ENG 364

Experiments on full-size piles undergoing lateral spreading conducted at E-Defense Shake Table Facility, Japan

by
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A report submitted to the Earthquake Commission

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Abstract

A large scale experiment was conducted on piles embedded in liquefiable sand adjacent to a quay wall, on the 23rd of March 2006 at E-Defense, the world's largest shake table. This experiment is described in detail, along with the preliminary analysis conducted to simulate the seismic response. The experiment was part of a larger project which aims to significantly improve the seismic performance of structures. A description of the other research conducted within this project is given, including the behaviour observed from full-scale tests on reinforced concrete buildings, wooden residential buildings and soil-foundation systems. The capabilities, features, researchers and the organisation behind the E-Defense facility are described, as well as opportunities for future international collaboration.

Acknowledgements

I wish to acknowledge the support received from the Earthquake Commission, firstly for travel expenses to Japan and secondly for financially supporting the ongoing research project conducted at the University of Canterbury. Thanks also must go to E-Defense for hosting us in Japan and also the Science University of Tokyo for their collaboration.

Contents

1 Introduction

1.1 E-DEFENSE

E-Defense, a large scale three-dimensional shake table located in Miki City, Japan, was opened in January 2005. It is the focus of a large research project aiming for significant improvement of the seismic performance of structures. This research project started in September 2002 and involves the experimental testing and advanced analysis of reinforced concrete buildings, soil-pile-structure systems and wooden residential buildings. As part of the project a large scale experiment was conducted on piles embedded in liquefiable sand adjacent to a quay wall, on the 23rd of March 2006.

1.2 REASONS FOR E-DEFENSE

The construction of E-Defense was extremely expensive, and this facility is also very expensive to operate. However recent earthquakes have demonstrated the need for such a facility and the large social and economic effects of these earthquakes justify the large investment. In the aftermath of the 1994 Northridge Earthquake and the 1995 Kobe Earthquake the inadequacies of our past understanding of earthquake loads and our resulting construction strategies, were highlighted. The Northridge earthquake resulted in the deaths of 51 people, with over 9000 people injured. The damage resulted in a direct cost of US$44 billion. The Kobe Earthquake caused 6433 deaths, and tens of thousands of people were left homeless. The economic consequences exceeded US$100 billion, some 2.5% of Japan's GDP at the time. The engineering profession was given a strong message that our understanding of the performance of full scale structures and complex 3D ground motion characteristics is incomplete.

1.3 CAPABILITIES OF E-DEFENSE

With the goal of improving this understanding, and acknowledging the enormous costs of recent earthquakes, Japan's National Institute of Earth Science and Disaster Prevention (NIED) has constructed the world's largest shaking table facility, known as E-Defense, shown in Figure 1. Located in Miki City, northwest of Kobe, E-Defense is the largest and most technologically advanced test facility in the world. It boasts a 20x15m shake table, which is capable of testing full scale structures of up to six stories in height and weighing 1200 tonnes. It contains a three dimensional motion simulator, capable of reproducing real earthquake ground motions and generating those of predicted serious earthquakes. It is able to generate Kobe-class strong motions even when fully loaded.

Figure 1. General view of E-Defense (NIED (website) 2006)

The facility enables full-size structures to be tested to failure under strong ground motions. As such it is regarded as the ultimate tool for verification of the seismic capacity of structures.

1.4 RESEARCH AT E-DEFENSE

Currently, NIED is conducting a research project titled "DaiDaiToku - Special Project for Earthquake Disaster Mitigation in Urban Areas". A topic within this project is the "Significant Improvement of Seismic Performance of Structures" which will test and analyse the seismic performance of

- reinforced concrete structures
- residential wooden buildings
- soil-pile-structure systems

The research work will be performed by a close collaboration of dozens of Japanese research organisations, universities, independent administrative institutions and private companies.

1.5 REPORT ORGANISATION

This report consists of three main parts. First, an overview of the facility is given; its capabilities, features, organisation and funding. Next a general description of the research performed at E-Defense, through the current DaiDaiToku Project, is given. One experiment within the DaiDaiToku project, a large scale test on piles undergoing lateral spreading, which was observed at E-Defense on March 23rd 2006, is then described in further detail. Finally a conclusion is given to summarise and comment on the future direction of E-Defense and the possibilities of future international collaboration.

