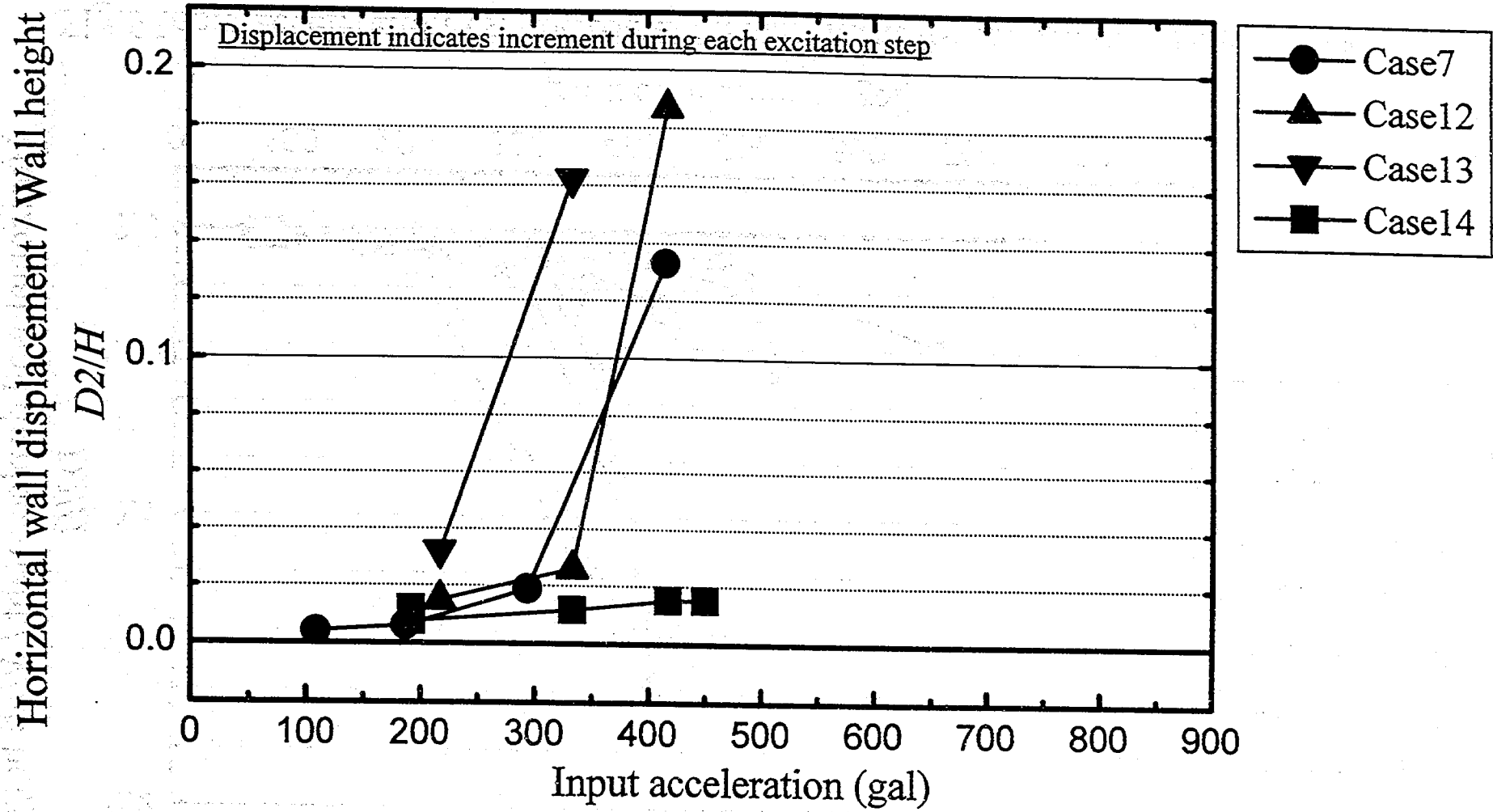


Fig.7 Input acceleration vs. horizontal displacement for the cantilever retaining walls

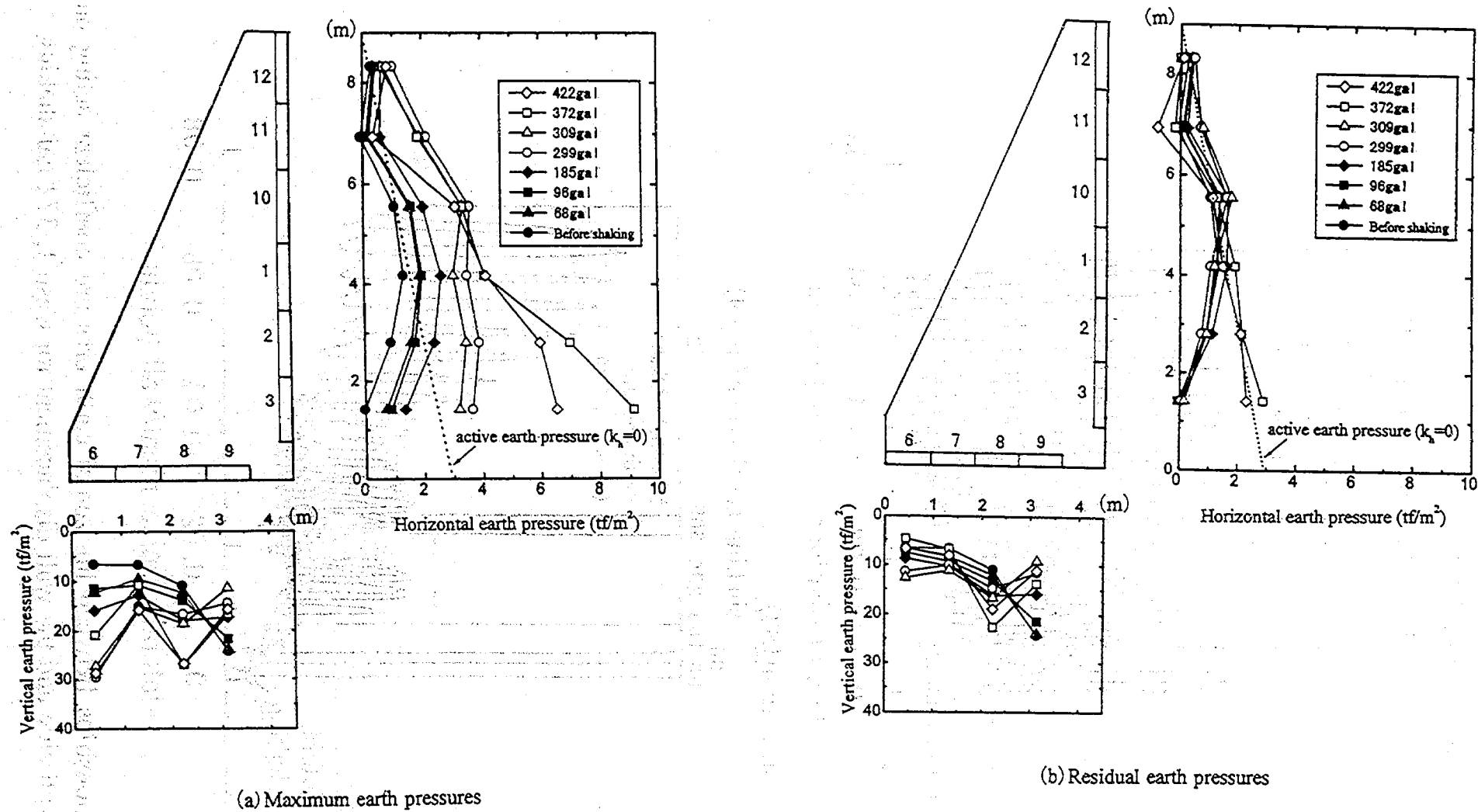


Fig.8 Distribution of normal earth pressures acting on the side and bottom of the gravity retaining wall for Case 1

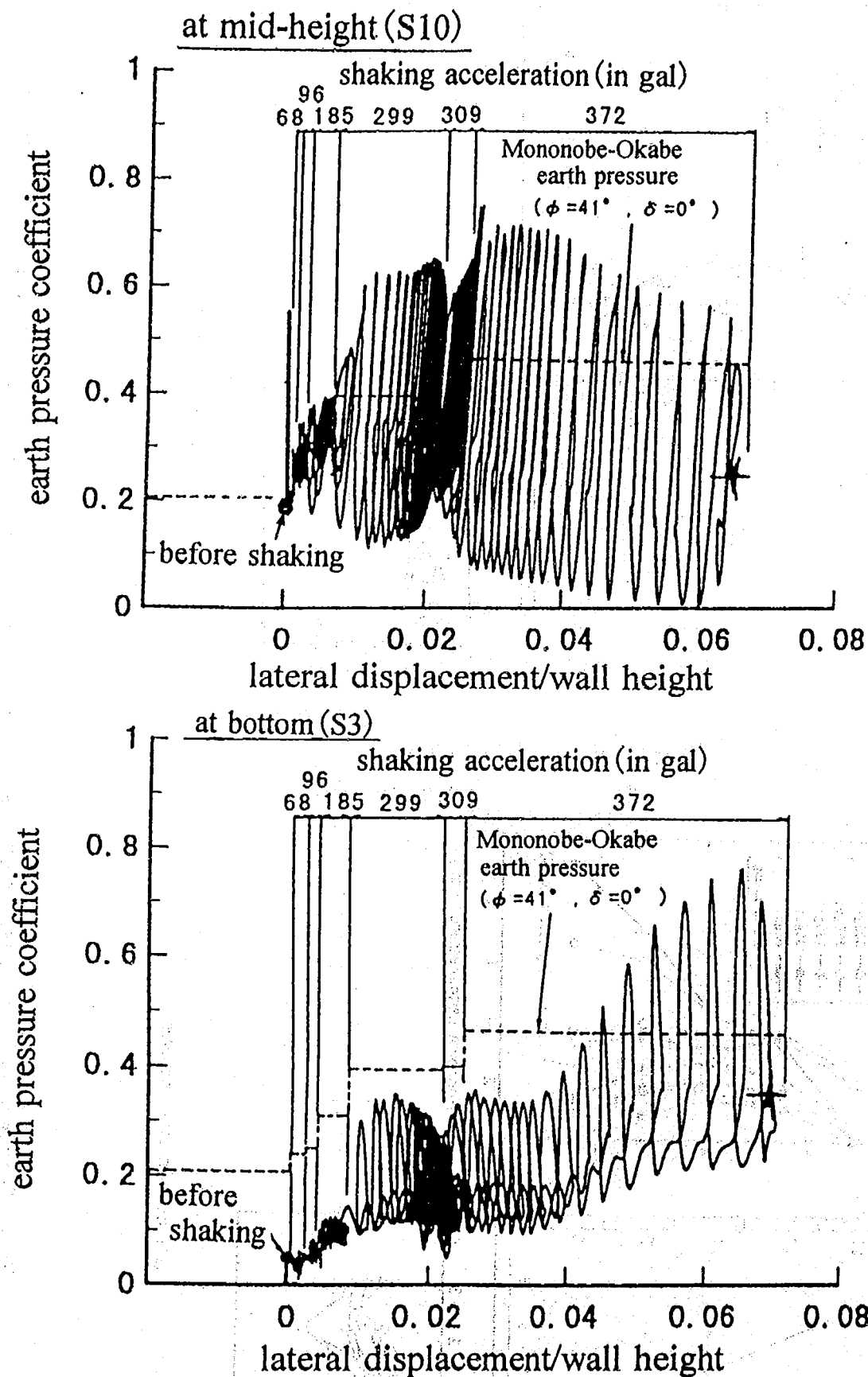
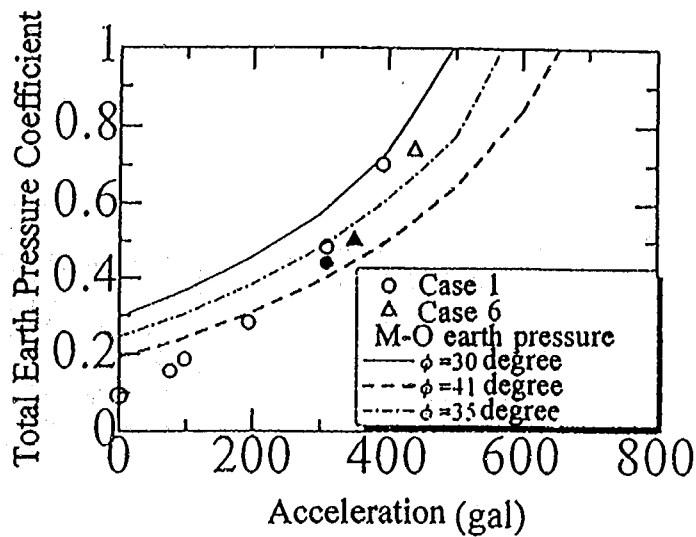
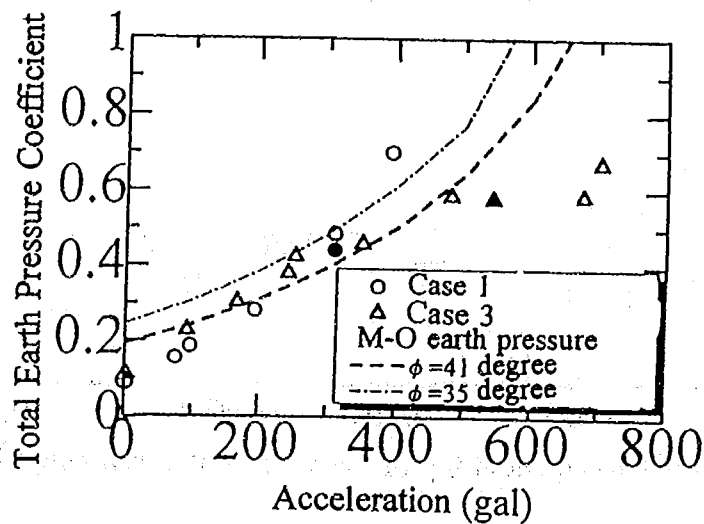


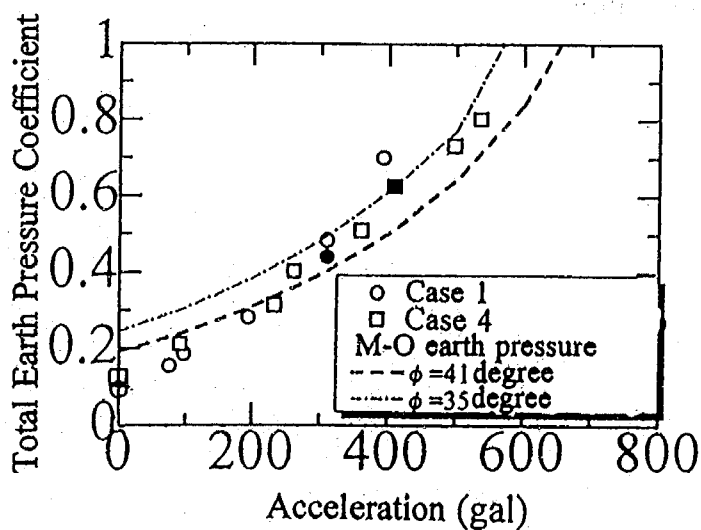
Fig.9 Relationships between horizontal earth pressure coefficient acting on the wall side and the wall displacement for Case 1, 372 gal shaking



(a) sinusoidal acceleration



(b) real earthquake acceleration, the Kobe-record



(c) real earthquake acceleration, the Aomori-record

Fig.10 Relationships between the amplitude of input acceleration and total earth pressure coefficient

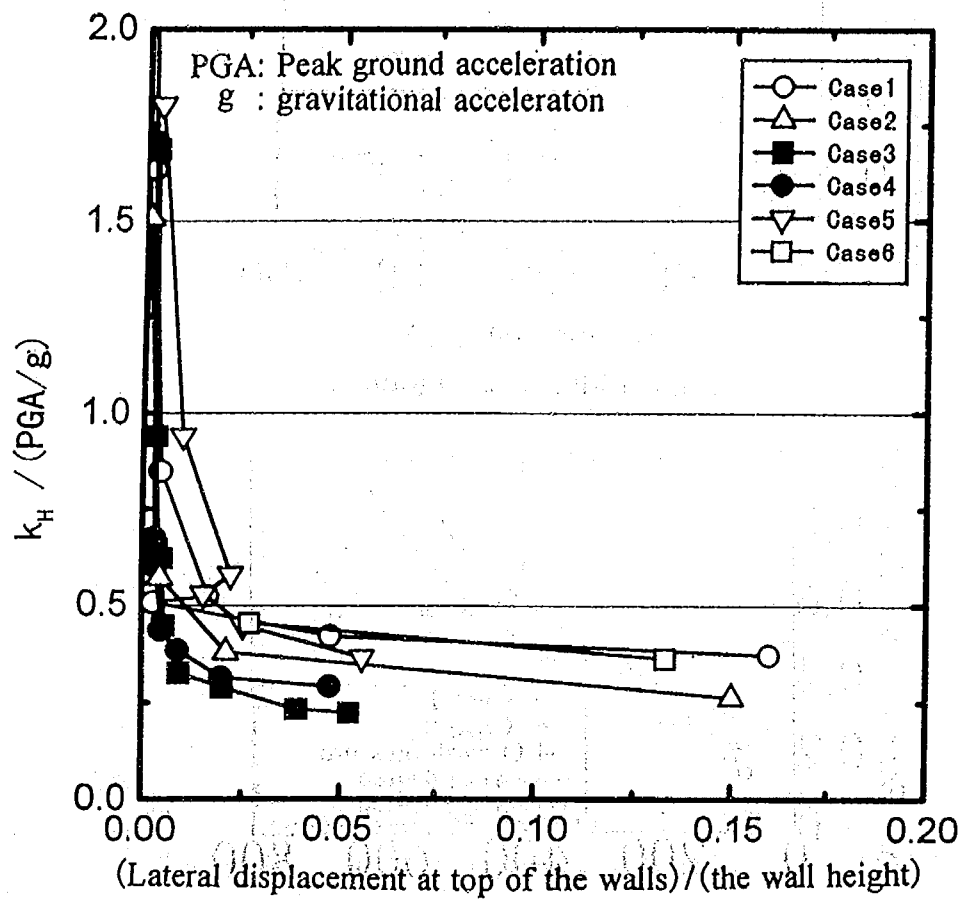


Fig.11 Relationship between the ratio of design seismic coefficient to PGA and allowable displacement

## The dynamic geotechnical centrifuge at PWRI

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**ABSTRACT:** A geotechnical centrifuge with a radius of 6.6 m has been developed at the Public Works Research Institute, Japan. This is the third centrifuge in PWRI. The main features of the equipment are; a radius of 6.6 m, an earthquake capability of 40 g-ton, and double swing platforms to enable clay consolidation preparation at a same time.

### 1 INTRODUCTION

The Public Works Research Institute (PWRI) constructed its first geotechnical centrifuge with a radius of 1.25 m in 1961 for use in earth dam stability studies. With increasing needs of geotechnical earthquake studies, the second centrifuge with a radius of 2.0 m was introduced in 1987 with a mounted shaker with a shaking capacity of 4 g-ton. This machine has been used for studies of liquefaction, seismic slope stability, soil improvement techniques, tunnel excavation, etc. Due mainly to the experimental ease compared with full-size tests or model shaking table tests, its operational ratio has been nominally 100 % for

these years.

The Hyogoken Nanbu earthquake, January 17, 1995, that hit the densely populated area caused devastating damages to human lives and properties. It also demonstrated various earthquake geotechnical problems. Immediately after the quake, the PWRI, the Ministry of Construction headquarters and the Government decided to construct a new centrifuge facility with a primary purpose of geotechnical earthquake studies. The PWRI initiated basic and detailed design works in May, 1995. Construction of the machine and the facility started in March, 1996 and completed in March, 1997.

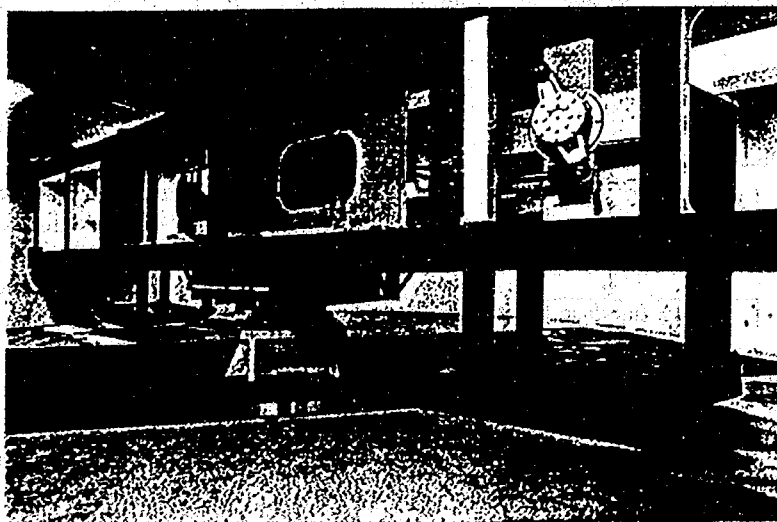


Figure 1. PWRI's geotechnical centrifuge

(unit : mm)

Table 1. Specifications of centrifuge

|                        |                                                    |
|------------------------|----------------------------------------------------|
| Effective Rotor Radius | 6.6 m                                              |
| Maximum Acceleration   | static test :<br>150 g<br>dynamic test :<br>100 g  |
| Payload Weight         | 5 ton                                              |
| Payload Capacity       | 400 g-ton                                          |
| Swing Platform         | static test : 2<br>dynamic test : 1                |
| Platform Space         | length : 2.4 m<br>width : 1.3 m<br>height : 1.0 m  |
| Data Acquisition       | static test :<br>100 ch<br>dynamic test :<br>80 ch |

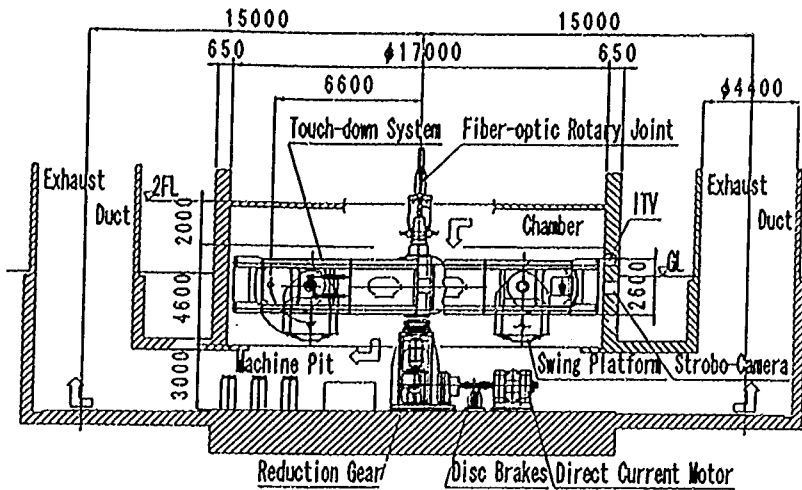


Figure 2. Side view of centrifuge

## 2 MAIN SPECIFICATIONS

The PWRI's centrifuge is shown in Figure 1. Its structure is similar to that of the Taisei centrifuge (Nagura et al. 1994). Figure 2 shows the side view of the centrifuge. The main parts of the centrifuge are housed in an underground reinforced concrete pit, 17.0 m in diameter and 4.6 m deep.

Table 1 provides the specifications. The main features of this centrifuge are:

- This is one of the largest dynamic centrifuges in the world.
- A shaking table is mounted which can precisely simulate strong earthquake motions by fixing the swing platform to the rigid rotating arm. This is called "Touch-down System" (Nagura et al. 1994).
- This centrifuge has two swing platforms at the both ends of the arm to facilitate soft clay consolidation at a same time, for experimental efficiency. The "Touch-down System" is adopted in only one of them.

## 3 CENTRIFUGE SERVICES

### 3.1 Rotary Joints

The PWRI's centrifuge is equipped with rotary joints which permit fluids to flow through the axis of the machine to the platform. There are a total of 8 passages, with 6 rotary joints for oil and 2 rotary joints for either water or air. The capacity of working pressure is 10 MPa for oil and 0.7 MPa for water or air.

### 3.2 Data Acquisition System

The data acquisition system is shown in Figure 3. For dynamic tests, the analogue signal outputs of transducers are amplified, digitized and converted to optical signals through the electro-optic converting unit mounted on the rotating arm. Through the multiport fiber-optic rotary joint (Nagura et al. 1994), the data are sent to the optic-electric converting unit in the control room and converted to electrical signals. The signals are recorded on the internal disk of a workstation. For static tests, the signal outputs of transducers are recorded on the static strain indicator on the rotating arm. The data are sent to the workstation through the GP-IB interfaces. The dynamic strain amplifier and the static strain indicator are controlled from the workstation through the GP-IB interfaces. Test results can be viewed on the workstation monitor during the experiment. This system can provide noise-free precise data.

### 3.3 In-flight Balancing System

In order to eliminate the imbalance forces of rotating arm produced during rotation, the in-flight automatic balancing system is provided. In-flight balancing is achieved by moving two balancing

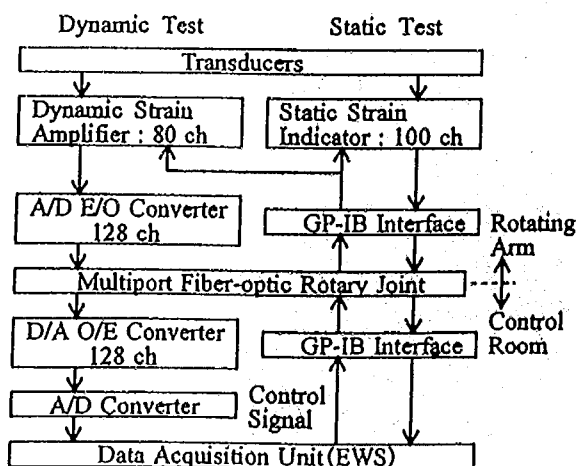


Figure 3. Data acquisition system

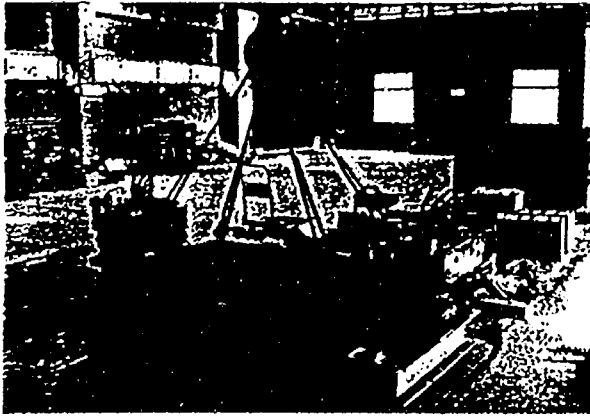


Figure 4. Shaking table

Table 2. Specifications of shaking table

|                                  |                                   |
|----------------------------------|-----------------------------------|
| Table Size                       | length : 1.7 m<br>width : 1.0 m   |
| Shaking Direction                | horizontal : 1                    |
| Type of Shaking Wave             | sinusoidal wave<br>irregular wave |
| Maximum Centrifugal Acceleration | 100 g                             |
| Maximum Shaking Acceleration     | 40 g                              |
| Maximum Velocity                 | 90 cm/s                           |
| Maximum Displacement             | $\pm 5$ mm                        |
| Capacity                         | 40 g-ton                          |
| Frequency Range                  | 10 - 400 Hz                       |

weights on the rotating arm toward the axis of the arm. When the balance of the arm is lost and the horizontal displacement of the rotating axis measured with a gap sensor becomes large, the imbalance error signals are delivered to the balancing weights and they are moved. The auto-balance capacity is 5 % of the maximum payload: 20 g-ton.

### 3.4 Monitoring System

While the arm is rotating, test models on the swing platform can be taken with the strobo-camera and the CCD camera which are mounted on the wall of the concrete pit and on the arm, respectively. Three video cameras are also set in the concrete pit mainly for human safety reasons.

### 3.5 Ventilation Unit

In order to prevent a temperature buildup inside the underground pit, three fans are used for air ventilation in the pit. These fans are set in the chamber above the pit. As shown in Figure 2 by arrows, the air flows along the rotating axis, from the chamber to the exhaust ducts.

## 4 SHAKING TABLE

### 4.1 Specifications

An electro-hydraulic shaking table shown in Figure 4 has been developed. Specifications of the shaking table are shown in Table 2. The shaking capacity is as large as that of the shaker at UC Davis (Kutter et al. 1994) which is the largest in the world. This shaking table provides 40 g shaking acceleration to 1 ton payload, which is equivalent to the maximum ground acceleration observed during the 1995 Hyogoken Nanbu earthquake when excited at a centrifugal acceleration of 50 g.

### 4.2 Earthquake Simulation System

This system is composed of the excitation device, the hydraulic supply system, and the excitation control system as shown in Figure 5.

#### 4.2.1 Excitation device

As shown in Figure 6, the excitation device is designed symmetrically so that the swing platform should not be inclined. Two actuators which have two servo valves in each are arranged on both sides of the shaking table. There are four actuators and eight servo valves in total. In order to measure the displacement of actuators and the acceleration

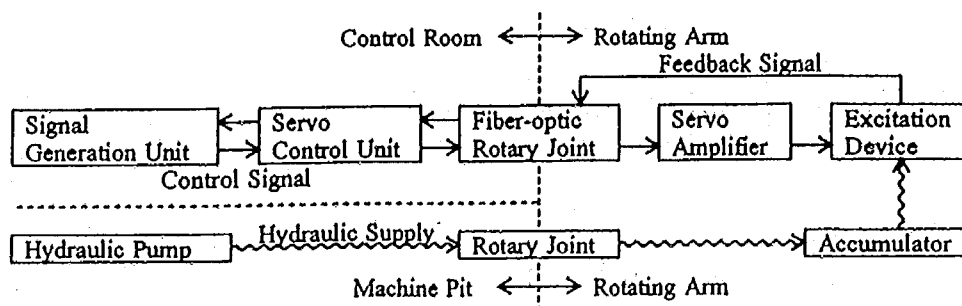


Figure 5. Earthquake simulation system

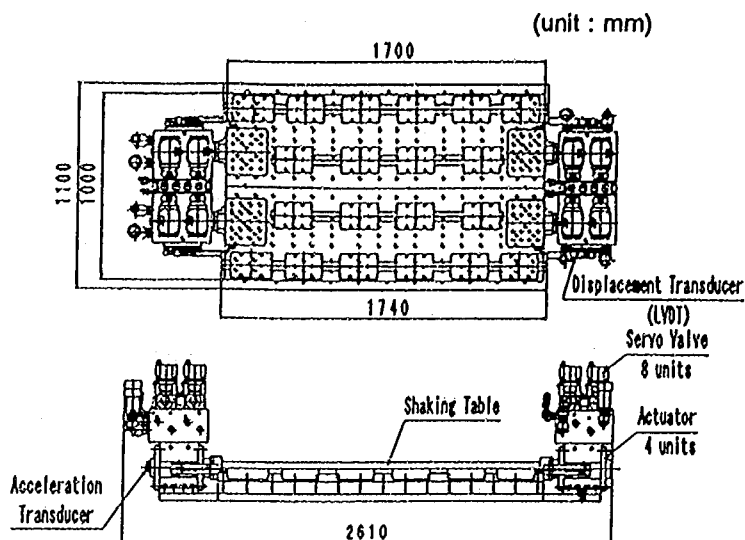


Figure 6. Plan and side views of excitation device

of shaking table for the control, LVDTs and accelerometers are installed.

Figure 7 shows the structure of the actuator and the servo valve. The servo valve is composed of a large-scale torque motor, a pilot spool and a main spool, and controls the quantity and the direction of flowing oil to the actuator according to the movement of a main spool. In order to make the excitation capacity larger, it is necessary to supply a large quantity of flowing oil to the actuator. Therefore the diameter of the main spool become large, and the pilot spool is installed in order to drive the main spool hydraulically. The pilot spool hangs on the one end of an armature in the direction of a centrifugal acceleration, and the counter weight is installed on the other. As a result, an unbalanced torque by the self-weight of the pilot spool and a friction torque by the vertical motion is decreased. The movement of the servo valve and the actuator is controlled by servo in the following sequence.

- The signal from the servo control unit is converted to the electric current by the servo amplifier, and the current is input to the torque motor.
- The torque motor swings the armature according to the current.
- The pilot spool is driven up and down by the swing of armature.
- The quantity and the direction of oil supplied to a main spool are controlled by the movement of the pilot spool, and the main spool is driven right and left.
- The quantity and the direction of oil supplied to the actuator are controlled by the movement of the main spool, and the actuator is moved right and left.
- The excitation power is transmitted to the shaking table.

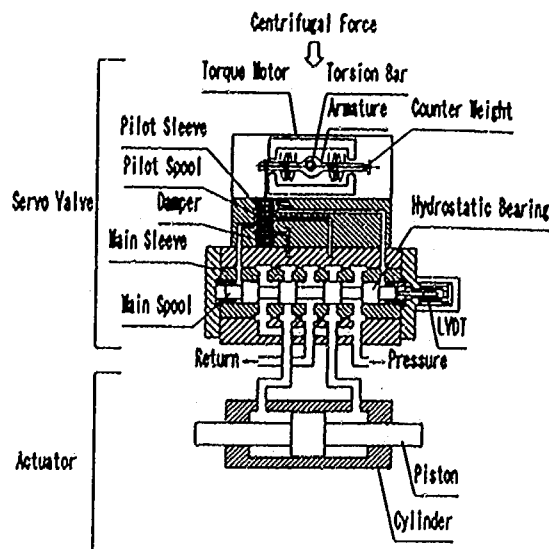


Figure 7. Actuator and servo valve

#### 4.2.2 Hydraulic supply system

When dynamic tests are performed, it is necessary to supply a large quantity of oil to the excitation device. Therefore, the oil is supplied continuously from the hydraulic pump in the machine pit to the accumulator on the rotating arm through the rotary joints. Pressurized oil in the accumulator is exhaled in time of shaking.

#### 4.2.3 Excitation control system

The excitation control system is composed of the signal generation unit and the servo control unit. The control signal is transferred to the excitation device through the optic rotary joint. The signal generation device has the functions to generate input signal and to correct the input signal so as to adjust response waves to target waves according to the feedback signal from the excitation device.

### 5 Performance

#### 5.1 Electric Power Consumption

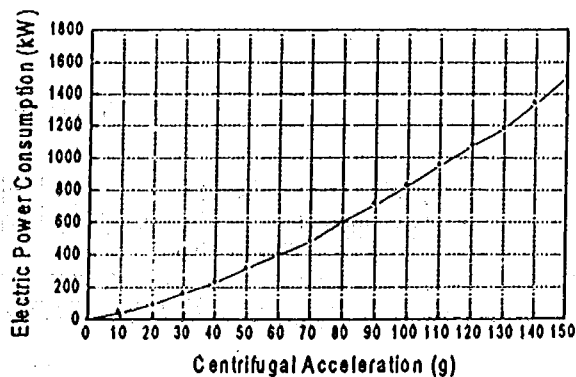


Figure 8. Electric power consumption

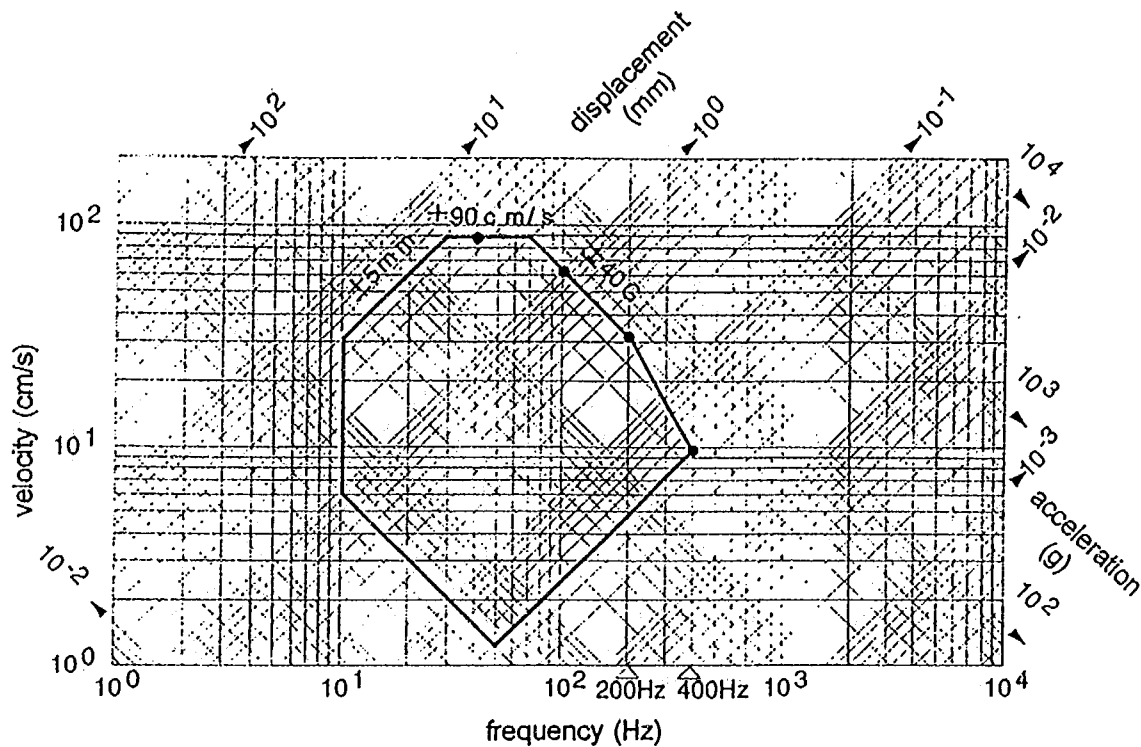


Figure 9. Shaking limit diagram

Figure 8 shows the relation between the centrifugal acceleration and the electric power consumption in a steady state. This centrifuge consumes 1,500 kW at a centrifugal acceleration of 150 g. The run-up times are 5.2 minutes to 50 g, 10.2 minutes to 100 g and 22.6 minutes to 150 g.

### 5.2 Proof Tests of Shaking Table

In order to investigate the performance of the shaking table, proof tests were carried out.

At first, sinusoidal waves were used under 100 g. Several points on the shaking limit line shown in Figure 9 were selected. Figure 10 shows the response acceleration waves of the shaking table with a weight of about 400 kg at frequencies of 40, 100, 200, 400 Hz, respectively. These input waves were not corrected. The figure indicates that response waves were distorted in the case of large

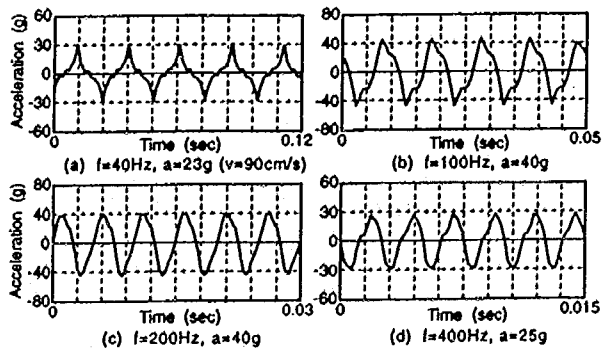


Figure 10. Response waves

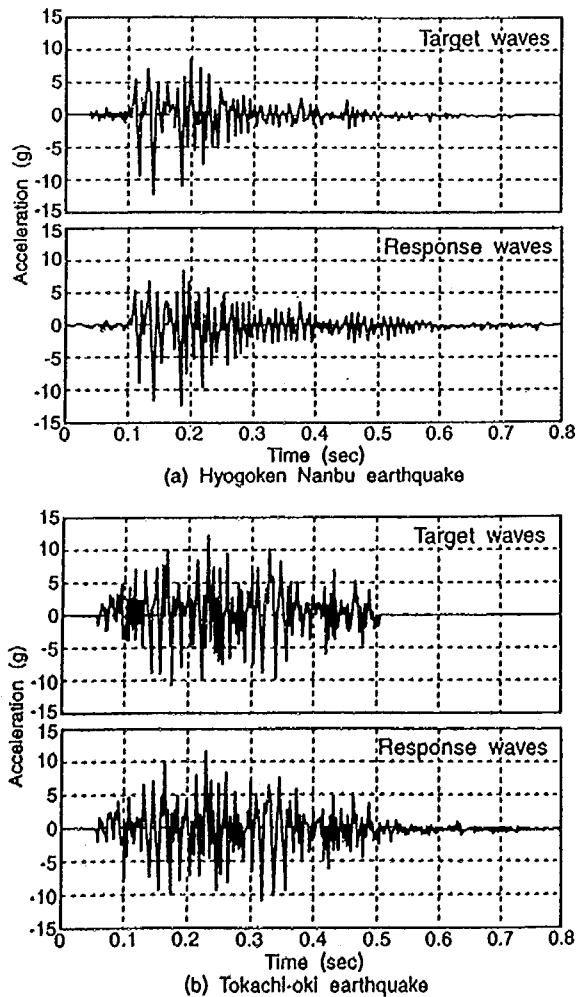


Figure 11. Comparison between target waves and response waves

displacement or extremely high frequency.

Next, two earthquake waves were used under 30 g: the NS component of the JMA Kobe acceleration record in the 1995 Hyogoken Nanbu earthquake, and the EW component of the Aomori Harbor acceleration record in the 1968 Tokachi-oki earthquake. Each of the target waves was scaled so that the maximum acceleration was 12 g and the predominant frequency became 60 Hz. Figure 11 compares the response acceleration waves of the shaking table with a weight of about 1.4 ton and the target earthquake waves. These input waves were corrected with the excitation control system. Figure 11 shows that the response waves almost coincides with the target waves.

#### ACKNOWLEDGMENTS

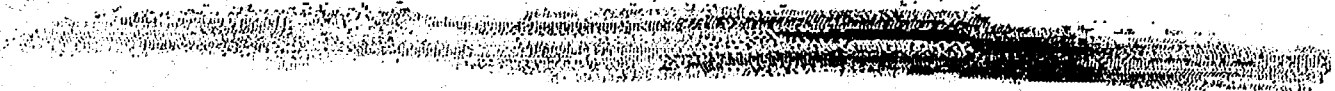
The authors are grateful to the support of the PWRI, Ministry of Construction for decision making and funding to construct the centrifuge.

They are also grateful to the staff of the PWRI for their contribution to determine the basic design. The guidance and advice from other public and private centrifuge centers are gratefully acknowledged. Construction of the machine and the facility by Hitachi Corporation and Taisei Corporation, respectively, in a very short time, is also acknowledged.

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- Kutter, B. L. et al. 1994. Design of a large earthquake simulator at UC Davis. *Proc. Int. Conf. Centrifuge 94*, Singapore. Balkema: 169-175.
- Nagura, K., M. Tanaka, K. Kawasaki & Y. Higuchi 1994. Development of an earthquake simulator for the TAISEI centrifuge. *Proc. Int. Conf. Centrifuge 94*, Singapore. Balkema: 151-156.

# SESSION 4



- *B. Kutter*: Remediation and Apparent Shear Strength of Lateral Spreading Centrifuge Models
- *Y. Taji*: Reproduction of Large - scale 1g Tests on Dry Sand Deposit and Pile Foundation using Centrifuge Modeling

## REMEDIATION AND APPARENT SHEAR STRENGTH OF LATERAL SPREADING CENTRIFUGE MODELS

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### ABSTRACT

Centrifuge models involving liquefaction have been tested with an ultimate goal to enable determination of the optimal extent of ground remediation to achieve a specified performance criteria. Models of a bridge site consisting of a deep, liquefiable sand layer overlain by a sloping clay layer with a free face at a river channel were tested at a centrifugal acceleration of 30 g. Densification of a limited region of sand and in-ground water barriers were evaluated as possible remediation treatments. The test results show that settlements were controlled, and lateral sliding was reduced but not adequately controlled by the limited remedial treatments.

Data from one of the untreated models is back-analyzed to evaluate the apparent shear strength, which controls the lateral spreading. The peak measured mobilized shear stresses are much larger than back-calculated strengths from (one-way and two-way) sliding block analyses. The two-way sliding block analyses were used to calculate ground oscillations as well as permanent displacements.

### INTRODUCTION

To control liquefaction consequences, a variety of soil improvement methods have been proposed. Two main uncertainties with all of the proposed procedures are the volume of soil that should be treated and the level of treatment required to achieve a desired level of performance. Limited case histories reported by Mitchell et al. (1995) indicate that ground improvement by densification has been successful where the treatment extended to a lateral distance equal to the depth of the treatment beyond the perimeter of the structure. Establishment of relationships between the extent of treatment and expected performance is one of the eventual goals of the research presented in this paper. This paper presents results from five centrifuge model tests, two of which included remediation by densification of a portion of the liquefiable sand.

Many case histories have been analyzed to back-calculate soil shear resistance to lateral displacement from earthquakes (Newmark 1965, Seed 1966, Idriss 1985, Seed 1987, Davis et al. 1988, Seed and Harder 1990, Dobry and Baziar 1993). Some case histories involved flow failure and others involved lateral spreading. In lateral spreading cases, one-way Newmark type sliding block analysis has been used to calculate permanent displacements during dynamic loading. In this paper, apparent shear strengths of laterally spreading soil are back-calculated by comparing measured displacements with one-way and two-way sliding block calculations.

### CENTRIFUGE EXPERIMENTS

Five centrifuge experiments have been completed as a part of a project titled, "Liquefaction Remediation at Bridge Sites". The models were constructed in the FSB (Flexible Shear Beam) model container and tested on the large centrifuge at UC Davis. The large centrifuge, the earthquake simulator, and model container are described by Kutter et al. (1994). All data presented in this paper are scaled to prototype values unless otherwise stated.

#### Model descriptions

A river and its flood banks with and without an abutment were modeled in five different centrifuge tests to study the effects of liquefaction remediation. As indicated in Figure 1, the soil profile consisted of a 6.0 m thick dense sand layer ( $D_r = 80\%$ ) at the bottom, a 9 m uniform thickness medium density sand layer ( $D_r = 50\%$ ), and a sloping surface deposit of overconsolidated clay. The river channel cuts through the surface deposit across the short dimension (east-west direction) of the model container. To the extent possible, the model dimensions were based on reasonable prototype dimensions in accordance with the scaling laws. As indicated in Figure 1, the clay layer was thickened at the boundaries to prevent leakage of pore pressures along the boundary. The entire container was tilted about 3% relative to the centrifugal acceleration field; the resultant slope of the clay surface was near zero on the north flood plane and 9% on the south flood plane.

No remedial work was involved in BAM01, BAM02 and BAM05. In BAM03, the medium density sand layer in the central third of the container, across the river channel was densified to full depth by pluviation to a relative density of about 80%. In practice, densification of soil directly beneath an abutment and river channel may be difficult. The zone of improvement in the model was chosen to be of sufficient size to produce a noticeable effect, to leave a substantial unimproved zone between the improved zone and the container boundaries and to keep the geometry simple.

In BAM04 an identical densified zone (as in BAM03) was included and an impermeable water barrier, a 19 mm thick (prototype scale) latex rubber sheet, was placed beside the densified zone. This barrier may be thought to

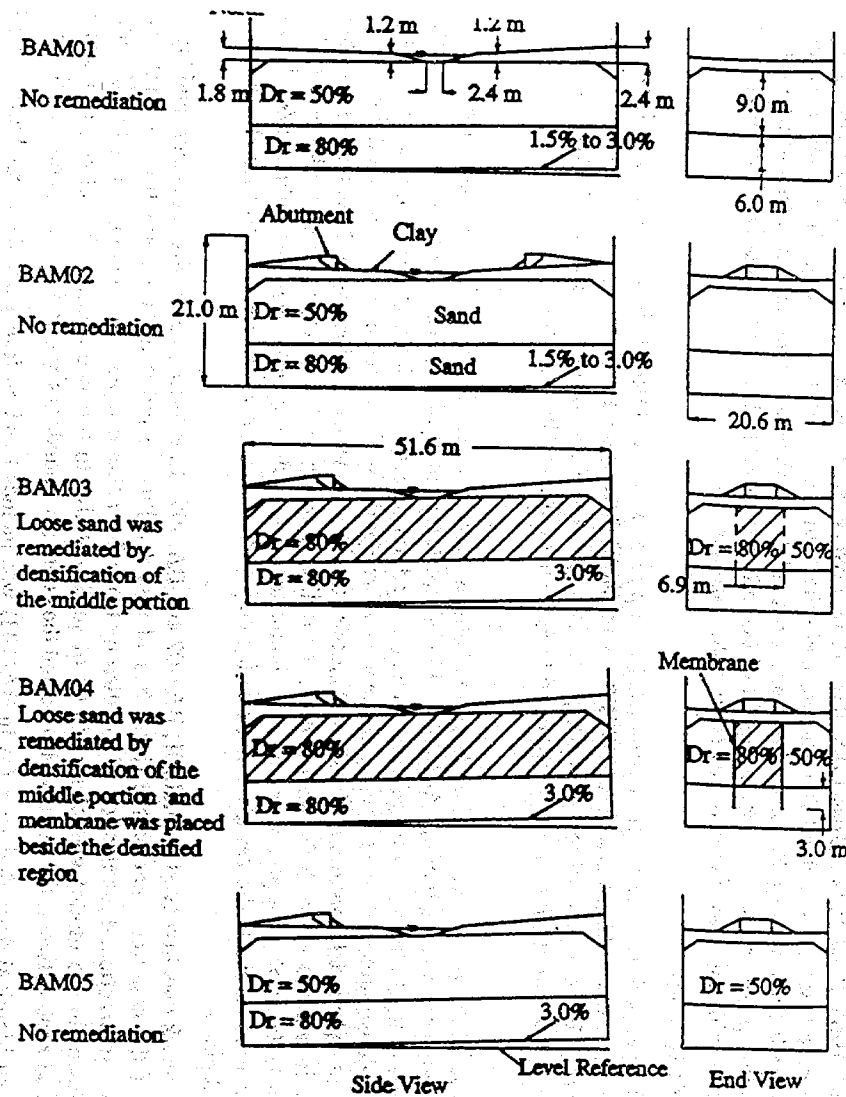


Figure 1: Model Configurations for BAM01 through BAM05 with prototype dimensions.

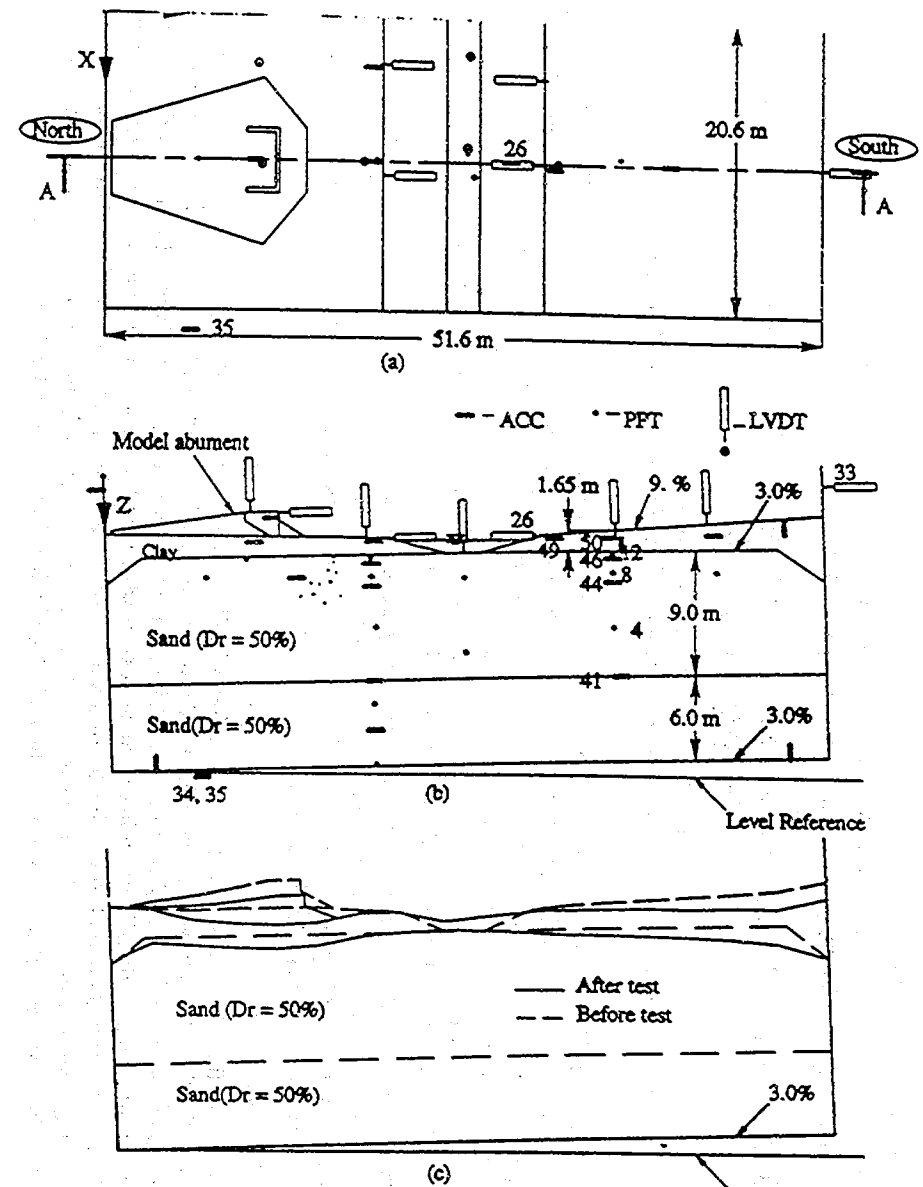


Figure 2: Details of the Model dimensions, instrument locations and deformed shape of BAM05. (a) Plan view, (b) Instrument locations on section A-A, (c) before and after profiles on section A-A.

represent a slurry trench barrier in the prototype; it was envisioned that an impermeable barrier would prevent pore pressures in the unimproved region from migrating into the improved region.

Figure 2 shows details of model BAM05. Sixty three instruments were used in the model. Entry (channel) numbers of selected instruments are shown in Figure 2b. Figure 2c shows soil profiles before and after the test. After the experiment, the model was dissected and it was noted from pre-installed spaghetti noodles and black sand columns that the medium density sand on the south side sheared toward the north side and the clay layer moved (relative to the medium density sand) towards the river channel. Details of model preparation are discussed by Balakrishnan et al. (1998a).

### Soil properties

Nevada sand (uniformly graded,  $D_{50} = 0.17$  mm) was used for the dense and the medium density sand layers. The corrected blow count,  $(N_1)_{60}$  for the medium density sand layer ( $D_r = 50\%$ ) was estimated as 11 from the relationship given by Kulhawy and Mayne (1990). The clay layer was consolidated from remolded San Francisco Bay Mud (LL = 90, PL = 38). The saturated unit weight of the medium density sand and clay are 19.3 and 15.5 kN/m<sup>3</sup> respectively.

### Shaking events

The shaking event sequence for all models (except BAM01) was: Small Step Wave (0.12 g), Small Kobe (0.13 g), Large Kobe (0.7 g), Intermediate Kobe (0.3 g), and finally the Small Step Wave event was repeated. The Large Kobe motion was similar to the motion recorded at 82 m deep at Port Island during the 1995 Hyogoken-Nambu earthquake. Figure 3 shows the acceleration response spectra of the achieved earthquake motions in BAM05. The similarity of base acceleration measured at the east and west sides of the model demonstrate the uniformity of the shaking table motions.

In the intermediate and Large Kobe events pore pressures equal to 100% of the overburden stress were recorded, but in the Small Kobe event about 55% to 85% was recorded in medium density sand layer. Complete sets of time histories for these models are documented in electronic and paper data reports (e.g., Balakrishnan et al. 1997).

### EXPERIMENTAL RESULTS

Figure 4a shows the recorded accelerations during the Large Kobe motion at different depths in BAM05. The base motion had a peak of 0.73 g, but all of the other accelerometers showed acceleration amplitudes greater than 1 g. The pore pressure records, plotted with results of nearby accelerometers, rapidly climb to the initial overburden pressure. Intermittent pulses of negative pore pressure (positive effective stress) coincide with large spikes in acceleration (shear stress). The initial values in the pore pressure records indicate the static pressure.

Figure 4b shows the dynamic displacements obtained by double integration of the digital acceleration records in the frequency domain (Wilson et al. 1998). In this process, a fifth-order high-pass acausal (no phase delay) butterworth filter with a cut off frequency of 0.15 Hz was used. Therefore, permanent displacements cannot be seen in Figure 4b. Prior to digitization, the analog signals were low pass filtered at 27 Hz. Figure 4b shows that the displacement was amplified and the phase was delayed with elevation. The amplification is especially clear for times after 15 seconds.

The influence of soil densification and the in-ground water barrier on settlements, lateral displacements and pore pressure developments was discussed in detail in Balakrishnan et al. (1998a).

However, some of the important observations are covered very briefly below.

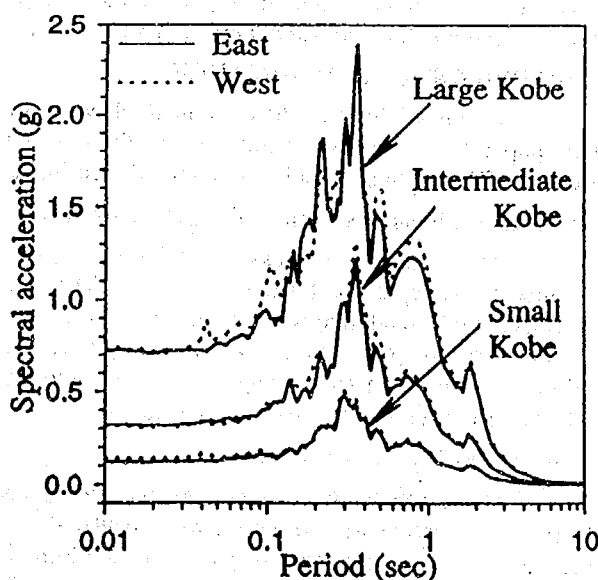


Figure 3: Base acceleration response spectra during the Kobe motions.

## Settlement

The sand surface settlements, summarized in Figure 5, were determined from post-test excavations and measurements after all the shaking events in models BAM01 to BAM05. Except the Small Step Wave motions (which caused immeasurable permanent deformation), the sequence of the earthquake motions was nearly identical for BAM02 through BAM05. From the post-test survey, settlements were measured at four locations indicated in Figure 5.

In Figure 5, settlements from the regions N-C and S-C (treated) in BAM03 and BAM04 show about 40% or more reduction relative to the untreated models BAM02 and BAM05. Comparing the two treated models, the settlements in BAM04 (with the membrane) are about 20% smaller than in BAM03 on the south side, and 40% smaller than BAM03 on the north side (beneath the abutment); perhaps this difference is of practical significance and can be attributed to the effect of the water barrier.

Comparing the cases without remediation, the settlement in BAM01 at S-C was about half the settlement of BAM05. The sand saturation was not checked in BAM01 and it may have been low. Also, the base motions in BAM05 were somewhat greater than those in BAM01 (Balakrishnan et al. 1998a). At the S-W (untreated) zone, the settlement in BAM05 is somewhat greater than in BAM03 and BAM04. The densification of the central portion may have helped to reduce the settlement of the adjacent untreated soil; alternatively this could be explained by random differences in soil density and input motion. These settlement measurements clearly show that the soil densification in a limited region reduces the settlement. Placing an impermeable barrier beside the densified zone appears to enhance the effectiveness of the improvement.

## Lateral displacement

Figure 6 shows vectors of lateral displacement at different locations. These data were obtained after all shaking events by measuring the displacements of the thumb tacks on the clay surface. The lateral sliding on the south side (steeper slope) was much larger than that on the north side, which had a free face at the river channel, but little to no surface slope. Perhaps due to remediation, the sliding in BAM03 was less than that in BAM05. As described in Balakrishnan et al. (1998a), the clay in BAM04 was significantly softer than in BAM03 and BAM05, thus the lateral deformations of the clay layer were about 20% to 40% greater than in the other tests. However, if we compare treated and untreated parts of a particular model we see that the sliding in the treated zone is consistently about 20% less than in the untreated zone. In BAM05, which had no remediation, the lateral sliding in the south side was almost identical in the central and west portions.

On the south side, without an abutment, the surface clay layer apparently tends to bridge across the narrow zone of densified sand; the sliding above the unimproved soil is transferred above the improved soil. This problem could be remedied by additional effort to attach the surface deposit to the improved soil (eg., with piles).

## APPARENT SHEAR RESISTANCE

In this study three different methods were used to estimate or bracket the apparent shear resistance of the soil. This paper focuses on the south side of the model BAM05 because it may be approximated as a two-dimensional problem. The north side included a three-dimensional abutment.

### Static shear stress based on initial geometry of the model

The static shear stress at the interface was calculated by considering the static equilibrium of clay blocks, assumed to be separated by tension cracks. The calculated static shear stress varied between 0.3 and 0.7 kPa at the interface. Additional analyses were carried out for a presumed failure surface parallel to the interface at the depth of 1.5 m from the top of the sand layer (midway between the accelerometers labeled En. 44 and 46 in Figure 2). The estimated static shear stress ranged between 2.2 to 4.5 kPa. As seen in Figure 2c the final ground surface was almost flat. The final static shear was not calculated.

### Shear stress-displacement loops

Using acceleration measurements and a procedure similar to that proposed by Zeghal et al. (1995), dynamic shear stress-displacement response was calculated at two locations of the model for the Large Kobe event (Figure 7a). At the interface, the shear stress was calculated from the acceleration of the clay (En. 50), and relative displacement was calculated by integrating the difference between En. 50 and En. 46. It should be noted that the small static shear stress described above should be added to the calculated dynamic stresses. To estimate the shear strain, the displacement could have been divided by the distance between accelerometers. But, the strains are likely to be non-uniform near the clay-sand interface.

The entire shear stress-displacement history at the clay-sand interface is given in Figure 7a for the Large Kobe event. The peak mobilized shear stress at the interface,  $\tau_{max} = 29$  kPa, corresponds to the largest negative spike in

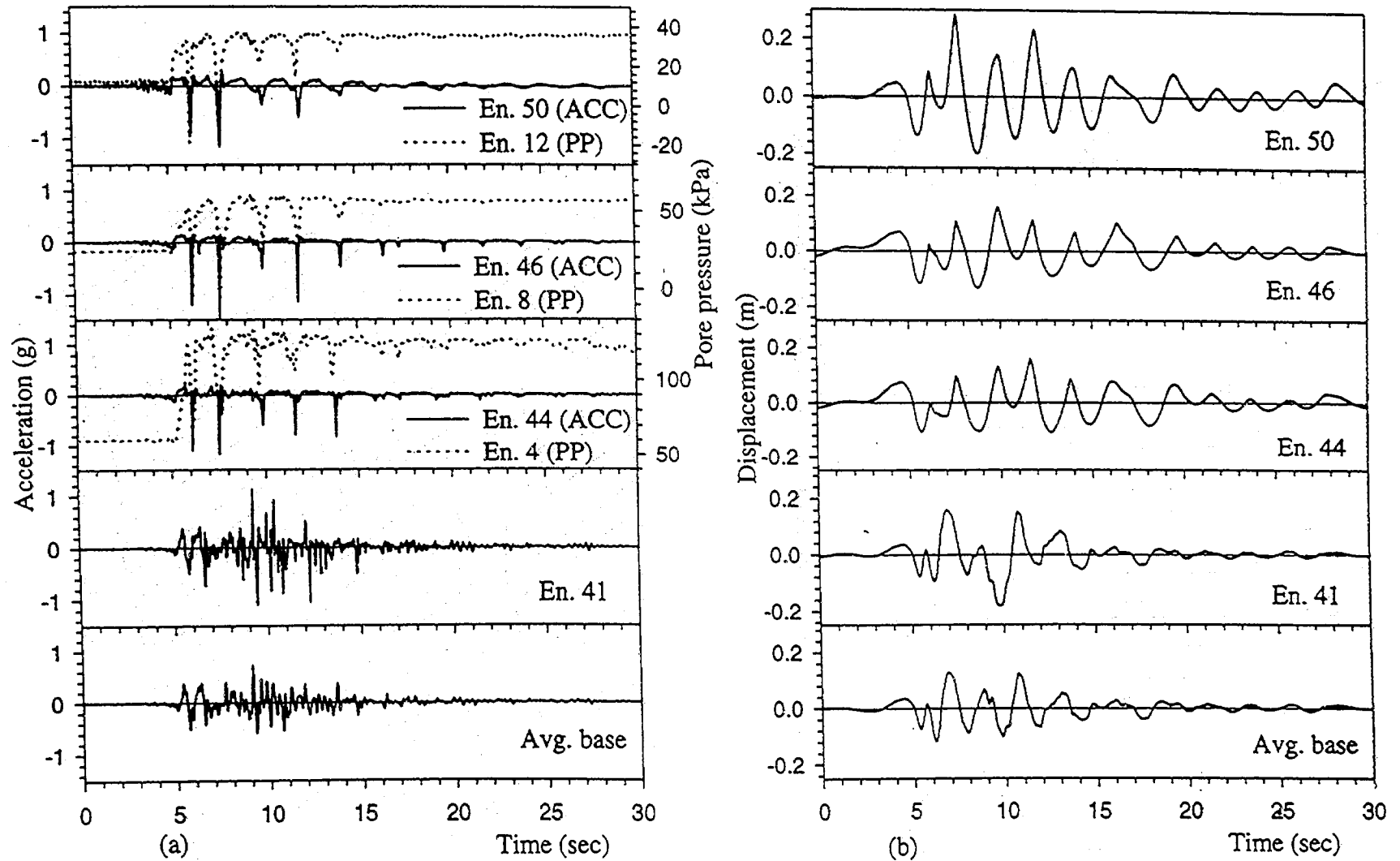


Figure 4: (a) Measured accelerations and pore pressures, and (b) calculated dynamic displacements from accelerations (high pass filtered with  $f_c = 0.15$  Hz) at different depths during the Large Kobe event in BAM05.

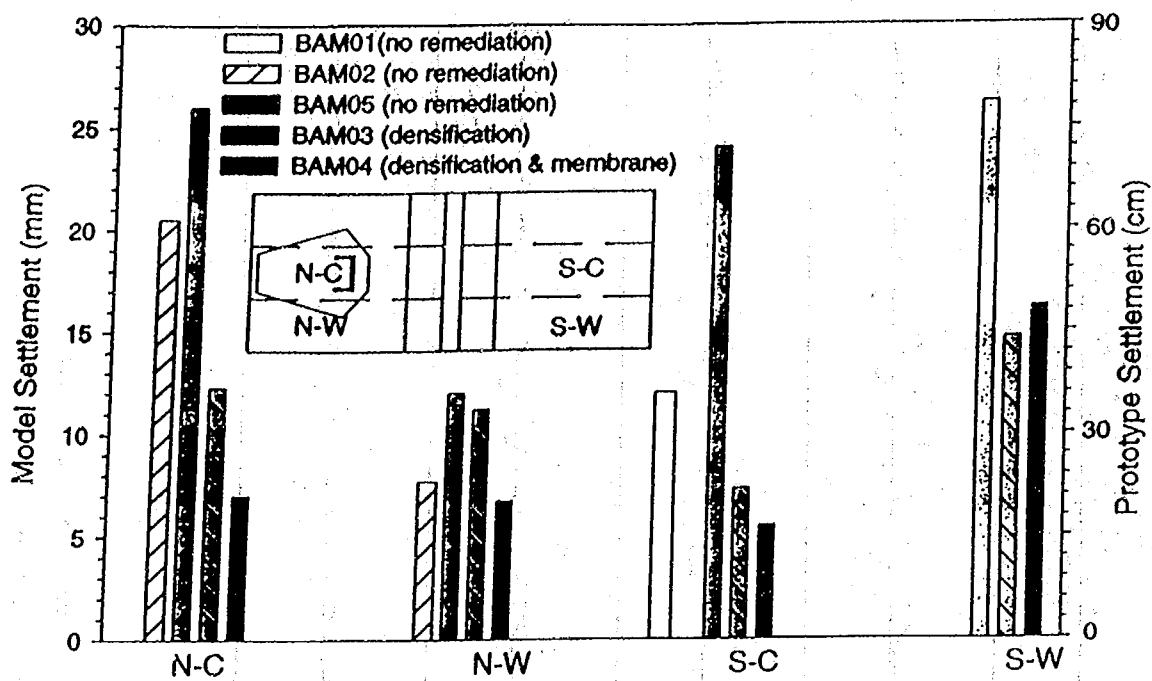


Figure 5: Settlements on sand surface for the models BAM01 through BAM05 at different locations.

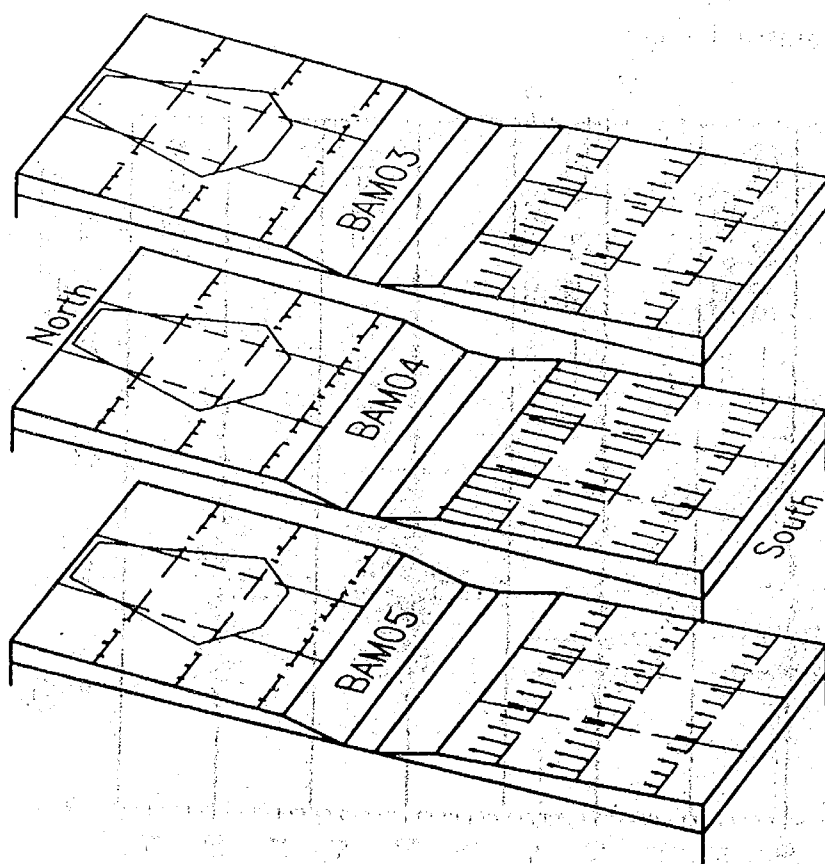


Figure 6: Horizontal displacement vectors (twice the length measured) at the clay surface in models BAM03, BAM04 and BAM05.

acceleration in Figure 4. The initial vertical effective stress,  $\sigma'_{v0} = 16.8$  kPa, indicates  $\tau_{max}/\sigma'_{v0} = 1.73$  which is possible considering the spikes of negative pore pressure shown in Figure 4. The shear stress-displacement history during the Large Kobe event in the sand layer is shown in Figure 7b. The shear stress was obtained from accelerations recorded by En. 50 and En. 46 and displacement was from integration of the difference between En. 46 and En. 44. In the sand,  $\tau_{max}/\sigma'_{v0} = 42/30.7 = 1.37$ .

#### Calculations from sliding block analyses

Newmark type sliding block analyses were used to back-calculate the apparent shear resistance of the liquefied soil in this model. The measured displacement of the clay layer from the LVDT (En. 26) indicates that the clay block moved in both the up-slope and down-slope directions while the permanent displacement was down-slope. This ground oscillation can only be captured if two-way sliding is considered in the analysis. In gently sloping ground, the ground oscillation could cause severe damage to super-structures and should be considered. For the back-calculations, the values of yield acceleration were adjusted by trial and error until good agreement between calculated and measured displacements was obtained. The comparisons between measured and calculated ground oscillation as well as comparisons between measured and calculated permanent displacement were used as criteria for evaluating the goodness of agreement.

As described by Balakrishnan et al. (1998b), fairly involved data processing techniques were required to obtain accurate measurements of dynamic and static clay displacements. Figure 8a shows the resulting measured displacement of clay relative to base. A similar procedure was used to determine the absolute measured displacement of the clay (Figure 8b).

For a two-way sliding block analysis, many combinations of up-slope and down-slope yield accelerations ( $k_{y,up}$  and  $k_{y,down}$ ) can be used to match the measured permanent displacement. For Figure 8, yield accelerations were selected by considering two extreme conditions, one with infinite up-slope shear resistance ( $k_{y,up} = \infty$ ) and the other with zero up-slope shear resistance. For zero up-slope resistance the yield acceleration was assumed equal to the ground slope:  $k_{y,up} = 0.03$ . In Figure 8a, the base acceleration was used as input to the sliding block calculation to match the measured displacement of clay relative to the base. The calculated displacements do not match the measured ground oscillation even with zero up-slope shear resistance. As shown in Figure 4, the dynamic displacement was amplified between the base and the surface. In retrospect, it is obvious that it would be impossible to predict the ground oscillation using a rigid-plastic sliding block analysis without accounting for the observed amplification.

To approximately account for the amplification, the measured acceleration of the clay could be used as input to the sliding block calculation. Figure 8b shows that calculated and measured ground oscillations and permanent displacements are in good agreement if the clay motion is used as input and  $k_{y,up} = 0.03$ . It is somewhat circular, however, to use the measured clay acceleration to predict the clay displacement, because the dynamic component of the measured clay displacement was also obtained from the measured clay acceleration.

It would be better if the acceleration from the top of the sand layer was used as input in an analysis to calculate the clay displacement relative to the top of the sand. Unfortunately, the permanent displacement of the top of the sand could not be measured. However, the measured dynamic displacement of the clay relative to the sand could be obtained from the accelerations (En. 50 - En. 46). Two different cases were considered to determine a range of values of  $k_{y,up}$  and  $k_{y,down}$  that provide reasonable agreement between measured and calculated dynamic displacements. One case (Figure 9a) assumed equivalent shear resistance in up- and down-slope directions, requiring the yield accelerations to differ by twice the slope angle ( $k_{y,up} = k_{y,down} + 2 \times 0.03$ ). The other case (Figure 9b) assumed zero shear resistance in the up-slope direction ( $k_{y,up} = 0.03$ ). With these restrictions, the yield accelerations were adjusted by trial and error until a range of yield accelerations could be identified that produced good agreement between measured and calculated displacements.

Yield accelerations back-calculated from the above mentioned cases were used to estimate the ranges of the apparent shear resistance during the Small, Intermediate, and Large Kobe events, and they are plotted in Figure 10. The apparent shear resistance was obtained by multiplying the yield acceleration ( $k_{y,down}$ ) by the thickness and unit weight of the overlying soil.

#### COMPARISONS OF THE SHEAR RESISTANCE

The stress-displacement histories in Figure 7 indicate that the soil resistance varies with the level of lateral displacement or strain. Most of the ground oscillation occurred at very low resistance and only a small percentage of the ground movement occurred at greater resistance. However, in the sliding block analysis a constant shear resistance for up-slope and down-slope movements was assumed to match the ground oscillation and permanent displacement.

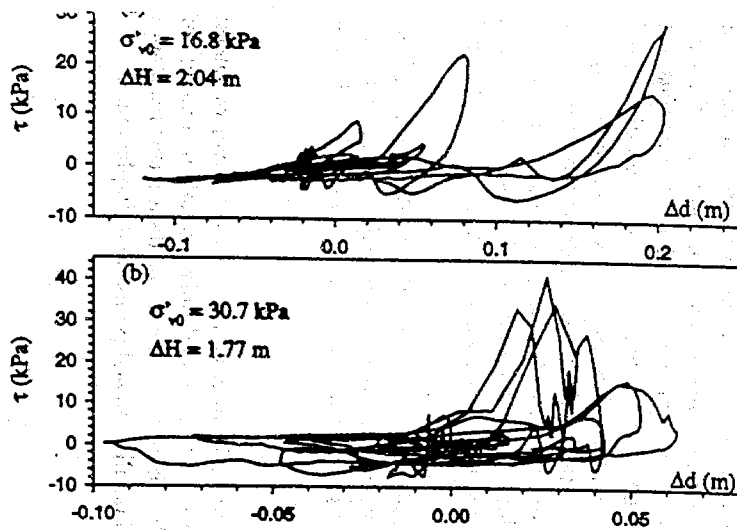


Figure 7: Shear stress-displacement loops during large Kobe event in BAM05 (a) at interface and (b) within sand layer.

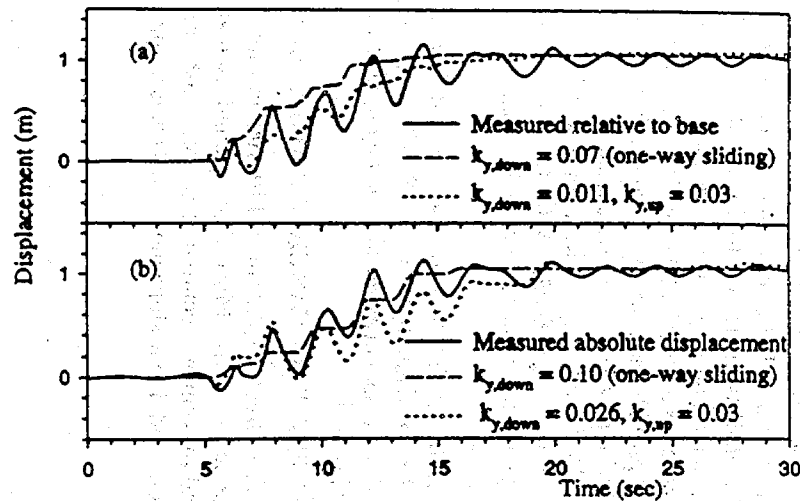


Figure 8: Sliding block analysis to match the permanent displacement of clay relative to the base in BAM05 (a) using the acceleration at the base and (b) using the acceleration in clay.

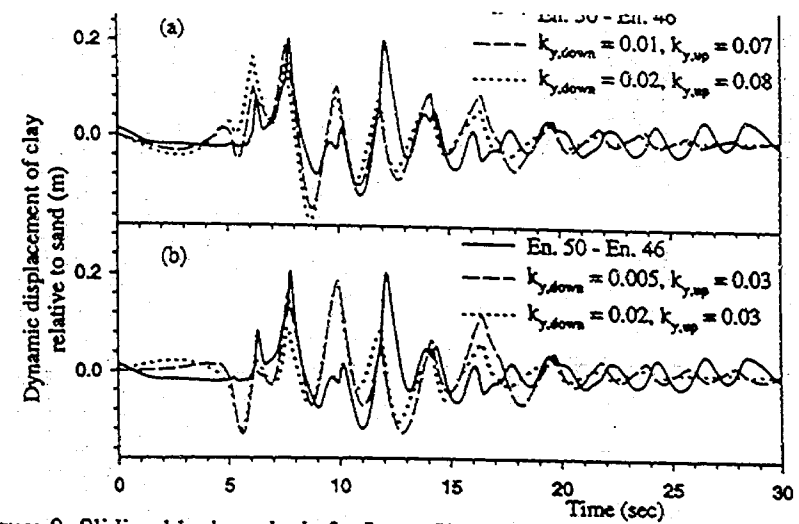


Figure 9: Sliding block analysis for Large Kobe event to match the ground oscillation relative to the sand layer using the acceleration in the sand with (a) equal up-slope and down-slope shear resistance and (b) no up-slope shear resistance.

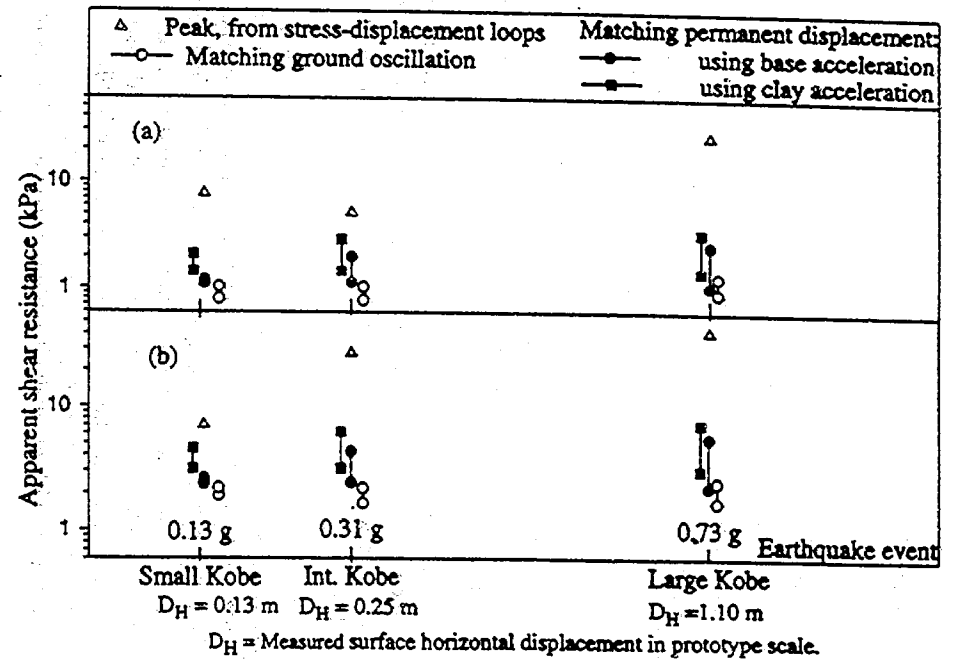


Figure 10: Comparisons of calculated apparent shear resistance from BAM05 (a) at interface and (b) within sand layer.

The range of the apparent shear resistance back-calculated from the sliding block analyses and the peak mobilized shear stress from the stress-displacement loops are plotted in Figure 10. Figure 10a shows the calculated apparent shear resistance at the interface, and Figure 10b shows the calculated apparent shear resistance within the top 1.5 m of the sand layer. Figures 10a and 10b indicate that the apparent shear resistance at the interface was less than that within the sand layer.

In Figure 10, for the apparent shear resistance determined by matching the measured permanent displacement, the plotted range in apparent shear resistance is bounded on the top by the case of one-way sliding, and on the bottom by the case of zero shear resistance to up-slope displacement. As mentioned earlier, in reality, the shear resistance at the sliding surface is not a constant for either up-slope movement or down-slope movement, therefore, exactly matching the ground oscillation using a constant shear resistance in the sliding block analysis is impossible.

The apparent shear resistance determined by matching only the dynamic ground oscillations are smaller than any other back-calculated apparent shear resistance. The peak mobilized shear stress from the stress-displacement loops are consistently larger than the back-calculated apparent shear resistance, and they fall around the range of residual strength of Seed and Harder (1990). The range of residual strength from Seed and Harder (1990) for a soil with an equivalent blow count,  $(N_1)_{60}$  of 11 is 8 kPa to 30 kPa. While the apparent shear resistance implied by Figure 10 may seem small, the normalized apparent shear resistance (divided by the initial effective overburden stress ( $\sigma'_{v0} = 16.8$  kPa at the interface, and 30.7 kPa), falls in a typical range between 0.05 and 0.25, for both the interface and the sand. Based upon matching ground oscillations, the normalized apparent shear resistance is near the bottom of the range 0.05 to 0.1, and based upon one-way sliding block analyses, the normalized apparent shear resistance is 0.1 to 0.25.

## CONCLUDING REMARKS

A series of model tests simulating liquefaction at a hypothetical bridge site has been conducted. Significant liquefaction-induced deformations were observed in the models. Two models included remediation by densification of a limited region of medium density sand; one of these models included an in-ground water barrier to isolate the remediated zone from pore pressures in the medium density soil. The settlements, lateral sliding and pore water pressures in regions of improvement were consistently reduced by the remedial work. Settlements were reduced by a factor of 2 to 4, and lateral sliding was reduced, but with less effectiveness. Based upon these tests it appears that settlements may be controlled by densification of a relatively narrow zone. But, to control lateral sliding, a larger zone may need to be improved. The surface deposit of non-liquefiable soil appeared to bridge across the improved region, transferring lateral movement from the unimproved soil via the surface deposit to the soil above the improved region. In cases like this, it would be important in practice to anchor the non-liquefiable surface deposits to the improved ground (with piles for example) to prevent movement with the unimproved ground. The presented model studies should be augmented by additional tests including different extents and magnitudes of improvement and including different combinations of treatment methods like densification, in-ground walls, and sand drains.

Two-way sliding block analyses were used to calculate the ground oscillation. A model with one-way sliding cannot capture the measured ground oscillations (approximately 0.6 m peak to peak during the Large Kobe event). To a superstructure, such large ground oscillations could be more damaging than the observed 1 m permanent displacement. In order to predict the ground oscillations using a sliding block model, the amplification of the base displacements by the soil deposit must be considered.

The peak mobilized shear stresses, calculated from measured accelerations, are affected by the dilative behavior of the medium density sand layer. The peak stresses coincide with the peak accelerations of the overlying soil which are shown to correspond to pulses of negative pore pressure (Figure 4a). Pulses of negative pore pressure are caused by the tendency for dilation at large strains (Figure 7). In order to predict reasonable permanent displacements, shear resistance much smaller than the observed peak stress must be used in a sliding block calculation.

## ACKNOWLEDGEMENTS

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# REPRODUCTION OF LARGE-SCALE IG TEST ON DRY SAND DEPOSIT AND PILE FOUNDATION USING CENTRIFUGE MODELING

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## 1. INTRODUCTION

Dynamic centrifuge model test technology has recently attracted considerable attention in the field of geotechnical earthquake engineering, mainly because it has the obvious advantage in reproducing the dynamic response of actual soil ground under the conditions of satisfying the similitude requirements concerning confined stress of soil. However, when soil models were prepared for centrifuge tests, it may not always be perfectly consistent with the actual soil ground. It is important to establish test procedures and techniques for reproducing the behavior of actual soil-structures system during earthquakes. Therefore, it is necessary that experimental techniques were improved to settle some problems for the reproduction.

Previously, dynamic centrifuge tests satisfying the similitude requirements were conducted in reference to unprecedented large-scale tests with laminar containers [1, 2] using dry sand [3], saturated sand [4], and a pile foundation in liquefied soil [5], and challenges in reproducing dynamic behavior of actual ground were discussed. In this paper, a test with reference to the past studies is reported to reproduce the dynamic behavior of a pile foundation structure and dry sand deposit.

## 2. LARGE-SCALE SHAKING TABLE TEST

### (1) Test apparatus

A shaking table test using a large laminar container was conducted in National Research Institute for Earth Science and Disaster Prevention. The table has size of 15.0 m x 14.5 m and its maximum payload is 500 t.

### (2) Laminar container

The laminar container has inner dimensions of 11.6 m in length, 6 m in height, and 3.1 m in width. The container consists of 29 rectangular frames made of H-shaped steel members (200 mm x 200 mm). The inner face of the

container was lined with a 5 mm thick rubber membrane to provide waterproofing and to protect rollers from the soil. Steel pipes with an outer diameter of 200 mm as rollers and Teflon sheets were placed between the each container frame to reduce the shear stiffness of the container.

### (3) Preparation ground

A large-scale shaking test model and locations of transducers is shown in Fig. 1. The soil material is  $\rho_{\max} = 1.74 \text{ t/m}^3$ ,  $\rho_{\min} = 1.39 \text{ t/m}^3$ ,  $D_{50} = 0.27 \text{ mm}$ ,  $U_c = 3.4$ ,  $F_c = 3\%$ , the river sand collected from Hokota, Ibaraki Prefecture. After dried naturally, the sand was conditioned to be slightly wet. The test sand was poured through a funnel from a height of 1 m. The sand deposit had an average density  $\rho = 1.69 \text{ t/m}^3$  and a water content  $w = 11\%$  at the beginning of the test.

### (4) Pile foundation structure

The foundation is a rigid structure, consists of stacked rectangular steel plates, which has a length of 3.0 m, a width of 2.5 m, a height of 0.26 m, and a weight of 15.6 t. The piles are used four (2x2) actual steel piles with an outer diameter of 318.5 mm, a thickness of 6.9 mm, and a length of 6 m with the center-to-center spacing 6 times of the diameter. The top of each pile was firmly fixed to the structure, and the bottom of each pile was pin-connected to the container base. Accelerometers were buried in the sand deposit, and strain gauges were attached to the steel pile surfaces to measure the bending strain.

### (5) Excitation

Excitation cases of the large-scale shaking tests are described in reference [6]. In this paper, the test result of the following conditions is reported: a structure weight of 15.6 t, a sinusoidal wave with incremental acceleration, an excitation frequency of 3 Hz, and a maximum acceleration of 600 gal.

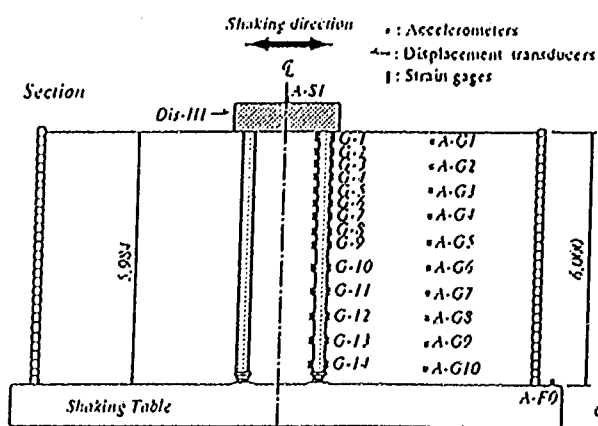


Fig. 1 A large-scale shaking test model and locations of transducers

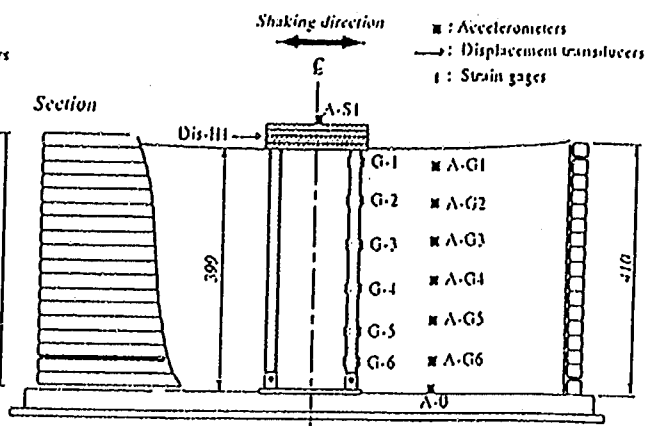


Fig. 2 A the centrifuge shaking test model and locations of transducers

Table 1 Similitude requirements used in the experiment

|                       |                               | Symbol   | Scale ratio   | Unit              | 1g      | Centrifuge             |
|-----------------------|-------------------------------|----------|---------------|-------------------|---------|------------------------|
| Sand                  | Thickness                     | $H_s$    | $1/\lambda$   | m                 | 5.984   | 0.399                  |
|                       | Density                       | $\rho_s$ | 1             | g/cm <sup>3</sup> | 1.69    | 1.69                   |
| Pile                  | Length of pile                | L        | $1/\lambda$   | m                 | 6.0     | 0.4                    |
|                       | Diameter                      | D        | $1/\lambda$   | mm                | 318.5   | 20(21.4)               |
|                       | Thickness                     | t        | $1/\lambda$   | mm                | 6.9     | 0.5(0.46)              |
|                       | Young's modulus               | E        | 1             | MN/m <sup>2</sup> | 206,000 | 206,000                |
|                       | Geometrical moment of inertia | I        | $1/\lambda^4$ | cm <sup>4</sup>   | 8,200   | 0.162                  |
|                       | Bending stiffness             | E·I      | $1/\lambda^4$ | MN·m <sup>2</sup> | 16.9    | 0.000300<br>(0.000334) |
|                       | Area                          | A        | $1/\lambda^2$ | cm <sup>2</sup>   | 67.55   | 0.300                  |
|                       | Normal stiffness              | E·A      | $1/\lambda^2$ | MN                | 1,392   | 6.18                   |
|                       | Mass                          | $m_r$    | $1/\lambda^3$ | kg                | 15,600  | 4.62                   |
|                       | Length                        | $L_r$    | $1/\lambda$   | m                 | 2.5     | 0.167                  |
| Exciting acceleration |                               | $\alpha$ | $\lambda$     | g                 | 0.4     | 6.1                    |

$$1/\lambda = \text{model} / \text{prototype} = 1/15$$

### 3. CENTRIFUGE SHAKING TABLE TEST

#### (1) Test apparatus

Model tests were performed using a dynamic centrifuge at the Institute of Technology, Shimizu Corporation. The specifications of the centrifuge have been described in details by Sato [7]. The arm radius of the centrifuge to the shaking table is 3.11 m, and is equipped with a shaking table of 950 mm x 650 mm. The table accommodates a maximum payload 300 kg at centrifugal acceleration of 50 g. The maximum acceleration of excitation is 5 g when the frequency of sinusoidal vibration is continuously changed in the frequency range of 50-350 Hz, and 10 g in the case of arbitrary waveform, such as a seismic wave. Excitation is driven by an electromagnetic system so that seismic waves can be precisely reproduced by the stable control of waveform even in a wide frequency range.

#### (2) Laminar container

The laminar container, which is geometrically similar to the one used in the large-scale shaking test, has inner dimensions of 80 cm in length, 41 cm in height, and 23 cm in width. The container consists of 18 stacked frames of rectangular hollow steel pipes with 20 mm in height, 40 mm in width, and 1.2 mm in thickness. The inner face of the container was lined with a 0.5 mm thick rubber membrane to provide waterproofing and to protect the bearings from the soil. The inertia effect of the container frames was minimized because of the light weight of the frames. Flat bearings of 2 mm in bearing thickness were installed between the rectangular frames to reduce the shear stiffness of the container. Acrylic plates were installed between the container frames to prevent the insertion of the membrane into the gaps between the each frames.

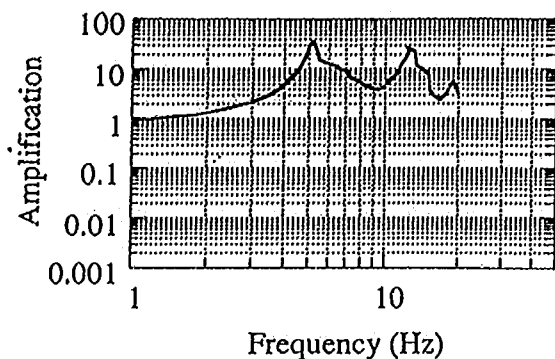


Fig. 3 Frequency transfer function of soil in the large-scale shaking test

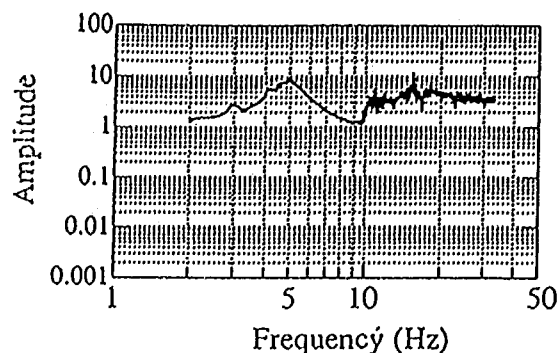


Fig. 4 Frequency transfer function of soil in the centrifuge shaking test

### (3) Similitude requirements

The similitude requirements used in the centrifuge test is summarized in Table 1. The ratio of the centrifuge test and the large-scale test is 1/15. The value of centrifugal acceleration was set to 15 g. In the time history and strain diagrams shown in this paper, the values of the centrifuge test were translated to the actual size based on the similitude requirements.

### (4) Preparation model ground

Some informations was previously known as need data to execute centrifuge experiment: reproducing the large-scale test results, which were the soil density, the natural frequency of the ground, the specifications of the pile foundation, and the input wave of the large-scale shaking test. So, the centrifuge test was conducted as a forecast experiment of actual sand deposit.

The centrifuge shaking test model and locations of transducers is shown in Fig. 2. The same sand as that used in the large-scale test, and the model was used with the same preparation method. The natural frequency of the model ground in the centrifuge test was made equal to that in the large-scale test. Although it was reported in a previous study [3] that the behavior of dry sand deposit could roughly be reproduced by using the same average soil density for large-scale and centrifuge tests, there is another concern that response behavior of sand deposit may be affected by inhomogeneity. It was therefore considered that the use of sand deposit having the same elastic response behavior is more appropriate to investigate overall behavior of an actual sand deposit.

A soil model with a slightly lower density than that of the large-scale test was prepared. The natural frequency was investigated by swept sinusoidal vibration with a

small acceleration of 0.1 g and a frequency range of 30-150 Hz, corresponding to 6 gal and 2-10 Hz under the prototype. Through vibrating compaction, a target value of 75 Hz, corresponding to 5 Hz under the prototype, was attained as the natural frequency of the sand deposit.

### (5) Pile foundation structure

The model iron piles satisfied the similitude requirements regarding the bending stiffness were adopted. Each pile has a diameter of 20 mm, a wall thickness of 0.5 mm, and a length of 40 cm. The same conditions as those in the large-scale test were adopted regarding the number of piles, pile arrangement, and the fixed method of the pile top and bottom.

### (6) Excitation

The input wave was reduced the time scale of that used in the large-scale test to 1/15. Although the input wave-forms of both tests were nearly the same, the maximum acceleration in the centrifuge test resulted in a smaller value of 400 gal.

## 4. COMPARISON OF EXPERIMENTAL RESULTS

### (1) Natural frequency of sand deposit

Microtremors were measured to investigate characteristics of sand deposit before the start of the large-scale shaking test, and the natural frequency was found to be 5 Hz as shown in Fig. 3.

Because microtremors could not be measured in the centrifuge test, a swept vibration test with a very small level of excitation was conducted. As stated in (4) of 3 and shown in Fig. 4, the natural frequencies of both sand deposits were in good agreement.