2 The E-Defense Facility

2.1 DESCRIPTION OF FACILITY

E-Defense is located in Miki City, a city northwest of downtown Kobe in the Hyogo Prefecture. The facility is quite complex and includes several buildings, as shown in Figure 2.

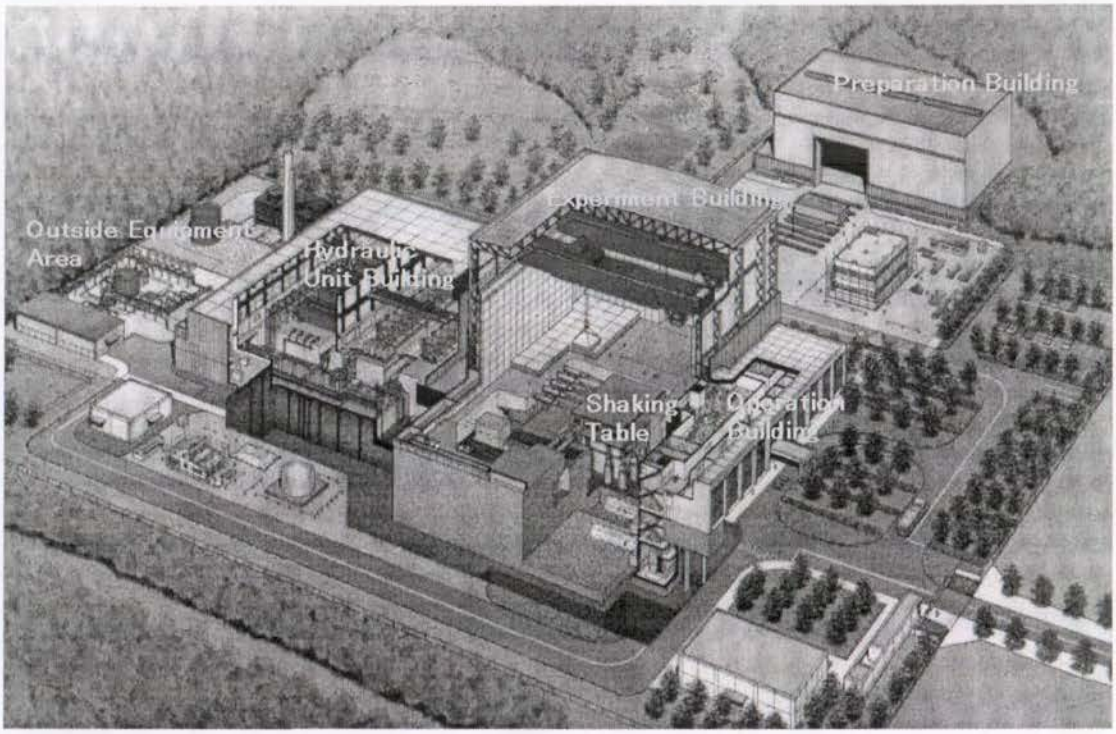


Figure 2. Layout of the facility (NIED (website) 2006)

The experiment building is the largest building and houses the shake table. It is 43m high and has an area of 5200m^2. The preparation building is where the experimental models are built and prepared, before their transportation to the experiment building. The preparation building is 29m high and has a plan area of 2200m^2. The hydraulic pumps, accumulators and gas engines that drive the shake table are located in the hydraulic unit building. The outside equipment area is the location of the main oil tank, water tank, filter house and cooling tower. The operation building contains the measurement and control rooms, as well as reception, conference rooms, offices and a library.

2.2 CONSTRUCTION

The construction began on the E-Defense facility in 1999 and was completed in early 2005. Considerable excavation and site levelling were undertaken in early 2000, which was followed by the laying of the experiment building's massive foundations in 2001. The shake table actuators were installed in January 2003, with the installation of the shake table itself being completed in May 2004. The construction schedule is shown in Figure 3. Figure 4-Figure 8 shows the construction at various stages.