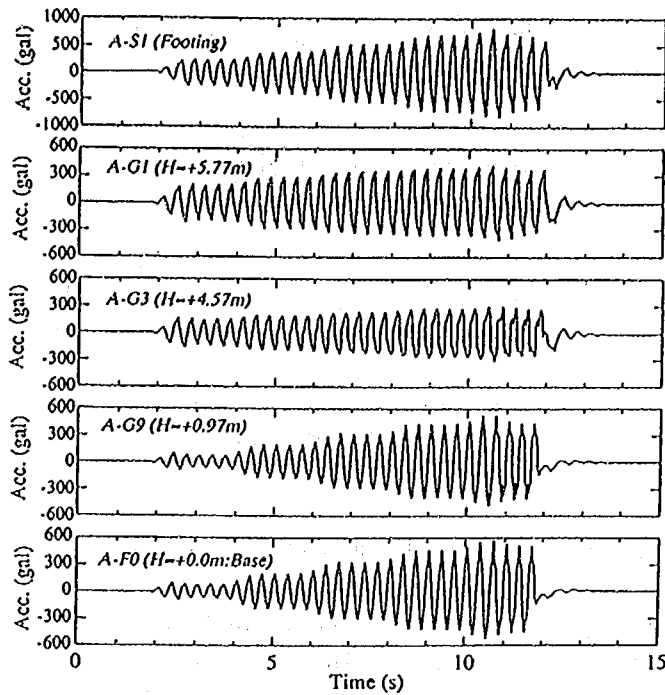


Fig. 5 Time history of the acceleration of soil in the large-scale shaking test

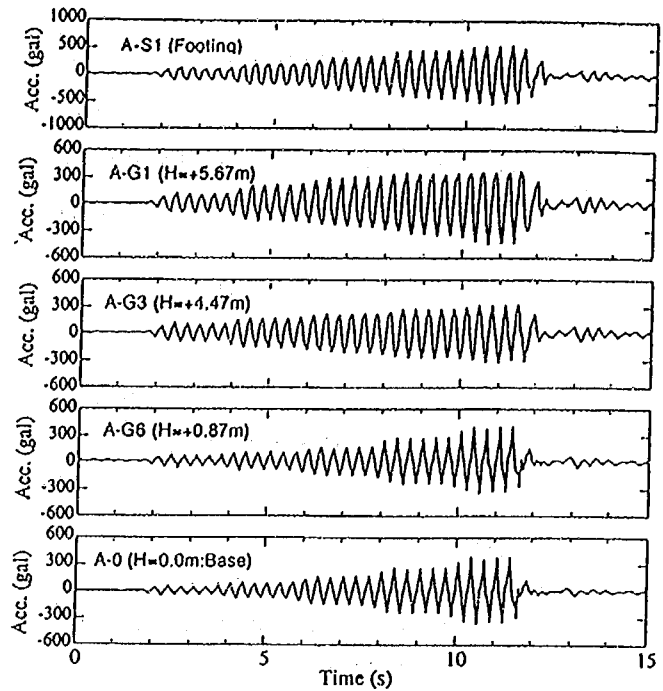


Fig. 6 Time history of the acceleration of soil in the centrifuge shaking test

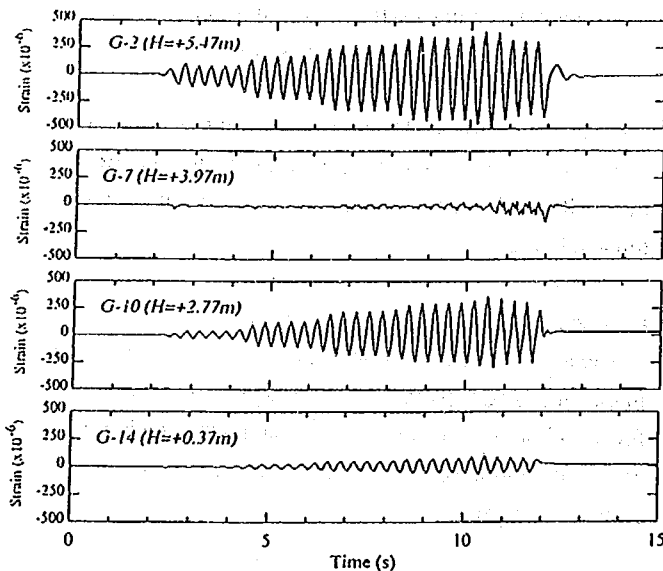


Fig. 7 Time history of the bending strain of piles in the large-scale shaking test

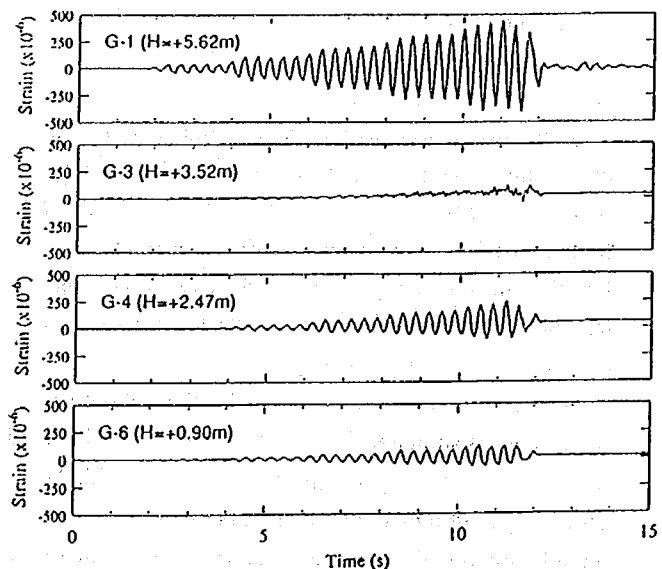


Fig. 8 Time history of the bending strain of piles in the centrifuge shaking test

## (2) Acceleration response of the soil and the pile foundation structure

The time history of acceleration in the large-scale shaking test is shown in Fig. 5, and that in the centrifuge shaking test is shown in Fig. 6. The centrifuge test resulted in a lower response acceleration at the footing because of the lower input acceleration of the shaking table. The two tests resulted in similar soil response acceleration even though the input waves had different

amplitudes. A trend of decreased response at the intermediate height of about 4.5 m was observed in both tests. The similarity of response accelerations in both tests in spite of different input imply which the degree of soil nonlinearity in the large-scale test was greater than that in the centrifuge test.

## (3) Bending strain of piles

The time histories of the bending strain of piles in the

# SESSION 5

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- *S. Yasuda*: Earthquake - Induced Settlement of an Isolated Footing for Power Transmission Tower
- *H. Kim*: Deformation Mechanism of Composite Breakwaters under Earthquake Shaking

## Earthquake-induced settlement of an isolated footing for power transmission tower

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**ABSTRACT:** Parameters which influence the settlement of an isolated footing for a power transmission tower are evaluated. A series of dynamic centrifuge tests and tests for obtaining fundamental properties of Toyoura sand saturated with different pore fluids (water, 50 cSt silicon oil) are carried out. From the test results, it can be said that the settlement of the footing varies in direct proportion to the amplitude of input acceleration, carrying load, thickness of liquefiable layer and cycles of input motion and in inverse proportion to relative density, thickness of unliquefiable layer, grain size and footing width. The predicted settlement is expressed in a conventional form by use of deviating factors relating to above-mentioned parameters.

### 1 INTRODUCTION

The influence of soil liquefaction on various facilities on the soft ground has been investigated by many researchers. Of these facilities, a power transmission tower is selected for this study. As the practical target of the study, an isolated footing of four-isolated-footing type foundation for a power transmission tower (shown in Figure 1) is taken into consideration. Evaluation of differential settlements among isolated footings is a focus of this study. From an investigation of power transmission towers damaged during several past great earthquakes, it is found that the maximum relative settlement of a footing of a foundation comprising four-isolated footings is about 50cm in order to repair the tower. In other words, the allowable differential settlement is equivalent to an angle of rotation of  $1/12$ , which is much larger than  $1/1200$  that is employed in practical design.

Accurate prediction of the relative settlement due to liquefaction may be necessary for evaluating the performance of the structures and countermeasures against liquefaction.

The purpose of the study is to evaluate parameters affecting the total settlement and to offer data that may be helpful for considering the mechanism of the settlement of the foundations due to liquefaction through centrifuge model tests.

### 2 TEST PROGRAM

The following are considered to be parameters influencing the settlement due to liquefaction.

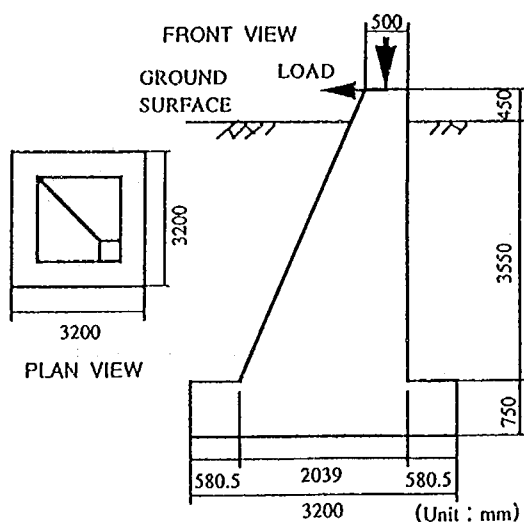


Figure 1 Target isolated footing

(1) Load. In the practical design of a tower for power transmission, wind load, the dead weights of a tower, insulators and electric wires, and the tensile force of electric wires, etc. are taken into account. Table 1 shows practical design loads and simplified loads for the tests.

(2) Relative density of the ground. The relative density is set at either 20, 40 or 60%. Most of the tests are performed at 40% relative density. For an unliquefiable layer, sand with 80% relative density is used.

(3) Thickness of liquefiable layer. The thickness of liquefiable layer, the distance from the bottom of the footing to the pedestal of the container, is 13.2m for the standard case. Thickness of 7m and 3m are also tested.

Table 1 Design Loads and simplified loads

| Practical design loads  |                     |                  |                                        |                                    | Simplified loads in the tests |                     |                       |
|-------------------------|---------------------|------------------|----------------------------------------|------------------------------------|-------------------------------|---------------------|-----------------------|
| Deviation angle of line | Wind Velocity (m/s) | Temperature (°C) | Compression Vertical (Horizontal) (tf) | Tension Vertical (Horizontal) (tf) | No                            | Vertical force (tf) | Horizontal Force (tf) |
| 11° 46'                 | 15                  | 15               | 37.51 (3.34)                           | 21.60 (3.34)                       | 1                             | 30                  | 0                     |
|                         | 0                   | 15               | 31.24 (2.48)                           | 15.33 (3.34)                       | 2                             | 40                  | 0                     |
| 0°                      | 15                  | 15               | 16.28 (0.86)                           | 0.37 (3.34)                        | 3                             | 10                  | 0                     |
|                         | 0                   | 15               | 10.01 (0)                              | —                                  | 4                             | -15 (Tensile)       | 2.5                   |

(4) Thickness of unliquefiable layer. As the existence of an unliquefiable layer restrains the settlement due to liquefaction, two cases were added in the tests in which a compacted layer ( $D_r = 80\%$ ) exists beneath the footing. The thickness of the unliquefiable layer is either 2m or 5m.

(5) Grain size. In addition to Toyoura sand, two kinds of silica sands, #5 and #8, are used for the ground material. In those cases, silicon oils with different viscosity are used so that each apparent coefficient of permeability coincides with the permeability of Toyoura sand.

(6) Input motion. A sinusoidal wave at a frequency of 1Hz (50Hz at 50g) is used for the tests. To vary the intensity of liquefaction, motions with different amplitudes of acceleration or different numbers of cycles were applied.

(7) Width of footing. The width of the footing is 3.2m, except two cases which use footings 2.5m-wide and 4.0m-wide under loads of 18.3tf and 46.9tf, respectively. Each load is determined so as to have the same intensity as the case of a 3.2m-wide footing under 30tf.

(8) Pore fluid. From the law of similarity, silicon oil at 50 cSt dynamic coefficient of viscosity should be used under 50g centrifugal field. One test with water as pore fluid is added to investigate the difference between the fluid materials (water and 50 cSt silicon oil).

### 3 TEST PROCEDURES

#### 3.1 Laboratory tests

##### (1) Tests for sand properties

General laboratory tests such as sieve analysis, specific gravity tests, maximum and minimum density tests were carried out according to the standard of Japanese Geotechnical Society.

##### (2) Permeability tests

A falling head permeability test was conducted at 50g.

##### (3) Cyclic triaxial tests

As silicon oil is used for pore fluid instead of water in the centrifuge tests, cyclic triaxial tests with 50cSt silicon oil are carried out.

The test conditions are as follows:

Input motion: sinusoidal wave with 0.1Hz

Back pressure:  $2\text{kgf/cm}^2$

Initial effective vertical stress:  $\sigma'_{v0} = 0.5\text{kgf/cm}^2$

Confining pressure: alternating

#### 3.2 Centrifuge model tests

All the centrifuge tests are performed at 50g centrifugal acceleration field. Transducers are installed during the model preparation, at locations shown in Figure 2.

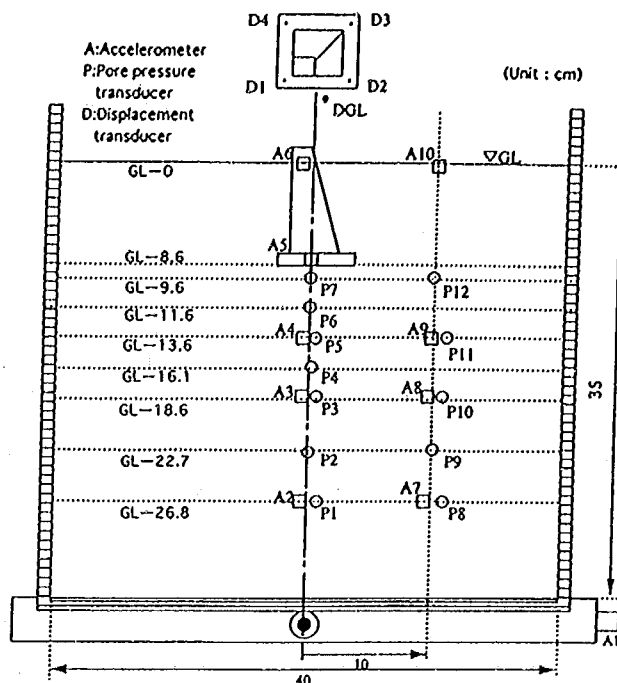


Figure 2 Typical location of transducers

Table 2 Summary of test conditions

| Phase | No  | Factors related to ground, input motion |               |                      |              | Factors related to footing |                      | Remarks                |
|-------|-----|-----------------------------------------|---------------|----------------------|--------------|----------------------------|----------------------|------------------------|
|       |     | Layer(m)                                |               | Relative density (%) | Input motion | Load (tf)                  | Width of footing (m) |                        |
|       |     | Liquefiable                             | Unliquefiable |                      |              |                            |                      |                        |
| I     | 1   |                                         |               |                      |              | 30                         |                      |                        |
|       | 2   |                                         |               |                      |              | —                          |                      | Without footing        |
|       | 3   | 13.2                                    | 0             | 40                   | 60           | -15                        |                      | Horizontal Load: 2.5tf |
|       | 4   |                                         |               |                      |              | 10                         |                      |                        |
|       | 5   |                                         |               |                      |              |                            |                      | Standard case          |
|       | 6   | 8.2                                     | 5             |                      |              | 30                         | 3.2                  |                        |
|       | 7   | 11.2                                    | 2             |                      |              |                            |                      |                        |
|       | 8   |                                         |               | 20                   | 90           |                            |                      | Loose                  |
|       | 9   |                                         |               |                      |              | 40                         |                      |                        |
|       | 10  | 13.2                                    | 0             | 40                   | 160          | 30                         |                      |                        |
|       | 11  |                                         |               |                      | 220          |                            |                      |                        |
|       | 12  |                                         |               |                      | 240          | 0                          |                      |                        |
| II    | 1   |                                         |               |                      |              | —                          | —                    | Without footing        |
|       | 2   |                                         |               |                      |              | 30                         | 3.2                  | Standard case          |
|       | 3   |                                         |               |                      |              | 18.3                       | 2.5                  | Same load intensity    |
|       | 4   |                                         |               | 40                   | 90           | 46.9                       | 4.0                  | Same load intensity    |
|       | 5   | 13.2                                    | 0             |                      |              | 10                         |                      |                        |
|       | 6   |                                         |               |                      |              | 30                         |                      |                        |
|       | 7-1 |                                         |               | 60                   |              | 30                         | 3.2                  | Dense                  |
|       | 7-2 |                                         |               |                      | 160          | 20                         |                      | Dense                  |
| 8     |     |                                         | 40            | 90                   |              |                            | Pore fluid: water    |                        |
| III   | 1   | 3                                       |               | 40                   |              |                            |                      |                        |
|       | 2   | 7                                       |               |                      |              |                            |                      |                        |
|       | 3   |                                         | 0             | (41.8)               | 90           | 20                         | 3.2                  | #5 Silica sand         |
|       | 4   | 13.2                                    |               | (42.2)               |              |                            |                      | #8 Silica sand         |

Data are stored in two ways : one is a short period data processing by the computer and the other is a long period data processing by the data recorder. The latter one is adopted for the pore pressure and the settlement time histories. Twenty five centrifuge tests were performed as shown in Table 2.

## 4 TEST RESULTS AND INTERPRETATION

### 4.1 Laboratory tests

#### (1) Tests for sand properties

The results of the laboratory tests are summarized in Table 3.

Table 3 Properties of sands

| Material                           | Toyoura Sand | #5 Silica Sand | #8 Silica Sand |
|------------------------------------|--------------|----------------|----------------|
| $\rho_s$ (g/cm <sup>3</sup> )      | 2.665        | 2.660          | 2.646          |
| $\rho_{dmax}$ (g/cm <sup>3</sup> ) | 1.635        | 1.746          | 1.520          |
| $\rho_{dmin}$ (g/cm <sup>3</sup> ) | 1.335        | 1.468          | 1.109          |
| $D_{50}$ (mm)                      | 0.161        | 0.420          | 0.096          |

#### (2) Permeability tests

The coefficient of permeability of the test ground is on the order of  $6.5 \times 10^{-3}$  cm/s under both water (1cSt) at gravitational force field and 50 cSt silicon oil at 50g centrifugal acceleration field. Therefore, the time relevant to dissipation in the model is 1/50 of that in the prototype, which is in agreement with the time relevant to dynamic behavior.

#### (3) Cyclic triaxial tests

The relationships between cyclic stress ratio and number of cycles to liquefaction are shown in Figure 3. The relationship is hardly affected by the difference of pore fluid, water or silicon oil, under a loading frequency of 0.1Hz.

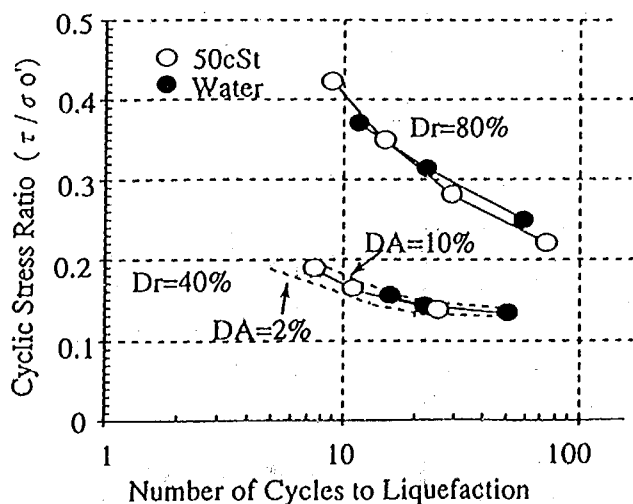


Figure 3 Results of cyclic triaxial tests

### 4.2 Dynamic centrifuge model tests

(1) Response acceleration The response acceleration time histories at various locations at Test II-2 are shown in Figure 4. They are arranged according to depth, from the base motion A1 to the motion at the ground surface A10 or that at the top of the footing A6. At 13.4m depth(A2, A7), liquefaction occurs during the first or second cycle of the loading. As a result, response accelerations at the shallower depths are not amplified. As will be mentioned later, this is one of the reasons why the settlement does not advance in proportion to the amplitude of input acceleration.

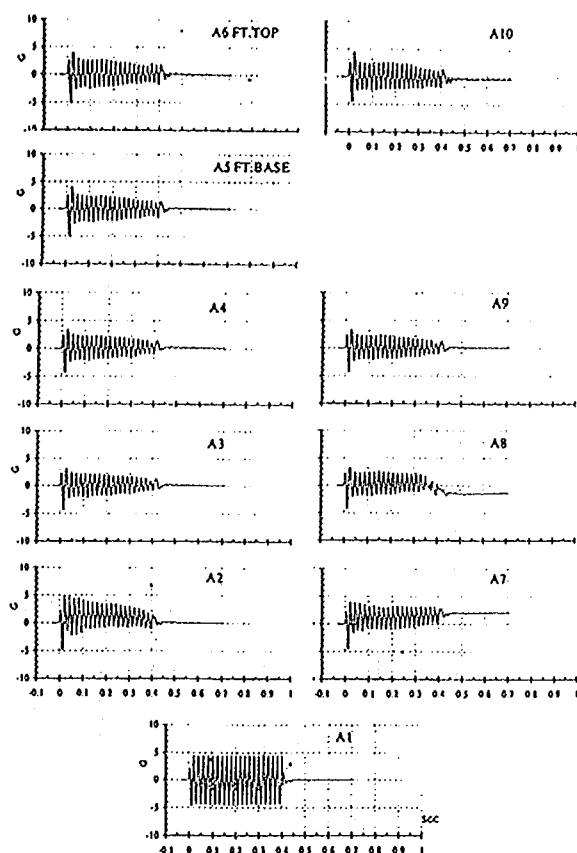


Figure 4 Acceleration time histories (Test II-2)

#### (2) Generation and dissipation of excess pore pressure.

The generation and dissipation of excess pore pressure at Test II-2 are shown in Figure 5. At deeper depths (P1, P2, P8, P9, P10), liquefaction occurs during the shaking, but the excess pore pressure ratios do not reach 100% at most shallower depths. Because of the higher effective stress due to the weight of footing and the load acting on the superstructure, excess pore pressure generation under the footing is less than that at a distance of 10cm from the footing center. After shaking, the excess pore pressure propagates upward. As a result, the secondary liquefaction occurs in the region near the

surface, except in regions whose stress state is affected by the existence of the footing. Excess pore pressure begins to decrease at deeper depth first, then it propagates upward. At the deepest two depths, the dissipation starts just after the end of shaking.

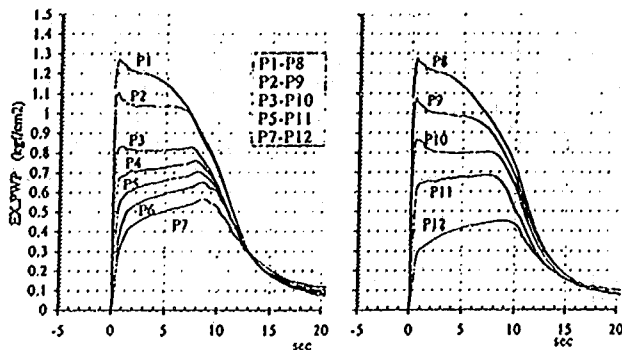


Figure 5 Excess pore pressure generation (Test II-2)

### (3) Settlement of the footing and ground surface

The main purpose of this study is to get information about the parameters which affect the settlement of the footing due to soil liquefaction. The results related to settlement are summarized in Table 4. From the tables, it is noticed that the settlements of both the footing and the ground surface mainly occur after the shaking, but not during the period of shaking.

#### • Effect of load

As shown in Figure 6, the settlements of ground surface both during and after the shaking are almost independent on the load applied. The final footing settlements of the two cases, -15tf and 10tf, are almost the same. At over 10tf of applied load, however, they have a tendency to increase in proportion to the applied load. This may be because the load intensity exceeds the residual strength of liquefied soil.

#### • Effect of relative density

As shown in Figure 7, the relative density influences the settlement of both the ground and the footings strongly, especially on the final settlement of both the footing and the ground surface.

- Surface Settlement (during shaking)
- -○- - Surface Settlement (final)
- Footing Settlement (during shaking)
- -■- - Footing Settlement (final)

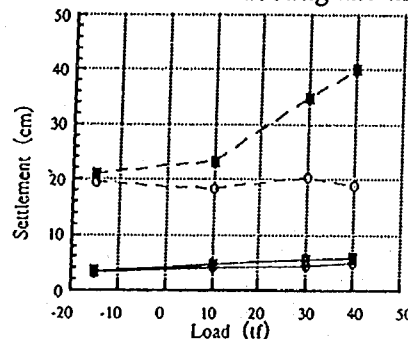


Figure 6 Effect of load

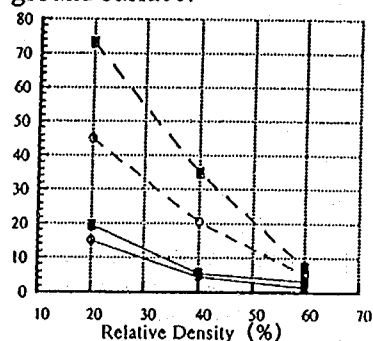


Figure 7 Effect of relative density

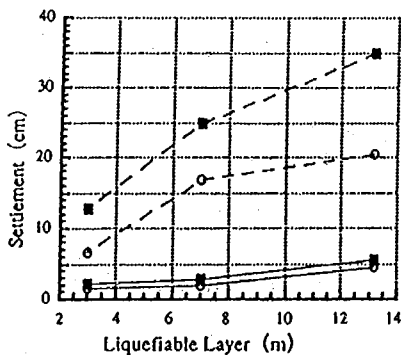


Figure 8 Effect of liquefiable layer

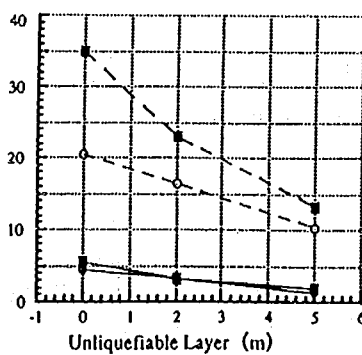


Figure 9 Effect of unliquefiable layer

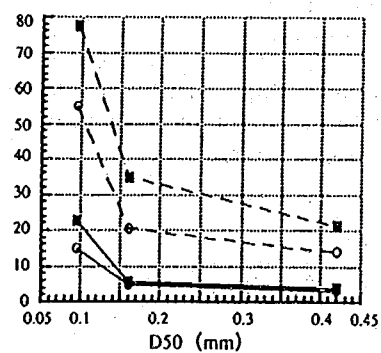


Figure 10 Effect of grain size

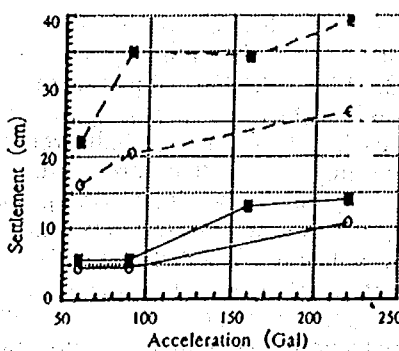


Figure 11 Effect of amplitude of acceleration

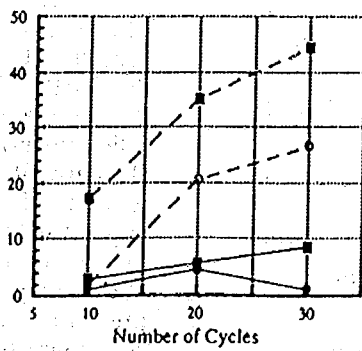


Figure 12 Effect of number of cycles

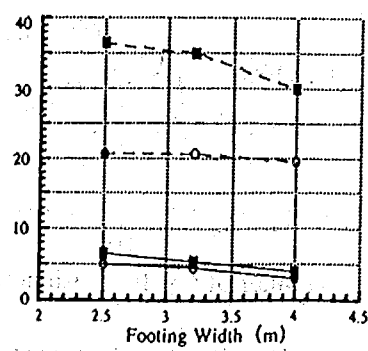


Figure 13 Effect of the footing width

Table 4 Summary of settlements

| Test No | Peak Acc. on the surface (Gal) | Observed settlement in the experiment (mm) |       |                |       | Converted prototype settlement (cm) |       |                |       |
|---------|--------------------------------|--------------------------------------------|-------|----------------|-------|-------------------------------------|-------|----------------|-------|
|         |                                | Surface                                    |       | Footing        |       | Surface                             |       | Footing        |       |
|         |                                | During shaking                             | Final | During shaking | Final | During shaking                      | Final | During shaking | Final |
| I-1     | 136                            | 0.9                                        | 3.2   | 1.1            | 4.4   | 4.5                                 | 15.8  | 5.5            | 22.0  |
| I-2     | 156                            | 1.0                                        | 4.1   | —              | —     | 5.0                                 | 20.5  | —              | —     |
| I-3     | 132                            | 0.6                                        | 3.9   | 0.7            | 4.3   | 3.3                                 | 19.6  | 3.5            | 21.0  |
| I-4     | 150                            | 0.8                                        | 3.7   | 0.9            | 4.7   | 4.1                                 | 18.3  | 4.5            | 23.4  |
| I-5     | 120                            | —                                          | —     | 1.0            | 6.2   | —                                   | —     | 5.0            | 30.8  |
| I-6     | 162                            | 0.3                                        | 2.1   | 0.4            | 2.6   | 1.7                                 | 10.5  | 2.0            | 13.2  |
| I-7     | 140                            | 0.6                                        | 3.3   | 0.7            | 4.6   | 3.2                                 | 16.4  | 3.3            | 22.9  |
| I-8     | 134                            | 3.1                                        | 9.0   | 4.0            | 14.6  | 15.3                                | 45.0  | 36.0           | 73.0  |
| I-9     | 184                            | 1.0                                        | 3.8   | 1.2            | 8.0   | 5.0                                 | 18.8  | 5.9            | 40.0  |
| I-10    | 134                            | —                                          | —     | 2.7            | 6.8   | —                                   | —     | 13.5           | 34.2  |
| I-11    | 122                            | 2.1                                        | 5.2   | 2.8            | 7.8   | 10.5                                | 26.0  | 13.9           | 39.0  |
| I-12    | 150                            | —                                          | —     | 1.8            | 6.9   | —                                   | —     | 9.0            | 34.7  |
| II-1    | 122                            | 1.1                                        | 4.1   | —              | —     | 5.3                                 | 20.3  | —              | —     |
| II-2    | 94                             | 0.9                                        | 4.1   | 1.1            | 7.0   | 4.5                                 | 20.5  | 5.5            | 35.0  |
| II-3    | 96                             | 1.0                                        | 4.1   | 1.3            | 7.3   | 5.0                                 | 20.5  | 6.5            | 36.5  |
| II-4    | 90                             | 0.6                                        | 3.9   | 0.8            | 6.0   | 3.0                                 | 19.5  | 4.0            | 30.0  |
| II-5    | 96                             | 0.2                                        | 0.4   | 0.6            | 3.5   | 1.0                                 | 1.8   | 3.0            | 17.3  |
| II-6    | 110                            | 0.2                                        | 5.3   | 1.7            | 8.9   | 1.0                                 | 26.5  | 8.5            | 44.5  |
| II-7-1  | 148                            | 0.3                                        | 1.0   | 0.6            | 1.5   | 1.3                                 | 5.0   | 3.6            | 7.5   |
| II-7-2  | 214                            | 0.5                                        | 2.4   | 0.9            | 3.1   | 2.5                                 | 12.0  | 4.5            | 15.5  |
| II-8-2  | 148                            | 6.2                                        | 6.2   | 10.3           | 10.3  | 31.0                                | 31.0  | 51.5           | 51.5  |
| III-1   | 80                             | 0.3                                        | 1.2   | 0.4            | 3.2   | 1.5                                 | 6.0   | 2.0            | 16.0  |
| III-2   | 90                             | 0.3                                        | 3.3   | 0.5            | 4.9   | 1.5                                 | 16.5  | 2.5            | 24.5  |
| III-3   | 120                            | 0.7                                        | 2.8   | 0.7            | 4.2   | 3.5                                 | 14.0  | 3.5            | 21.0  |
| III-4   | 196                            | 2.8                                        | 10.9  | 4.5            | 15.3  | 13.0                                | 54.5  | 22.5           | 76.5  |

#### • Effect of liquefiable layer

The settlement during the shaking seems to be instant and the amount of settlement depends on the thickness of the liquefiable layer. The settlement after the shaking also depends on the thickness of the liquefiable layer. That appears in the difference of the settlement proceeding time during which the secondary liquefaction due to excess pore pressure propagation occurs at the upper part of the layer. (See Figure 8)

#### • Effect of unliquefiable layer

As shown in Figure 9, the existence of an unliquefiable layer is effective in restraining settlement.

#### • Effect of grain size

As the index to express grain size, 50% diameter of soil particle  $D_{50}$  is adopted. The larger  $D_{50}$  is, the smaller the settlement becomes. This is because in larger  $D_{50}$  ground the decrease in strength and stiffness due to liquefaction diminishes, and the amount of settlement after the shaking is also affected by the excess pore pressure dissipation time.

#### • Effect of amplitude of acceleration

Figure 11 shows that the increment of the final settlement from 60Gal to 90Gal input changes significantly, but the other increments are relatively small. This can be explained by the following reasons. In the case of 60Gal input, liquefaction occurs only in a deep region and does not propagate towards the

ground surface. In the case of over 90Gal input, however, a liquefaction region does not extend abruptly during the shaking and the liquefied region extends gradually according to the process of excess pore pressure dissipation.

#### • Effect of number of cycles of loading

Settlements both at the ground surface and at the footing are small during the shaking, and the difference between them is also small. The final settlements both at the ground surface and at the footing increase according to the increase in the number of cycles. (See Figure 12)

#### • Effect of width of the footing

The intensity of the load applied at the base of the footing is kept constant in each test. The results are shown in Figure 13. The settlement decreases gradually, as the footing width increases. This may be because the region where the liquefaction resistance increases by effective stress extends in proportion to the width of the footing.

#### (4) Influence of pore fluid on the occurrence of liquefaction

From the viewpoints of the generation and dissipation of excess pore pressure and settlement, the difference between the behavior of water used as pore fluid and silicon oil used is shown in Figure 14. In the silicon oil test, excess pore pressures at some depths increase continuously even after shaking, and dissipation starts at around 8 seconds. The tendency of settlement coincides with that of the excess pore pressure generation and dissipation; settlement increases even after shaking, and it comes to an end when the excess pore pressure starts to dissipate. In the water test, on the contrary, excess pore pressures at all depths start to dissipate even during the shaking, whether they reach 100% excess pore pressure ratio or not. At the moment the shaking finishes, the settlement stops. From the acceleration time histories, it is recognized that the stiffness, once softened, tends to recover due to the dissipation of excess pore pressure during the shaking.

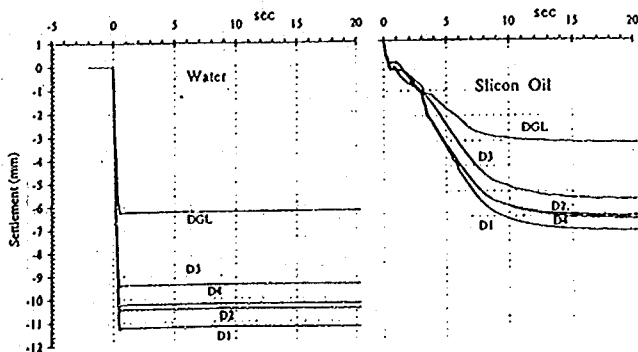


Figure 14 Influence of pore fluid on liquefaction

## 5 SETTLEMENT PREDICTION BY CONVENTIONAL FORM

The settlement of the footing for the power transmission tower due to liquefaction is assumed to be expressed by the following equation.

$$S = S_0 \cdot C_1 \cdot C_2 \cdots C_m \cdot D_1 \cdot D_2 \cdots D_n \quad (1)$$

where,

$S$  : Settlement

$S_0$  : Settlement under particular standard condition

$C_1 \sim C_m$  : Deviating factors from  $S_0$  relevant to ground and input motion

$D_1 \sim D_n$  : Deviating factors from  $S_0$  relevant to the target structure

The following factors can be taken into account as an example.

- Thickness of liquefiable layer ( $C_1$ )
- Thickness of unliquefiable layer ( $C_2$ )
- Relative density ( $C_3$ )
- Grain size ( $C_4$ )
- Amplitude of acceleration ( $C_5$ )
- Number of cycles of loading ( $C_6$ )
- Width of the footing ( $D_1$ )
- Load intensity ( $D_2$ )
- Penetration depth ( $D_3$ )

The following condition is set as a standard and effect of each factor from the standard is shown in Figure 15.

$C_1$  : 13.2m,  $C_2$  : 0m,  $C_3$  :  $D_r = 40\%$ ,  $C_4$  : Toyoura sand,  $C_5$  : 90Gal,  $C_6$  : 20,  $D_1$  : 3.2m,  $D_2$  : 2.9tf/m<sup>2</sup>,  $D_3$  : 4.3m.

If the site investigation and laboratory tests for the parameters are conducted, the settlement can be predicted from equation (1) and Figure 15. For better prediction, however, some modifications based on a review of field data from real earthquakes may be necessary.

## 6 CONCLUSIONS

Dynamic centrifuge model tests and some element tests were conducted to obtain data and information for evaluating the settlement of a power transmission tower.

The results are summarized as follows.

1. From the permeability test and the centrifuge tests for comparing the influence of pore fluid during liquefaction, it is preferable to use silicon oil, instead of water, under the centrifugal field. When silicon oil is used as the pore fluid, settlements take place mainly after the shaking, not during the period of shaking.
2. It is noticed that liquefaction occurs at the deep zone and extends upward due to the propagation of excess pore pressure. Therefore, even after the shaking liquefaction can propagate and the settlement continues until the excess pore pressure starts to dissipate.
3. In spite of the excess pore pressure generation, there exists an unliquefiable zone below the footing mainly because of relatively high effective stress due to the footing and the load.
4. From the parametric tests, an empirical procedure for predicting the footing settlement was indicated, although it may need some modifications.

## ACKNOWLEDGMENT

This study was organized by the Association for Development of Earthquake Prediction. The authors would like to express their appreciation to the people who joined the discussion and assisted them in centrifuge tests.

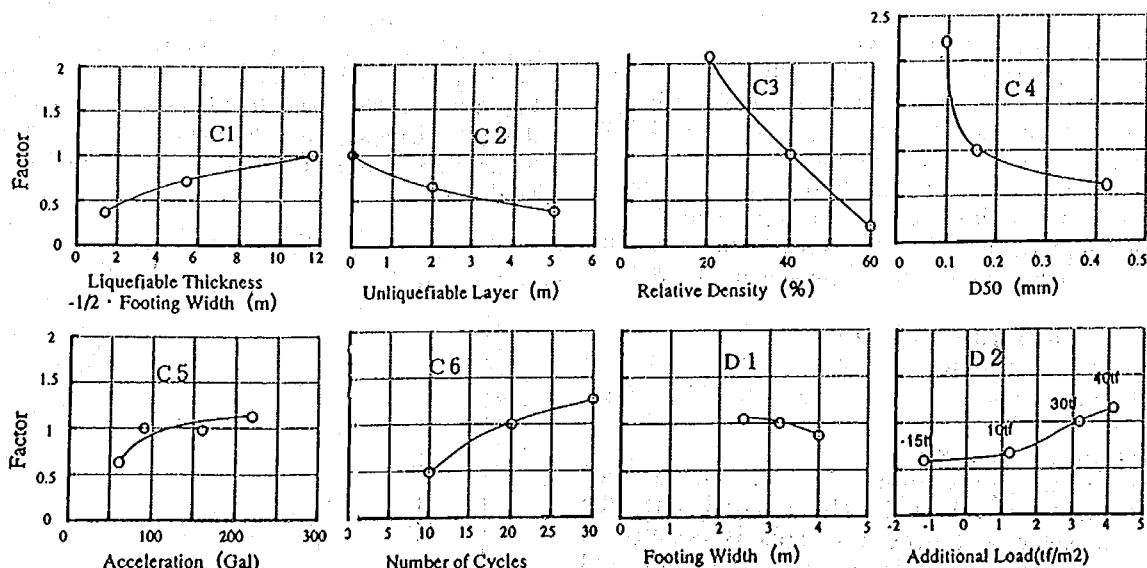


Figure 15 Deviating factors

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## PROBLEMS TO BE SOLVED

### 1. Mechanism of settlement of the footing

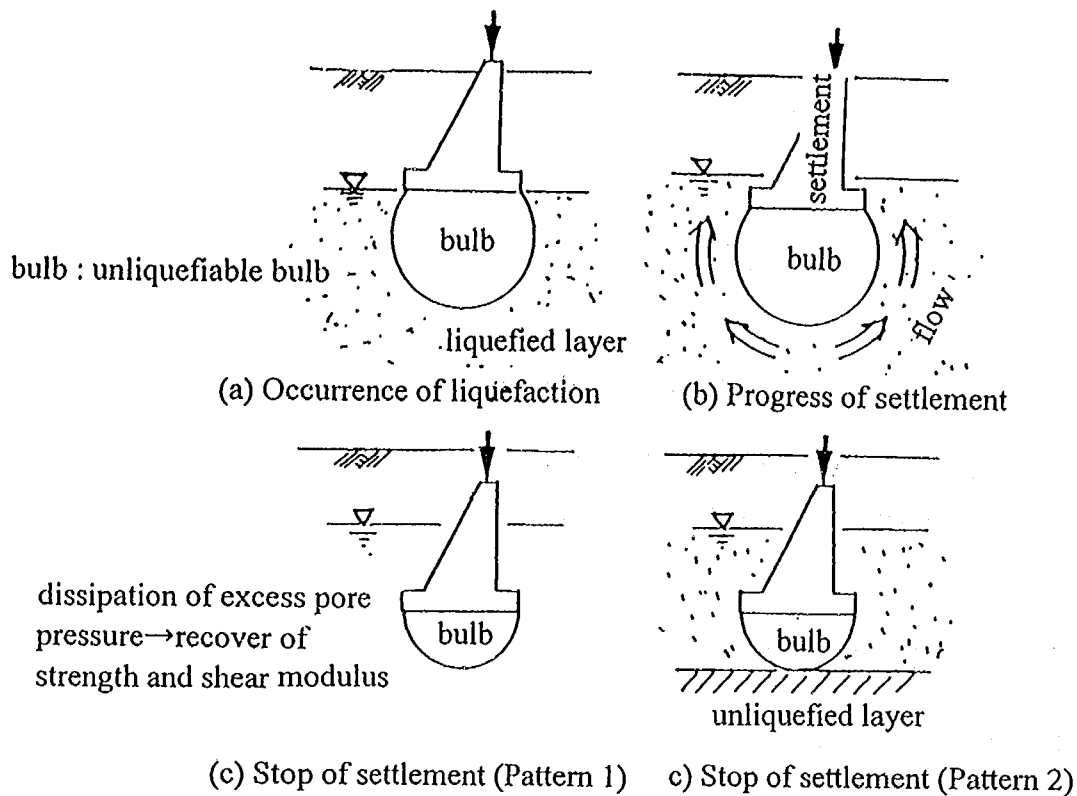


Fig. 1 One idea of the mechanism of settlement

### 2. Applicability of the test results

- Test results :  $S_0=35\text{cm}$
- Settlement of power transmission towers during the 1964 Niigata earthquake :  $S_0=66\text{cm}$
- Settlement of buildings during the 1964 Niigata earthquake :  $S_0 \div 2.5\text{m}$  (?)

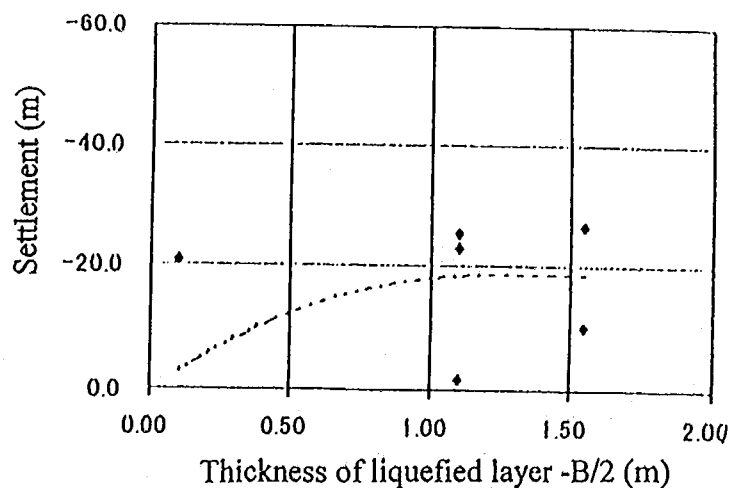


Fig. 2 Relationship between settlement of power transmission towers and thickness of liquefied layer during the 1964 Niigata earthquake

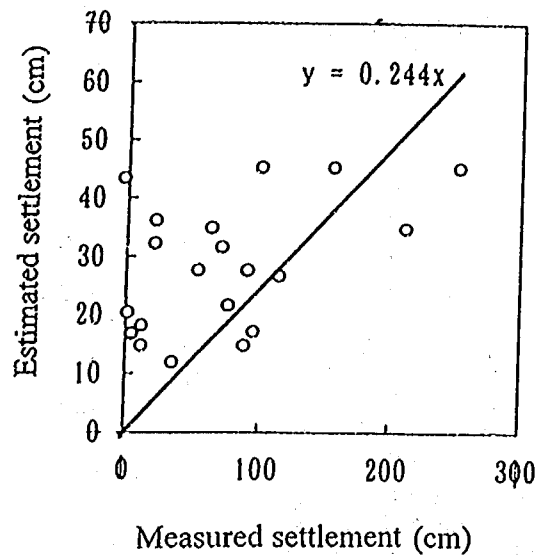


Fig. 3 Relationship between measured settlement of buildings during the 1964 Niigata earthquake and estimated settlements

3. Appropriate material for silty sand

4. Reliability of test results under very strong shaking

5. Economical countermeasures for existing footings

# Deformation Mechanism of Composite Breakwaters under Earthquake Shaking

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**ABSTRACT:** A series of centrifugal shaking table tests on composite breakwaters resting on saturated sand deposits were performed with viscous scaling introduced. In all the tests, relative density of the saturated sand was varied over a wide range. The experimental results indicate that in each shaking test, the caisson continuously settled with time during the main earthquake shaking. The ratio of the final caisson settlement to the thickness of a given supporting sand layer decreased almost linearly with increasing relative density of the supporting sand layer. These results suggest that the main mechanism responsible for the settlement of the caisson was cyclic shear deformation of the loosely packed soil beneath the caisson/mound system.

## 1 INTRODUCTION

In the standard practical design of composite breakwaters, engineers consider the effect of wave forces on the stability of the breakwater system, particularly slippage and overturning of caissons, along with bearing-capacity failure of the rubble mound and foundation soil. No special consideration has been given to the effect of earthquake shaking. Nevertheless, extensive damage occurred to the composite breakwaters in the port of Kobe due to the 1995 Hyogoken-Nambu earthquake [Sekiguchi, Kita et al., 1996; Port Bureau Ministry of Transport et al., 1997]. This fact emphasizes the significance of the earthquake-resistant design of such marine gravity structures and may result in the revision of the current design practices. Most of the composite breakwaters in Kobe Port were constructed on a foundation of fill sand which replaced the clay layer of the seabed. The caissons settled into the rubble mounds and pushed the rubble mounds outward. The maximum settlement observed was 2.6m at breakwater No.7 in Kobe Port.

The stability of composite breakwaters under earthquake shaking presents a range of problems involving dynamic soil-water-structure interactions, depending on the relative importance of such factors as [Sekiguchi and Kita, 1996]:

- Dynamic fluid pressure on both sides of the caisson
- Slippage at the caisson-rubble mound interface
- Initial stress state in the rubble mound
- Liquefaction of soil near the toe of the rubble mound
- Cyclic shear of the foundation soil beneath the caisson due to earthquake shaking
- Mass transport between the rubble mound and the foundation soil

In order to investigate into some of these problems, the authors performed a series of centrifugal shaking table tests on composite breakwaters resting on sand deposits. In what follows, the principal results will be presented and discussed.

## 2 CENTRIFUGE SHAKING-TABLE TESTS

All the centrifuge shaking-table tests were conducted at a steady state centrifugal acceleration of 30g. The arrangement of the model and the transducers used are shown in Fig.1. Foundation sand and rubble mound used in the models were Keisa No. 7 and Keisa No. 4, respectively. The specific gravity, the minimum void ratio and the maximum void ratio of Keisa No. 7 are 2.69, 0.67 and 1.15, respectively. The specific gravity, the minimum void ratio and the maximum void ratio of Keisa No. 4 are 2.64, 0.702 and 1.038, respectively. Sand (Keisa No. 7) was poured into the model container in a controlled manner to achieve a range of void ratios ( $e$ ) or relative densities ( $D_r$ ). At appropriate stages, transducers were set into position in the soil.

A trapezoidal mound was made on level ground after the foundation ground was consolidated for about 30 minutes in a steady state centrifugal acceleration. The mound was 120mm wide at the crest, 35mm high and had side slopes of 1:1.5. The mound was heavily densified. A rigid caisson of 60mm  $\times$  100mm  $\times$  100mm rested on the mound. The caisson had a mass of 1.42kg.

Three laser-type displacement gages were used to measure the vertical displacements at two points on top of caisson and the horizontal displacement at one point of the caisson.

Colored sand, which had the same physical properties as the foundation soil, was installed to form grids, and the deformation patterns due to earthquake shaking were recorded by taking photographs. To observe possible mass transport between the mound and foundation soil, a layer of colored sand was placed to form the uppermost layer of the ground.

Viscous scaling was introduced so as to match the time scaling laws of vibration and consolidation. Specifically, silicon oil with a viscosity coefficient of 30cSt was used as the pore fluid.

Each centrifuge model was subjected to 20 cycles of sinuso-

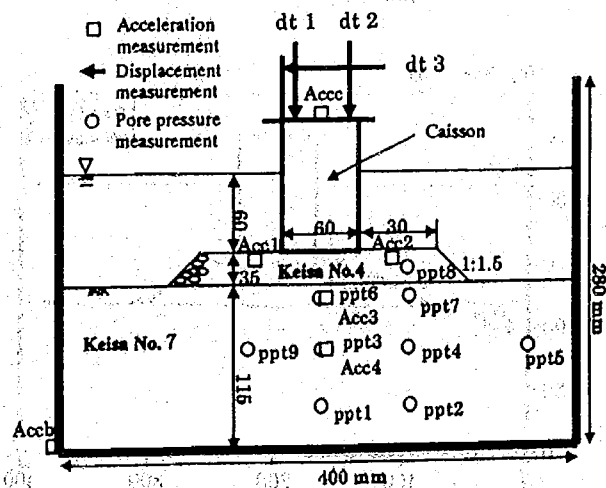


Fig. 1 Cross-section of a centrifuge model

dal excitation at a frequency of 30Hz, with a peak acceleration of about 3 gravities.

### 3 EXPERIMENT RESULTS

A total of six centrifugal shaking table tests were performed, with different relative densities  $D_r$  of the foundation sand (Table 1).

Table 1. Centrifuge shaking-table tests performed

| Case No. | $D_r$ (%) | Case No. | $D_r$ (%) |
|----------|-----------|----------|-----------|
| KH03     | 25        | KH06     | 52        |
| KH04     | 33        | KH07     | 95        |
| KH05     | 81        | KH08     | 75        |

#### 3.1 Plastic deformation pattern of ground due to shaking

The patterns of deformation of four breakwater foundations obtained after shaking is shown in Fig.2. In the case of the loosest foundation soil ( $D_r=25\%$ ), the caisson tilted significantly and underwent very large settlements. When a supporting sand layer densified highly to a relative density of 81%, little settlement was induced. In the intermediate values of  $D_r$ , the caisson settled vertically into the foundation sand. Note that the supporting sand block below the caisson compressed remarkably and the ground surface in the open areas upheaved.

It is noteworthy that the compression of the mound was negligibly small except for the  $D_r=25\%$  case. Therefore, the settlement of caisson arised from the compression of a column of foundation soil.

#### 3.2 Time histories of caisson settlement

Three time histories of caisson settlements are shown in Fig.3. Note that the curves by displacement gages dt1 and dt2 almost overlapped in Figs.3(b) through Fig.3(d). In fact, each case rocked evenly during shaking and settled vertically. As a result, the residual horizontal displacement was very small. In cases KH04 and KH06, the caisson continuously settled with time during the main earthquake shaking. In Case KH05 with a densely packed sand ( $D_r=81\%$ ), the settlement of the caisson terminated in the early stage of the main earthquake shaking.

It is evident that with increasing  $D_r$  of the foundation soil, the caisson underwent a smaller final settlement. This important aspect can be more clearly seen in Fig.4, where the caisson settlement is normalized by dividing by the thickness  $D$  of the foundation soil and is plotted against the relative density  $D_r$  of the foundation soil. The caisson resting on a loosely packed sand layer ( $D_r=25\%$ ) underwent an  $S/D$  ratio as high as 22%. It is interesting to note that the  $S/D$  ratio of the caisson decreased almost linearly with increasing relative density of the supporting sand layer, realizing an  $S/D$  ratio of 1% when the supporting sand layer densified to a relative density of 95%.

#### 3.3 Excess pore pressures in the supporting sand layer

Three time histories of excess pore pressures, located along the centerline at mid depth of the foundation soil for KH04, are shown in Fig.5. Excess pore pressure ppt5 is typical of the free field response. The foundation soil in the free field reached in a state of liquefaction. Therefore, dissipation of the excess pore pressure in this position (ppt5) after shaking was slower when compared with that beneath the caisson. The pore pressure of supporting sand layer (ppt3) beneath the caisson was smaller in

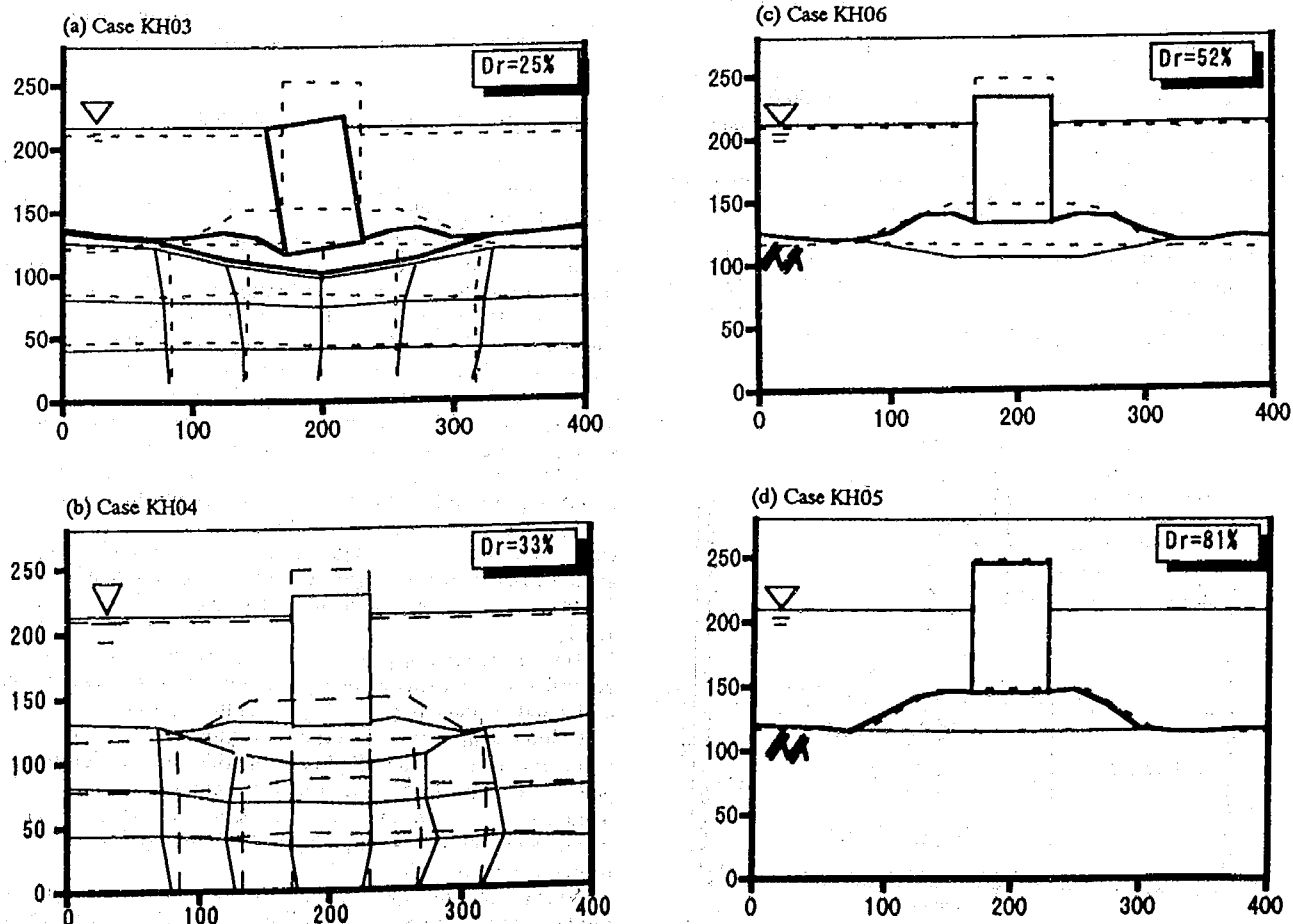


Fig. 2 Deformation of composite breakwater induced by earthquake shaking

value, and it was initially negative due to dilatancy. The pore pressures under the foundation continued to increase after the shaking due to migration of the pore fluid from the free field or from the deeper ground.

The maximum excess pore pressures  $u_{max}$  measured at different positions are plotted in Fig. 6 against the horizontal coordinate  $x$  from the center line of the caisson. The solid lines in Fig. 6 indicate the horizontal variations of initial vertical effective stresses  $\sigma'_{v0}$  for the five cases indicated, which were calculated on the basis of the theory of elasticity [Poulos and Davis, 1974]. In the loose or medium dense sand cases, the maximum excess pore pressures observed in the free field (ppt5) reached the levels of the initial vertical effective stress indicated. However, in the soil beneath the caisson (ppt3), the maximum excess pore pressure  $u_{max}$  corresponded to 40% of  $\sigma'_{v0}$  or less.

#### 4 DISCUSSION

This section will discuss the deformation mechanism of the composite breakwaters under earthquake shaking due to the centrifugal shaking table tests described in the preceding section.

Note that the caisson continuously settled with time during the main earthquake shaking, and that when shaking was stopped, the caisson immediately stopped settling. The dissipation of excess pore pressures in the ground after shaking was rather slow. These results suggest that the main mechanism responsible for the settlement of the caisson was cyclic shear deformation of the soil beneath the caisson/mound system.

According to the results of cyclic torsional undrained shear tests on saturated sand, a significant amount of vertical compression can develop in the course of undrained cyclic shearing [Sekiguchi, Tanaka et al. (1996)]. The cyclic shear deformation under these undrained conditions can be thought as a cause of the settlement of the caisson structure. The important thing is that even if the supporting sand below a heavy rigid structure does not undergo complete liquefaction, a significant settlement due to the compression of the supporting sand may be brought about according to the mechanism suggested above.

#### 5 CONCLUSIONS

A series of centrifugal shaking table tests were performed on composite breakwaters resting on sand deposits. The main results of obtained are as follows.

- 1) The caisson rocked evenly during shaking and settled vertically, except when the foundation soil was very loosely packed.
- 2) When the caisson rested on a very loosely packed sand layer ( $Dr=25\%$ ), it underwent the ratio of settlement  $S$  to the sand thickness  $D$  as high as 22%. With increasing relative density of the supporting sand layer, the  $S/D$  ratio of the caisson decreased almost linearly, realizing an  $S/D$  ratio of 1% when the supporting sand layer densified to a relative density of 95%.
- 3) The measured maximum pore pressures  $u_{max}$  in the soil below the caisson were smaller in value than the initial vertical effective stresses  $\sigma'_{v0}$ , irrespective of the relative density of the foundation soil. In contrast, the maximum excess pore pressures  $u_{max}$  measured in the free field rose to the level of  $\sigma'_{v0}$  when the sand deposit was in the state of loose or medium dense, indicating the occurrence of liquefaction in the free field.
- 4) The seismic instability of the foundation soil was a form of repeated bearing-capacity failure, although the soil in the free field took the form of liquefaction.
- 5) The caisson continuously settled with time during the main earthquake shaking, with no significant dissipation of excess pore pressures. These results suggest that the main mechanism responsible for the settlement of the caisson was a form of repeated bearing-capacity failure, permitting the vertical compression to accompany the lateral expansion that was superimposed on the simple shear type of straining.

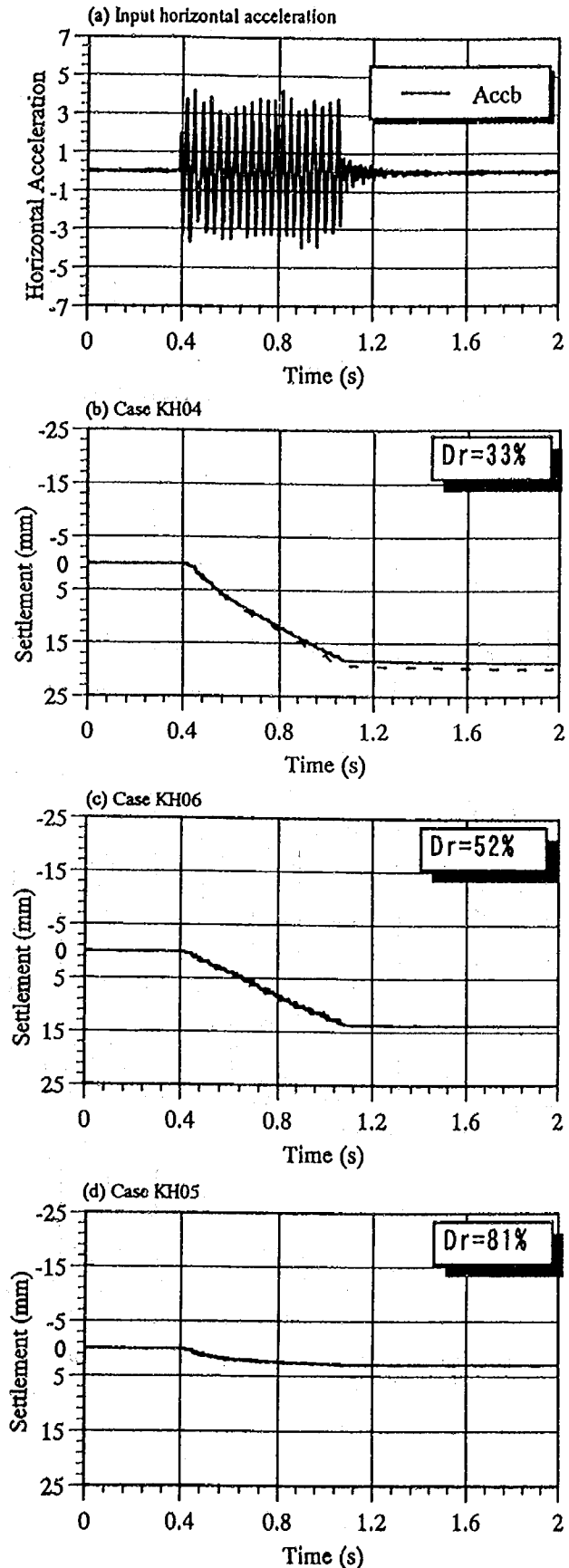


Fig. 3 Settlement time histories of the crest of caisson

Key: ———— disp. gage I      ———— disp. gage II

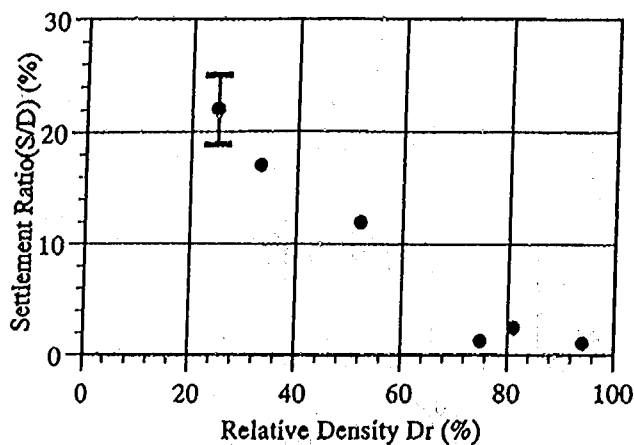


Fig. 4 Relationship between settlement ratio and relative density

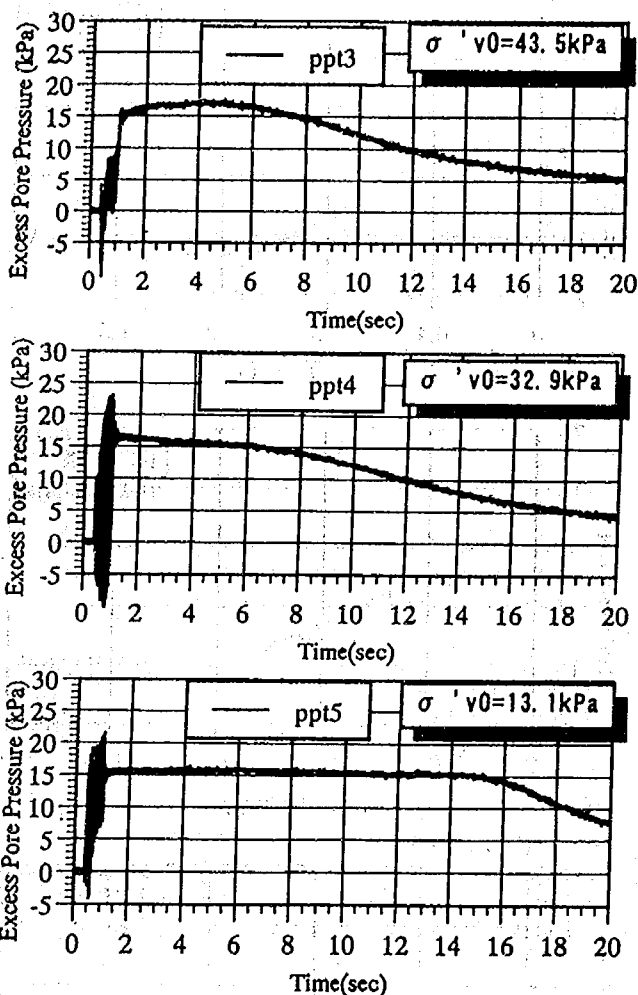


Fig. 5 Time histories of excess pore pressure at mid depth of the foundation soil

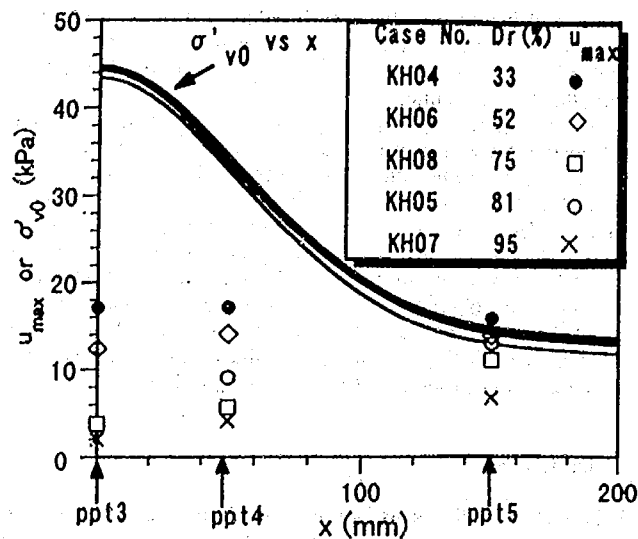


Fig. 6 Horizontal distributions of  $u_{max}$

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# SESSION 6

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- *I. Towhata*: Cyclic Triaxial Tests on Crushed Mud Stone and Effects of Ageing
- *S. Steedman*: Investigations on the Behavior of Liquefying Soils

## CYCLIC TRIAXIAL TESTS ON CRUSHED MUD STONE AND EFFECTS OF AGEING

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Hwang Chung-Yuan, Master Student, University of Tokyo

### ABSTRACT

Cyclic triaxial tests were carried out on crushed mud stone in order to understand the reason why the south halves of Rokko and Port Islands did not suffer liquefaction damage during the 1995 Kobe earthquake. A special attention was paid to ageing effects which was reproduced somehow in the laboratory within a short research period by heating the specimens at 90°C. It was found that the contained fines content not only increased the liquefaction resistance directly but also induced ageing effects by which the liquefaction resistance further increased.

### INTRODUCTION

The 1995 Kobe earthquake caused significant liquefaction in many man-made islands. It is noteworthy, however, that the south half of the Rokko Island did not suffer the effects of liquefaction (Fig.1). This lack of liquefaction damage is partly due to the soil improvement. But attention should be paid to the fact that areas without soil improvement did not have liquefaction either. The present study concerns with the reason for this.

Figure 2 illustrates the years of land filling in Rokko Island. The south half was constructed later than the north half. Ohta et al. (1997) discussed that the seabed clayey soil under such a young artificial island was in the state of underconsolidation and that its soft nature made difficult the full transmission of the seismic energy to the surface fill. The reduced intensity of the surface motion, consequently, decreased the possibility of liquefaction. Tanaka (1998) made a similar point as well.

In contrast to the viewpoints of seismic response as quoted above,

Area of liquefaction and sand boils detected in air photos

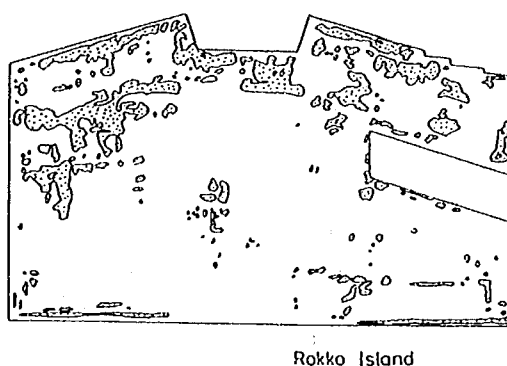


Fig.1 Area of liquefaction in Rokko Island (Development Bureau of Kobe City, 1995).

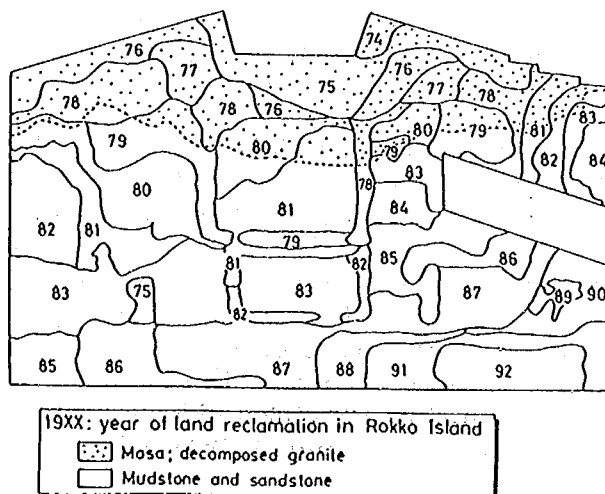


Fig.2 Years of land filling in Rokko Island (Development Bureau of Kobe City, 1995).

the present paper is interested in the soil properties in the concerned area. Fig.3 reveals the types of fill materials in Rokko Island. It is found in this figure that the south of the island where liquefaction was not significant is made of crushed mud stone as well as sandstone that were borrowed from sites of Kobe group of formations. On the other hand, the north part which were affected by liquefaction is made of decomposed granite. In contrast to the decomposed granite which is basically a silicate material, similar to sand to some extent, the mud stone is composed of clay minerals which may behave in different manners from sand during earthquake shaking.

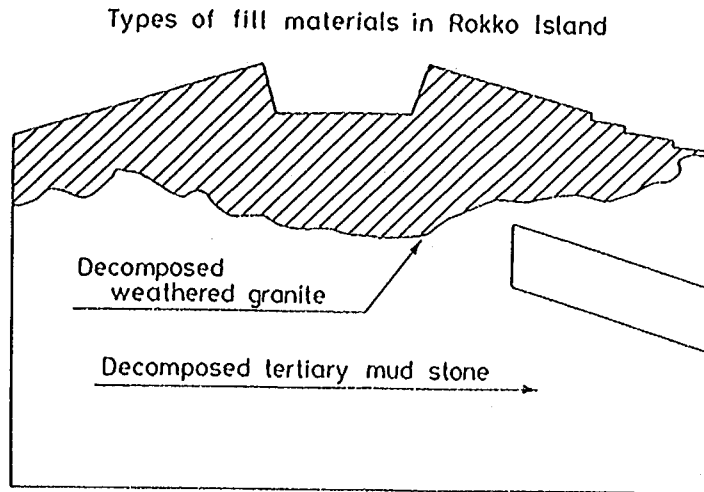


Fig.3 Types of fill materials in Rokko Island (Development Bureau of Kobe City, 1995).

There are two important points in seismic behavior of clayey materials. The first point is the effects of fines content. Hyodo and Uchida (1998) showed that the liquefaction resistance of sand increases as the plasticity index becomes greater. The second point is the effects of ageing. Wakamatsu et al. (1990) assessed the magnitude of the maximum acceleration at many artificial islands where liquefaction did and did not occur during past earthquakes. It was shown that the more aged subsoil required a greater surface acceleration to liquefy, suggesting that the resistance to liquefaction increased with age.

The effects of age or the duration time of consolidation on dynamic strength of clean sand have been studied by carrying out cyclic undrained tests in laboratories (Mulilis et al. (1977) as well as Tatsuoka et al. (1988)). It was shown that the strength increases with time, although the reason for the ageing effects is not known yet. Goto (1994) heated specimens of Toyoura sand with and without clay contents at 80°C for two weeks. He showed that the liquefaction resistance increased to some extent.

### REPRODUCTION OF AGEING EFFECTS BY CONSOLIDATION UNDER HIGH TEMPERATURE

Consolidation of clay and clay minerals under elevated temperatures is attracting engineering concern recently because heated consolidation seems to reproduce the effects of long-term consolidation within a short period of time.

Figure 4 indicates a consolidation test on MC clay which is similar to kaolin. It is seen there that the volume of a

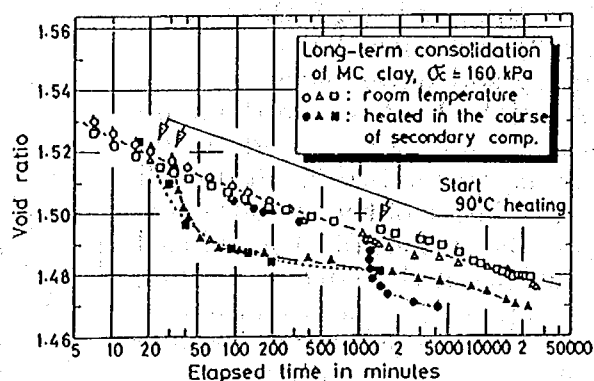


Fig.4 Effects of heating on consolidation of MC clay.

specimen suddenly contracted upon 90 °C heating in the course of secondary compression. It appears that the heating shortened the time required for volume contraction. Heating of at maximum 200 °C, however, does not change the nature of clay minerals (Towhata et al., 1993).

Undrained triaxial compression tests on MC clay was conducted in order to study the effects of heating on stress-strain behavior of clay. A special triaxial device provided with a heater was employed (Fig.5). Under elevated temperatures, the normally consolidated specimens developed the greater shear strength (Fig.6) and less excess pore water pressure (Fig.7). Thus, the material properties were improved by heating.

A more detailed study on reproduction of ageing effects by means of heating is presented below. Consolidation tests with and without heating were conducted on reconstituted specimens of sodium bentonite together with its mother stone which were collected at Tsukinuno bentonite mine in Yamagata. A site geologist stated that approximately six million years have passed since the original material became bentonite. For preparation of reconstituted samples, slurry of bentonite was lightly compressed in a large box for either 5 months or 3 years, after which specimens were trimmed out and were subjected to consolidation tests under room temperature or 90 °C. Specimens of over six-million-year-old age were directly trimmed from the mother stone and were placed in oedometers.

The void ratio ( $e$ ) versus effective stress ( $P'$ ) relationship is indicated in Fig.8. The primary consolidation of reconstituted samples in the laboratory was not able to reduce the void ratio to the values of the mother stone. Under the high temperature, in contrast, it was possible to attain the void ratio of the mother stone more easily because the heating caused more volume contraction. Fig.9 indicates the

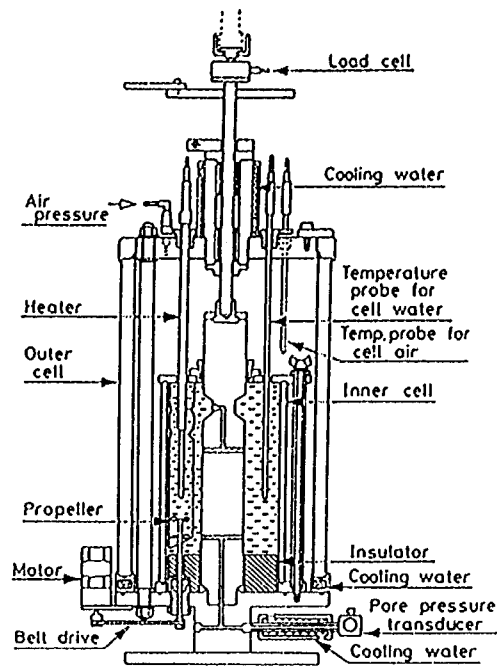


Fig.5 Triaxial shear device with heater.

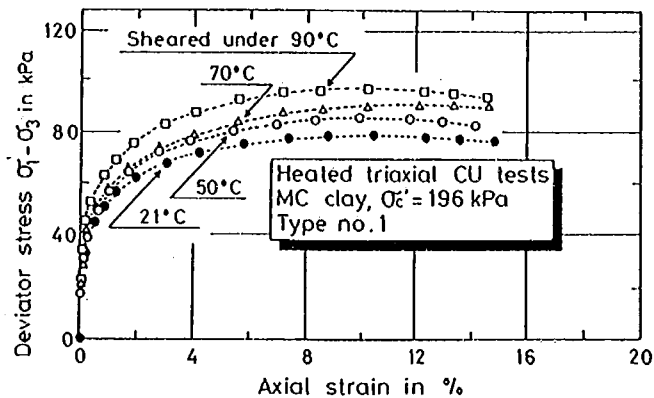


Fig.6 Undrained stress-strain behavior of MC clay under high temperature.

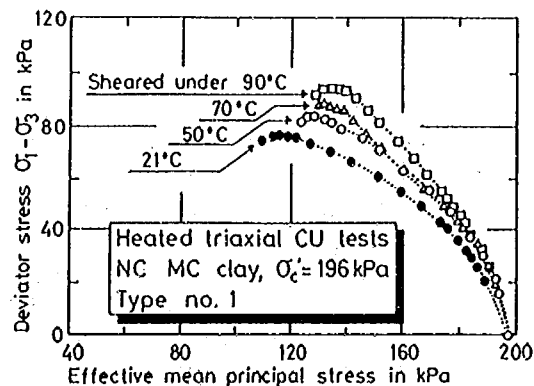


Fig.7 Undrained stress-path behavior of MC clay under high temperature.

volume compressibility,  $m_v$ . It appears that consolidation under  $90^\circ\text{C}$  was able to attain the similar  $m_v$  as that of the mother stone, while consolidation under the room temperature could not. Consequently, it seems reasonable to state that heated consolidation can reproduce the behavior of aged soil within shorter period of time.

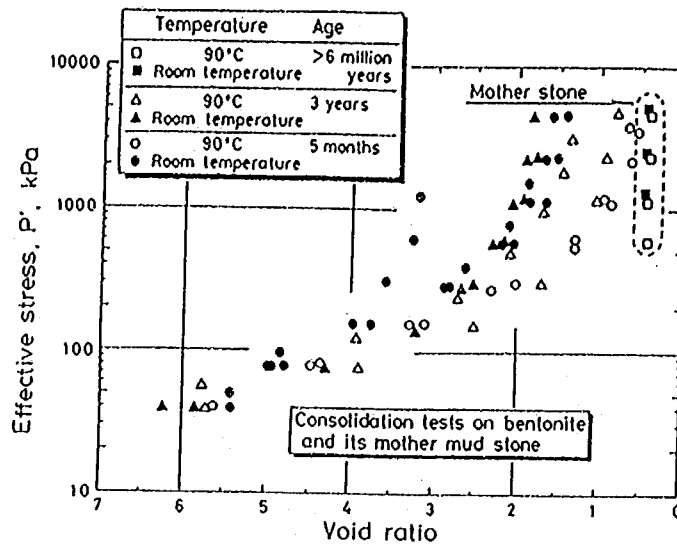


Fig.8 Measured  $e$ - $P'$  relationships of bentonite together with its mother stone.

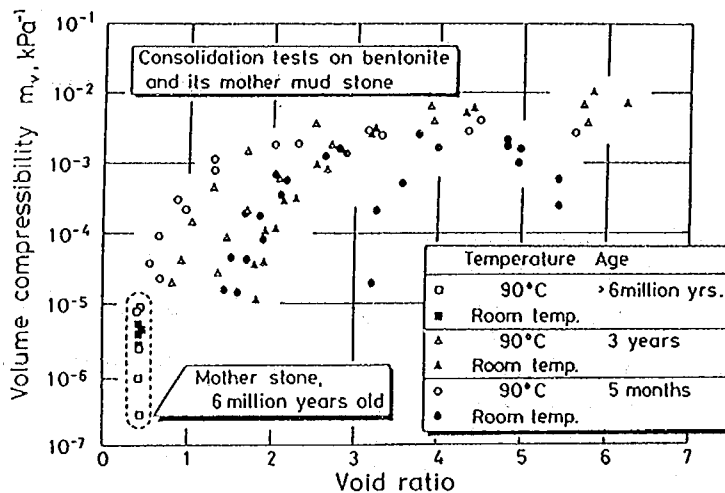


Fig.9 Volume compressibility of bentonite together with its mother stone.

## CYCLIC TRIAXIAL TESTS ON CRUSHED MUD STONE

A block of mud stone was collected from the Ikawa-Dani area in Kobe where the type of tertiary rock was similar to those employed for reclamation of Rokko Island. The collected stone was crushed into small particles which were used in liquefaction tests. For comparison purposes, Toyoura sand was tested as well. Both materials were of similar mean grain size of 200 microns, while the crushed mud stone had 14.1 % fines content. The maximum and minimum void ratios of the crushed mud stone are 1.86 and 1.19, respectively. The specimens of crushed mud stone included clay minerals and were expected to show an ageing effects, while Toyoura sand made of silicate minerals does not. Both materials were pluviated in air to form a specimen of 40% relative density together with 5cm in diameter and 10cm in height. Specimens were saturated with  $B > 0.95$  and were isotropically consolidated under 100kPa. The period of undrained cyclic loading with a prescribed stress amplitude was two minutes for one cycle.

The duration of consolidation was 1 day, 3 days, or 6 days. When heating at 90 °C was planned, 3 hours per day was spent for it. Hence, the total duration of consolidation was same whether or not a specimen was heated. Fig.10 indicates the volume change of the specimens during consolidation. While the void ratio of Toyoura sand did not decrease very much, the crushed mud stone exhibited a significant decrease of void ratio, increasing the relative density from 40% to, on average, 80%. This densification appears to contribute to the increased liquefaction resistance of the material as will be shown later on.

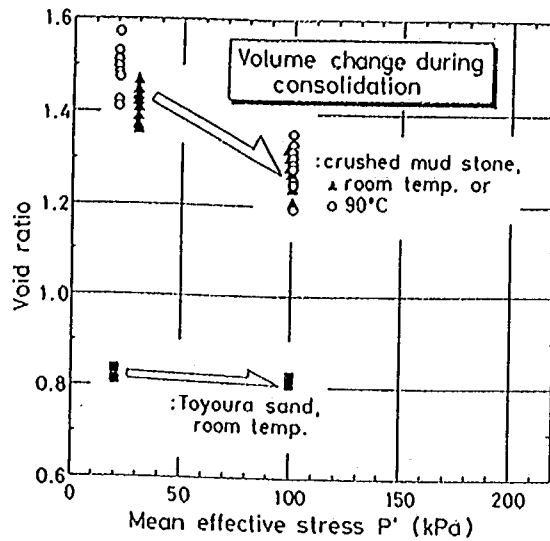


Fig.10 Volume compression of crushed mud stone during consolidation.

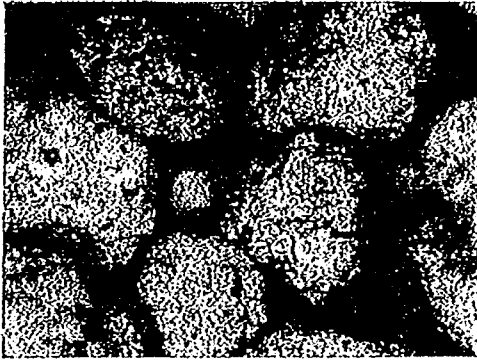


Photo 1 Microscopic appearance of the crushed mud stone.



Photo 2 Microscopic appearance of Toyoura sand.

The significant volume compressibility of the mud stone was probably related to the fines content. Photos 1 and 2 reveal the microscopic photographs of the two materials. Apparently the particles of the crushed mud stone are covered by fines while those of Toyoura sand are not. It seems that the fine particles at the surface helped attain a large void ratio during air pluviation. It is further possible that the volume contraction upon consolidation was substantial in the fines, making the final relative density 80% and achieving a good resistance against liquefaction.

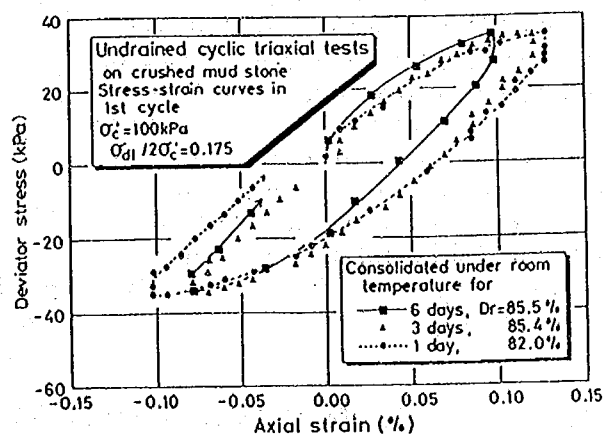


Fig.11 Undrained stress-strain behavior of crushed mud stone with three different consolidation time.

Figures 11 and 12 compare the undrained cyclic behavior of the crushed mud stone without heating. The data from the first cycle of loading, in particular, are employed here. Although the initial condition prior to cyclic loading was identical,

the specimen of 6-day consolidation was stiffer than others as evidenced by its smaller strain amplitude. Moreover, it developed the least excess pore water pressure among the three.

Liquefaction curves of the tested materials are shown in Fig.13 where the number of cycles required to develop the double amplitude of axial strain equal to 5% are employed. Firstly, the stress ratio of the crushed mud stone consolidated under the room temperature is substantially greater than the stress ratio of Toyoura sand. This is due to the increased relative density during consolidation. Most probably, the fill of Rokko Island became dense similarly during consolidation under the effects of its own weight. By examining the data more in detail, it is found that the specimen after one-day consolidation (with the symbol of ●) has the lowest liquefaction resistance, and the longer period of consolidation up to 6 days increases the resistance; see ▲ and ■. Moreover, the heating for three hours per day further increased the resistance. Although six days with heating (□) was not long enough to reproduce the whole effects of ageing, it is still possible to reproduce the significant increase of the liquefaction resistance that occurred during years after construction of Rokko Island. The increase of stress ratio at 20 cycles is illustrated in Fig.14.

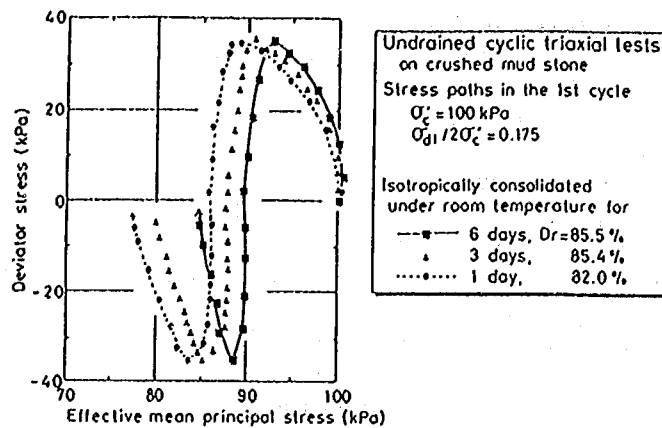


Fig.12 Stress path of crushed mud stone with three different consolidation time.

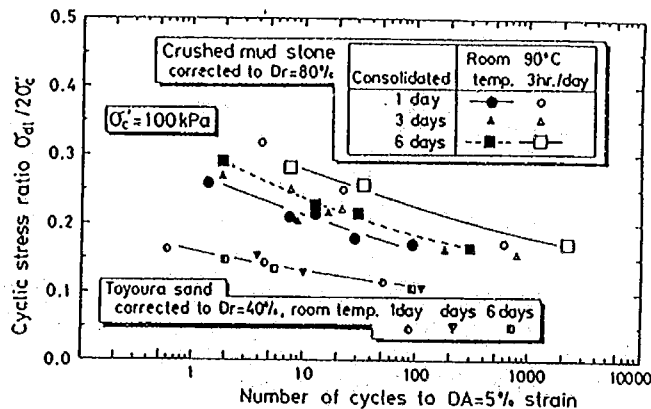


Fig.13 Measured liquefaction resistance.

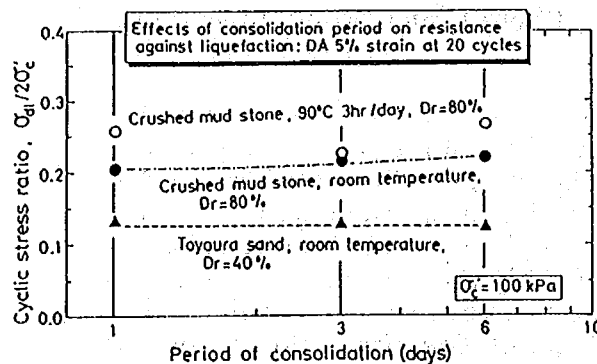


Fig.14 Increase of liquefaction resistance of crushed mud stone with consolidation period.

## INCREASE OF PLASTICITY WITH TIME IN CRUSHED MUD STONE

When tests with 6-day heating were terminated, one of the specimens was used to measure the post-testing fines content. It was found to be 36.3% that was much greater than the initial value of 14.1%. Hence, it was considered possible that the original grains of mud stone were decomposed into finer material and that the increased fines made the specimen more resistant against cyclic loading.

With the view point as above, the long-term change of the material property was investigated by placing the grains of crushed mud stone in water for seven months. Both pure water and salt water (salt concentration being similar to that in sea water) at room temperature were used. After seven months, liquid limit tests were carried out. Same tests were done also on triaxial specimens after liquefaction tests as well.

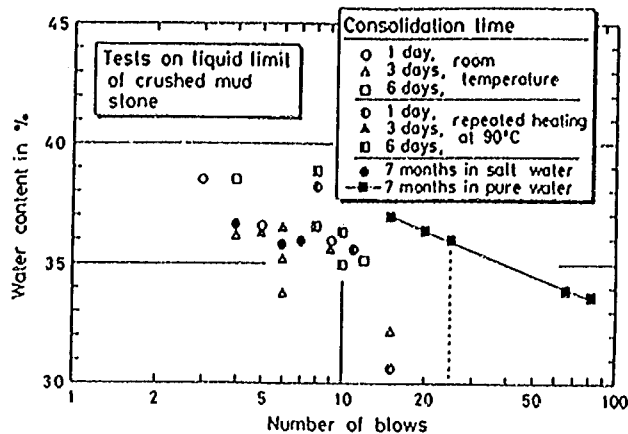


Figure 15 manifests the results of liquid limit tests. Firstly it

Fig.15 Increase of plasticity with time.

appears, although being not very evident, that the data points from heated specimens lie above those without heating. Hence, probably the fines content increased to some extent in six days. However, this increase was not enough to change the material from NP (non-plastic) to plastic. Conversely, seven months in pure water made the material (■ in the figure) plastic with LL equal to 36%. Since the real Rokko-Island subsoil had been in water for even a longer period, there was more chance to have a greater plasticity and a reduced possibility of liquefaction. It is noteworthy in Fig.15 that seven months in "salt" water did not change the mud-stone grains to a plastic material. This is because the separation of clay crystals in salty water was difficult due to the reduced size of the electric double layer at the surface of crystals. It has been theoretically considered that the thickness of the double layer decreases in water with ions (van Olphen, 1991). When the double layer is thin, clay grains can easily get contact with each other and are likely to form bigger particles that make the crushed mud stone more similar to sand. Authors suppose at this moment that the ground water in Rokko and Port Islands were fresh at the time of the earthquake, because the original salty sea water in the pore was gradually replaced by the fresh rain water.

## CONCLUSIONS

The lack of significant liquefaction in the south halves of Rokko and Port Islands during the earthquake in 1995 were investigated from the view point of material properties. Cyclic triaxial tests were carried out on crushed mud stone which were similar to what composed the liquefaction-resistant part of the island. A special attention was paid to the effects of ageing on the liquefaction resistance. Accordingly, the following conclusions were drawn as the reasons for the lack of liquefaction in Rokko Island.

- 1) The crushed mud stone includes a certain amount of fines content which can improve the resistance against liquefaction.
- 2) The fines content causes significant volume contraction upon consolidation, leading to the increased liquefaction resistance.
- 3) Liquefaction resistance further increases with the period of consolidation (ageing).
- 4) Heating during consolidation makes this ageing effects more remarkable, probably reproducing the long-term change of the material property in a

short period of time.

- 5) Fines content increases with time and increases the liquefaction resistance. This phenomenon is more significant when the pore water is fresh.

## ACKNOWLEDGMENT

The tested mud stone obtained from the Ikawa-Dani site where the type of the mud stone is similar to those employed for the construction of the islands. Mr. S.Okada of Kisojiban Consultants Company made a kind arrangement for this preparation of the material. The authors deeply appreciate his kind support.

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# INVESTIGATIONS ON THE BEHAVIOR OF LIQUEFYING SOILS

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**ABSTRACT:** Improved physical evidence is needed of the processes and mechanisms involved in the occurrence of liquefaction as soil strength deteriorates to a residual state to allow the development of new, more refined analyses for dam safety and more cost-effective and safe seismic remediation. Reliable physical evidence can only come from field or equivalent-field data of behavior under well known and defined conditions. It has been concluded that the only practical route by which this can be achieved is through the use of the centrifuge. This paper describes the centrifuge experiments being undertaken to capture data of equivalent prototype behavior under dynamic loading and preliminary results. The research is part of the ongoing U.S. Army Engineer Earthquake Engineering Research Program (EQEN) at the U.S. Army Engineer Waterways Experiment Station (WES) and is using the dynamic base shaking capability of the unique WES research centrifuge. Our objectives are: (1) improve the simplified liquefaction methodology correction factors,  $K_0$  and  $K_{0r}$ , because this method will always be used with in-situ measurements for initial field investigations and for verification of dam and foundation improvements, (2) define better the behavior of liquefying soil and effects on dam behavior, (3) expand our earthquake case history data base with better known conditions and responses, and (4) provide progressive results for immediate use in U.S. Army Corps of Engineers projects. The approach of this research to investigate the behavior of liquefying soils is to use laboratory testing, analytical procedures and centrifuge dynamic model testing.

## 1 INTRODUCTION

This paper will present the brief background and the problem, objective, description of work and preliminary results for the EQEN investigation of the behavior of liquefying soil materials. The state-of-practice assumptions concerning the occurrence and behavior of liquefied soil, residual strength behavior, and dam stability and deformations are not valid as shown by field behavior, numerical analyses and physical model test results. Our objectives are to improve the simplified liquefaction methodology correction factors, define better the behavior of liquefying soil and effects on dam behavior, expand our case history data base with better known conditions and responses, and provide progressive results for immediate use in Corps projects. The approach of this research to investigate the behavior of liquefying soils is to use laboratory testing, analytical procedures and centrifuge dynamic testing.

## 2 ENGINEERING PROBLEM

### 2.1 Current state-of-practice

The present state-of-practice for determining the performance of soils that undergo earthquake-induced shear-strain and consequently pore water pressure buildup in saturated soils is to determine if a soil will or will not liquefy. The performance and safety of structures are based on rigid sliding-block/slices limit-equilibrium methods to determine slip-plane stability and deformation using residual strengths in the case of liquefaction.

In-situ methods such as Standard Penetration Test (SPT), Cone Penetration Test and shear wave velocity measurements are used to define the triggering of liquefaction. The potential for liquefaction relies on empirical correlation between the penetration resistance and the performance of soil deposits in past earthquakes (Seed, 1979). The database on triggering liquefaction is based on data from level ground where

surface evidence occurred. Soil conditions were shallow at overburden pressures less than about 95.8 kPa (about 4.6m of depth). The data base is normalized to an earthquake of magnitude 7.5. Corrections for other earthquake magnitudes have to be made (Seed & Harder, 1990, NCEER, 1996).

To extend the database to depths representative of foundation soils under dams requires the use of correction factors (Seed & Harder, 1990) for the effects of overburden pressure and initial static shear-stress. Correction for high overburden,  $K_\sigma$ , is based on laboratory test results of the ratio of cyclic-shear-stress, CSS, to cause liquefaction at an overburden effective-stress state,  $\sigma'_o$ , to that at a  $\sigma'_o = 95.8$  kPa. Correction for initial static shear-stress,  $K_\alpha$ , is based on the ratio of CSS to cause liquefaction with initial static shear-stress applied to that at no initial static shear-stress.

A factor of safety, FSL, against the occurrence of liquefaction, defined as 100 percent pore water pressure, can be calculated as:

$$FSL = f[(N_1)_{60}] K_m K_\sigma K_\alpha / \tau_{eq}$$

Where  $(N_1)_{60}$  is from SPT relations,  $K_m$  is earthquake magnitude or duration correction factor and  $\tau_{eq}$  is a percentage of peak earthquake induced shear stress.