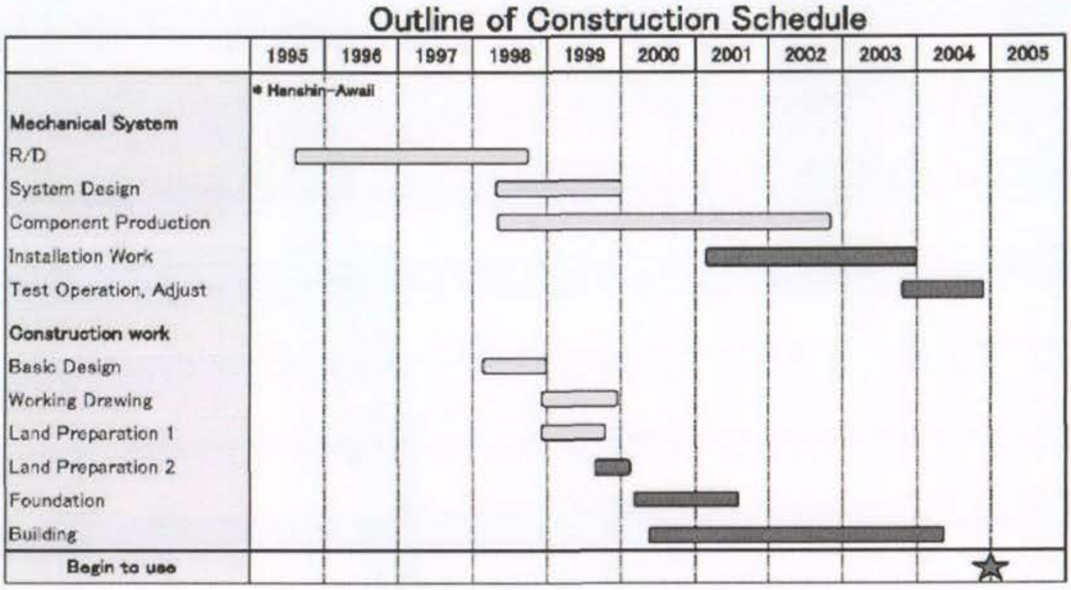


Figure 3. Construction schedule for E-Defense (NIED (website) 2006)

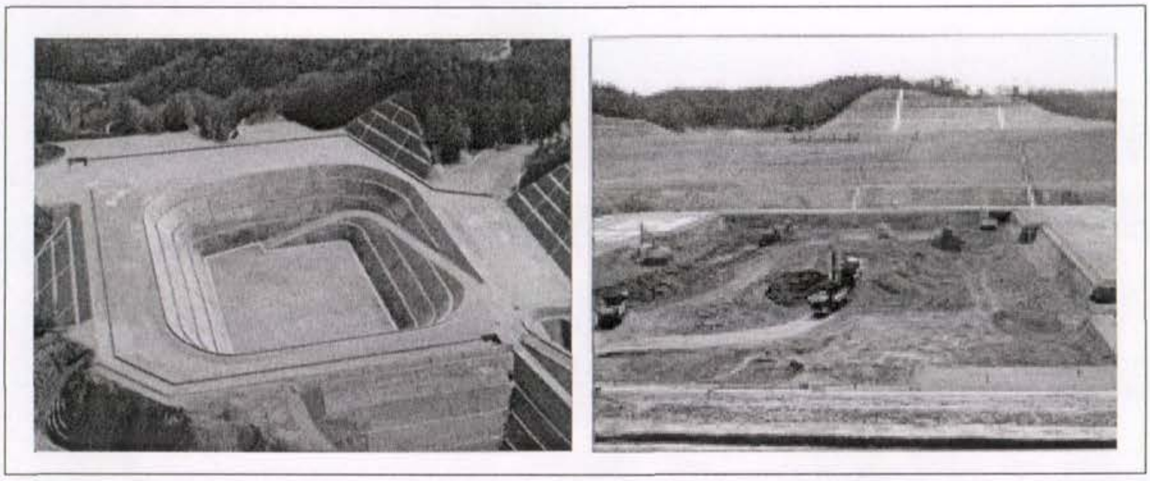


Figure 4. Site preparation and levelling (NIED (website) 2006)

The foundations for E-Defense are very large, to enable a rigid platform for the shake table to operate at full power and yet keep movements relative to the table to a minimum.