The FSL can be related to percent excess pore water pressure.

The correction factor  $K_\sigma$  has a large influence for dam foundations. It can reduce the CSS to cause liquefaction to about 45 percent of its in-situ value at pressures about 670 kPa that would exist beneath an embankment dam about 30.5m high. Alternatively, if we were deriving SPT criteria for remedial treatment of this dam to limit the potential level of earthquake generated pore water pressures, the  $K_\sigma$  factor would cause an increase of more than double in the required penetration values to be measured in the field. Clearly, the correction factor can have a major impact on the potential for triggering liquefaction or the excess residual pore water pressures and on the cost of remediation.

The correction factor  $K_\alpha$  can also contribute significantly to the reduction of the in-situ strength. However, for relative densities above 45-50 percent,  $K_\alpha$  can have a positive effect on the in-situ strength. The  $K_\sigma$  relationships are not well defined.

In application of the state-of-practice, we inherently

make the following assumptions: (1) the soil is always undrained, (2) liquefaction occurs instantaneously and the soil shear strength jumps to residual state, (3) residual strength is constant with monotonic loading, (4) liquefaction is independent of soil zone thickness, permeability, or boundary conditions, (5) liquefaction is independent of when the earthquake peak energy arrives, (6) behavior of the liquefied soil and its resultant effects on a dam are independent of the soil zone thickness, permeability, and boundary conditions, (7) dam stability and deformation are controlled by slip-planes independent of the liquefied soil zone thickness and behavior, and (8) non-liquefied soil at a site is unaffected by the earthquake. Field behavior, numerical analyses, and physical model tests show that these assumptions are invalid.

## 2.2 Research problem

Current studies for more thorough evaluation of liquefaction and for remediation design and analysis have shown serious limitations in the state-of-practice. The state-of-practice can force us to make costly excessive remediations when possibly no action is required, but it can also lead us to make unsafe conclusions in other cases.

Significant progress is being made in the development of numerical methods for analysis of liquefaction and the consequences. However, the engineering profession will most likely always use empirical correlation (Seed's or others) of in-situ measurements versus potential liquefaction, pore water pressure generation and earthquake response. Every time a site is evaluated for a seismic design or potentially liquefiable soil is improved and a dam remediated, in-situ measurements will be made. A value/range of in-situ measurement to achieve will be specified for a construction/remediation contractor. Some in-situ measure will be used to judge soil conditions/improvement and seismic safety of a dam or site. Therefore, improvement in our current state-of-practice and the empirical correlations between in-situ measures and performance of soil deposits has to be made.

Current studies of the seismically-induced deformation behavior of dams indicate that as soils are progressing toward liquefaction (pore water pressure is increasing and shear strain is occurring) significant deformations of the structure can occur. Failure (damaging levels of deformation) can develop significantly before the complete initial liquefaction stage (100 percent pore water pressure ratio) is

reached. Depending on specific conditions involving the location, depth and extent of liquefying soil and the driving forces, a structure may fail at only 50 percent strength reduction. In this case, remediation to assure safe performance is required to prevent serious damage significantly before an initial liquefaction condition and a residual strength stage are arrived at in the soil.

The problems in the current state-of-practice stem mainly from the fact that we do not know for the sites that have liquefied and constitute the empirical basis for analysis: (1) the exact and complete soil conditions and profiles, (2) the real behavior that occurred in the soils during and after the earthquakes or the various influences on the behavior, (3) that the assumed non-liquefied soils (used in comparison to liquefied soils) did not develop pore pressures or strains and change state during the earthquakes, and (4) whether artificial and possibly incorrect conditions in laboratory testing may have led to conclusions not totally applicable to the field behavior.

Improved definition and physical evidence is needed of the processes and mechanisms involved as a soil progresses to liquefaction and residual strength. This is needed to allow refined analyses for dam safety and more cost-effective and safe remediation design and analysis. Because we can make various assumptions coupled with methodologies and numerical analyses that can give solutions or answers to most anything, the reality-check of solutions must come from field or equivalent-field data of behavior under well known and defined conditions.

The earthquake response database needs to be expanded with more complete data to provide: (1) the necessary advance in the state-of-practice, (2) a basis for modification and improvement of current methodology and assumptions, and (3) definition of the physical processes and mechanisms involved in the liquefaction process and resultant effects on dam behavior. This would also provide the fundamentals and basis for development of new methodology and analyses. New methodologies have to be based on correct mechanisms and processes.

Current specific needs for more thorough earthquake engineering analyses can be identified from examination of the last two decades of experience in seismic evaluation of embankment dams and the serious limitations that arise when remediation design is attempted. Studies involving liquefaction, stability, and seismic-induced deformation behavior of dams raise serious questions that impact the safe performance and needs for remediation (e.g., Ledbetter and Finn, 1993, Ledbetter, et. al., 1994, Finn and Ledbetter, 1991, Finn, et. al., 1991 and 1994, Vaid and

Chern, 1985, Vaid and Thomas, 1994, Bryne and Harder, 1991). For example, Vaid and Thomas (1994) demonstrated that  $K\sigma$  for specific sand types may be substantially less (more than a factor of 2) than suggested by Seed and Harder. Pillai and Byrne (1994) showed for the foundation materials of Duncan Dam, a  $K\sigma$  of 0.6 compared with 0.4 from the Seed and Harder relation. This made a difference between recommending remediation and no remediation. There is a large spread of  $K\sigma$  relations and data by various researchers that can influence whether to remediate a dam at costs in the tens of million's of dollars or not remediate.

For the past thirty years, research has primarily concentrated on the triggering of liquefaction of soils both in the laboratory and in field sites. A significant database has resulted of very important and necessary information concerning the stress-state triggering of liquefaction, cyclic load stress-ratio and the dynamic properties of soils. Newmark, 1965, stressed that what counted was whether the deformations that a dam suffered during an earthquake were tolerable or not. Peck, 1992, stated that of all measures of safety, the most directly applicable results are the anticipated deformations.

Due to the lack of knowledge and experience involving the behavior of liquefiable soils under field conditions, we are forced to make, for the critical safety of dams, simplifying assumptions concerning behavior. We are forced to disregard possible significant controlling influences such as permeability, boundary layers and pore water pressure dissipation and migration.

Some specific needs are: (1) well defined and complete shear stress-strain response curves for earthquake loading including the residual strength portion, (2) strains within a problem soil mass, (3) effects of soil zone thickness, permeability, and boundary conditions, (4) influence of adjacent soil materials and of their permeabilities, (5) dissipation and movement of excess residual pore pressure both during and after an earthquake, (6) redistribution of stresses as a soil is losing strength, (7) interaction of remediation materials and adjacent soil, (8) dynamic response of remediation materials and of remediated zones, (9) improved  $K\sigma$  and  $K\alpha$  factors for the field evaluation of remediation achievement and for improved first estimates of liquefaction potential, (10) effects of strong aftershocks and (11) dam internal behavior and failure mechanisms in response to earthquake loading and strength degradation.

### 3 DESCRIPTION OF RESEARCH

This research is being conducted by investigations employing laboratory testing, analytical procedures, and centrifuge testing. Physical modeling of field problems generally involves the construction and experimentation with scale models of a prototype or field structure. The response of the model to a physical perturbation such as cyclic loading, earthquake, ocean waves, explosion or even freezing or groundwater changes can be measured and interpreted in terms of the field problem under consideration. Realistic model behavior data can provide valuable information for designers about failure mechanisms and long term performance.

Scaling relationships can be derived from physical principles which interpret centrifuge model experimental investigation data in prototype terms (Schofield 1980 and 1981, Schofield and Steedman 1988). These are summarized in Table 1.

Table 1. Scaling Relations

| PARAMETER                   | FIELD      | CENTRIFUGE MODEL ( $N_g$ ) |
|-----------------------------|------------|----------------------------|
| Stress                      | $\sigma$   | $\sigma$                   |
| Strain                      | $\epsilon$ | $\epsilon$                 |
| Length, displacement        | $L$        | $L/N$                      |
| Area                        | $A$        | $A/N^2$                    |
| Force                       | $F$        | $F/N^2$                    |
| Volume                      | $V$        | $V/N^3$                    |
| Mass                        | $M$        | $M/N^3$                    |
| Energy                      | $E$        | $E/N^3$                    |
| Frequency                   | $f$        | $Nf$                       |
| Velocity                    | $v$        | $v$                        |
| Acceleration                | $a$        | $a/N$                      |
| Time (for inertial events)  | $t$        | $t/N$                      |
| Time (for diffusion events) | $t$        | $t/N^2$                    |

The wide range of physical phenomena that may be created in a centrifuge and the quality and detail of the data that may be captured from a model-in-flight clearly provides substantial opportunities for engineering research studies. The Corps of Engineers has been

involved with centrifuge modeling since the 1970s, and the recent development of the WES Centrifuge Research Center in Vicksburg demonstrates the Army's full commitment towards the use of physical modeling as an integral part of engineering analysis.

Soil behaves non-linearly and is stress-state dependent. Centrifuge testing is the best, most practical, most economical, and the only method for properly investigating and verifying earthquake induced prototype behavior in soil.

#### 3.1 The WES Centrifuge

The design specification of the centrifuge followed a review of available academic facilities which had shown that none were able to routinely conduct experimental models of the large field structures and problems with which the Corps is principally concerned (Ledbetter, 1991). The facility was therefore designed with a unique operating envelope. It was required that model containers should be easily placed on and off the centrifuge and these requirements led to the design of a beam centrifuge with a swinging platform based on the French designed Acutronic 661, 665, and 680 series of geotechnical centrifuges. Other centrifuges in this Acutronic family include one at Rensselaer Polytechnic Institute, Troy, New York, at the Centre for Cold Oceans Resources Engineering, St. Johns, Newfoundland, and several in Japan and Europe. Figure 1 shows the WES Centrifuge in its containment structure; key parameters are listed in Table 2 and Figure 2 shows the performance envelope relating payload capability to centrifugal acceleration. Prototypes of the order of 300 m in breadth and 300 m deep can be simulated in models subjected to up to 350 gravities (around 3500 m/s<sup>2</sup>).

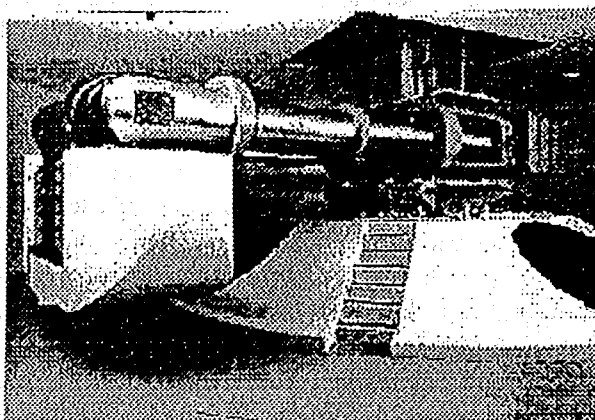


Figure 1. WES centrifuge.

Table 2. Key Characteristics of the Army Centrifuge

| Army Centrifuge    |              |
|--------------------|--------------|
| Radius to platform | 6.5 m        |
| Payload at 143 g   | 8000 kg      |
| Payload at 350 g   | 2000 kg      |
| Capacity           | 1144 g-tonne |

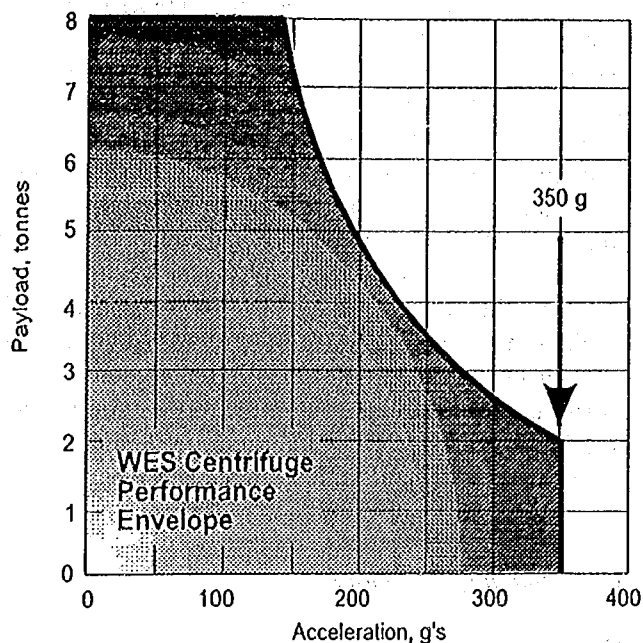


Figure 2. WES centrifuge performance envelope.

A wide range of modeling capabilities was required and actuators and appurtenances have been designed to fulfill these requirements. Models can be constructed in a range of containers that depend on the geometry of the problem under investigation (plane strain, axisymmetric, three dimensional, etc.) and the type of phenomenon to be recreated.

Recent world-wide interest in centrifuge modeling has concentrated on geotechnical applications; however, the WES centrifuge facility has a much broader mission. The WES centrifuge will address research needs in physical modeling across the full range of engineering applications. Investigations are possible under climatic conditions ranging from desert to polar to ocean regions. Key capabilities include: (1) force and displacement controlled load systems, (2) earthquake simulation, dynamic vibration loading and water wave generation, (3) blast loading, (4) model package environmental control, (5) in flight

manipulation of the model materials and fluids, (6) miniature instrumentation, cameras and in flight site interrogation, and (7) high speed data acquisition and control systems.

The new capabilities that will flow from the centrifuge will depend on the ingenuity of its users and the design of its appurtenances. In this respect the design of the centrifuge itself is merely one component of the development of new capabilities in physical modeling.

The uniquely large stored-angular-momentum (SAM) shaker (Madhabushi, et. al, 1996) (Figure 3 & 4) designed to be capable of exciting dynamic motions up to 200+ g will be used with the equivalent shear beam box (Figure 5) in this research.

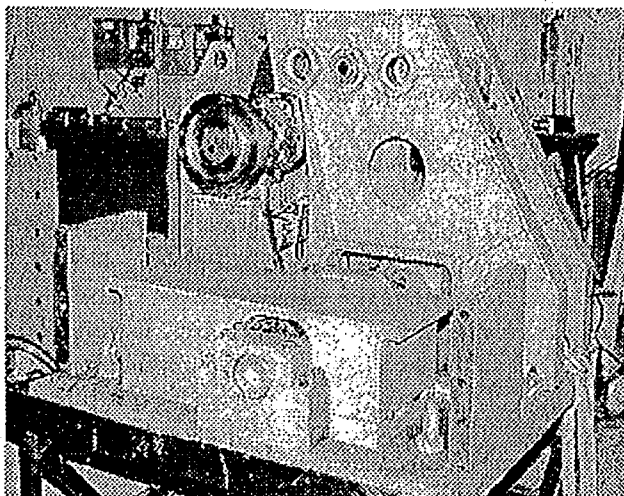


Figure 3. View of SAM actuator without support walls.

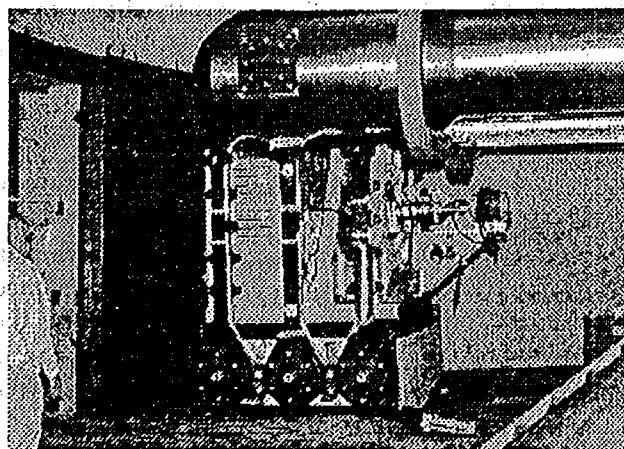


Figure 4. Assembled system on WES centrifuge.

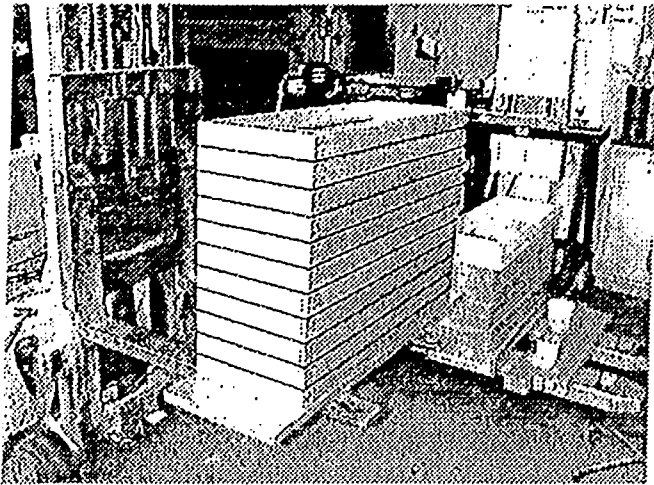


Figure 5. Stacked ring equivalent shear beam container.

### 3.2 Earthquake research

Centrifuge testing is being used to produce prototype response under laboratory conditions and with measured base shaking input motions and propagation. The experiments are to: (1) obtain high-quality detail and data for earthquake behavior case histories, (2) investigate the  $K\sigma$  and  $K\alpha$  correction factors under dynamic loading by shaking of the continuum rather than by laboratory element tests, (3) measure and investigate pore pressure behavior and response under various conditions during and after earthquakes, (4) measure and investigate strain, deformation, and softening behavior during and after earthquakes, (5) measure and investigate the controlling effects of permeability on pore pressure behavior during and after earthquakes, and (6) measure and investigate the effects of boundary conditions on the behavior of liquefying soil. Each centrifuge test will yield results relevant to several of the objectives in defining the behavior of liquefying soils and effects on dam response.

Laboratory testing will be used to: (1) develop and define the complete shear stress-strain curve for earthquake-induced liquefying soil behavior including the residual portion and the development of simplified methods for obtaining the curve, (2) determine the properties of soils used in the centrifuge dynamic tests for both pre and post test conditions, (3) determine the complete shear stress-strain curve for the soils used in the centrifuge dynamic tests and (4) conduct cyclic tests for the laboratory-based determination of  $K\sigma$  and  $K\alpha$  for comparison to the centrifuge dynamic test results.

Analytical procedures employing both numerical dynamic-analysis and conventional static-analysis methods will be used for: (1) analysis of the centrifuge results as an equivalent prototype and as a bridge between the results and existing field case history data, (2) identification of anticipated important parameters and sensitivities for consideration in the centrifuge dynamic tests, (3) careful design of centrifuge dynamic tests to maximize the results and to obtain results from each test that are applicable to several of the objectives, (4) analyses of pre, during and post dynamic test conditions, (5) extrapolation and comparison of centrifuge test results to limit the number and extent of needed tests and (6) important assistance in the necessary understanding and definition of the physics of the mechanisms and processes being investigated.

Results from the work of other researchers studying residual strength, causes of liquefaction and flow of liquefied soil will be incorporated and collaboration will be made. However, this study is unique in that it addresses the fundamental effective-stress behavior and mechanisms of soil as it is in the process of liquefying and not just the end point of a liquefied state. The entire process of liquefaction and the controlling conditions are key to eventual accurate predictions of anticipated deformations; how much and where.

### 3.3 Centrifuge test plans

The first tests and studies are concerned with evaluating  $K\sigma$  in centrifuge tests of the behavior of level ground for comparison with laboratory values. Two soils, Ottawa and Nevada sand, and a mixture with silts are planned for the tests. Both of these soils are well known with extensive laboratory investigations. Another soil or mixture with silt will be used for low permeability and will be placed at a non-liquefiable density to bound/confine liquefiable test soil layers. It may also be used to investigate a deep foundation layer beneath the liquefiable soil and a transition from the rigid base through which the input motion propagates to the test layers.

At least three test layer depths and possibly five will be used for each soil at different relative densities and at prototype overburdens of 95.8 to 575 kPa.

Pore pressure and acceleration transducers placed redundantly are used to measure behavior.  $K\sigma$  can be calculated for each layer in ratio to the top layer at 95.8 kPa. Several dynamic excitations will be used during each test period to determine the effects of dynamic excitation history and previous liquefaction on soil and  $K\sigma$  behavior.

Following the initial  $K\sigma$  test program, a similar centrifuge test program will be developed for  $K\alpha$  behavior. The tests will be conducted with embankment dam loading to induce initial static shear stresses in the foundation layers.

Test programs will also be conducted in similar fashion to get equivalent prototype data addressing the influences of permeability, boundary conditions and layer thickness and liquefiable soil zone thickness on the behavior of liquefying soil. This will yield significant effects on dam response both external and internal; strains, softening, response during and post earthquake, global behavior and failure modes.

#### 4 RESEARCH EXPERIMENTS RESULTS

This section presents typical and preliminary results from  $K\sigma$  centrifuge experiments. Figure 6 shows a typical instrumented model of Nevada sand with a loose layer beneath a denser layer that imposes a 95.8 kPa overburden pressure at 50 g centrifugal acceleration. The sand layers were rained in place at a relative density of 49% in the bottom and 74% in the top. The instrumentation of pore-pressure and accelerometer transducers is shown.

Figure 7 shows time history raw data from pore-pressure transducers and accelerometers.

Figure 8 shows the Fourier transform combined with harmonic wavelets analysis mapping (Butler, 1998). Conventional methods of signal analysis using Fourier transforms provide frequency information that is averaged over the entire length of the acceleration time history. This method of analysis fails to address the changing spectral content of a non-stationary signal that is produced by non-linear dynamic events and consequently, adversely effects attempts to validate numerical modeling techniques. The harmonic wavelet technique for analyzing the changing frequency response of a physical system will provide significant new insights into the characteristics of soil behavior and open new windows of understanding into its complex dynamic response.

Figure 9 shows a typical acceleration amplification profile as a function of base input motion. Figures 10 through 13 present shear-stress-ratio and number of equivalent uniform cycles to liquefaction for a number of centrifuge experiments. Figure 10 is for vertical overburden confining pressures of 0.36 tsf; Figure 11 is for 0.8-1.6 tsf, and Figure 12 is for 2-2.6 tsf. Figure 13 presents a summary of the liquefaction data. Also included on these figures is data representing the response of the soil with an over-consolidation ratio of

2.5 that was induced by running the centrifuge to higher acceleration and back to the test acceleration of 50 g. The implied result is that over-consolidation does cause a different response.

#### 5 CONCLUSIONS

The research described in this paper will provide definition and clarification of the  $K\sigma$  and  $K\alpha$  correction factors for use in the evaluation of dam remediation and in the initial determination of liquefaction potential. The results of this program will enhance significantly our understanding of dam response and our capability to determine and define liquefaction occurrence, its potential to cause damage, and the need for mitigation. Economic benefits will result in improved analysis, design, and remediation requirements. The threat of liquefaction will be lessened through better understanding of the process and conditions for earthquake induced strength loss and liquefaction. Significant enhancement of the data base of earthquake response will be made which will provide; (1) modification and improvement of current methodology and assumptions, and (2) definition of processes and mechanisms for development of new analysis methodology.

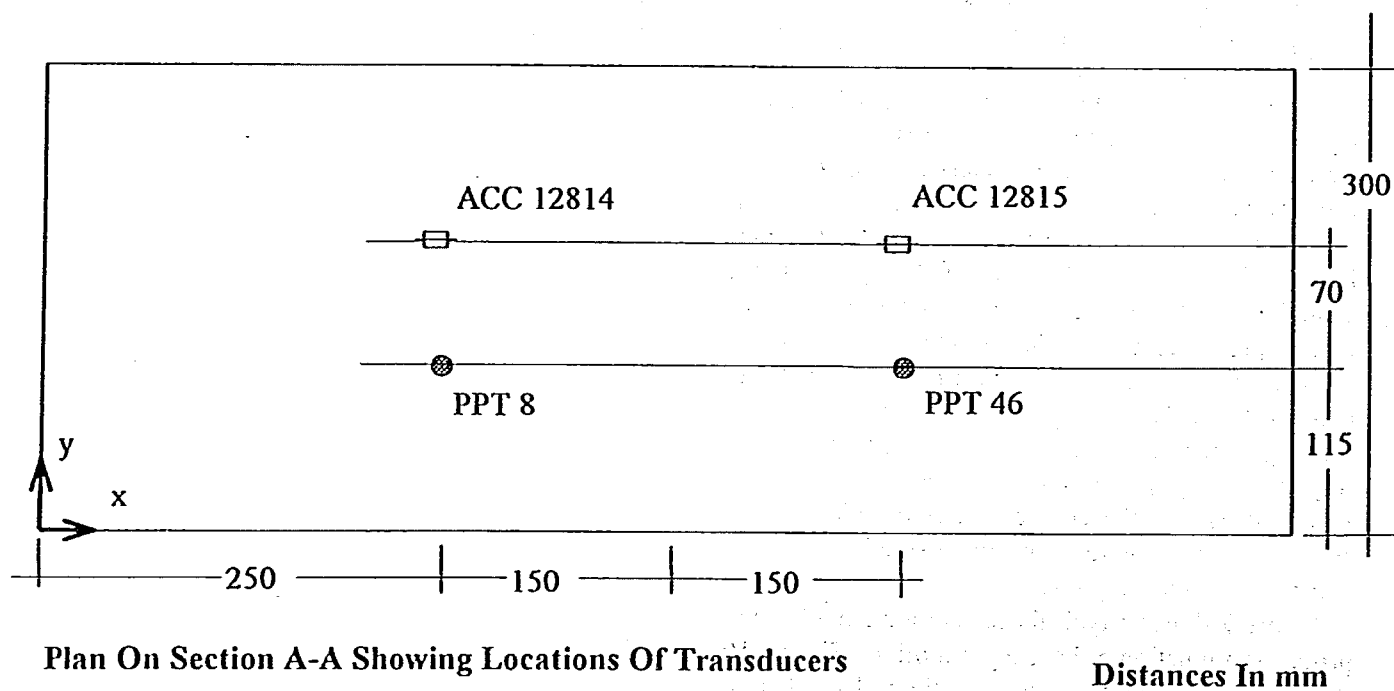
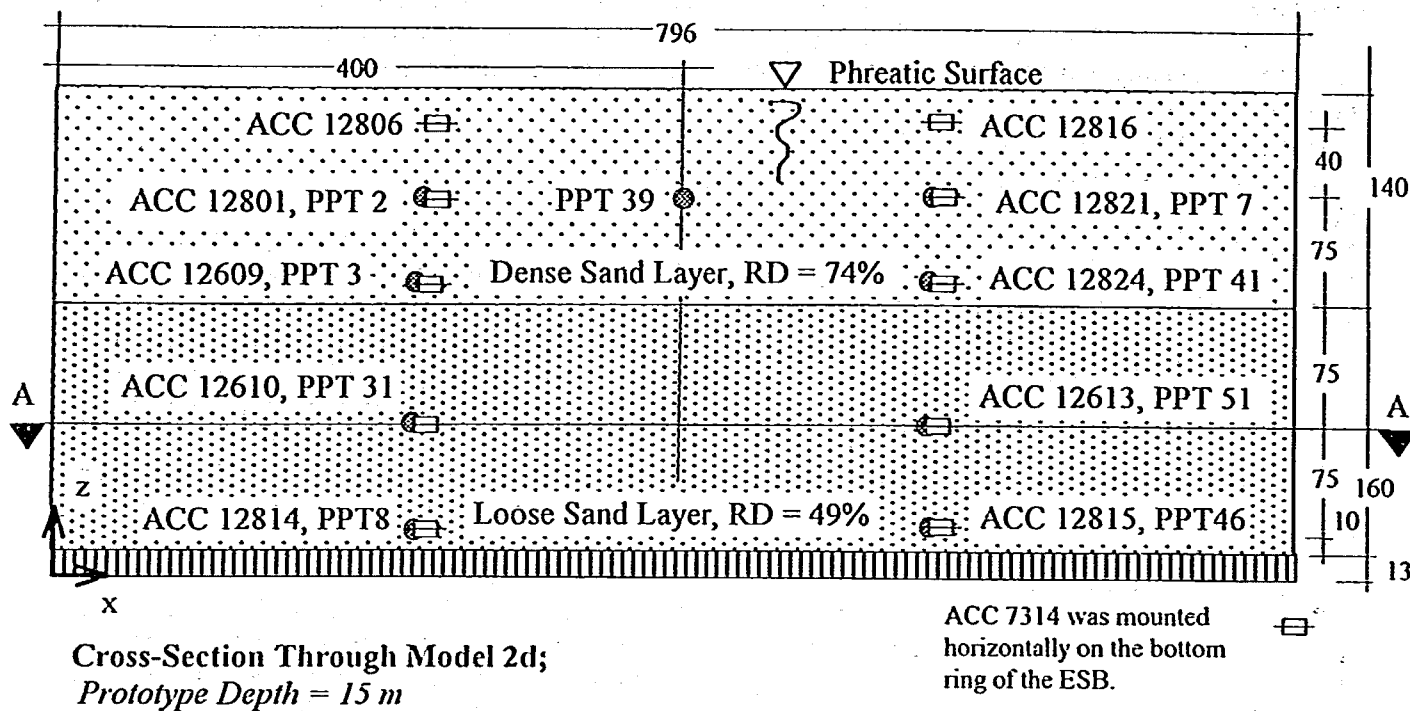


Figure 6. Typical model cross-section for 95.8 kPa effective overburden.

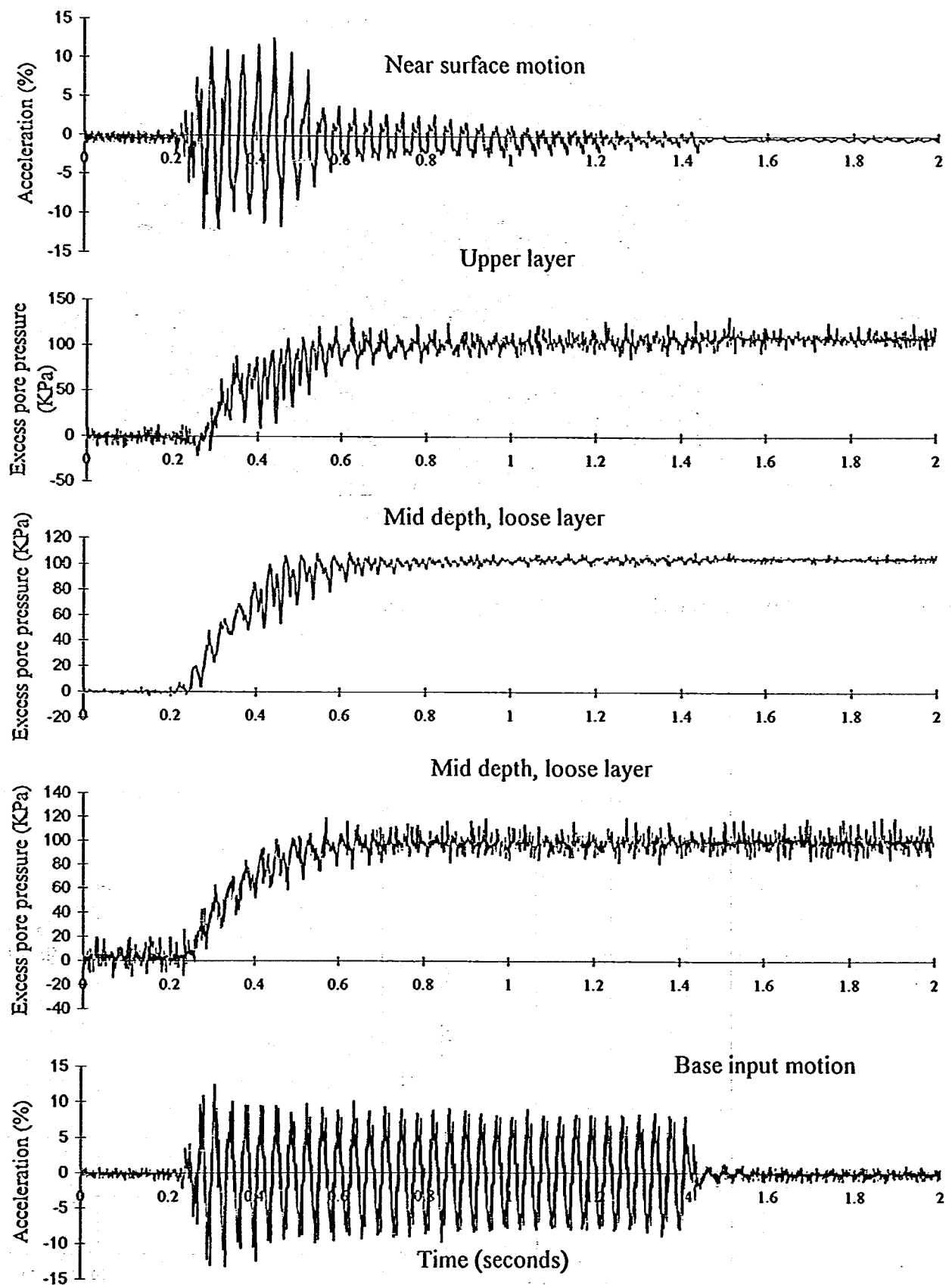


Figure 7. Time history raw data from pore pressure transducers and accelerometers.

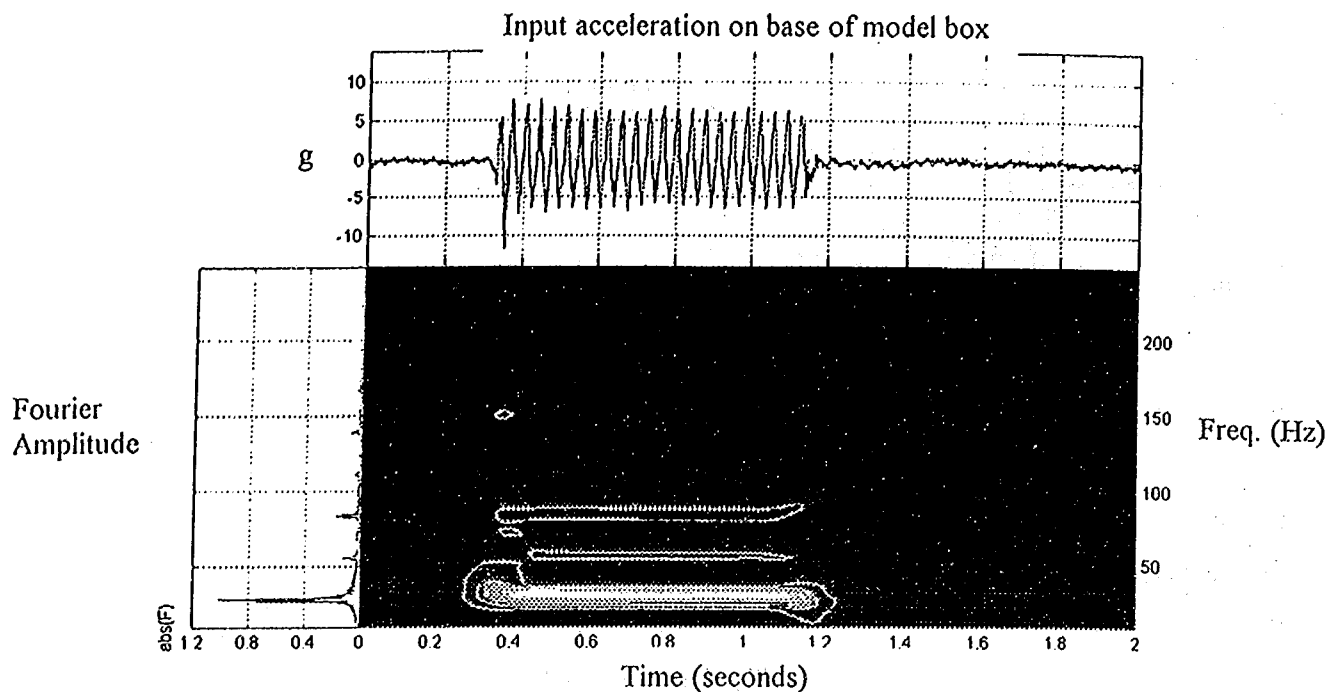


Figure 8. Fourier analysis and wavelet map of a base input motion.

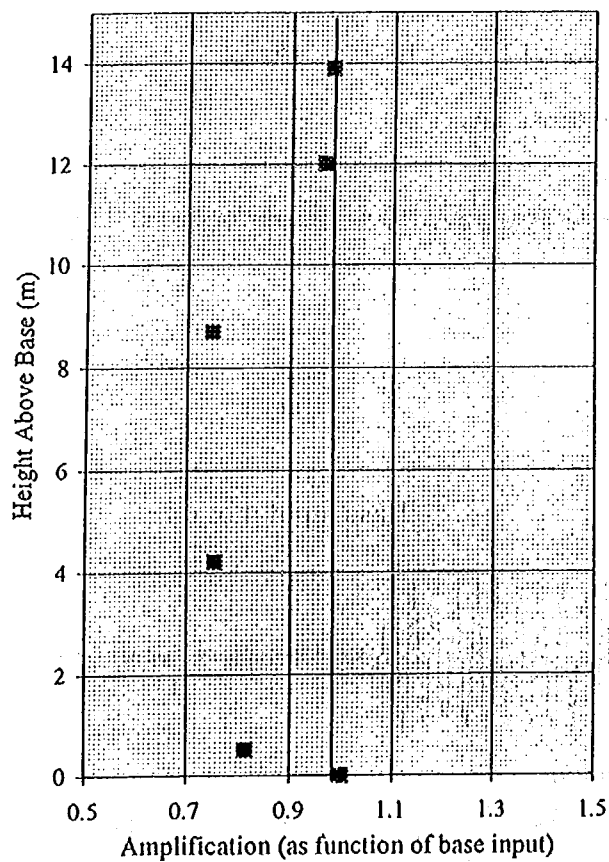


Figure 9. Typical acceleration profile.

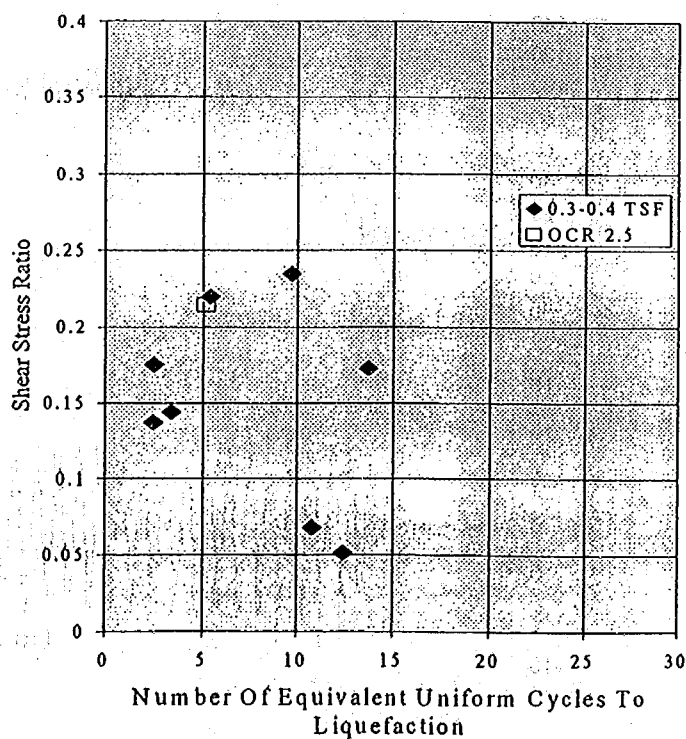


Figure 10. Liquefaction data at 0.36 tsf overburden.

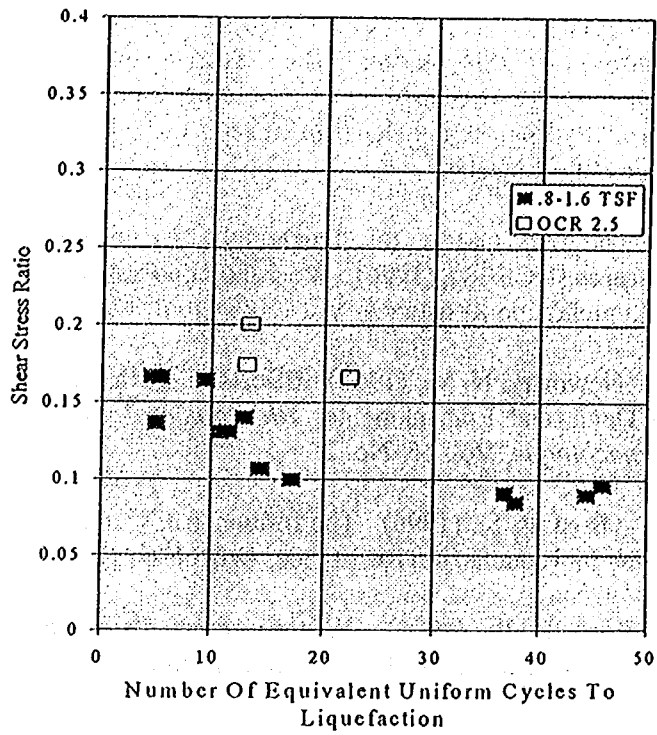


Figure 11. Liquefaction data at 1-1.5 tsf overburden.

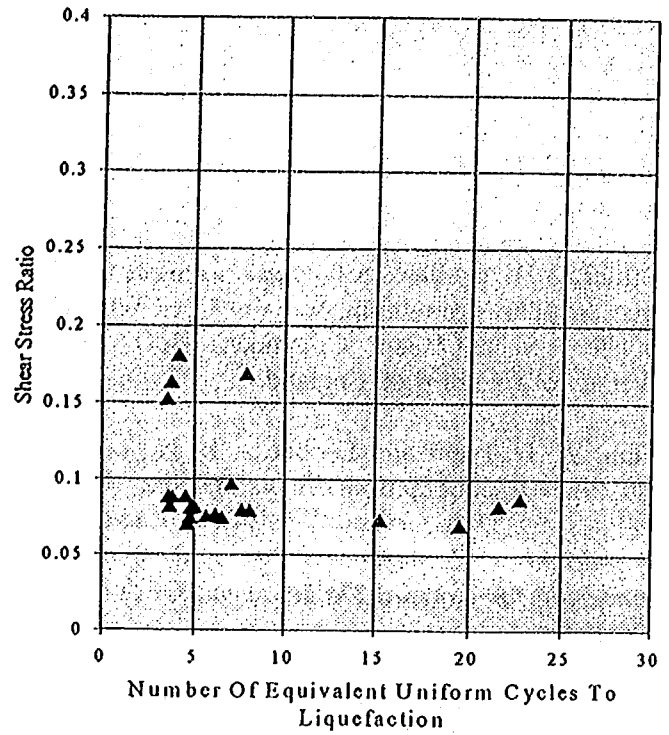


Figure 12. Liquefaction data 2-2.26 tsf overburden.

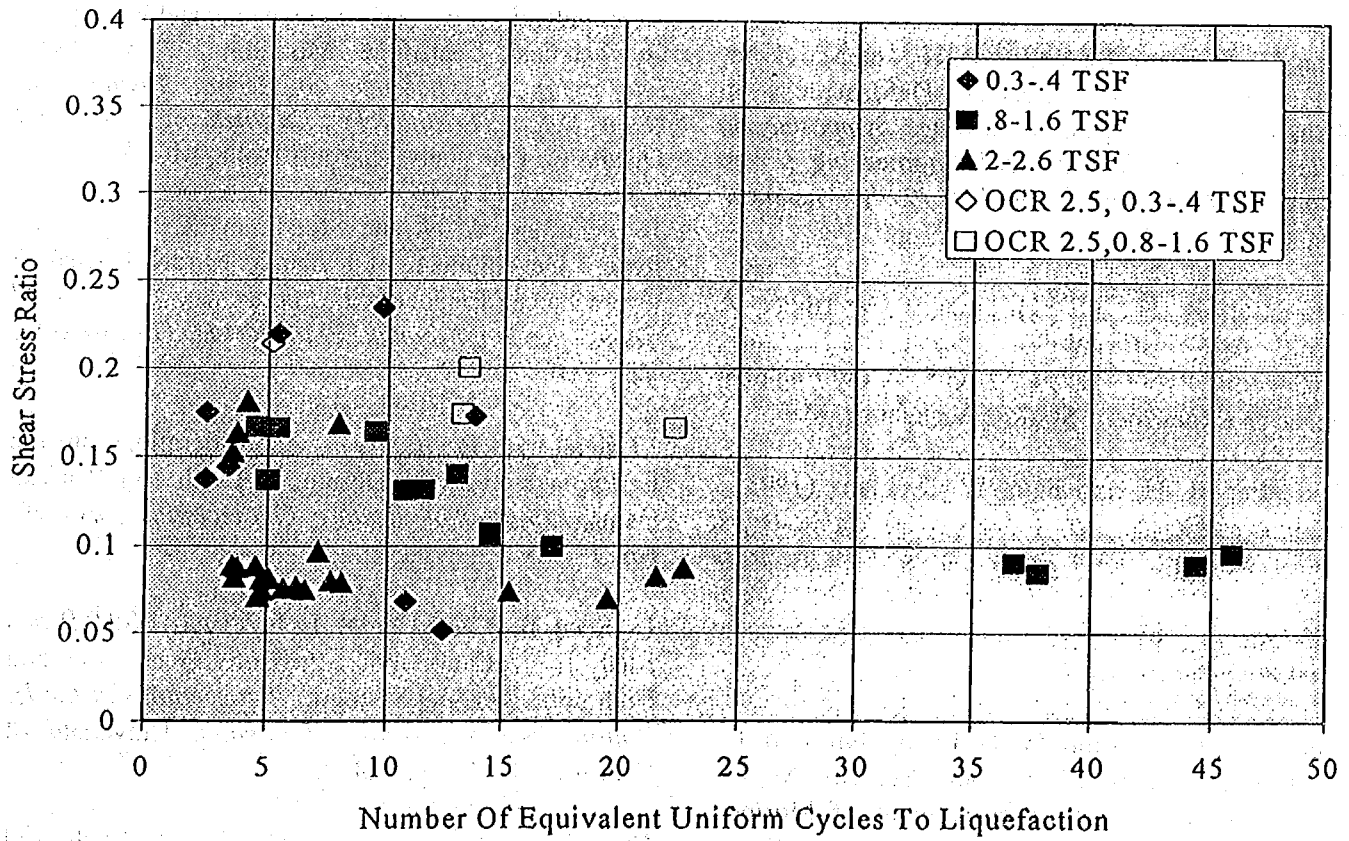


Figure 13. Summary of liquefaction data from centrifuge experiments.

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Centrifuge Research Center  
U. S. Army Engineer Waterways Experiment , Vicksburg MS, USA

The Centrifuge Research Center, located at the Waterways Experiment Station (WES), supports the Corps of Engineers and other Federal Agencies and collaborates with industrial, commercial, academic, and international organizations to address novel and demanding engineering and scientific problems worldwide.

The centrifuge can recreate a wide range of field phenomena and environments under laboratory conditions, generating realistic data to verify and validate computer simulations and engineering analyses. Conditions imposed in the field are replicated in physical models that are subject to the full prototype self-weight stresses and strains.

Table 1. Key characteristics of the Army centrifuge.

|                     |             |
|---------------------|-------------|
| Gravity range       | 10 to 350 g |
| Platform radius     | 6.5m        |
| Payload at 143g     | 8000kg      |
| Payload at 350g     | 2000kg      |
| Centrifuge capacity | 1144 tonnes |

The Army centrifuge is unique in its range of capabilities and breadth of research applications. Primary thrusts of research are in the areas of:

- Blast effects
- Contaminant and groundwater migration
- Earthquake response of earth structures
- Vehicle mobility and pavements
- Physics of frozen soils and water
- Hydraulic and coastal processes
- Soil-structure-interaction

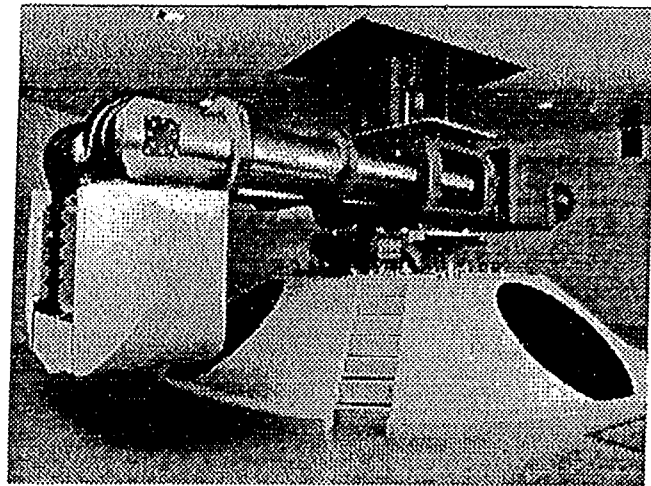


Figure 1. WES centrifuge.