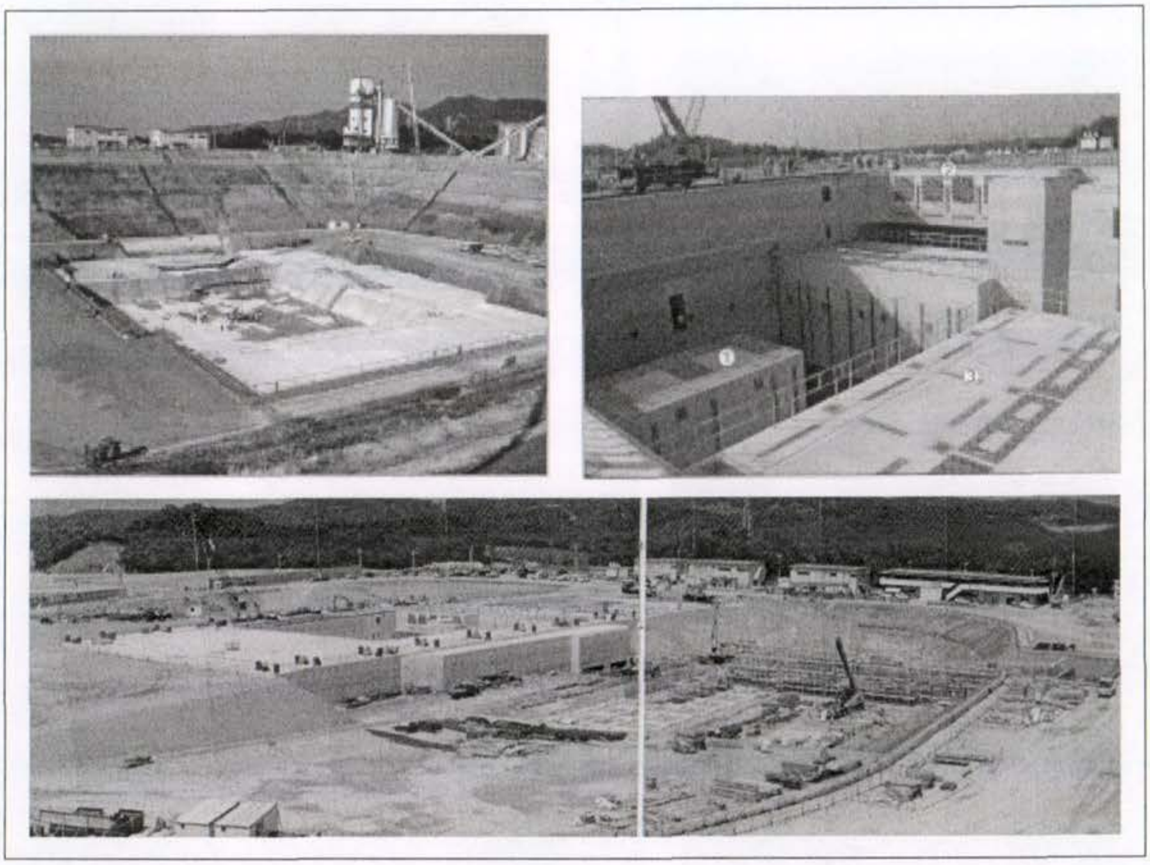


Figure 5. Construction of building foundations (NIED (website) 2006)

The buildings at E-Defense are all steel frame construction, and are designed to be earthquake resistant.