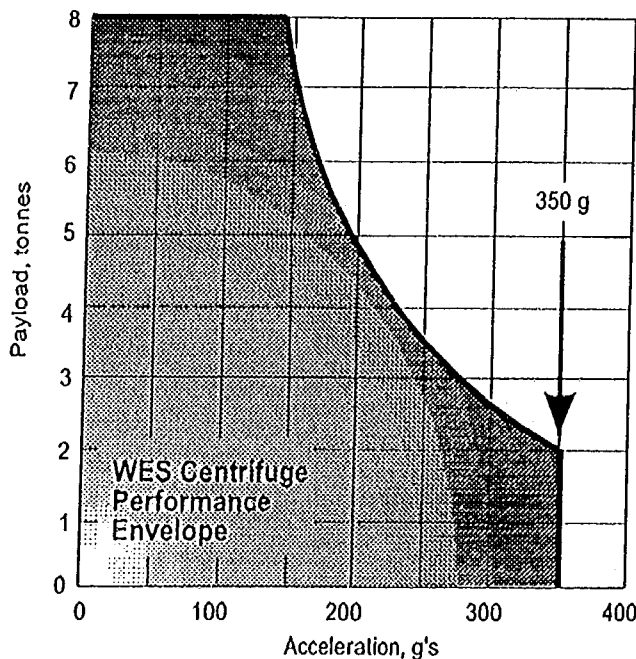


Figure 2. WES centrifuge performance envelope.

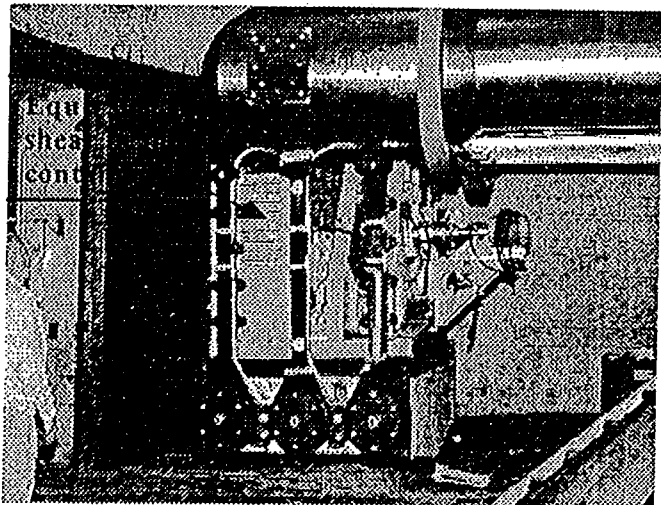


Figure 3. Dynamic load actuator, stored angular momentum, and equivalent shear beam container.

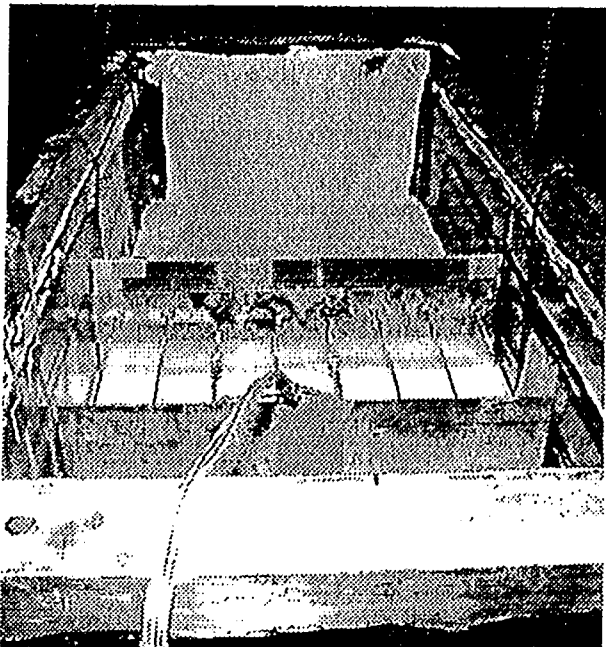


Figure 4. Model sheet pile wall with anchor wall; dynamic load experiment.

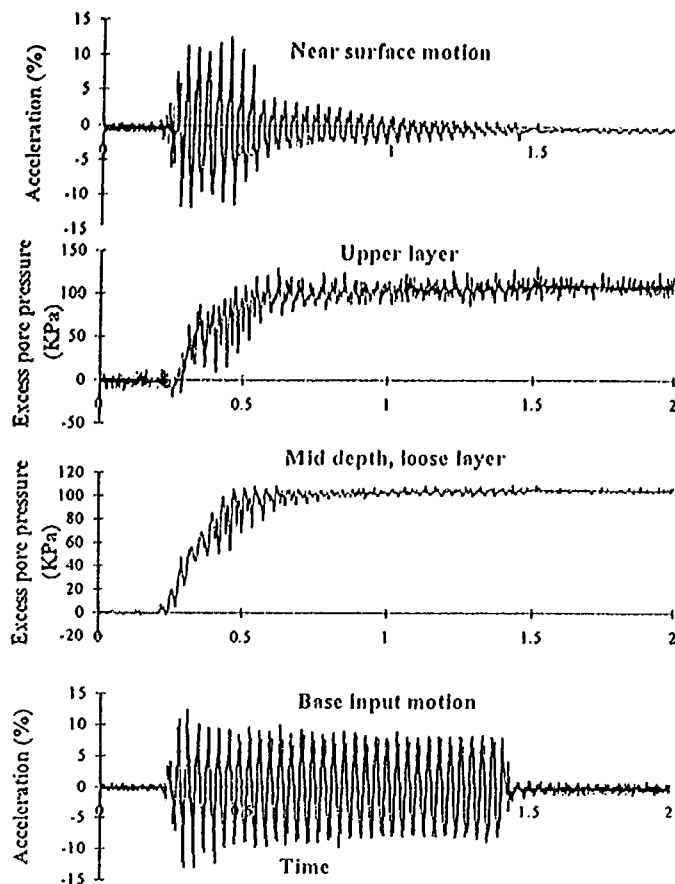


Figure 5. Dynamic load experiment time history raw data from pore pressure transducers and accelerometers.

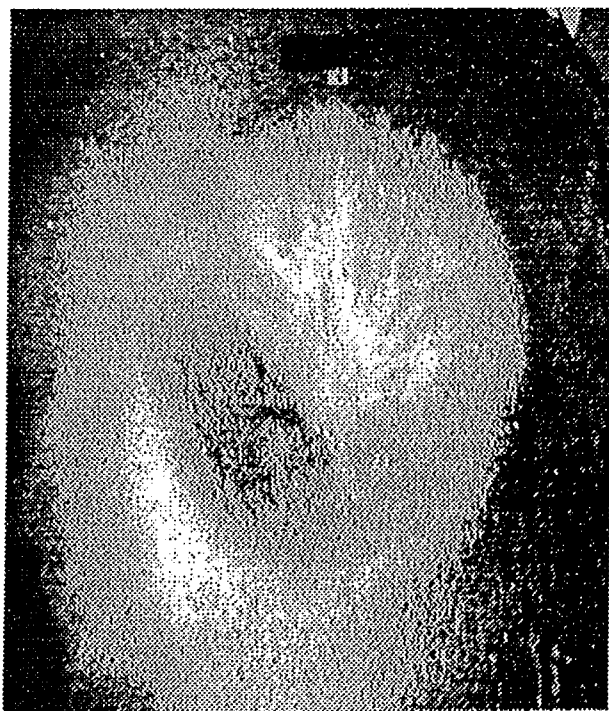


Figure 6. Precision-scale cratering experiment conducted at 50 g centrifugal acceleration.

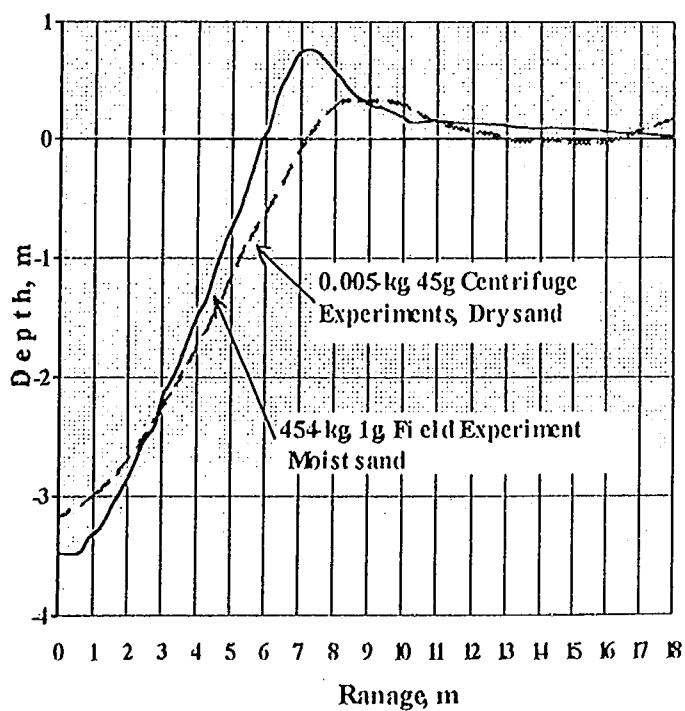


Figure 7. Comparison of crater profiles, field and centrifuge precision-scale experiments

# SESSION 7

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- *W. Vanadit-Ellis*: The U.S. Army Centrifuge: Earthquake Investigations
- *T. Namilkawa*: A Study on Building Foundation with Deep soil Cement Mixing Method for Thermal Power Plant - Verification of the Design Procedure by Centrifugal Experiments -
- *R. Ledbetter*: Research with the U.S. Army Centrifuge



Wipawi Vanadit-Ellis, Gary D. Butler

*U.S. Army Engineer Waterways Experiment, Vicksburg, MS, USA*

## Introduction

Modern centrifuge technology has afforded engineers the opportunity to study the dynamic behavior of soils including the phenomena of earthquake induced liquefaction. Such studies provide insight into the overall soil-fluid structure response with a degree of flexibility that cannot be acquired with large scale field testing. Usage of these modeling techniques by the US Army Corps of Engineers focus on the design, construction, and remediation of structures in areas susceptible to earthquake perturbations. Today, numerous U.S. Army facilities around the world, both above and below the ground surface and including critical and non-critical structures, have been and will be located in areas of high seismicity. With this in mind, dynamic site response criteria and analysis methods are needed to evaluate existing facilities and provide design criteria for future construction. Based on such research needs, investigations in the areas of dynamic soil behavior, soil-structure interaction, and earthquake engineering are now a major thrust of the Army centrifuge.

## Shaker System

The Army centrifuge has a self-contained shaker system that has been designed

specifically for use with the Acutronic 684-1 centrifuge, Figure 1. The Corps current centrifuge earthquake modeling capabilities are based on years of centrifuge modeling experience from worldwide industrial and academic institutions. The evolution of these techniques to the current state-of-the-art has now produced the capability to investigate the dynamic behavior of soil structures at large depths with a consistency that previously could not be achieved. This is accomplished using the combination of new techniques with respect to input motion, model boundary conditions, data actuation acquisition, and digital signal analysis.

The actuation system comprises two major components. One is a support system for the shaker that is mounted between the centrifuge booms and the other is the shaking unit that is mounted directly on the centrifuge platform. The system mounted between the booms is an air driven lubrication system for the shaking unit. The primary function of the system is to supply a high-pressure oil/air mixture to the high-speed bearings of the system, which are used to support the drive shaft containing the rotating flywheel assembly. Secondly, the system also contains a set of solenoid valves that control the delivery of air pressure to the systems pneumatic centering mechanism that repositions the model container after each shake.

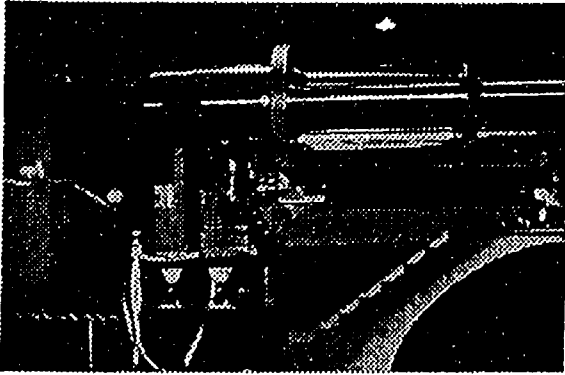


Figure 1. Shaker system on swing platform.

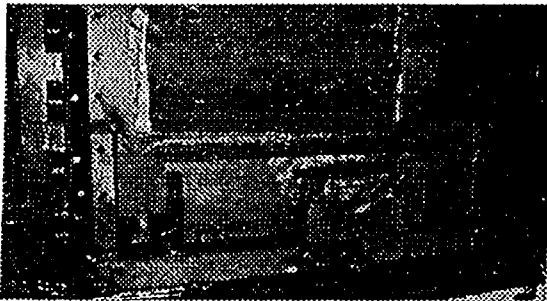


Figure 2. Shaking platform.

The swing of the centrifuge contains the primary shaking system. This system is composed of the driving motor, flywheels, drive train, and shaking platform. All components are mounted to the large steel base of the structure, which approximately covers the total surface area of the centrifuge's swinging platform. The system was designed to rest on a 4 inch thick layer of rubber padding which serves to isolate vibrations created in the actuator from the centrifuge, Figure 2. As seen in Figure 3, a schematic shows the principal components of the shaking system. An electric motor is used to power the drive shaft of the system to the desired operational frequency. The drive shaft is composed of two flywheels, two counterbalance units, and a timing disc for speed control. The middle of the shaft incorporates an eccentric cam with a throw

of 10mm. As the shaft rotates, a linkage serves to transmit the reciprocal motion from the eccentric to a rocker arm, which in turn causes its rotation around a fixed pivot point.

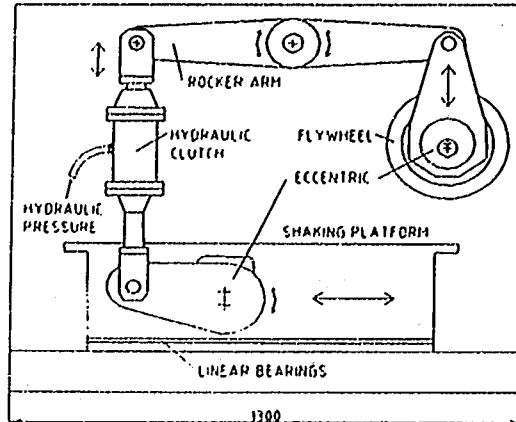


Figure 3. Schematic of the WES SAM system.

To control the displacement amplitude, the system was manufactured with two different rocker arms. The output motion from the rocker arm results in the reciprocation of a large rod inside a hydraulic clutch. To deliver the shaking energy to the model container, the hydraulic clutch must quickly couple the motion from the rocker arm with the remaining sections of the shaker's drive train. The clutch has the ability to transfer a maximum 5.5 tons of axial thrust, which is dependent upon the input hydraulic pressure, and coefficient of surface friction within the clutching system. Once the clutch is engaged, the flywheel energy is transferred as a sinusoidal motion through the remainder of the system's drive train to the shaking platform of the system. Based on the setting of rocker arm the input motion can have plus or minus values of 49mm, 1.47mm, and 4.41mm.

## Model Container

It is desired that during the centrifuge earthquake event, dynamic stresses and strains are created in the soil model that are similar to those found in a soil layer of infinite lateral extent and finite depth under dynamic prototype conditions. The scaling of the dynamic event is well known (Steedman, 1995) and shows that by increasing the  $N$  scaled models shaking frequency by  $N$  at  $N g$ 's will reproduce the dynamic shear stress and strain distribution seen at the prototype scale. The creation of these stresses and strains are the result of vertical and horizontal shear forces imposed on the soil during horizontal base shaking. Their magnitude and frequency are a function of both the input motion and the soils physical properties. Ideally, these forces should be identical on every vertical plane during excitation.

To properly simulate these effects, the centrifuge earthquake model container must help to sustain the complimentary shear stresses in the soil model during the shaking event. Failure to do so leads to the creation of distortions in the overall stress field which results in a dynamic rocking effect in the soil. To avoid this and other container boundary effects, Schofield and Zeng developed the design criteria for an ideal model container for earthquake centrifuge modeling (Schofield and Zeng, 1989). The result was the equivalent shear beam container.

The design of the equivalent shear beam container was based on the assumption that an isotropic column of soil responds as a continuous system and can be idealized as an equivalent shear beam. Therefore, it was reasoned that a container

and corresponding soil structure with an equivalent dynamic stiffness would respond with similar shear displacement profiles under base excitation, reducing the dynamic soil-container interaction effects.

The ESB was constructed from 11 rectangular aluminum frames separated by flexible layers with shear sheets attached to the end walls. The base length of the container is 1198 mm and the base width is 358 mm with a maximum soil and fluid carrying capacity of 336 kg. The rubber stiffness was designed so that during base shaking, the end walls will have the same deflection as the liquefied soil in the model container. Direct shear and dynamic tests were performed on various rubber specimens as a method of matching the lateral stiffness of the model container and the soil, Figures 3 and 4.

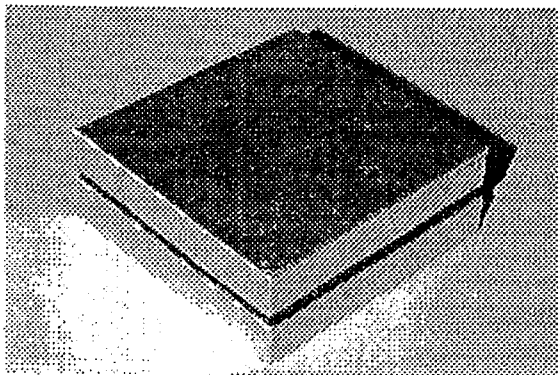


Figure 3. Rubber specimen for direct shear test.

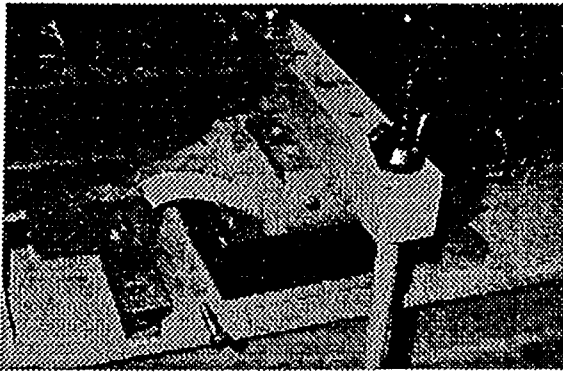


Figure 4. Direct shear test.

The shear modulus of the rubber is determined as a function of anticipated cyclic strain and excess pore pressure. The chosen specimen, with an acceptable shear modulus for the construction of the ESB, was automobile windshield sealant. Despite the hysteresis effects, the combination of a moderately high shear modulus and excellent sealing properties make it the best material for construction of the ESB, Figure 5.

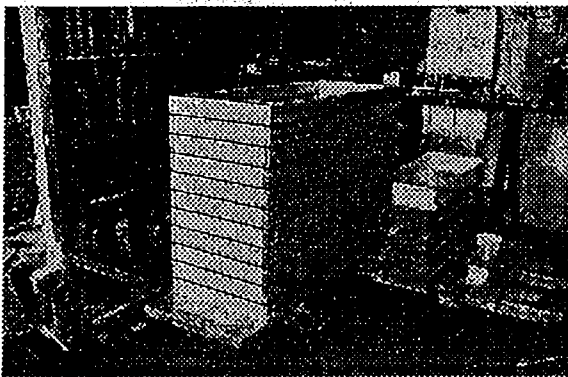


Figure 5. ESB model container.

This design seems to satisfy most of the requirements for modeling soil behavior; different boxes may be needed for to match different soil stiffness being tested.

#### Data Acquisition System

The Army centrifuge has 2 identical

earthquake data acquisition system, Figure 6.

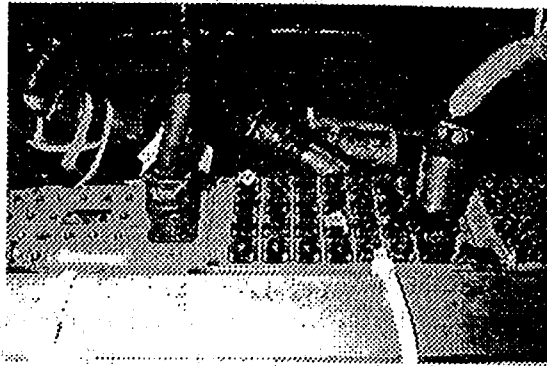


Figure 6. Onboard data acquisition system.

Each capable of recording 32 channels, measuring differential signals at maximum sampling rate of 5000 samples per second per channel. The input spans  $\pm 10$  volts measuring 16 bits of resolution. Typically in earthquake centrifuge modeling, one data acquisition system measures up to 32 accelerometers and the sister system measures up to 32 other transducers. The size of the Army centrifuge allows the data acquisition hardware to be physically mounted on the centrifuge between the booms and close to the test specimen; this proximity to the test specimen reduces possible noise generation that can occur when hardware is mounted further away. An onboard computer that collects, stores, and downloads the data from memory to a host computer for post analysis controls the hardware.

#### Method of Analysis

The fast Fourier transform (FFT) is one of many techniques of analyzing a time-varying signal  $f(t)$  by filtering out its frequency components. However, if at some point in the lifetime of  $f(t)$ , there is a local oscillation or a changing frequency

response, this will contribute to the calculated Fourier transform  $F(\omega)$ , but its location in time will be lost. There is no way of knowing whether the value of  $F(\omega)$  at a particular  $\omega$  derives from frequencies throughout the life of  $f(t)$  or just a few selected periods. This applies to acceleration time histories taken from centrifuge earthquake models. During such tests, the shear stiffness of the soil structure changes with void ratio, effective stress and shear strain (Schofield and Zeng, 1992). The rate and magnitude of the changing properties depend on the initial boundary conditions as well as the character of the base input motion. These changes in physical properties therefore result in a non-linear frequency response of the soil whose changing dynamic characteristics are lost with the FFT. Such deficiencies of Fourier analysis are overcome with the use of wavelet analysis which provides an alternative way of breaking a signal down into its constituent parts of frequency as a function of time (Newland, 1993).

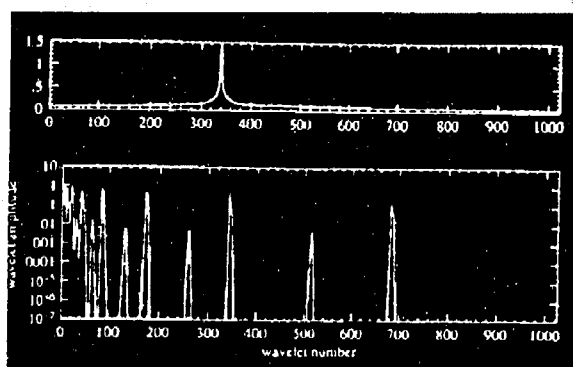


Figure 7. Discrete wavelet transform.

Plots in Figure 7; (a) is an arbitrary test function, sampled on a vector length 1024, (b) is the absolute value of the 1024 wavelet coefficient produced by the discrete wavelet transform (DWT). The

dotted curve plots the same amplitudes when sorted by decreasing size. The (DWT), like the FFT, is a fast, linear operation that operates in a data vector whose length is an integer power of two, transforming it into a numerically different vector of the same length. Also like the FFT, the wavelet transform is invertible and in fact orthogonal. Both FFT and DWT, therefore, can be viewed as a rotation in function space, from the input space (or time) domain to a different domain. For the FFT, this new domain has a basis functions that are the familiar sines and cosines. In the DWT, the basis functions are called wavelets. Unlike sines and cosines, individual wavelet functions are quite localized in space; simultaneously, like sines and cosines, individual wavelets are localized in frequency or more precisely characteristic scale. Unlike sines and cosines, which define a unique Fourier transform, there is not one single unique set of wavelets; in fact there are many possible sets. Roughly, the different sets of wavelets make different trade-offs between how compactly they are localized in space and how smooth they are. DWT, using harmonic wavelets, provides a good method for analyzing complex transient vibration data generated by earthquake experiments. Outputs from DWT analysis is a contour plot, Figure 8, with a corresponding mesh diagram, Figure 9, which shows the dominant effect of frequency. The emphasis of this method is that it is size-dependent, multi-resolution and allows researchers to invest resources on part of the signal that has the most information.

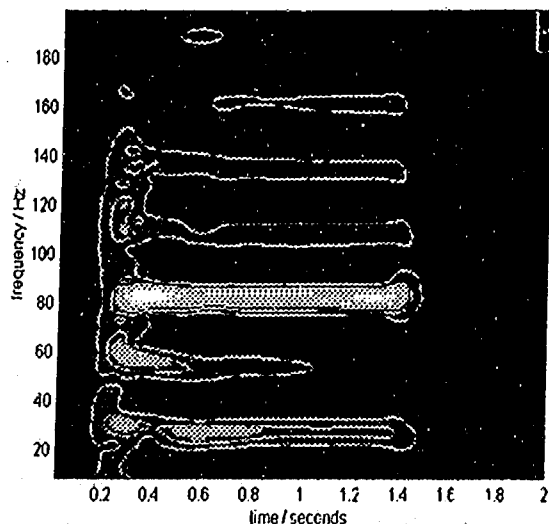


Figure 8. Time-frequency map of changing frequency.

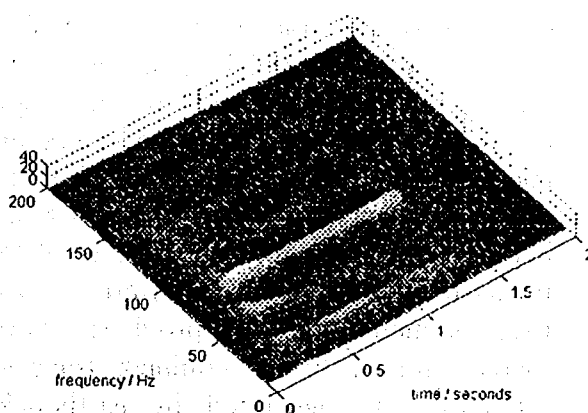


Figure 9. Corresponding mesh diagram for the contour plot in Figure 8.

## Conclusion

Extensive research activities to study seismic response have been successfully conducted on the Army centrifuge since the facility's inauguration in 1996. Verifications of systems were conducted in the early phase of operation and were modified accordingly. One such modification is an innovative design, patent is pending, to overcome self-weight

problem that prevents solenoid valves from functioning properly at high acceleration. A series of calculations and tests concerning the design stiffness of the equivalent shear box were also performed that led to the reconstruction of the box using windshield sealant. The electric motor that is currently being used on the Army centrifuge is being replaced by an hydraulic motor, again, to overcome the self-weight problem that plagues electrical motors at high accelerations. Continuing research efforts in the area of earthquake centrifuge modeling are shared with other U.S. Army Corps of Engineers Districts and academia.

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A STUDY ON BUILDING FOUNDATION WITH DEEP SOIL CEMENT MIXING METHOD  
FOR THERMAL POWER PLANT  
- DESIGN PROCEDURE AND ITS VERIFICATION BY EFFECTIVE STRESS ANALYSIS -

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**Abstract**

Design procedure of "improved soil ground (ISG)" using the Deep soil Mixing for large Building (DMB) method is proposed in order to solve the recent requirement of the construction cost reduction for thermal power plants. It makes possible to use the direct foundation avoiding the pile foundation with liquefaction countermeasure such as the sand compaction piles. This method was introduced several years ago for small buildings but was not applied to large buildings since there were not enough data of experiments and real projects. Since then related data have been accumulated. Therefore the design studies have started recently. A trial construction was conducted. This report presents numerical studies of the design procedure using the data and the dynamic effective stress analysis, which can take into account ground liquefaction phenomena. The numerical study was supported by the centrifugal experiment and reported in the separate paper.

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## Introduction

Thermal power plants are mostly constructed near the seaside in Japan. The soil ground there consists of sandy layers. Liquefaction potential of these area is usually high. On the other hand demands of cost effective construction for the plant is also high. The foundation accounts for large proportion of the construction cost. In order to solve this requirement for the cost reduction, the Deep Soil Mixing method for large Building (DMB) method was developed. The DMB method is the method to harden all the soil ground under a building by the Deep Soil Mixing method (DSM method). It makes possible to use the direct foundation avoiding the pile foundation with liquefaction countermeasure such as the sand compaction pile.

The DSM method is the method of mixing soil ground with cement using a vertical lod which has few propelas at the tip as shown in Figure 1. At the very beginning of the DSM development, it was used to harden the sludge at sea bed but not used for structural purpose. It is a recent development to use the DSM for the structures such as foundation of buildings and bridges. There are 2,795 case histories of structural use till 1997 (CDM Research Group, 1997).

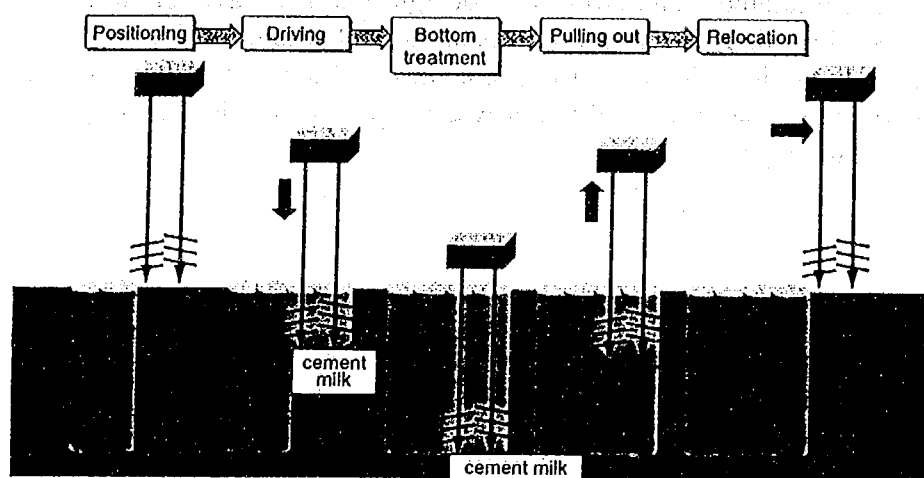


Figure 1. Soil improvement method of deep soil mixing

Design procedure for the large structures such as a thermal power plant has not been established yet. Therefore the structural safety of the DMB method studied. A trial construction has been conducted to verify the quality of the ISG. The material properties were obtained from the ISG. In order to verify the safety of DMB method subjected to earthquake, we used two approaches. One is the finite element method based on effective stress analysis, which can take into account liquefaction during earthquake. The other is a simple calculation. This simple calculation method was chosen as a design method of the DMB method after the method is proven to give equivalent results to the finite element analysis.

## Outline of Numrical Study

Numerical studies have been made to determine the seismic intensity acted on the ISG and to study structural safety of the ISG made by the DMB method. Parameter survey has been done in respects of thickness of the ISG, stiffness of the ISG, building types and input motions. Two real sites of shallow and deep ground were chosen for the soil layer. Heavy old power plant and light thermal power plant building were chosen for the building types.

Table 1 shows cases for the study of the seismic intensity. In this study, influence of earthquake was mainly surveyed. Table 2 shows cases to evaluate the structural safety of the ISG.

**Table 1 Cases for seismic intensity**

| depth to bearing layers | Stiffness of the Improved soil ground (kPa) | model | Rinkai 1992 (Tokyo Bay area) | Japan Architectural Center 1994 | EI Centro 1940 NS | TAFT 1952 EW |
|-------------------------|---------------------------------------------|-------|------------------------------|---------------------------------|-------------------|--------------|
| shallow layer           | 588,000                                     | 1-D   | O                            | O                               | O                 | O            |
|                         |                                             | 2-D   | O                            |                                 |                   |              |
|                         | 882,000                                     | 1-D   | O                            | O                               |                   |              |
|                         |                                             | 2-D   | O                            |                                 |                   |              |
| deep layer              | 588,000                                     | 1-D   | O                            | O                               |                   |              |
|                         |                                             | 2-D   | O                            | O                               |                   |              |

**Table 2 Cases for safety evaluation of the improved soil ground**

|            | depth of improved ground | depth of base foundation | width of improved ground | stiffness of the improved soil ground (kPa) | building | model        | Equivalent linear analysis | Effective stress analysis |
|------------|--------------------------|--------------------------|--------------------------|---------------------------------------------|----------|--------------|----------------------------|---------------------------|
| Short dir. | -12.5m                   | 5.9m                     | 77m                      | 588,000                                     | Heavier  | SYM59W77-G6  | O                          | O                         |
|            |                          |                          |                          | 882,000                                     |          | SYM59W77-G9  | O                          |                           |
|            | -30.0m                   | 4.8m                     | 72m                      | 588,000                                     |          | SYM48W72-G6  | O                          | O                         |
|            |                          |                          |                          | 588,000                                     |          | DYM48W72-G6  | O                          | O                         |
|            | -12.5m                   | 5.9m                     | 77m                      | 588,000                                     | Lighter  | SCM59W77-G6  | O                          |                           |
| Long dir.  | -12.5m                   | 5.9m                     | 356m                     | 588,000                                     | Lighter  | SCM59W356-G6 | O                          |                           |

**Table 3 Properties of the shallow ground layer model**

| Layers        | Thickness (m) | Wet density | Cohesion (kPa) | $\phi$ (degree) |
|---------------|---------------|-------------|----------------|-----------------|
| Sand          | 1.4           | 1.90        | 0              | 35              |
| Silty sand    | 11.1          | 1.90        | 0              | 35              |
| Bearing layer |               | 1.98        | 0              | 40              |

## Numerical Methods Used

The dynamic effective stress FE analysis code MuDIAN was used in this numerical study to take into account the liquefaction phenomena. That analysis code is based on Biot's theory for two-phase material (Biot 1956) and uses the densification model for the constitutive law (Shiomi et al, 1993).

## Numerical Modeling and Properties

The configuration of the foundation and the thermal power plant is shown in Figure 2. The size of the ISG is about 350m long, 60m wide and 12m height. Average overburden of the building including the basemat is about 200 kPa.

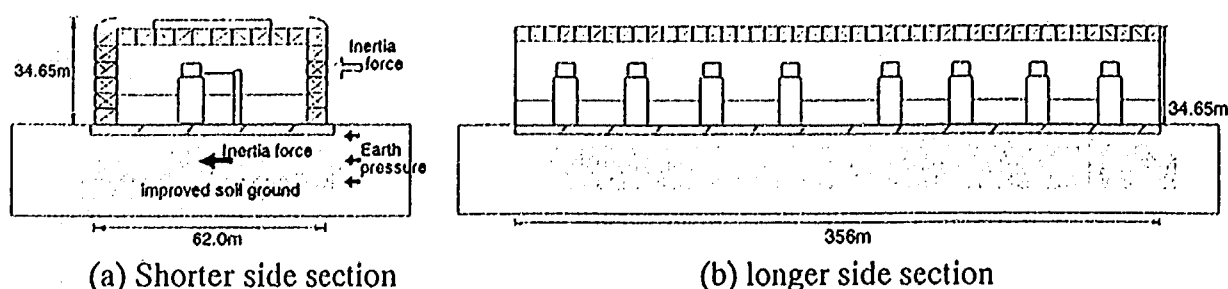


Figure 2. Outline of a thermal power plant

## Numerical models

Finite element mesh of shallow soil layer model is shown in Figure 3. Properties of the lighter building are summarized in Table 4 and Table 5. The building is modeled by shear beam elements, which shear modulus was 7.94 kPa. The building basemat was assumed rigid.

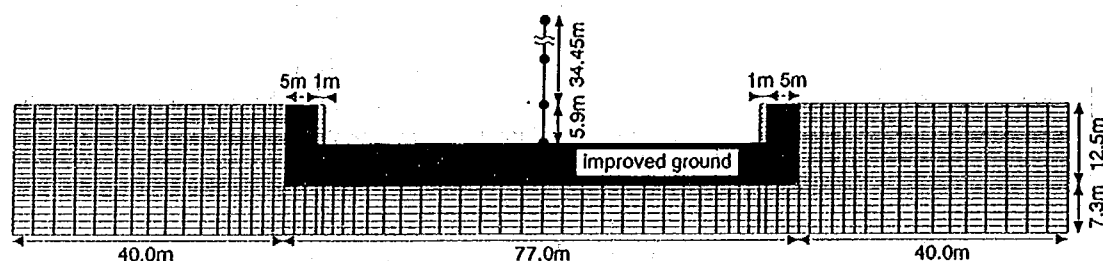


Figure 3 Configuration of numerical model and mesh (shallow layer model)

Table 4 Equivalent beam model of a building

| Element No. | 1         | 2         | 3         | 4         | 5        | 6       |
|-------------|-----------|-----------|-----------|-----------|----------|---------|
| Height(m)   | 29.1~34.4 | 24.0~29.1 | 18.5~24.0 | 13.0~18.5 | 7.0~13.0 | 0.0~7.0 |
| Long dir.   | 92.4      | 57.5      | 165.2     | 74.1      | 108.5    | 153.3   |
| Short dir.  | 3.53      | 8.51      | 9.5       | 11.2      | 17.4     | 25.8    |

(sectional area, unit:  $\times 10^{-4} \text{m}^2/\text{m}$ )

Table 5 Property of mass

| Elem. No.  | 1    | 2     | 3    | 4    | 5     | 6     | 7     | 8     |
|------------|------|-------|------|------|-------|-------|-------|-------|
| Height(m)  | 34.5 | 29.1  | 24.0 | 18.5 | 13.0  | 7.0   | 0.0   | -5.9  |
| Long dir.  | 42.6 | 187.4 | 88.0 | 46.6 | 241.5 | 222.6 | 1914. | 2681. |
| Short dir. | 7.82 | 34.4  | 16.1 | 8.5  | 44.3  | 40.8  | 363.5 | 528.9 |

(unit : Mg/m)

## Properties of soil layers

Soil properties were obtained from the real sites where the thermal power plants were located. Initial shear modulus  $G_0$  was determined from the shear wave velocity test. Reduction curve of shear modulus  $G$  vs. strain was obtained from the element tests of the soil samples. Figure 4 shows the  $G/G_0$  curve at the depth G.L. - 4.06m for the shallow ground layer model. The shapes of the reduction curves for the other depth were almost same. The strain dependency of damping

factor was also obtained from the same test. Poisson ratio was assumed as 0.3. The soil properties are summarized in Table 6

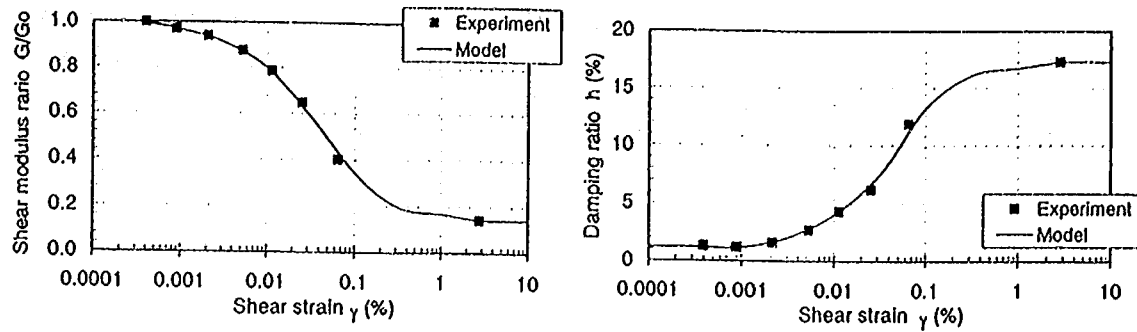


Figure 4 G/Go and h curve G.L. -4.06m

Table 6 Soil properties

| Soil type  |     | depth (m) | N-value | density (Mg/m³) | Share wave velocity (m/sec) | Poisson's ratio | Friction Angle (degree) | Cohesion (kPa) | Permeability (cm/sec) | RI20 ( $\sigma_d/2\sigma_v$ ) | Soil particle density | Porosity |
|------------|-----|-----------|---------|-----------------|-----------------------------|-----------------|-------------------------|----------------|-----------------------|-------------------------------|-----------------------|----------|
| fine sand  | SFs | -1.40     | 11      | 18.60           | 150                         | 0.33            | 38.6                    | 0.01           |                       |                               | 2.729                 | 0.4806   |
|            |     | -2.70     | 11      | 18.60           | 150                         | 0.33            | 38.6                    | 0.01           | 4.50E-3               | 0.3263                        | 2.729                 | 0.4806   |
|            |     | -4.40     | 7       | 18.60           | 150                         | 0.33            | 38.6                    | 0.01           | 4.50E-3               | 0.2537                        | 2.729                 | 0.4806   |
|            |     | -5.30     | 3       | 18.60           | 150                         | 0.33            | 38.6                    | 0.01           | 4.50E-3               | 0.1705                        | 2.716                 | 0.4767   |
| silty sand | SFs | -6.80     | 3       | 19.47           | 160                         | 0.33            | 38.6                    | 0.01           | 4.50E-3               | 0.1652                        | 2.716                 | 0.4248   |
| fine sand  | As1 | -8.80     | 3       | 19.47           | 160                         | 0.33            | 45.2                    | 7.64           | 8.79E-3               | 0.3857                        | 2.716                 | 0.4248   |
| fine sand  | As2 | -11.90    | 27      | 17.50           | 360                         | 0.33            | 38.4                    | 9.60           | 5.73E-3               | 0.1922                        | 2.692                 | 0.5355   |
| silty clay | Dc1 | -13.05    |         | 19.35           | 360                         | 0.33            | 15.6                    | 428.26         | 1.00E-6               | 0.2524                        | 2.713                 | 0.4314   |
| fine sand  | Ds1 | -14.60    |         | 19.35           | 360                         | 0.33            | 41.8                    | 27.05          | 1.23E-2               |                               | 2.713                 | 0.4314   |
|            |     | -19.80    |         | 17.61           | 430                         | 0.33            | 37.2                    | 39.59          | 5.28E-4               |                               | 2.679                 | 0.5253   |

### Properties of improved soil ground

Material properties of the ISG were obtained from samples of a trial construction. The average shear modulus at 60 days after construction was about 1,058MPa. Considering that the value of 588MPa often was used for design, two cases of 588MPa and 882MPa were studied. Poisson ratio 0.33 and damping coefficient 0.035 were used. Strength of the ISG determined based on the uni-axial strength. The average strength of the uni-axial tests was 2.94MPa. The design strength  $F_{sc}$  is third of fifth. Material strengths for design are shown in Table 7.

Table 7 Material strength of the deep mixing soil

|                 | Compression (kPa)   | Shear (kPa)                                 |
|-----------------|---------------------|---------------------------------------------|
| Permanent load  | $1/3 F_{sc}$ (588)  | $0.75 \times 1/5 \times 1/3 F_{sc}$ (88.2)  |
| Seismic level 1 | $2/3 F_{sc}$ (1176) | $0.75 \times 2/5 \times 1/3 F_{sc}$ (176.4) |
| Seismic level 2 | $F_{sc}$ (1764)     | $0.75 \times 1/5 F_{sc}$ (264.6)            |

## Input motions

Input motions used in this study were El Centro1940, Taft 1952, Rinkai 1992 and JAC 1994. Rinkai 1992 is an artificial earthquake defined as an input motion at Edogawa Layer ( $V_s = 300 - 500$  m/sec) in Tokyo Bay area. JAC 1994 is also an artificial earthquake for the seismic design defined by Japan Architectural Center in 1994. Figure 6 shows the time history and the response spectrum of the Rinkai 1992.

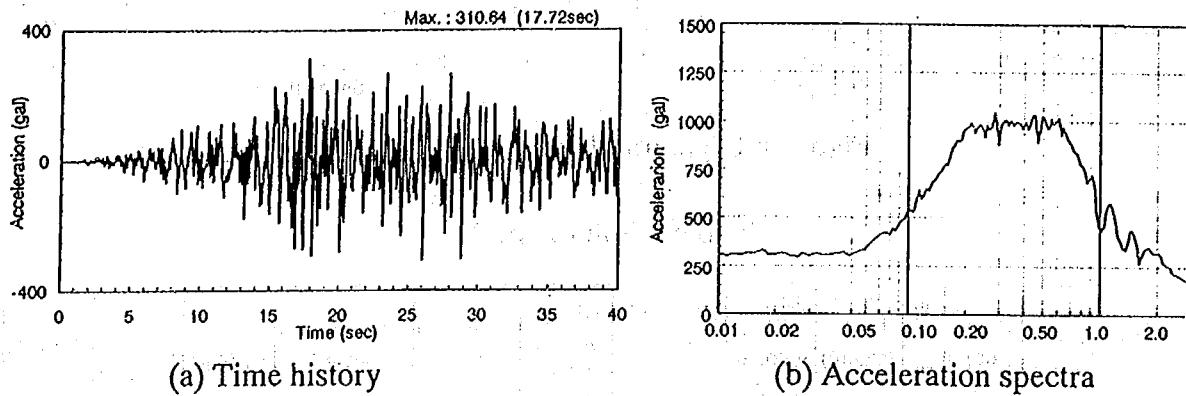


Figure 5 Rinkai 1992 artificial earthquake

## Evaluation of ISG Safty

Numerical results are critically examined for the safty of the ISG in respect of the items shown in Table 8. The load to be used for the design procedure was determined.

Table 8 Items for safety check of improved soil ground

|             |                 | Design level of the improved soil ground                             |                                                                                                        |
|-------------|-----------------|----------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------|
| Condition   | Design level    | For permanent load                                                   | Seismic level 2 for building                                                                           |
| Check items | Design load     | Load of building<br>Self-weight of the ISG                           | Load from building<br>Seismic intensity<br>Self-weight of the ISG<br>Earth pressure (skeleton + water) |
|             | External safety | Reaction stress of bearing layer                                     | Reaction stress of bearing layer<br>Slip of ISG, Overturning of ISG                                    |
|             | Internal safety | Max. pressure<br>Reaction force to building<br>Vertical shear stress | Max. pressure<br>Reaction force to building<br>Horizontal shear stress<br>Vertical shear stress        |

ISG : Improved Soil Ground

## External force

Maximum external forces acted on the ISG were examined. There were three external forces to be considered. They are inertia force due to self-weight and seismic intensity, earth pressure due

to static and seismic force, and force from building. At first, the seismic intensity for Tokyo Bay area was studied. Load from building was determined from the earthquake response analysis for building in which the base boundary was fixed.

### Seismic intensity

Seismic intensity was determined from the maximum response acceleration of the ISG subjected to the Rinkai 1992 earthquake. The earthquake was defined as the level two earthquake around Tokyo Bay area. The results are shown in Figure 6. Figure 6 (a) shows the maximum response acceleration profile in the different location of the ISG. The maximum acceleration ranged from 248gal to 299gal. Figure 6(b) shows the maximum response acceleration normalized by the acceleration at the input motion applied. The maximum amplification factor was 1.12. Seismic intensity for the ISG was determined as 0.3 to cover these results.

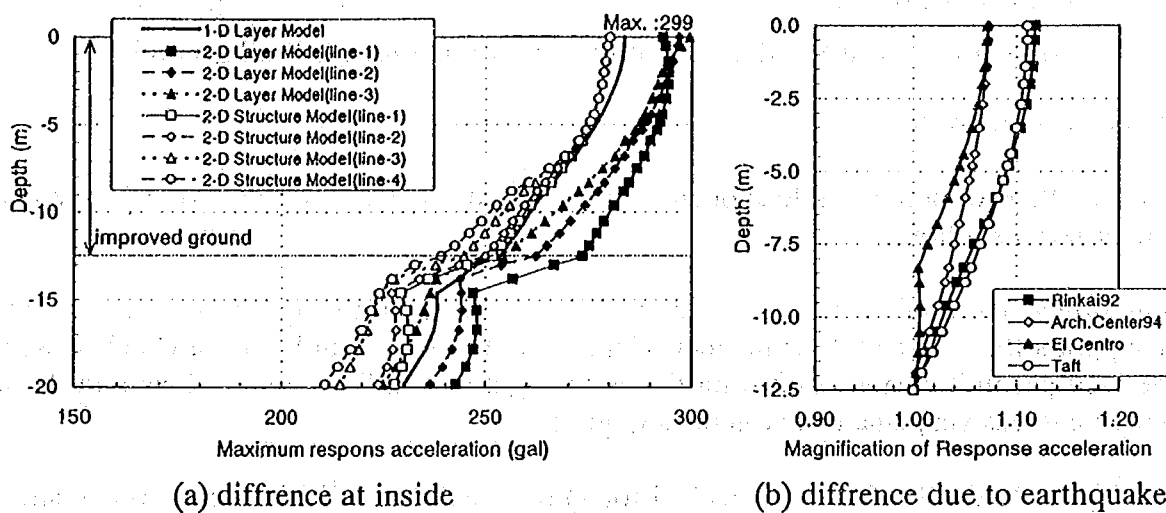


Figure 6 Maximum response acceleration

### Earth pressure due to liquefaction

External force acts on the side boundary of the ISG was evaluated for the case that the surrounding soil layer was liquefied. Figure 7(a) shows the maximum pressure profile. The left-hand side figure shows the excess pressure profile and the dynamic component of the total earth pressure (excess pressure + dynamic horizontal component of the effective stress) that acted on the left and right hand sides respectively. The maximum profile of the total earth pressure acted on the ISG was calculated by subtracting the earth pressure acted on the left hand side from the one acted on the right hand side at each time step and taking the maximum of them. The profile is shown in Figure 7(b). It is interesting that the maximum earth pressure profile of each side was almost same as the maximum force profile acted on the ISG. This maximum force was slightly higher than the earth pressure that was obtained the empirical Westergaard's formula for dynamic water pressure. The maximum pressure profile did not occur at the same time, so that the earth pressure in liquefaction case of the side soil layers for the static design analysis can be determined by the Westergaard's formula.

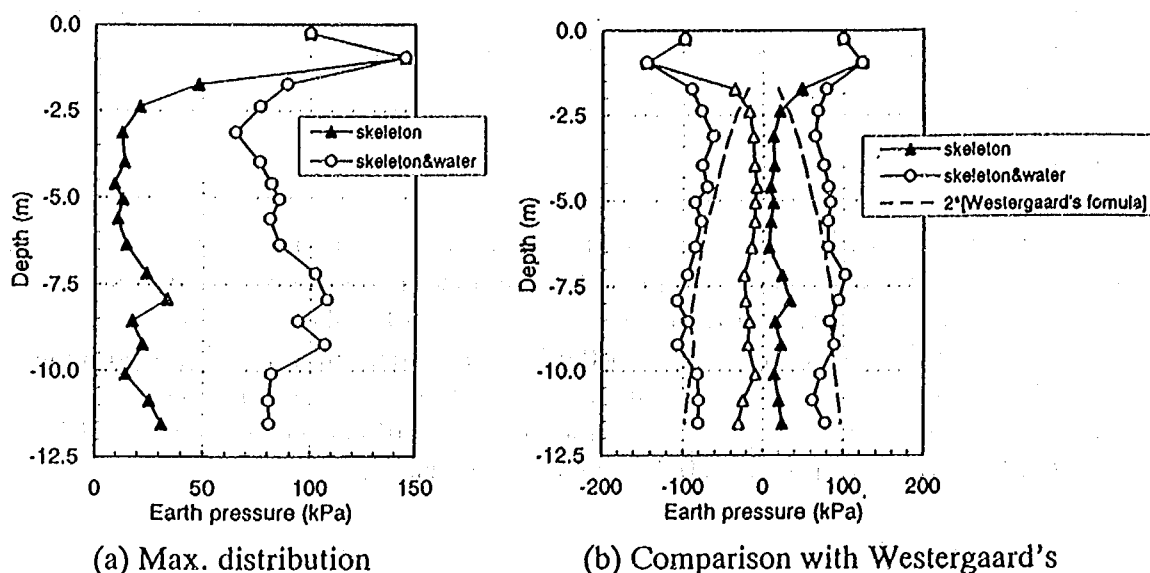


Figure 7 Pressure at the side boundary of the improved soil ground

### Safety for permanent load

Safety of the ISG for permanent load obtained by the static analyses was not critical. The results are summarized as follows.

*External safety.* External safety was examined for 1) the bearing capacity of the soil layers, which supports the ISG and 2) settlement of the ISG. The first item has been checked with an average load per unit area accumulating the self-weight of buildings, foundations and the ISG. The settlement was very small and can be negligible.

*Internal Safety.* Internal safety was checked for 1) the maximum effective compressive stress in the ISG, 2) subgrade reaction to the building and--- 3) shear stress. The results of the FE analyses were about one third of the compression strength for 1) and 2). The shear stress is much less than the shear strength. The result data are summarized in Table 9.

### Safety for Seismic Load

#### External Safety

The external safety of the ISG was examined for 1) possibility of slip failure at the boundary between the ISG and the support soil layer, 2) subgrade reaction at the bearing layer and 3) overturning of the ISG. The safety factor for the slip failure was as large as 1.5 shown in Table 9. The safety factors for the subgrade reaction were about three times larger than the compression strength in the case of the shallow layer model. The possibility of the overturn was very little because of the building aspect ratio so this check item can be neglected from the design procedure.

#### Internal safety

In order to check safety of the ISG itself, the maximum stresses at the local point were checked. At first, normal stress was checked. The maximum normal stress naturally occurs at the bottom

edge of the building or of the ISG as shown in Figure 8. The profile of the normal stress along the horizontal line was almost straight including the effect of the building rotation although it was very little. This may be a typical behavior of the thermal power plant. Thus normal stress can be calculated from the gravity force of building and self-weight and rotational force from the building assuming the constant distribution of the force. This mechanism can be used for the design. The value obtained by the design procedure was reported later.

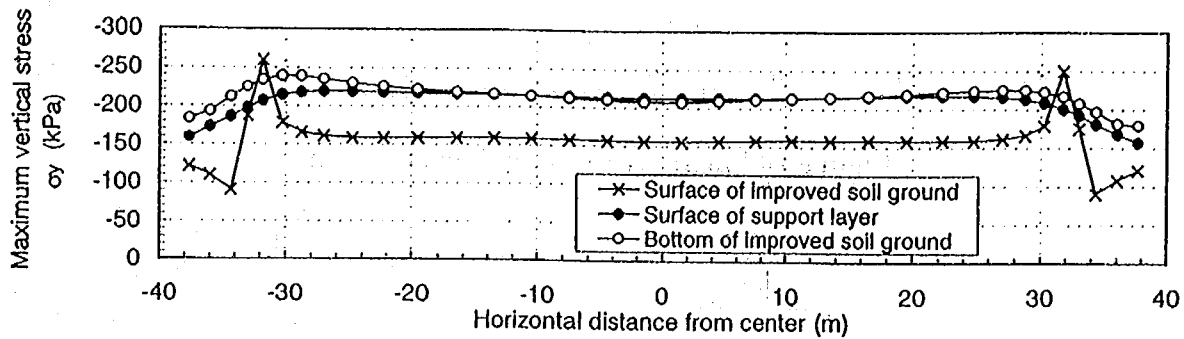


Figure 8 Stress distribution of vertical component

Figure 9 shows the maximum of the maximum-shear-stress at time. The largest contour line was nearly horizontal. So footing type of failure mechanism was not triggered by seismic force. Failure line may be horizontal near the bottom or vertical near the side edge of the ISG. The both were examined. Figure 10 shows the distribution along the horizontal line. As the slip line analysis, the accumulated shear stress per a unit length along the line was compared with the shear strength. In the vertical direction, the values were comparable and less than the failure strength as shown in Figure 11. In the horizontal direction, the result of the calculation was much higher than the material strength. Therefore it is concluded at this point, only vertical line can be checked in the design procedure.

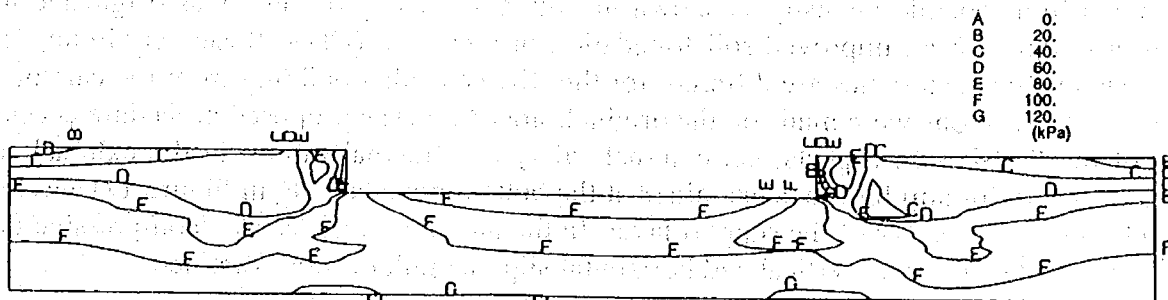


Figure 9 Maximum value of the maximum shear stress

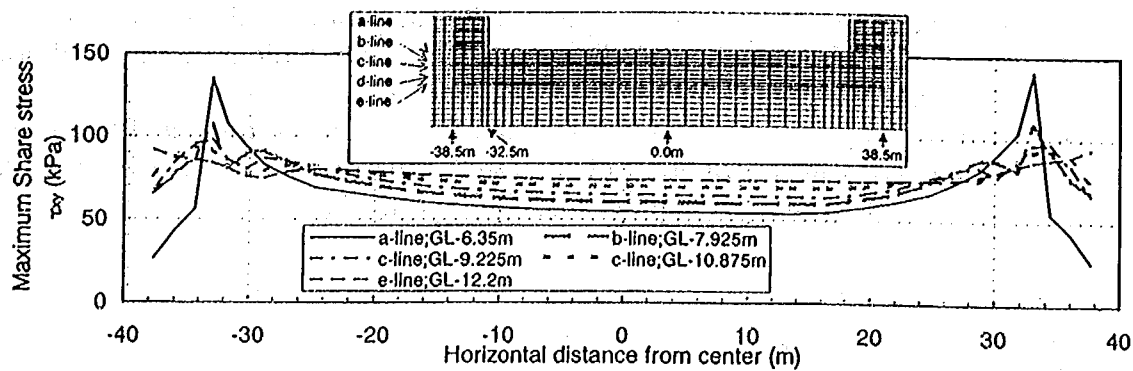


Figure 10 Shear stress along horizontal line

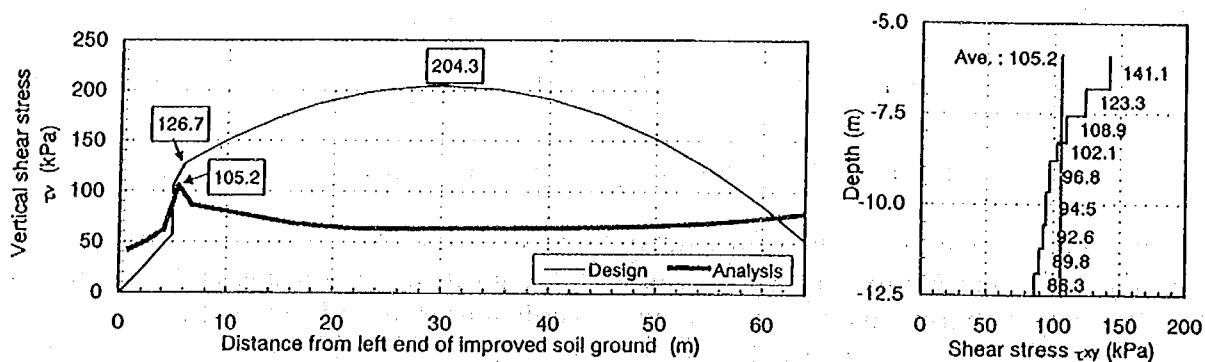


Figure 11 Shear stress distribution along the vertical line

## Design Procedure

The proposed design procedure is based on a kind of slip line analysis. Its external forces are determined from seismic intensity as shown in Figure 12. The procedure was originated in the design procedure for an improved soil foundation of a caisson (CDM Research Group, 1993). Since dynamic characteristics are different for the ISG beneath a building from the caisson type problem, modifications were made on the original procedure. The proposed procedure checks the safety of the ISG in two aspects; i.e. external safety and internal safety. In the external safety check, the safety for slip failure is examined at the bottom boundary ( $F_s$  in Figure 12) and for the subgrade reaction capacity of the support layer. In the internal safety check, "compressive failure of the improved soil" and "vertical and horizontal slip line failure" are examined.

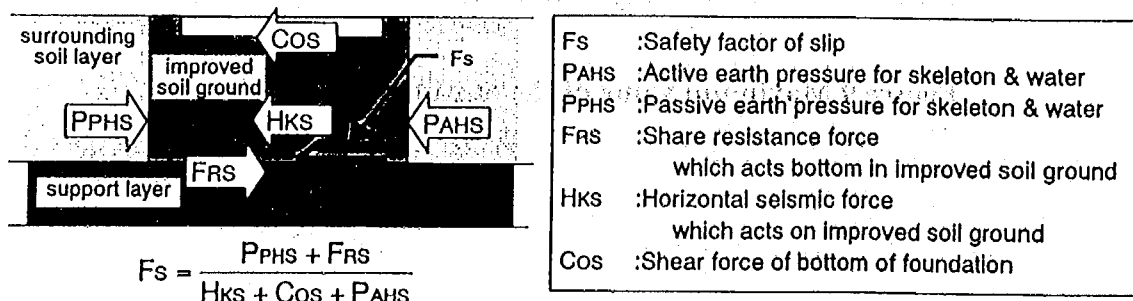


Figure 12. External force used in the design procedure

### Verification of design procedure

To examine of the design procedure, the all results obtained by the design procedure were compared with that of the results of FE analyses. As shown in Table 9(only results of seismic load are shown), the ratios (FE analyses. over design procedure) are very close to one, except the vertical shear stress. For the vertical shear stress, the design procedure may give too conservative results for some cases. Further research is necessary to improve the accuracy of the design procedure but the proposed design procedure can be accepted in respect of structural safety.

**Table 9 Comparison of design procedure and FEM**

| Check items               |                 |                                 | Ground kind   | Building type | Side  | Analysis | Design | Anal. /Design |
|---------------------------|-----------------|---------------------------------|---------------|---------------|-------|----------|--------|---------------|
| At lead-carrying capacity | External safety | Subgrade reaction for ISG (kPa) | Shallow layer | Lighter       | Long  | 221      | 224    | 0.98          |
|                           |                 |                                 |               |               | Short | 239      | 269    | 0.89          |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 293      | 326    | 0.90          |
|                           |                 |                                 |               | Heavier       | Short | 513      | 688    | 0.75          |
|                           |                 | Slip [safety factor]            | Shallow layer | Lighter       | Long  | 1.69     | 1.57   | —             |
|                           |                 |                                 |               |               | Short | 1.91     | 1.43   | —             |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 2.01     | 1.44   | —             |
|                           |                 |                                 |               | Heavier       | Short | 1.99     | 2.10   | —             |
|                           | Internal safety | Maximum contact pressure (kPa)  | Shallow layer | Lighter       | Long  | 221      | 224    | 0.99          |
|                           |                 |                                 |               |               | Short | 239      | 269    | 0.89          |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 290      | 326    | 0.89          |
|                           |                 |                                 |               | Heavier       | Short | 646      | 688    | 0.94          |
|                           |                 | Subgrade reaction (kPa)         | Shallow layer | Lighter       | Long  | 247      | —      | —             |
|                           |                 |                                 |               |               | Short | 310      | —      | —             |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 330      | —      | —             |
|                           |                 |                                 |               | Heavier       | Short | 310      | —      | —             |
|                           |                 | Horizontal shera force (kPa)    | Shallow layer | Lighter       | Long  | 81       | —      | —             |
|                           |                 |                                 |               |               | Short | 78       | —      | —             |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 92       | —      | —             |
|                           |                 |                                 |               | Heavier       | Short | 186      | —      | —             |
|                           |                 | ViERTICAL shera force (kPa)     | Shallow layer | Lighter       | Long  | 147      | 176    | 0.83          |
|                           |                 |                                 |               |               | Short | 105      | 205    | 0.51          |
|                           |                 |                                 | Deep layer    | Heavier       | Short | 116      | 241    | 0.48          |
|                           |                 |                                 |               | Heavier       | Short | 142      | 253    | 0.56          |

### Conclusion

The safety of the ISG by the deep soil mixing method for very large structures was examined by the static and the earthquake response analyses. The analyses concluded that the following stresses should be checked and are sufficient for the safety evaluation of the ISG. Following these results a design procedure based on seismic intensity method was proposed. The results of the analyses also reflected to determine the seismic intensity and other external forces such as the earth pressure to the side boundary of the ISG. The followings are the critical point for the safety and should be checked by the design procedure. 1) subgrade reaction of the ISG to bearing layer, 2) slip failure at the bottom boundary for external safety, 3) maximum contact pressure at the bottom of ISG, 4) the subgrade reaction and 5) average shear stress along vertical line for internal safety. It is also found that the soil ground improved by the deep soil mixing method can safely be constructed for the case the bearing layer is comparatively shallow layer.

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A STUDY ON BUILDING FOUNDATION WITH DEEP SOIL CEMENT MIXING  
METHOD FOR THERMAL POWER PLANT  
- VERIFICATION OF THE DESIGN PROCEDURE BY CENTRIFUGAL EXPERIMENTS-

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**Abstract**

The objective of this research is to experimentally verify the design procedure of improved soil ground by deep soil-cement mixing ground. The improved soil ground makes possible to use the direct foundation of a thermal power plant. The experiment was done by model tests under centrifuge acceleration field of 100G with the scale model 1/100<sup>th</sup> of actual size. Experimental research was conducted to determine the external forces during earthquake which act on the improved soil ground. Experimental model consisted of a turbine building, base mat, improved soil ground, liquefiable sand layer and bearing layer. This model was for the shorter section of the structure. The first natural frequency of the model was 1.8 Hz. Input motion was sinusoidal wave and the maximum acceleration was 200 gal at prototype scale. Acceleration of each part, pore water pressure of surrounding sand layer, earth pressure acting the improved soil, displacement of the improved soil and surface of the sand layer were measured. Test result showed that sand layers were liquefied and their maximum acceleration records were less than the bearing layer. There was permanent deformation at surrounding area of the improved soil. Lateral earth pressures at rest were changed. Both phenomena were largely due to liquefaction phenomena. The measured external forces act on the sides and bottom agreed well with the force which was calculated by the design procedure assuming that the seismic intensity for design is equivalent to the measured from the maximum acceleration of the top of the improved soil ground. Under the condition that the surrounding soil of the improved soil was liquefied, the measured distribution of the earth pressure acting to the improved soil ground showed good agreement with the one calculated by the design procedure.

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## Introduction

Cost effective construction method and structure systems are earnest demand for thermal power plants as mentioned so that a new method of construction for foundation was proposed. Its design procedure is examined by the numerical method but it is also necessary to verify its applicability with experimental studies. The thermal power plant is as large as 60m times 350m so that a scale down modeling is inevitable. To obtain better similarity rule for geotechnical problem, a centrifugal test is most recommended method for soil layer since confining pressure can be reproduced in the range of real site. A centrifuge apparatus with 6.5 m arm which can produce centrifugal force 500ton\*G is used. The models simulating soil ground and turbine building of a thermal power plant are set on the shaking table in the centrifuge apparatus. A transient tests were conducted with the shaking table. In this paper, the outline of the centrifuge apparatus, modeling, test conditions and results are reported.

## Outline of Instrument

### Centrifuge apparatus

At First, outline of centrifuge apparatus is briefly reported. Size and overview of the apparatus is shown in Figure 1 and Photo 1 respectively. It is a beam type centrifuge apparatus. A platform is installed at the tip of two beams and a counter weight is set at the other side of the beams. Test specimen is set on a platform. (Suzuki et al, 1994)

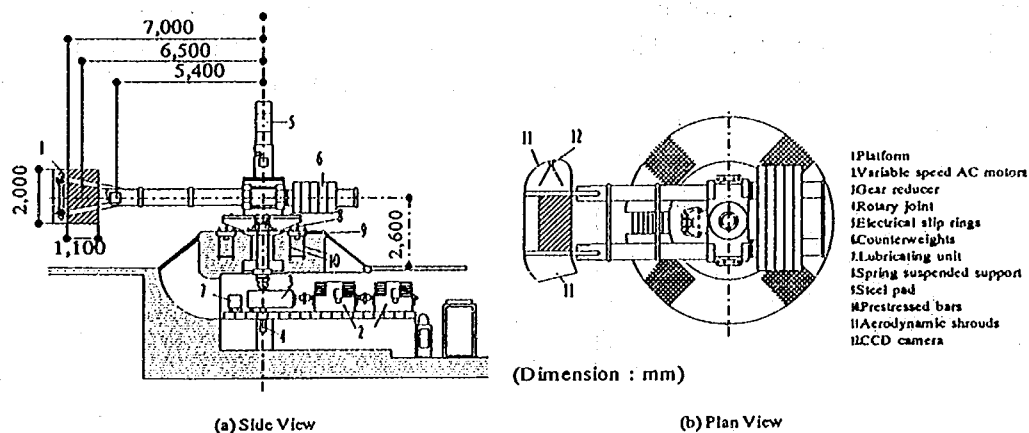


**Figure 1. Centrifuge apparatus.**

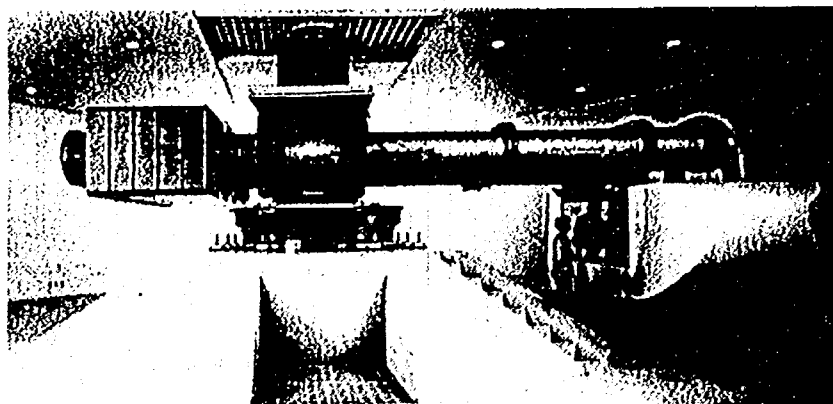


**Photo 1. Overview of Takenaka Centrifuge Apparatus.**

## Shear box

Configuration of the laminated shear box used here is shown in Figure 2. The size is 1002mm long, 302mm width and 345mm height. Inside of the share box is covered by thin rubber membrane ( $t=1\text{mm}$ ). Thickness of the shear ring is 15mm and laminated with a gap of 1mm. Flat cage bearings are installed between the share rings. Six braces are set to prevent perpendicular movement. This shear box is set on the shaking table.

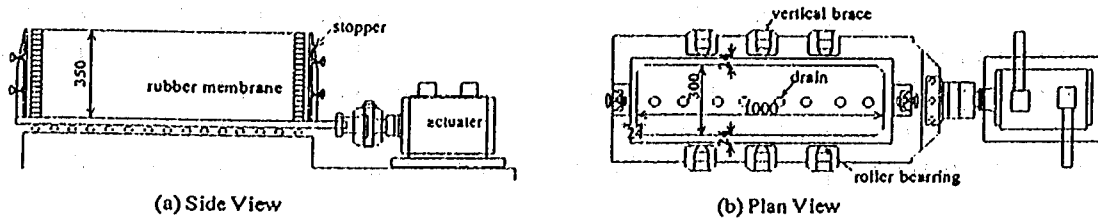


Figure 2. Configuration of shear box.

## Experimental Modeling

### Modeling

Two models were tested; i.e. a flat soil layer model and a soil structure model which includes the improved soil ground as shown in Figure 3. The first model was tested to survey the characteristics of the soil layers. The second model was for verification of the design procedure of the DMB (Deep cement and soil Mixing method for Building foundation) method of construction. (Masuda et al, 1997)

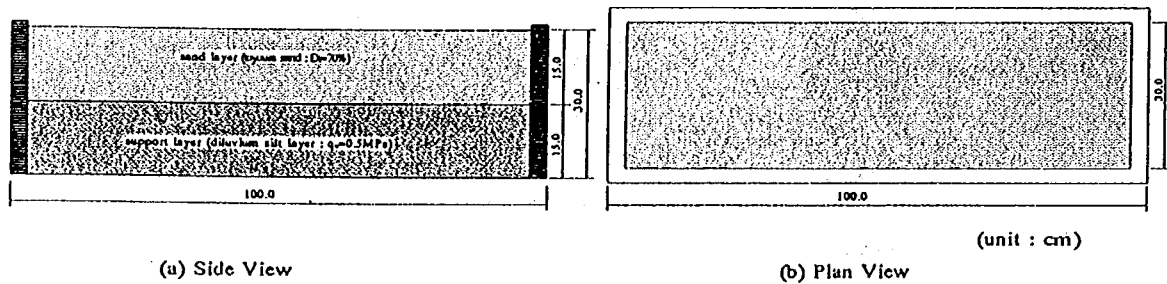


Figure 3. Soil layer model.

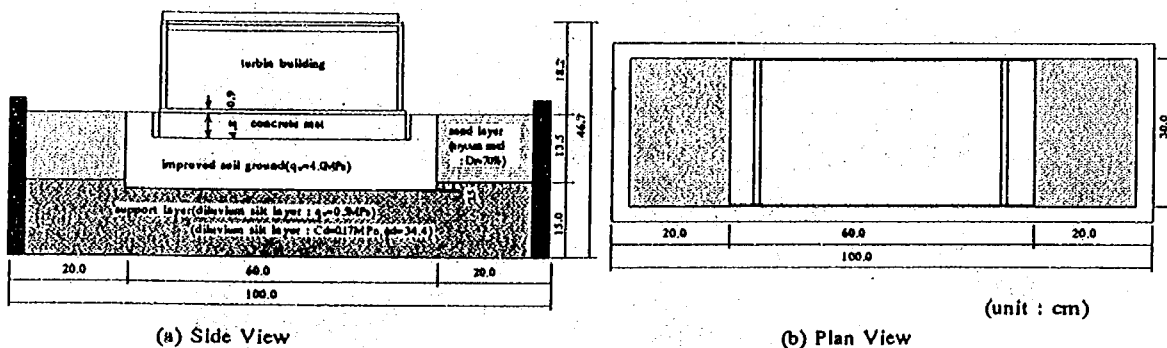


Figure 4. Soil structure model with improved soil ground.

The simple soil layer model consisted of two kinds of soil materials. The upper layer was made of Toyoura Sand (relative density is 70%) in which density liquefaction was expected. The lower layer was made of diluvium silt layer. Heights of the both layers were 15 cm. The soil structure model consisted of a turbine building, concrete mat and improved soil surrounded by liquefiable soil layer and bearing layer. The model size was 200cm long and 47cm high that is one of hundredth of the prototype model. The support soil layer was 15cm which model a diluvium silt layer. The liquefiable sand layer was 13cm on the support soil layer. The improved soil part was embedded 1.5 cm into the support soil layer. The shorter section of the turbine building was modeled. The gravity center and mass weight are made considering similarity rule. The first natural frequency of the model was 1.8 Hz which frequency is for an actual turbine building. Natural frequency of the turbine model is 1.0 Hz.

### Materials

Soil layer was made of Toyoura Sand (relative density is 70%). Improved soil ground was made of mortar (its unconfined strength was 4.0 MPa). Two cases were examined the support soil layer. One was diluvium sand layer, the other was diluvium silt layer. Diluvium sand layer was consist of Toyoura Sand, cement, bentonite and water with mixing (its weight ratio was 1:0.06:0.06:0.38, its cohesion was 0.17 MPa, its internal friction angle was 34.4 degree). Deluvium silt layer was consist of Nogoya Silt, cement and water with mixing (its weight ratio was 1:0.04:0.15, its unconfined strength was 0.5 MPa). Physical properties of Toyoura Sand and Seabed Silt are shown in Table 1 and Figure 5. Silicon oil is used for pore fluid instead of water. Its viscosity is 97.3 cs at 25 degree (Centigrade) and density is 0.966.

**Table 1. Physical properties of Toyoura Sand and Seabed Silt.**

| Sample Name                              |                        | Seabed Silt | Toyouura Sand |
|------------------------------------------|------------------------|-------------|---------------|
| Soil Particle Density( $\text{g/cm}^3$ ) |                        | 2.666       | 2.638         |
| Grain Distribution                       | Gravel (%)             | 0.0         | 0.0           |
|                                          | Sand (%)               | 19.9        | 98.4          |
|                                          | Silt (%)               | 56.8        | 1.6           |
|                                          | Clay (%)               | 23.3        | 0             |
|                                          | Uniformity Coefficient | -           | 1.62          |
|                                          | Mean Grain Size        | 0.0236      | 0.178         |
| Max Density ( $\text{g/cm}^3$ )          |                        | -           | 1.659         |
| Min Density ( $\text{g/cm}^3$ )          |                        | -           | 1.341         |

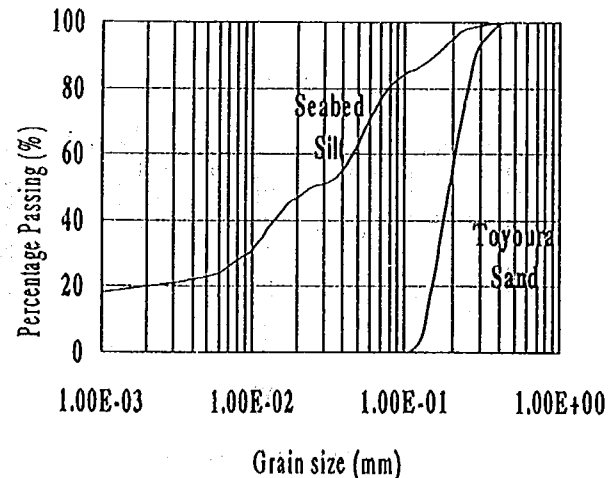


**Figure 5. Grain size distribution of Toyoura Sand and Seabed Silt.**

### Input motion

Input motion was sinusoidal wave of 20 cycles and the maximum acceleration was determined by pre-run test targeting all sand layer liquefied. They were about 200 gal at prototype scale. The frequencies applied were in the range between 1.6 Hz and 1.8 Hz that were first natural frequencies of the soil layer. The acceleration recorded at surface of the shaking table is considered as an actual input motion and reported as test results.

## Measurement

### Location

Accelerations and pore water pressures at each depth, displacement of the soil surface and structure were measured for both models. Locations of the measurement are indicated in Figure 6 and Figure 7. Earth pressures acting to the improved soil ground and foundation were measured for the soil structure model.

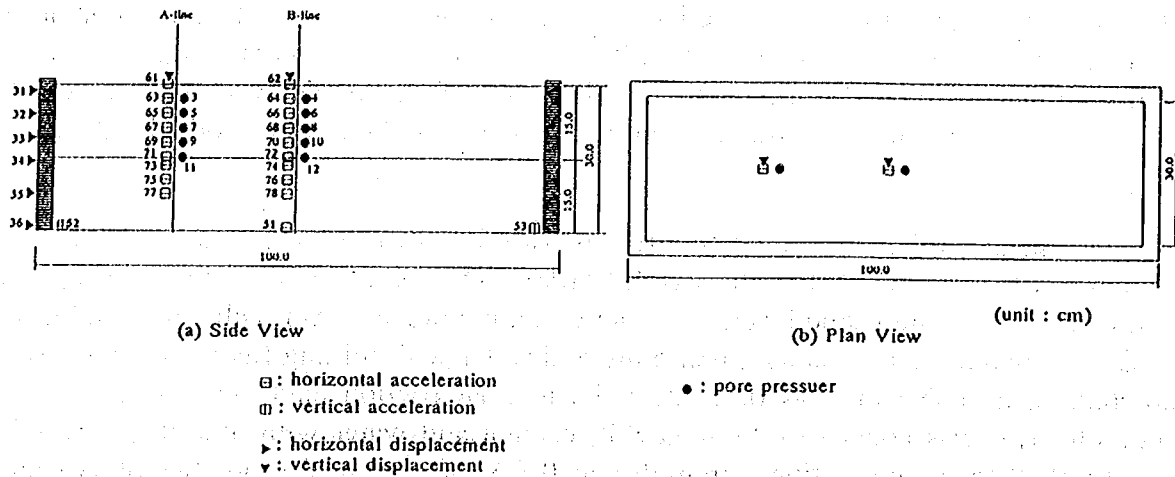


Figure 6. Location of measurement for soil layer model.

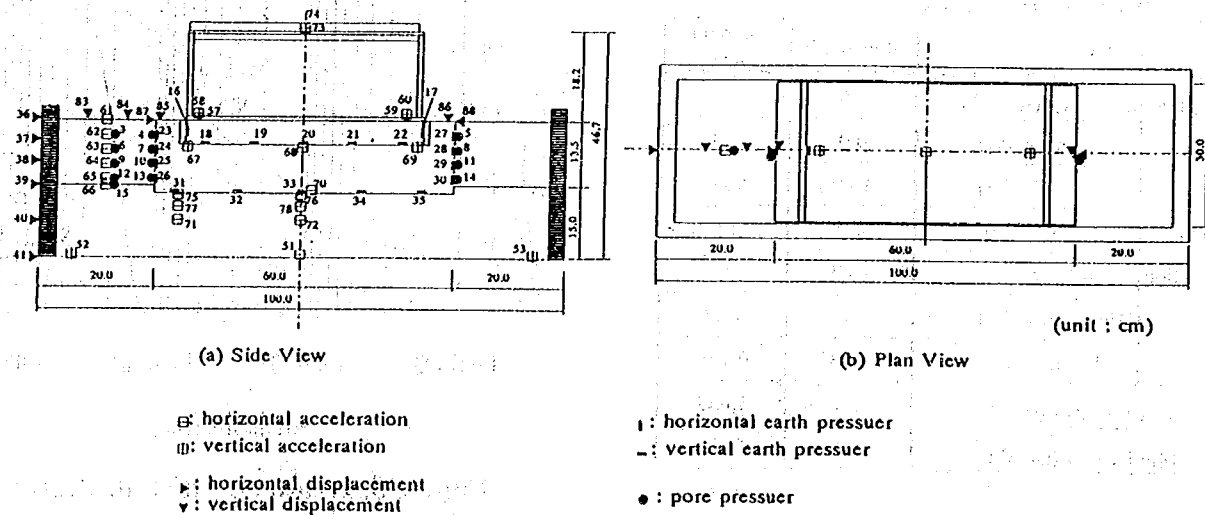


Figure 7. Location of measurement for soil structure model.

Sampling interval was 0.3 msec. Data gathering from the models to the computers outside of the centrifuge apparatus was done through slip ring.

## Results

### Layer Model

To know the dynamic behavior at the linear range, a small excitation was applied to the soil

layer model. Figure 8 shows the transfer function of soil surface against the shaking table. A major resonance frequency dominates the dynamic characteristics.

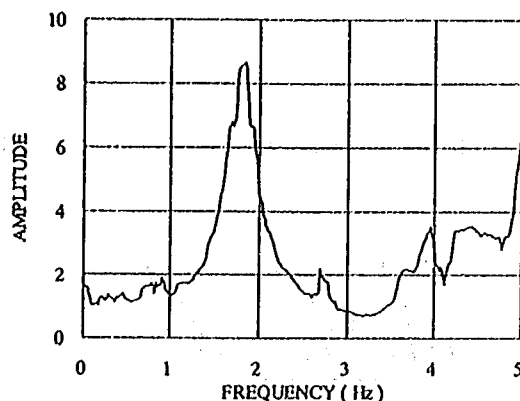


Figure 8. Transfer functions of the soil layer model.

Figure 9 shows the time history of the input motion to the soil layer model. The amplitude of the first 3 cycles were reduced to avoid an impact like force. The wave shape near peak amplitudes were sharpened due to nonlinear behavior of the soil layer and that the actuator power could not control it although the input motion intended to input was a sinusoidal wave. The frequency was chosen near the first resonance frequency. This is a part of the reason the input motion was influence at the peak amplitude by the dynamic behavior of the soil layer.

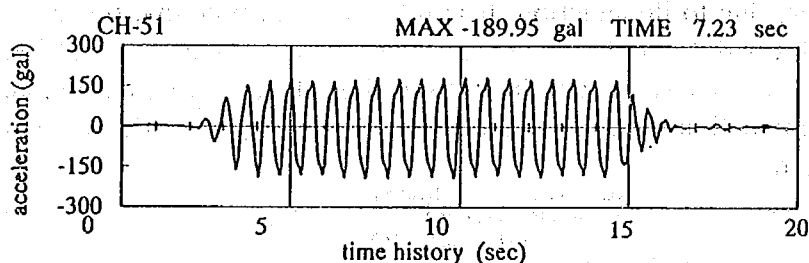


Figure 9. Time history of input motion (max. 189.95 gal).

Figure 10 shows the response acceleration of the surface of the soil layer model (record of CH61). It become nearly zero after 1.5 cycle since no acceleration force is transferred after liquefaction took place under the layer of measured point. The liquefaction took place from the surface to the bottom. The acceleration becomes zero after 1.5 cycle at the surface and then 2.5 cycle at CH63, 3.5 cycle at CH65, and 4 cycle at CH67. This indicates that the speed of pore water pressure built up is similar but initial vertical stresses are larger at deeper layers so it took longer time to reach the stresses at the deeper place. Time history of the pore water pressure shows that tendency (Figure 11). Figure 12 shows that bearing layer was not liquefied.

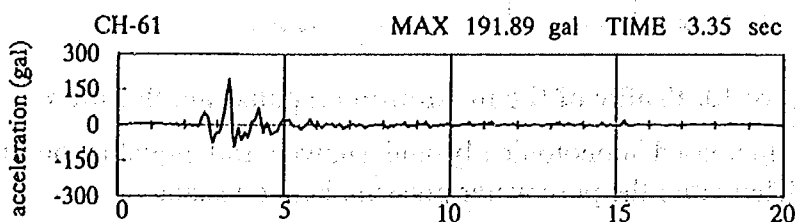


Figure 10. Time history of acceleration at Channel 61.

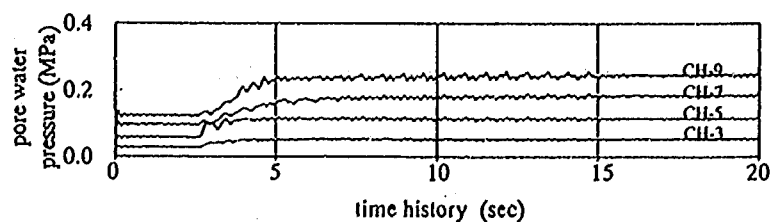


Figure 11. Time history of pore water pressure at Channel 3,5,7,9.

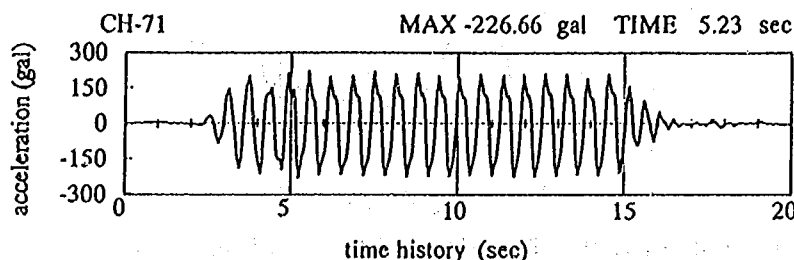


Figure 12. Time history of acceleration at Channel 71.

Figure 13 shows the profile of the maximum response acceleration at depth. There is no significant amplification in the layer although the excitation frequency is near the resonance frequency. The reason of this may be that the stiffness of the soil layers was reduced due to strain hardening and liquefaction. Typically the accelerations near the surface were decreased than the deeper layers due to liquefaction. It is interesting to note that the acceleration of the lower part of sand layer did not show significant decrease of their acceleration although the layers liquefied. This is because the acceleration of those layers experienced more than one or two cycle before liquefaction took place and reached the larger amplitude while the material strength was remained.

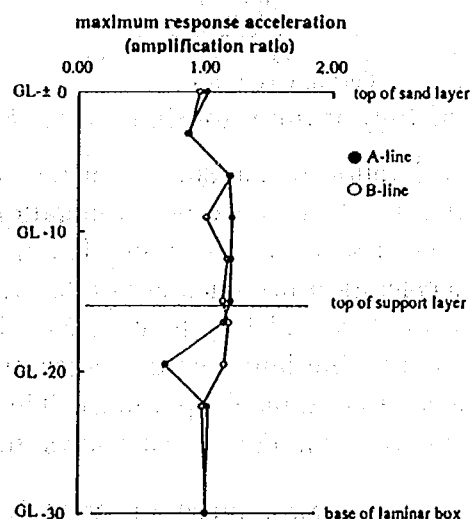


Figure 13. Profile of the maximum response acceleration

Pore water pressure increased monotonically and showed full liquefaction at 4 second as shown in Figure 14. Thereafter the pore water pressure kept constant.

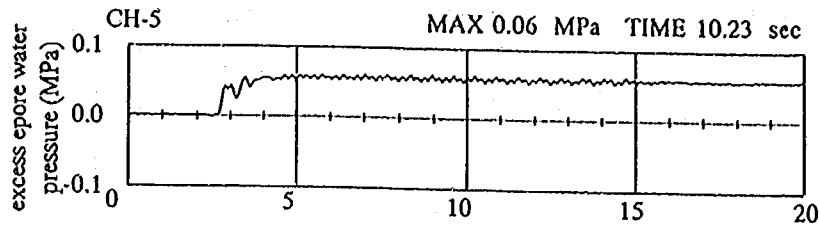


**Figure 14. Time history of pore water pressure at middle of the sand layer**

Figure 15 shows that all sand layer (0 – 15m) has liquefied. It happened at about 4 second from the surface to bottom. After then the pore water pressure kept constant.

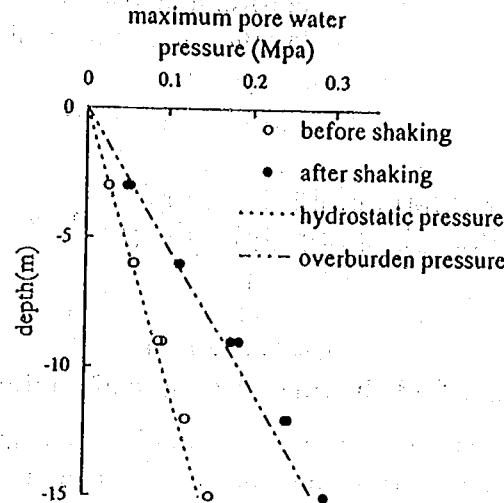


**Figure 15. Profile of maximum pore water pressure**

Permanent displacement of the laminar box is shown in Figure16. Large displacement occurred after 5 second although the full liquefaction was observed at 4 seconds. A random response took place after the input motion stopped. During input motion active, cyclic displacement at constant amplitude was observed.

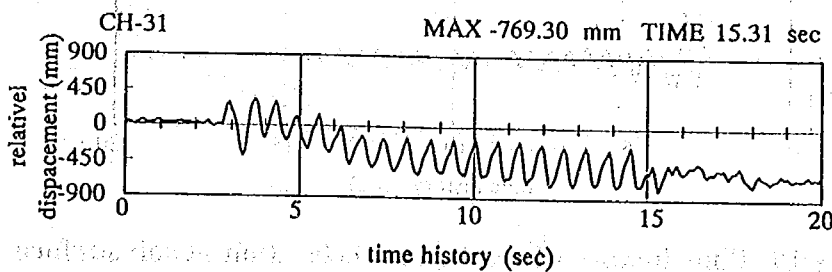


**Figure 16. Time history of the displacement at Channel 31**

Surface settlement was about 37 mm for A line and 14.7mm for B line. The settlement started at about 4 second and continued even after shaking finished. This tendency is little different from the horizontal displacement.

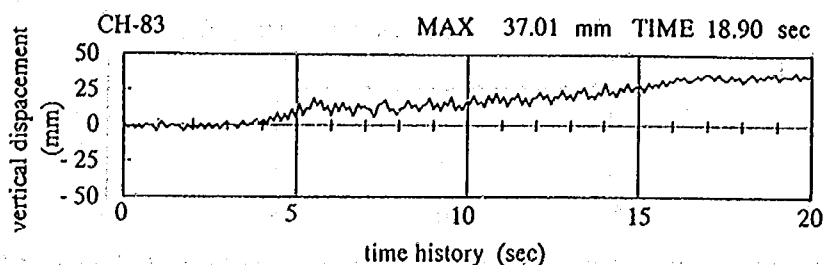


Figure 17. Time history of surface settlement (Channel 83).

### Improved soil model (Sand)

The results of the shaking table test on the centrifugal apparatus were discussed here. Figure 18 shows the acceleration at the base of the laminar box. A sinusoidal wave acceleration was applied except a little disturbance at the first 5 cycles.

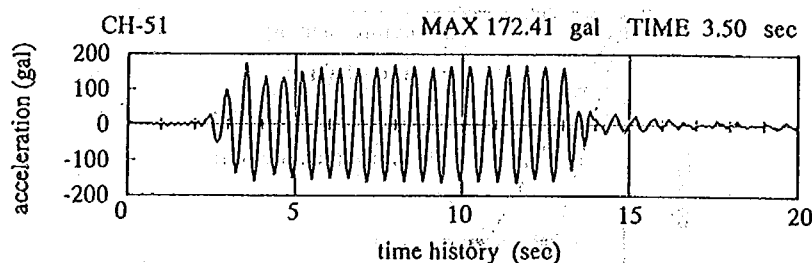


Figure 18. Time history of at base of laminar box

Response acceleration at the surface of the surrounding area is shown in Figure 19. The amplitude showed irregular behavior at the first to 4<sup>th</sup> cycle. Thereafter constant sinusoidal wave was seen but with the phase delay (180 degree). It indicated the drastic change of the soil stiffness due to liquefaction at the very early stage of the excitation. Figure 20 shows the acceleration at boundary between sand and bearing layer (very dense sand). The amplitude increased at beginning and reached the peak at 5.31 sec. Then it decreased little till about 7 second. After then constant cyclic behavior continued. There was a little phase delay in these duration. This also indicates some decrease of material stiffness of the dense layer.

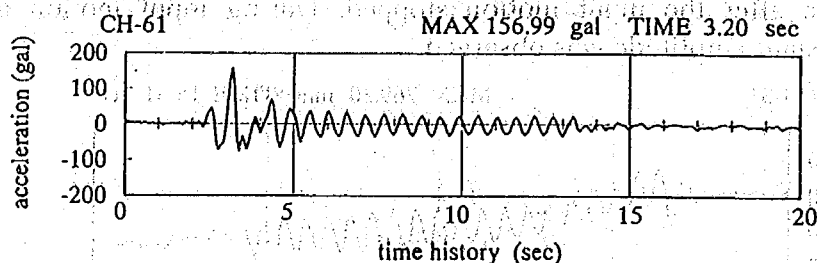


Figure 19. Time history of response acceleration at soil surface

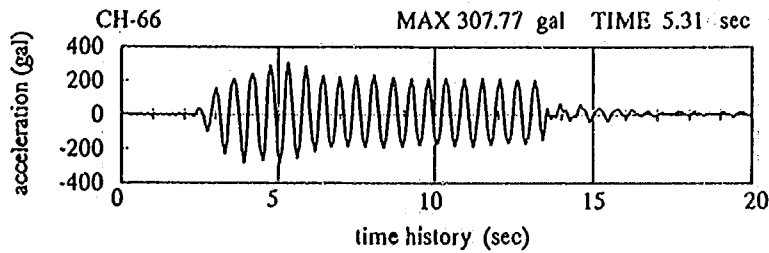


**Figure 20. Time history of response acceleration at support soil layer**

Figure 21 shows that pore water pressure reached the full liquefaction at very early stage but oscillated during the excitation. The reason of this may be due to an interaction with the improved soil ground.

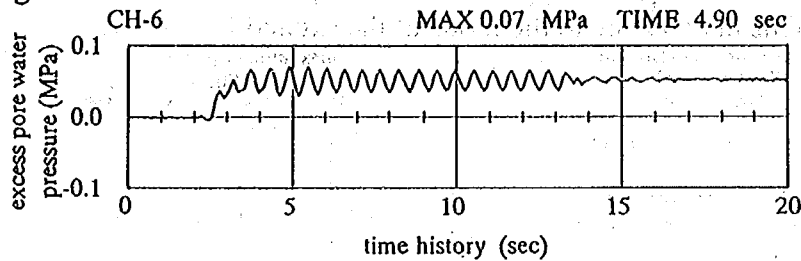


**Figure 21. Time history of pore water pressure at surrounding soil layer.**

Figure 22 shows that all sand layer (0 – 15m) has liquefied. It happened at about 4 second from the surface to bottom. After then the pore water pressure kept constant.

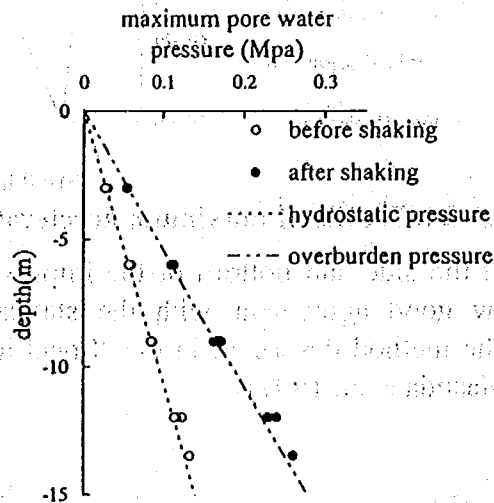


**Figure 22. Profile of maximum pore water pressure**

Figure 23 show time history of the displacement at the top of the laminar box. Large displacement was developed during the first 7 cycles. After then a little deformation was observed.

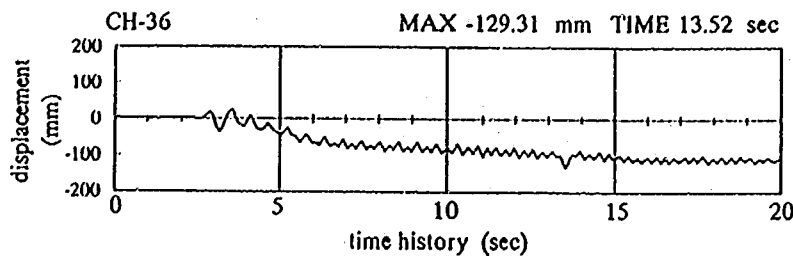


Figure 23. Time history of displacement at top layer

Figure 24 shows the maximum accelerations along the two lines. One is at the center of building (a) and the other is center of surrounding area (b). They are small near the surface of the surrounding area indicating that liquefaction took place there. The structure parts also show small values. This may be due to the mode at the excited frequency.