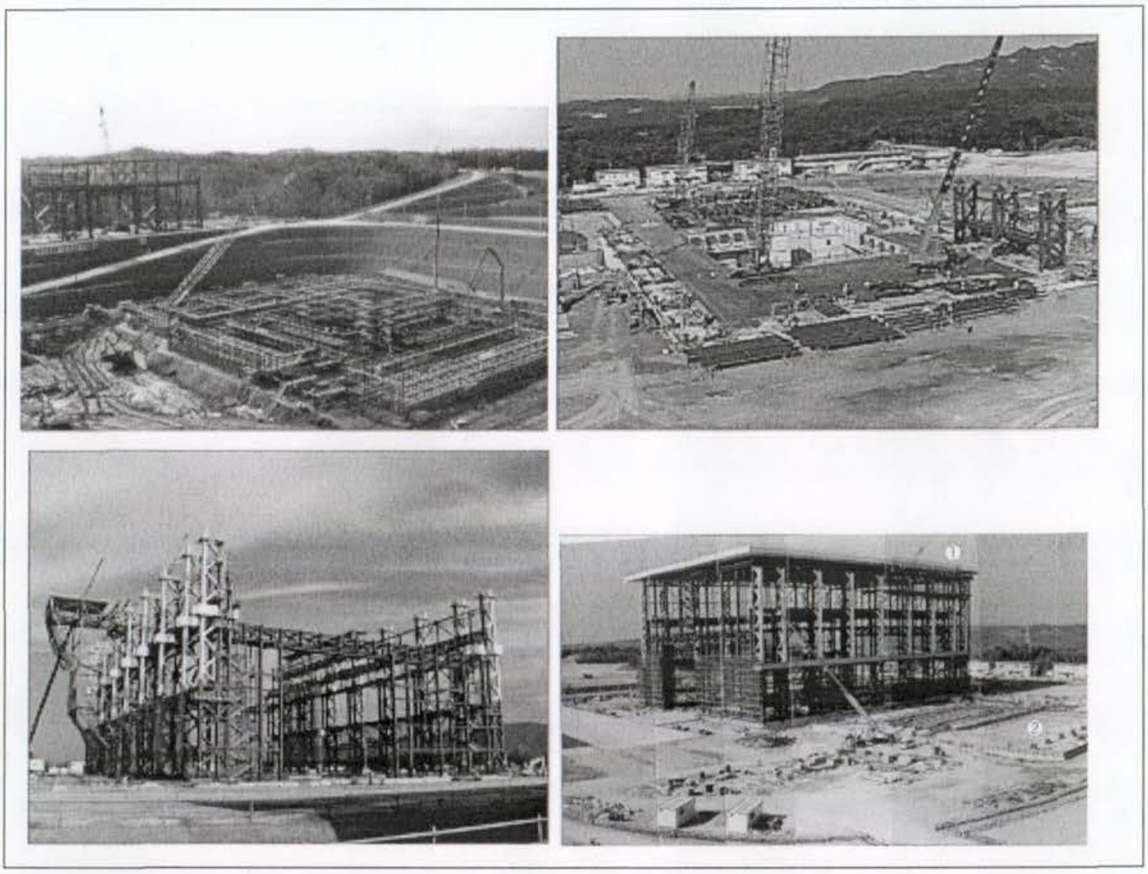


Figure 6. Construction of the experiment building (NIED (website) 2006)

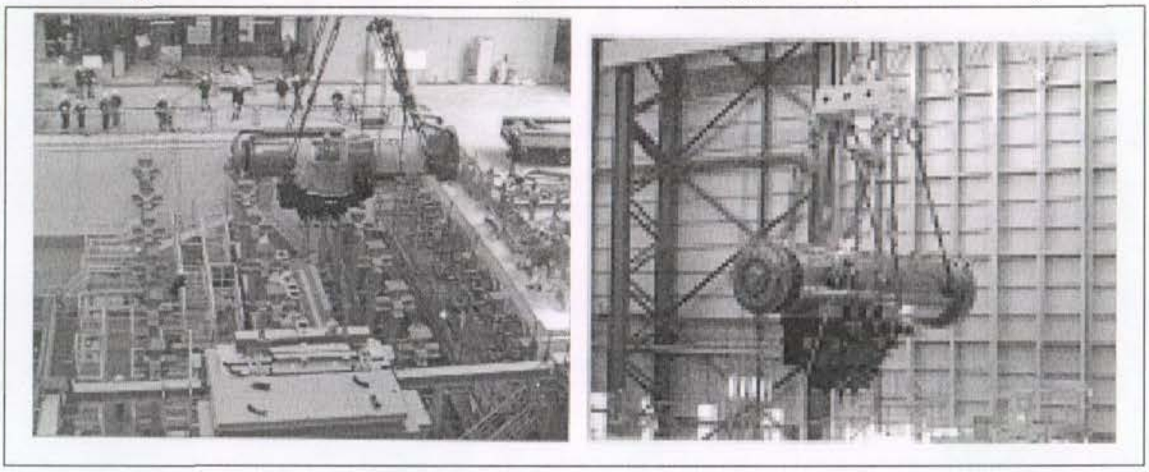


Figure 7. Installation of the actuators (NIED (website) 2006)

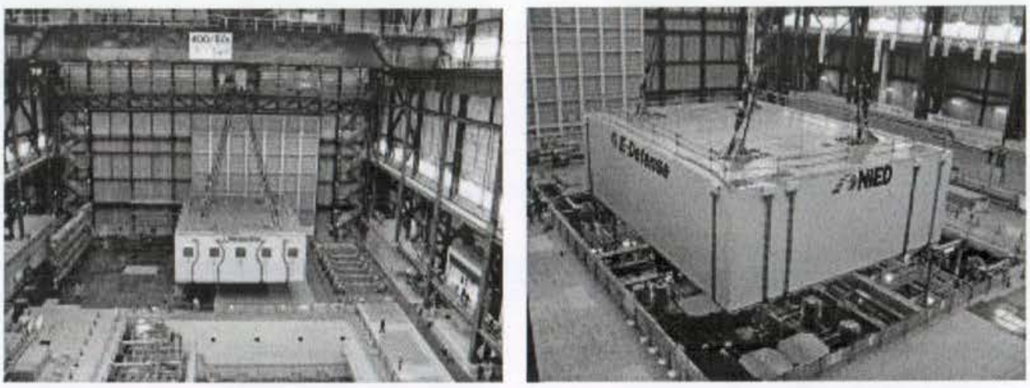


Figure 8. Installation of shake table (NIED (website) 2006)

2.3 3-D LARGE-SCALE SHAKE TABLE

2.3.1 Performance indicators

The shake table is made up of 32 steel blocks welded together on site. It is 15 x 20m in plan area and 5.5m thick and weighs 750 tonnes. The shake table has the capability to apply 3D motions of extremely large intensity (displacement of 1m and nearly 1g acceleration as summarised in Table 1) and therefore is the largest shake table in the world. A photograph of the shake table is shown is Figure 9. The figures shown in Table 1 indicate that the shake table is capable of modelling the performance of real structures subjected to realistic strong ground motions:

- The large size and payload of the shake table enables full size structures to be tested.
- The 3D motion generator can accurately simulate the ground motions of past strong earthquakes.
- The large accelerations, velocities and displacements possible can reproduce even very strong earthquakes such as the 1995 Kobe earthquake.

Table 1. Specifications of the 3D Full-Scale Earthquake Testing Facility (Sato and Inoue 2004)

Size	20x15m		
Maximum Test Weight	1200 tonnes		
Driving Type	Accumulator Electric Oil Control		
Shaking direction	X	Y	Z
Max. Acceleration (at max. loading)	0.9g	0.9g	1.5g
Max. Velocity	130cm/s	200cm/s	70cm/s
Maximum Displacement	±50cm	±100cm	±50cm
Overturning Moment	>15 000t-m	>15 000t-m	>4 000t-m

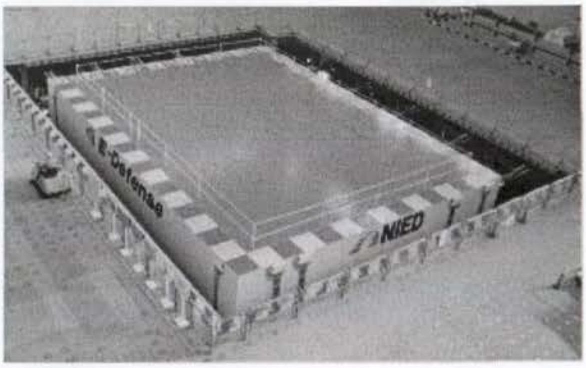


Figure 9. Photograph of shake table (NIED (website) 2006)

2.3.2 Actuators

The shake table is powered by 24 super size actuators, shown in Figure 10, each capable of moving ±1m whilst supporting 120 tonnes of load.