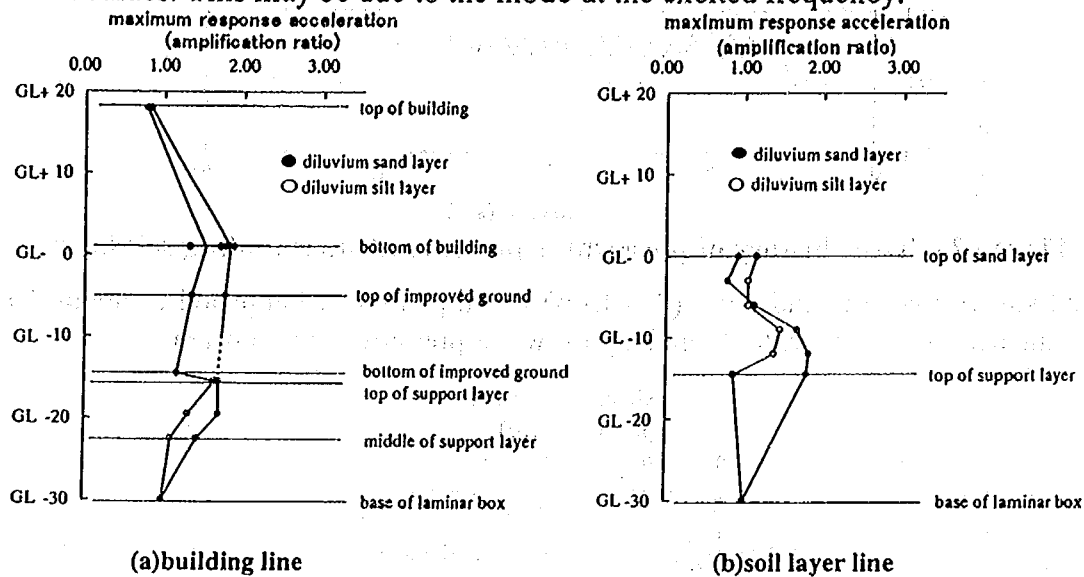


Figure 24. Profile of maximum acceleration

Earth pressure distribution at the side and bottom of the improved soil ground is shown in Figure 25. The values show good agreement with the statically calculated value using maximum accelerations by the method described in the "Specification for Highway Bridges part V -Seismic Design-" (Masuda et al, 1997)

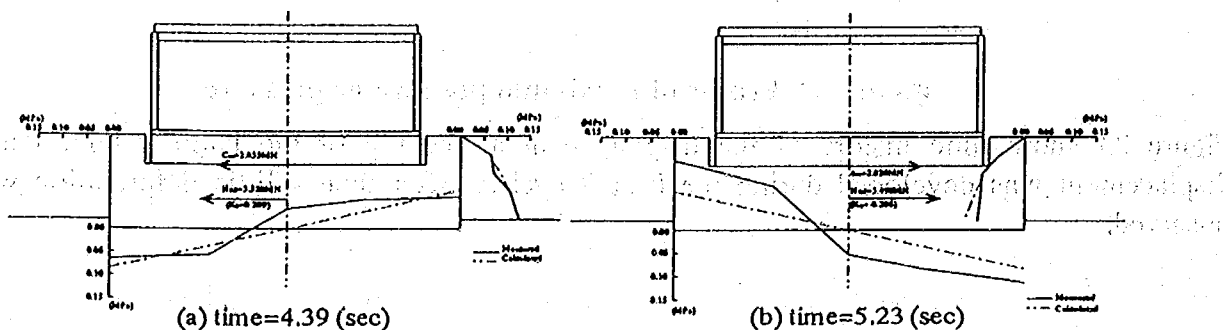


Figure 25. Pressure distribution, at time 4.39 and 5.23 sec.

### Random excitation and sinusoidal acceleration

The experimental model test has been done at a frequency so it is necessary to translate the behavior to random behavior. A numerical study has been done to simulate this. Figure 26 shows the solution of random and sinusoidal excitation. The frequency for the sinusoidal excitation was about 0.8 of the first natural frequency. That is the same proportion taken at the experimental test. The response of the sinusoidal excitation is about double of the random excitation. The value obtained by the design procedure was much less than those. This can be understood by the relationship between the maximum value acceleration and the seismic intensity.

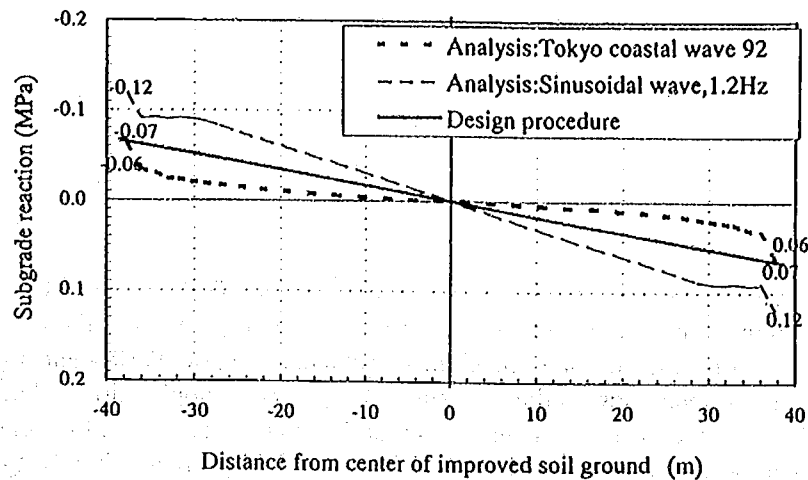


Figure 26. Comparison of random and regular excitation.

### Concluding Remarks

Assumption of the design procedure of the improved soil ground using the deep soil mixing ground with cement was studied by the shaking table test under the centrifugal force (100G). The distribution of the earth pressure was examined for the side and bottom boundary of the improved soil ground. The pressure distribution of the experiment proved the distribution used in the design procedure. The seismic intensity for the design model can be statically calculated using the value obtained by the experiment. But it is necessary to consider the randomness of the earthquake.

### References

1. Japan Highway Association: "Specification for Highway Bridges part V-Seismic Design", 1993.
2. Masuda A. et al: "A Study on building foundation with Deep Cement Mixing method for thermal plant ( part 9, Design Methodology)", *Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan*, 1997, 667-668.
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## Research With The U.S. Army Centrifuge

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**ABSTRACT:** The U.S. Army Engineer Waterways Experiment Station (WES), located in Vicksburg, Mississippi, has recently installed a uniquely powerful and large centrifuge. Although much of the recent world-wide interest in centrifuge modeling has concentrated on geotechnical problems, the new WES centrifuge facility will address research needs in physical modeling across the full range of engineering applications. The purpose of this paper is to describe the WES centrifuge, to discuss current research being conducted in the centrifuge facility and our vision for the future. The research described here will focus on field problems involving dynamics particularly earthquake work and blast loading. However, the WES centrifuge is uniquely broad-based in its research applications; many types of field problems, involving fluid-flow, heat transfer, and cyclic and static loading are being addressed in a wide range of fields: geotechnical, structural, environmental, hydraulics, coastal engineering, and engineering for cold regions. Some initiatives in these areas are also described.

## 1 INTRODUCTION

The WES has a long history of physical modeling. Its pioneering research in the early 1930s in areas of hydraulics engineering and flood control used scaled physical models carved in the loess soil. Hydraulic physical modeling experiments which replicated such historic locations as the Los Angeles Harbor, the New York Harbor, Niagara Falls, and the Old River Control Structure on the Mississippi River have been conducted throughout its 65 years of service to the Army and the Nation.

During the past several decades, with the advent of analog and digital computers, WES moved into the numerical modeling arena. With the rapid advance in computer technology leading to PC's and supercomputers, research is now being conducted using numerical modeling techniques including finite difference, finite element, discrete element, and discontinuous deformation analysis. Recent accomplishments include the development of a three-dimensional water quality model for the entire Chesapeake Bay, the use of complex three-dimensional

codes to investigate cratering and air-blast effects from explosive events, and the use of two-dimensional effective stress finite element analysis for predicting large deformations occurring in embankment dams as the result of earthquake-induced liquefaction in the dam and/or its foundation material.

The WES centrifuge is capable of spinning a payload of up to 8 tonnes and achieving accelerations of up to 350 gravities (g). This centrifuge provides a physical modeling capability that greatly enhances research techniques. Case histories and/or physical model experiments are required to validate numerical computer codes. Physical models have the advantages of being controllable, relatively inexpensive, and provide better known conditions than field cases.

Researchers can better control the material properties and boundary conditions of physical models. The cost of centrifuge experiments is far less than full-scale field experiments which may involve hundreds of thousands or millions of dollars.

## 2 MODELING FIELD PROBLEMS IN A CENTRIFUGE

Physical modeling of field problems generally involves the construction and experimentation with scale models of a prototype or field structure. The response of the model to a physical perturbation such as cyclic loading, earthquake, ocean waves, explosion or even freezing or groundwater changes can be measured and interpreted in terms of the field problem under consideration. Realistic model behavior data can provide valuable information for designers about failure mechanisms and long term performance.

In many engineering fields, particularly geotechnical engineering, gravitational effects dominate performance. Where the behavior of a material is stress dependent, such as with the engineering properties of soil, then simple scaling of a field problem and experimental investigations on the laboratory floor will give erroneous results. For example, in a slope failure it is common to observe cracking near the ground surface where the confining stresses are low but the shear stresses are high, rupture along a slip surface at greater depth and finally flow at depth as the confining pressure increases still further and the shear stress ratio reduces to a value below the critical state or characteristic threshold value, (Figure 1).

If one were to attempt to model the slope failure in Figure 1 on the laboratory floor at 1/50 scale without the benefit of a centrifuge, the effective confining stresses in the soil would be 50 times less than those same stresses in the field and hence could bring the full depth of soil in the model into a region where tensile fracture (or cracking) dominates.

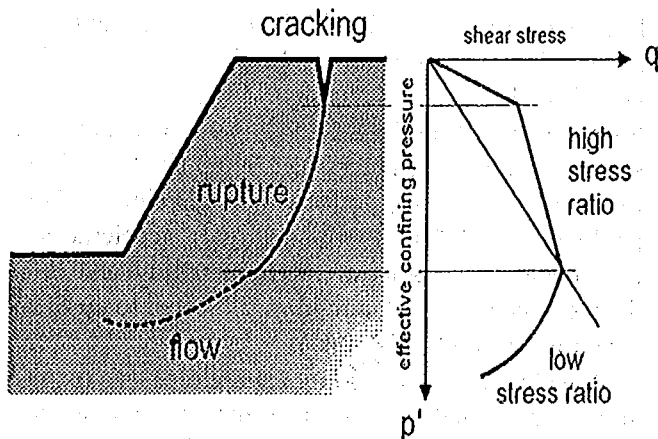


Figure 1. Ratio of shear stress to confining pressure governs soil behavior.

The use of a centrifuge however, overcomes this modeling error by increasing the weight of each particle in the model by an equivalent number of gravities. Generally the linear scale of the model and the number of gravities are chosen to be identical. Therefore, a 1/50 scale model of a 20 m high earth slope would be 400 mm high. Under a steady acceleration of 50 gravities (approximately  $500 \text{ m/s}^2$ ), it would have identical self-weight stresses at equivalent points in the model and in the prototype. The behavior of the miniature slope will then mimic the behavior of the prototype. Each soil element in the centrifuge model is subject to the same stress path as its equivalent element in the full-scale prototype.



Figure 2. Failure of a slit slope in Vicksburg loess.

Figure 2 shows a silt slope failure in remolded Vicksburg loess, carried out on the WES centrifuge. Details of the failure can be seen, including the cracking near the top of the slope, and rupture along the slip surfaces that run nearly parallel to the slope. The response shown is typical of loess behavior with no deep-seated flow movement.

Scaling relationships can be derived from physical principles that interpret the centrifuge model experimental investigation data in prototype terms (Schofield 1980 and 1981, Schofield and Steedman 1988). These are summarised in Table 1.

Table 1. Centrifuge scaling relations.

| PARAMETER                   | FIELD      | CENTRIFUGE MODEL (Ng) |
|-----------------------------|------------|-----------------------|
| Stress                      | $\sigma$   | $\sigma$              |
| Strain                      | $\epsilon$ | $\epsilon$            |
| Length, displacement        | L          | L/N                   |
| Area                        | A          | A/N <sup>2</sup>      |
| Force                       | F          | F/N <sup>2</sup>      |
| Volume                      | V          | V/N <sup>3</sup>      |
| Mass                        | M          | M/N <sup>3</sup>      |
| Energy                      | E          | E/N <sup>3</sup>      |
| Frequency                   | f          | Nf                    |
| Velocity                    | v          | v                     |
| Acceleration                | a          | a/N                   |
| Time (for inertial events)  | t          | t/N                   |
| Time (for diffusion events) | t          | t/N <sup>2</sup>      |

As a technique for solving engineering problems, the use of the centrifuge was first pioneered in the United States (US) and in the Soviet Union during the 1930s (some references claim the concept of a centrifuge model can be traced to the 19th century or earlier). Although the idea did not fully develop in the US at that time, the Soviets recognized its potential in the area of weapons effects and developed a major program of centrifuge modeling. It is understood that this continued until the recent political changes.

In the west, the development of centrifuges for study of geotechnical problems was led by academic researchers in the United Kingdom (UK) committed to the study of soil as a plastic material. A number of centrifuge centers developed in the UK, US and in Europe during the 1970s and 1980s primarily attached to universities or research institutions.

Miniature instrumentation and, recently, the development of affordable advanced data acquisition systems has revolutionized the potential for centrifuge models. Model experiments can now provide detailed quantitative information on the response of a model at many different points for comparison with numerical models or analytical solutions. Miniature instrumentation has already been developed which can measure displacements, pore fluid pressures,

accelerations, resistivity, wave height, shear wave velocity, and so on. 'In-flight' site investigation tools can perform cone penetration probes, vane shear experiments or other explorations of the 'site'. Miniature cameras can observe the behavior of the structure in close up. Systems can load the soil and structures, trigger earthquakes, generate water waves, freeze ice sheets, carry out staged construction, release contaminants or detonate explosives.

The wide range of physical phenomena that may be created in a centrifuge and the quality and detail of the data that may be captured from a model in-flight clearly provides substantial opportunities for engineering research studies. The Corps of Engineers has been involved with centrifuge modeling since the 1970s, and the recent development of the WES Centrifuge Research Center in Vicksburg demonstrates the Army's full commitment towards the use of physical modeling as an integral part of engineering analysis.

### 3 THE WES CENTRIFUGE

The design specification of the centrifuge followed a review of available academic facilities which had shown that none were able to routinely conduct experimental models of the large field structures and problems with which the Corps is principally concerned (Ledbetter, 1991). The facility was therefore designed with a unique operating envelope. It was required that model containers should be easily placed on and off the centrifuge and these requirements led to the design of a beam centrifuge with a swinging platform based on the French designed Acutronic 661, 665, and 680 series of geotechnical centrifuges. Other centrifuges in this Acutronic family include one at Rensselaer Polytechnic Institute, Troy, New York, at the Centre for Cold Oceans Resources Engineering, St. Johns, Newfoundland, and several in Japan and Europe. Figure 3 shows the WES Centrifuge in its containment structure; key parameters are listed in Table 2 and Figure 4 shows the performance envelope relating payload capability to centrifugal acceleration. Prototypes of the order of 300 m in breadth and 300 m deep can be simulated in models subjected to up to 350 gravities (around 3500 m/s<sup>2</sup>).

A wide range of modeling capabilities was required and actuators and appurtenances have been designed to fulfil these requirements. Models can be constructed in a range of containers that depend on the geometry of the problem under investigation (plane strain, axisymmetric, three dimensional, etc.) and the type of phenomenon to be recreated.

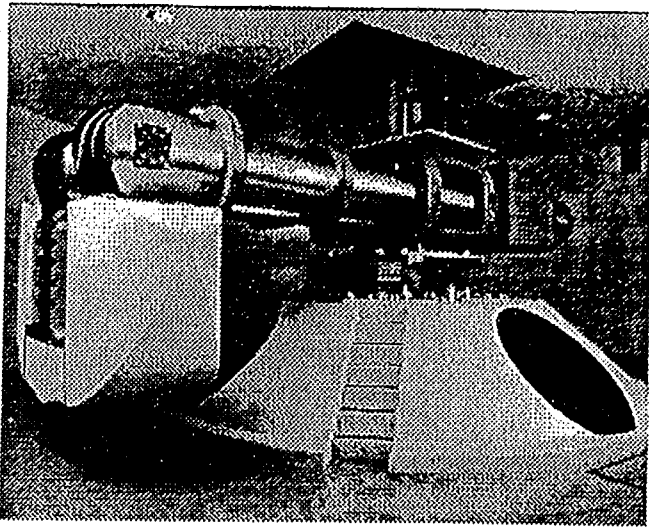


Figure 3. The U.S. Army centrifuge at the Waterways Experiment Station.

Table 2. Key characteristics of the Army centrifuge.

|                     |             |
|---------------------|-------------|
| Gravity range       | 10 to 350 g |
| Platform radius     | 6.5m        |
| Payload at 143g     | 8000kg      |
| Payload at 350g     | 2000kg      |
| Centrifuge capacity | 1144 tonnes |

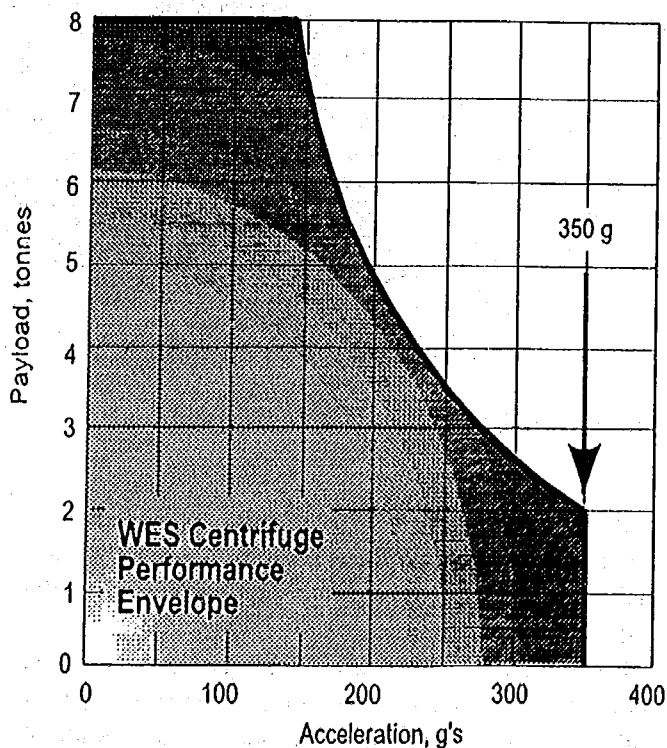


Figure 4. WES Centrifuge performance envelope.

Recent world-wide interest in centrifuge modeling has concentrated on geotechnical applications; however, the WES centrifuge facility has a much broader mission. The WES centrifuge will address research needs in physical modeling across the full range of engineering applications. Investigations are possible under climatic conditions ranging from desert to polar to ocean regions. Key capabilities include: (1) force and displacement controlled load systems, (2) earthquake simulation, dynamic vibration loading and water wave generation, (3) blast loading, (4) model package environmental control, (5) in flight manipulation of the model materials and fluids, (6) miniature instrumentation, cameras and in flight site interrogation, and (7) high speed data acquisition and control systems.

The new capabilities that will flow from the centrifuge will depend on the ingenuity of its users and the design of its appurtenances. In this respect the design of the centrifuge itself is merely one component of the development of new capabilities in physical modeling.

#### 4 CENTRIFUGE MODELING OF DYNAMIC PROBLEMS

The use of centrifuges to address field problems involving dynamics dates back many years. As noted above, the Soviets made use of centrifuges as early as the 1930s and 1940s for the study of weapons effects. WES, which first used centrifuge modeling for cratering studies in the 1970s, has now developed the capability for carrying out such explosive investigations on its new centrifuge (as discussed further below).

In the field of earthquake modeling, Professor Mikasa at the University of Osaka pioneered the development of the first centrifuge 'earthquake' actuator by using a tilting mechanism to create a pseudo-static lateral acceleration field on a soil model in flight, (Mikasa et al. 1969). Steedman (1991) provides a review of the application of the centrifuge to dynamic geotechnical studies which highlights some of the important achievements of Japanese researchers in earthquake centrifuge modeling.

Since the 1980s, a variety of actuators have been developed around the world for the simulation of earthquake-like base shaking on centrifuge models. These have ranged from sophisticated units using servo-hydraulic systems to provide broad band shaking input to less mechanically complex systems that can apply a broadly sinusoidal base shaking motion of a

given duration and frequency. In general, limitations posed by the high gravitational field in which the equipment must work and the high development cost has restricted the use of servo-hydraulic systems to low g centrifuge applications and small models. At WES, the requirement to conduct experimental models of large field problems has led to the development of a robust mechanical shaker designed to operate at over 200 gravities, significantly higher than any earthquake centrifuge modeling on other beam centrifuges carried out to date. Early indications, using a prototype version of the system developed together with Cambridge University, have provided encouraging results, as seen in Figure 5.

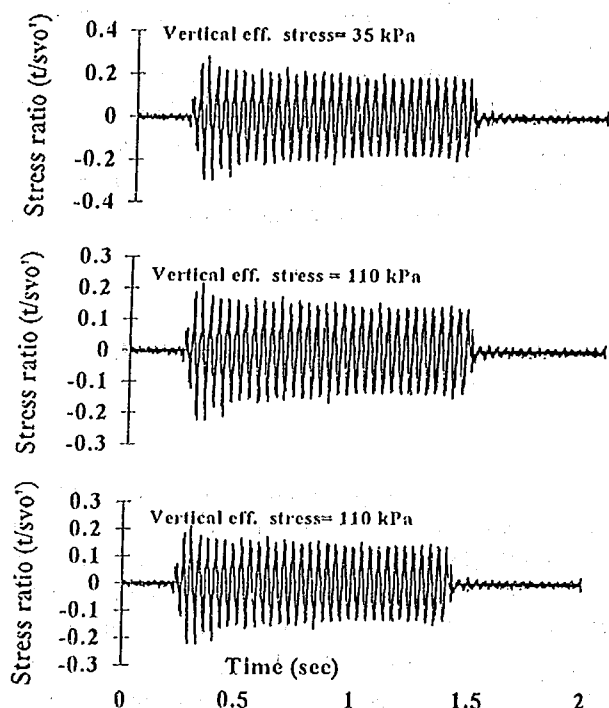


Figure 5. Shear stress ratio at two depths in a WES soil model under dynamic loading at 50 g.

The WES earthquake shaker (Figure 6) uses the angular momentum stored in high speed rotating flywheels to provide the energy source for the shaking motion. Power is transferred from the flywheels to the base plate beneath the model via a continuously oscillating shaft which is briefly grabbed by a high-speed clutch for the desired duration of excitation. Shown in Figure 7 is the earthquake shaker mounted on the centrifuge. Figure 8 shows the equivalent-shear-beam laminar model container that is 900 mm in length, 650 mm in height and 355 mm in width. This container incorporates shear sheets on each of the end

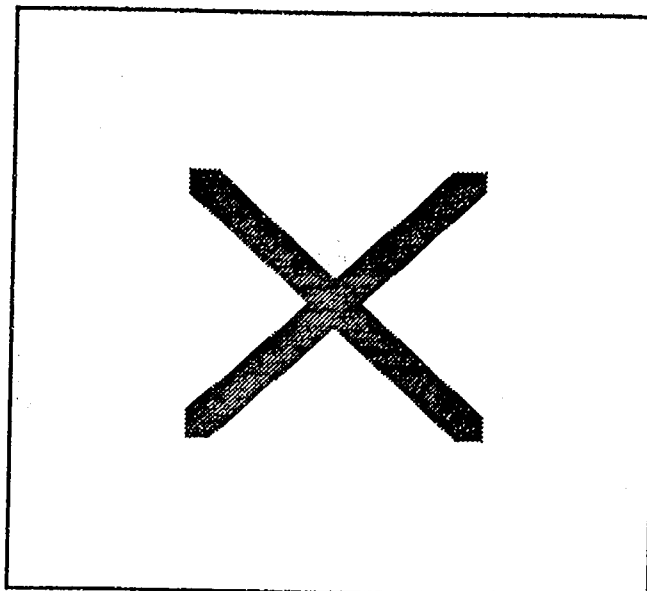


Figure 6. The WES dynamic load actuator.

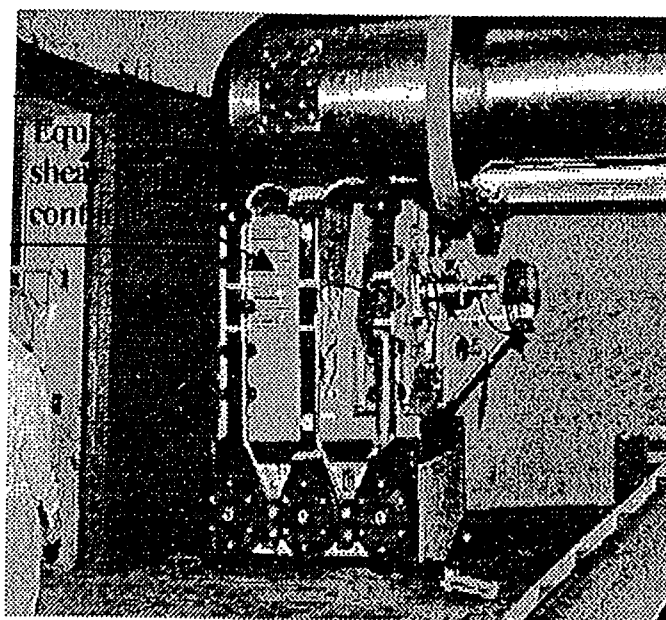


Figure 7. Earthquake shaker on the centrifuge

walls to transfer the complementary shear stresses generated on vertical planes in the specimen by base shaking in the horizontal direction. This improves the uniformity of shear.

From Table 1 it is seen that as the g level is increased it is necessary to increase the frequency of shaking and simultaneously to reduce the amplitude of motion. At high g levels these small displacements may become comparable to the manufacturing tolerances of the moving parts; particular attention has been paid to

this issue in the design of the WES shaker. The resulting

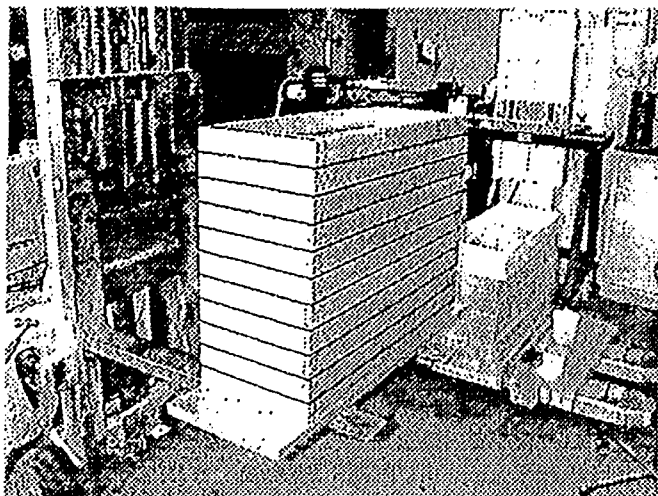


Figure 8. Equivalent shear beam model container.

system is robust and economic, and will provide a pre-programmed duration and frequency of shaking.

In fields such as the seismic design of retaining walls, centrifuge model studies have provided valuable insights into the mechanisms underlying the onset of permanent movement and the ultimate failure of different forms of wall. Conventional approaches to the prediction of permanent movement of monolithic walls have been investigated and refined using centrifuge model behavior data (Steedman and Zeng 1996). Other studies have investigated the point of application of the dynamic force increment and the consequences of the development of excess pore pressures and liquefaction in the backfill behind anchored walls (Steedman and Zeng 1990). Figure 9 shows a model of an anchored sheet-pile wall experiment conducted at WES for Oregon State University.

A key area of interest to WES has been the study of liquefaction and the dynamic response of embankments and dams. WES has made extensive use of centrifuge modeling in this field to provide realistic data for the validation of numerical codes and other analyses and a substantial program of research is planned for the new facility. Figure 10 presents typical results from recent experiments in a test program investigating the effects of confining stress on liquefaction that could have a significant impact on dam earthquake stability and safety.

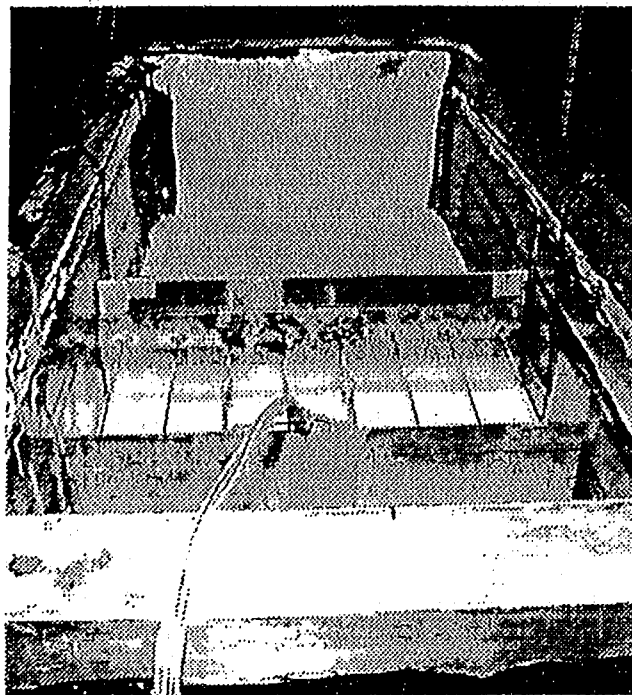


Figure 9. Model anchored sheet pile wall dynamic load experiment.

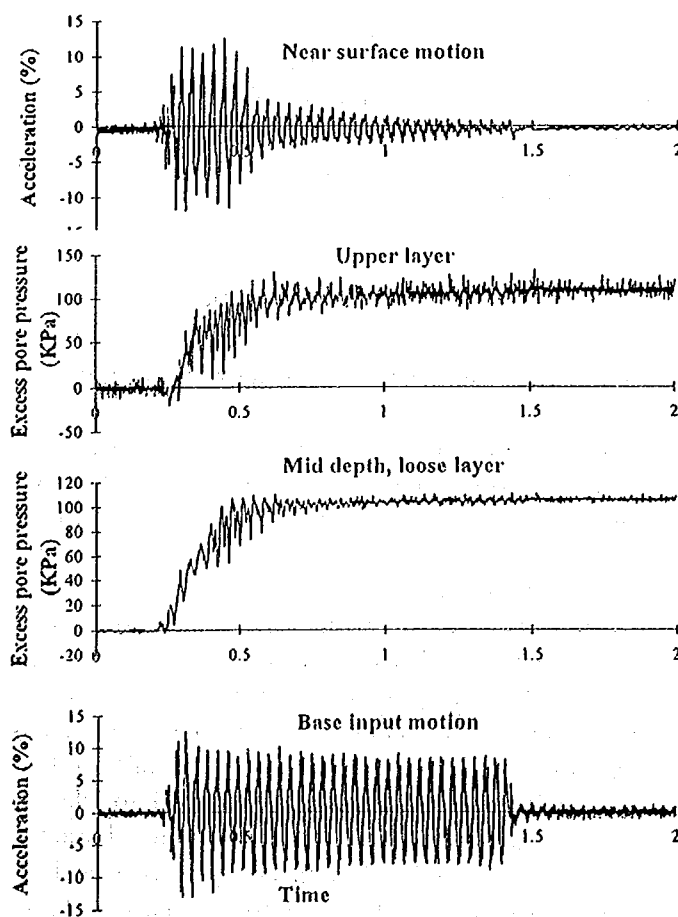


Figure 10. Dynamic load experiment time history raw data from pore pressure transducers and accelerometers.

The WES involvement and interest in centrifuge earthquake engineering experiments and studies dates back about twenty years to collaboration with the University of Cambridge (Morris 1979). WES conducted experiments in the 1980s of models including dry and saturated embankments with and without surface supported and embedded structures.

An important advantage of centrifuge earthquake experiments is the wealth of data and information that can be acquired. The measured responses shown in Figures 10 cannot be obtained by any other means under the equivalent prototype stress states and provide valuable insight to behavior under earthquake excitation and for numerical model development and verification.

Other fields involving dynamic loading that have been addressed using the centrifuge include dynamic loading of foundations (>machine vibration), (see Laue and Jessberger 1994), and wave loading on coastal defences and seabed deposits (Sekiguchi et al. 1994).

The modeling of instability of sand beds caused by water waves at  $g$  has been hampered by the need for very large or deep wave tanks. At high  $g$  it becomes practical to adopt viscous scaling of the pore fluid because of the small size of the wave tank, and researchers at Kyoto University have shown that using this technique, wave induced liquefaction can be reproduced in a sand bed deposit, (Sekiguchi et al. 1994). This class of problem will also be addressed in future at WES.

## 5 CURRENT RESEARCH AT WES

A number of research projects using the centrifuge are currently underway at WES. In the following paragraphs, a few of these investigations are described which demonstrate the Corps interest in a wide range of applications. Some of these fields are entirely new to centrifuge modeling.

In the field of earthquake engineering, a large study has commenced to investigate aspects of the prediction of liquefaction. This has the potential for a major impact on the design of remedial measures for earth dams in seismic areas in the US. To date, the factors which strongly influence the prediction of a soil to liquefy and its residual strength as used for design purposes have been evaluated only on laboratory element test data and fortuitous field performance data. The new research approach is to use the new large earthquake shaker mounted on the WES centrifuge to conduct a series of models experiments, monitoring the development of excess pore fluid pressure in the

foundation soils directly as a function of the density, amplitude of shaking and number of load cycles (Figure 10). This approach will not need to extrapolate design factors from small laboratory samples and may lead to significant improvements in design and analysis techniques.

A second research program is concerned with the gravity driven mixing and flow of immiscible fluids as oils seep into the natural groundwater environment. The purpose of the models is to acquire data suitable for the validation of numerical models of DNAPL (denser than water non-aqueous phase liquid) movement in groundwater, the development of which have been hampered by the lack of realistic experimental or field data. In a centrifuge model under high gravity, the time taken for a diffusion event is reduced by the square of the model scale over the equivalent time in the field. This occurs because of the small physical size of the model and the higher hydraulic gradients. This is a strong advantage for any experiments investigating long term diffusion type problems as the centrifuge can achieve in a few hours what would take many years to tens of years of monitoring in the field.

The first centrifuge experiments undertaken by the Corps in the 1970s were in the field of blast crater modeling. More recent work has studied the effects of blast and blast induced liquefaction on structures. Figure 11 shows a blast precision-scale cratering experiment model configuration. Figure 12 shows a crater formed in dry sand in the blast chamber on the WES centrifuge, with a buried charge equivalent to 500 lb. of high explosive (PETN). The model, at 45 gravities, has a crater diameter of 300 mm, equivalent to 13.5 m in the field. Figure 13 shows a cut cross-section through the crater and model of Figure 12. Shown in Figure 14 is a comparison of crater profiles from field experiments and centrifuge precision-scale experiments.

The WES centrifuge has a substantial capability for modeling the effects of explosions on a wide range of structures, again benefiting from the capability to operate at up to 350 gravities. In the scaling of blast phenomena, energy scales with  $N^3$ , Table 1, and hence 1 gm of high explosive at 100 gravities will generate an event equivalent to a 1 tonne explosion in the field. At 350 gravities, large explosions can be replicated. The program of work in blast modeling includes simulation of field events to demonstrate internal consistency of the models and the effects of explosions on a range of structures and facilities in different ground conditions.

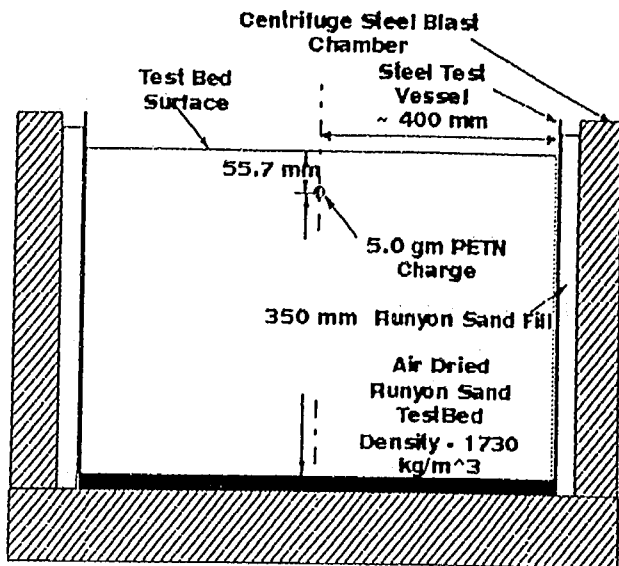


Figure 11. Precision-scale cratering experiment model.

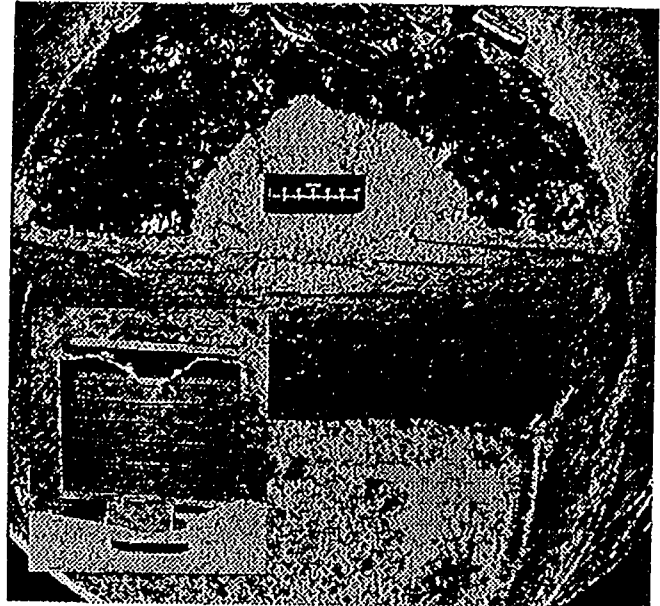


Figure 13. Cross-section through the crater of Figure 12.



Figure 12. Precision-scale crater experiment conducted at 45 g centrifugal acceleration.

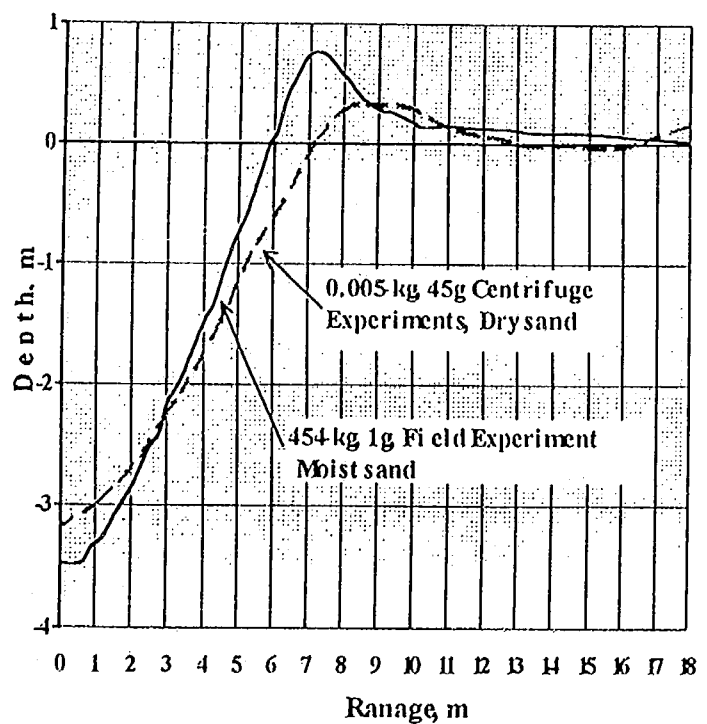


Figure 14. Comparison of crater profiles, field and centrifuge precision-scale experiments.

Engineering for cold regions is another field of interest to the Corps and in this area studies have commenced concerning the effects of ice forces on structures with the growing of an ice sheet under increased gravity in a specially designed insulated container. Earlier studies at Cambridge University have shown that ice grown under increased gravity in the laboratory has an internal structure like that of a multi-year ice sheet in the Arctic, with a profile of snow on the surface, random crystals at shallow depth and vertically oriented crystals below that. If ice sheets of similar form to those found at sea can be formed in a centrifuge model then this pioneering research may provide a new route altogether for the development of engineering solutions for cold regions engineering.

In the environmental area, a research program has been initiated to investigate the consolidation of dredged material and the consequential damage to capping layers. Initial experiments have addressed the long-term settlement of the material. Continuing experiments are investigating the movement of contaminants. As was noted earlier, small volumes of material in a centrifuge can replicate large volumes in the field. For contaminant migration investigations, this means that even small samples, which can be readily managed in the laboratory, can provide a basis for realistic modeling and simulation of the behavior of the full-scale prototype. Samples can be used this way rather than simply used to acquire parameters in a laboratory element test. Experiments on the WES centrifuge have replicated 20 years of contaminant migration.

## 6 VISION

The development of the WES Centrifuge Research Center marks the transition of centrifuge modeling from a pure geotechnical engineering research tool to a modeling technique of wide application in engineering work. The Centrifuge Research Center aims to become the worlds leading centrifuge modeling facility supporting the Corps of Engineers, the Army, the Department of Defense, other federal agencies and other nations, collaborating with academic and industrial organizations and addressing novel and demanding engineering problems world-wide.

In order to make this vision a reality, WES is developing vehicles to foster partnerships with researches and colleagues from Government, Industry and Academia. The Corps has an agreement with the National Science Foundation to facilitate such partnerships. WES is also developing vehicles to foster

partnering with owners, operators, contractors, and designers who have problems that could be addressed using data obtained from the results of centrifuge studies.

## 7 CONCLUSIONS

The WES is continuing its tradition of 65 years of physical modeling with the acquisition of the most powerful centrifuge known to date and the development of unique applications and appurtenances for conducting research. Centrifuge investigations are possible across a wide range of engineering fields. Research is being conducted in earthquake engineering, environmental problems concerning groundwater flow, geotechnical engineering, blast phenomena, cold regions and ice formation, dredge material disposal behavior, and contaminant migration. Research is planned in the engineering areas of soil-structure interaction (behavior of large lock structures), hydraulics and coastal (wave loading on beaches and seabeds), and airfield pavements. The Corps is convinced that numerical and physical modeling must progress together to solve todays and future engineering problems. The centrifuge will allow us to investigate phenomena not previously feasible including the performance of large civil engineering structures up to and beyond their design limits and long term prediction of environmental problems.

The WES centrifuge modeling capability marks the transition in centrifuge development from academic research to broad application in engineering research and problem solving. The increase of gravity in a centrifuge and its influences on physical processes provides great benefit for modeling field phenomena. The opportunities for application to physical processes are limited only by imagination.

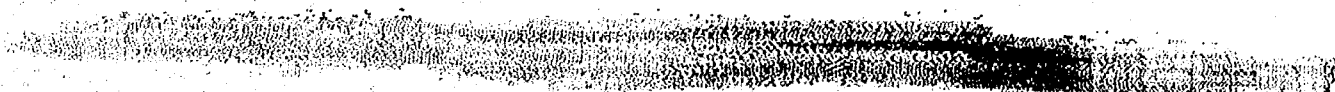
## 8 ACKNOWLEDGMENTS

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# APPENDICES



# 1. PROGRAM SCHEDULE

September 28, 1998

at Conference Room (8F) PWRI

**10:00-10:20 Opening ceremony**

Opening remarks by *K. Yamamoto*, Deputy Director-General, PWRI

Remarks by *M. Sharp*, U.S.-side Acting Chair of Task Committee H

Remarks by *O. Matsuo*, Japan-side Chair of Task Committee H

**10:20-12:10 Session 1**

Chair: R. Ledbetter

*R. Dobry*: Soil Liquefaction Lateral Spreading and Effects on Foundations

*M. Hamada*: A Consideration on Mechanism of Large Ground Deformation Caused by Liquefaction.

*R. Phillips*: Modeling Slides Due to Flow Liquefaction

**12:10-12:20 Photograph (Entrance of PWRI Main Building)**

**12:20-13:00 Lunch (Conference Room, 1F)**

**13:00-14:10 Session 2**

Chair: M. Hamada

*L. Finn*:

*A. Yashima*: Prediction of Seismic Behavior of Embankment Founded on Liquefiable Soil

**14:10-14:30 Coffee Break**

**14:30-15:40 Session 3**

Chair: S. Steedman

*H.-Y. Ko*: Centrifuge Modeling of Retaining Walls With Dry and Saturated Backfill

*O. Matsuo*: Seismic Performance of Rigid Retaining Walls

*T. Tsutsumi*: The Dynamic Geotechnical Centrifuge at PWRI

**15:40-16:00 Coffee Break**

**16:00-17:20 Short trip for centrifuge facility and large scale 3D shaking table facility**

**18:30-20:30 Reception at Ushiku Chateau (hosted by Director General of PWRI)**

## September 29

### 9:30-10:40 Session 4

Chair: M. Okamura

*B. Kutter*: Remediation and Apparent Shear Resistance of Lateral Spreading Centrifuge Models

*Y. Tajir*: Reproduction of Large - scale 1g Tests on Dry Sand Deposit and Pile Foundation using Centrifuge Modeling

### 10:40-10:50 Coffee Break

### 10:50-12:00 Session 5

Chair: R. Dobry

*S. Yasuda*: Earthquake - Induced Settlement of an Isolated Footing for Power Transmission tower

*H. Kim*: Deformation Mechanism of Composite Breakwaters under Earthquake Shaking

### 12:00-13:00 Lunch (Conference Room, 1F)

### 13:00-14:10 Session 6

Chair: M. Sharp

*I. Towhata*: Cyclic Triaxial Tests on Crushed Mudstone and Effects of Ageing

*S. Steedman*: Investigations on the Behavior of Liquefying Soils

### 14:10-14:30 Coffee Break

### 14:30-16:15 Session 7

Chair: I. Towhata

*W. Vanadit-Ellis*: Application of Time Frequency Analysis to Strong Motion Data

*T. Namikawa*: A Study on Building Foundation with Deep Soil Cement Mixing Method for Thermal Power Plant — Design Procedure and its Verification by Effective Stress Analysis —

A Study on Building Foundation with Deep Soil Cement Mixing Method for Thermal Power Plant — Verification of the Design Procedure by Centrifugal Experiments —

*R. Ledbetter*: Earthquake Investigation on US Army Centrifuge

### 16:15-16:30 Closing ceremony

Closing remarks by *R. Ledbetter*, Waterways Experiment Station

Remarks by *M. Yasuda*, Director, Earthquake Disaster Prevention Research Center, PWRI

## 2. List of Participants

### Participants From Japan Side

**Dr. Yasutake Inoue**  
Director-General,  
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Japan

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Public Works Research Institute  
Japan

**Dr. Toshinori Kyosai**  
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Public Works Research Institute  
Japan

**Dr. Shigeo Kamio**  
Geotechnical Counselor,  
Public Works Research Institute  
Japan

**Dr. Masahiko Yasuda**  
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Research Center,  
Public Works Research Institute  
Japan

**Mr. Osamu Matsuo**  
Head, Soil Dynamics Division,  
Public Works Research Institute  
Japan

**Dr. Mitsu Okamura**  
Soil Dynamics Division,  
Public Works Research Institute  
Japan

**Mr. Tatsuya Tsutsumi**  
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Public Works Research Institute  
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**Mr. Tetsuya Sasaki**  
Soil Dynamics Division,  
Public Works Research Institute  
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**Mr. Shuichi Tamoto**  
Soil Dynamics Division,  
Public Works Research Institute  
Japan

**Ms. Yukiko Saito**  
Soil Dynamics Division,  
Public Works Research Institute  
Japan

**Mr. Hitoshi Yoshida**  
Head, Fill-type Dam Division,  
Public Works Research Institute  
Japan

**Dr. Yoshikazu Yamaguchi**  
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Research Institute  
Japan

**Mr. Jun Matsuura**  
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Public Works Research Institute  
Japan

**Mr. Noboru Okabe**  
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Public Works Research Institute  
Japan

**Mr. Hiroshi Kobayashi**  
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**Prof. Ikuo Towhata**  
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University of Tokyo  
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Japan

**Prof. Masanori Hamada**  
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**Prof. Yasushi Sasaki**  
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**Mr. Hideki Nagatani**  
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**Dr. Hideyuki Mouri**  
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Forestry and Fisheries  
Japan

**Dr. Chandan Ghosh**  
Banarash Hindu University  
India

**Mr. Hayoung Kim**  
Kyoto University  
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## Participants From U.S. Side

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**Prof. Bruce Kutter**  
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**Dr. Richard Ledbetter**  
Waterways Experiment Station  
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**Dr. Ryan Phillips**  
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**Mr. Michael Sharp**  
Waterways Experiment Station  
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**Dr. Scott Steedman**  
Andrew N. Schofield & Associates  
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**Dr. Wipawi Vanadit-Ellis**  
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