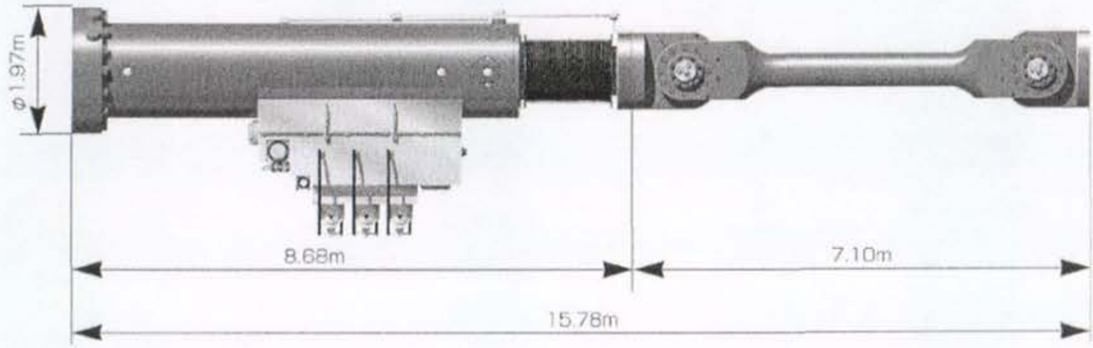


Figure 10. Actuator schematic and dimensions (NIED (website) 2006)

There are five actuators each in the x- and y- directions, and 14 actuators in the z-direction. Figure 11 shows a schematic drawing of the shake table and actuators.

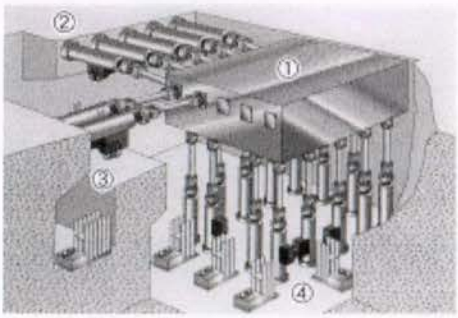


Figure 11. Schematic of shake table and actuators (NIED (website) 2006)

2.3.3 Data Acquisition

A sophisticated measurement and recording system is available within the facility, enabling detailed information on the response, damage and collapse of structures to be recorded and analysed. A total of 960 channels of data can be acquired and stored, with a sampling frequency of 2 kHz. A digital video system is able to take and store videos from various angles, and the images can be displayed on a 100-inch screen, as well as multiple plasma monitors. Ultra high speed cameras are available to measure complex three dimensional motions occurring during large deformations. Figure 12 shows views inside the operation room, with the image recording system and monitors.

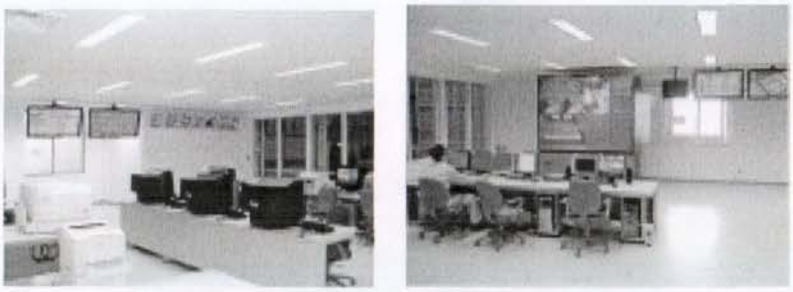


Figure 12. Operations room (NIED (website) 2006)

2.4 ORGANISATION

NIED is a government research agency in Japan. Their aim is to realise a robust society, which defends human life from disasters, learns lessons from previous disasters and promotes sustainable development. Earthquake research is hence only one facet of their operations, which also include research on

- Snow and ice
- Volcanoes
- Landslides
- Floods and storms
- Natural disaster prediction
- Resilient organisations and lifelines

Within the earthquake research programme, specific topics include

- Real-time analysis of the source process of large earthquakes
- Mechanism of earthquakes based on the state of the stress, earthquake source parameters and Earth's structure.
- Research on the seismic activity in the Kanto and Tokai districts
- Research on earthquake occurrence mechanisms
- Research on the transmission and utilization of real-time earthquake information
- Research on societal systems resilient against natural disasters
- Deployment of Asia-Pacific Hazard-mitigation Network for Earthquakes and volcanoes project
- Significant improvement of the seismic performance of structures

E-Defense is one of the facilities operated by NIED to achieve these goals. Other such facilities include a large scale rainfall simulator, a cryospheric environment simulator and numerous wind tunnel facilities.

The goal of E-Defense is to contribute to improving the seismic performance and design of structures. To most effectively utilise the facility, NIED has established a management council and a utilisation committee. The management council includes members from government, academic institutions and private companies. The council is responsible for the medium and long term management plans, with a goal of more effective management of the facility. The utilisation committee is made up of active researchers from various fields of earthquake engineering, who discuss the research plans and test results.

Figure 13 shows the organisation for the operation of E-Defense. NIED, based in Tsukuba, maintains overall control of the facility, but the management council and utilisation committee are responsible for the direction of the research performed at the facility. The day to day running of E-Defense itself is performed through the Hyogo Earthquake Engineering Research Center, a supporting consortium.

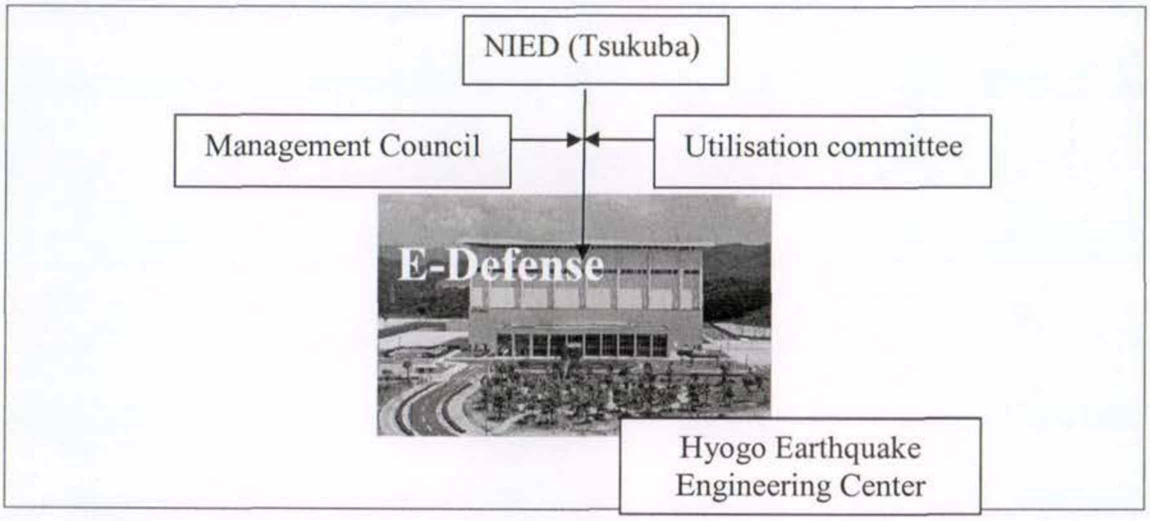


Figure 13. Organisation for the operation of E-Defense (Sato and Inoue 2004)

2.5 OPERATION

A general chart showing the current and future uses of E-Defense is shown below in. Table 2. The current research project, DaiDaiToku, will continue until March 2007, alongside a joint US-Japan research project on steel buildings due to finish at the end of 2007. A Japanese research programme will begin in 2007 also.

Table 2. Current and future utilisation of E-Defense (Sato and Inoue 2004)

Project	2005	2006	2007	Beyond
DaiDaiToku				
US-Japan				
Japanese Research Proj.				
Others				

3. DaiDaiToku – A Special Project for Earthquake Disaster Mitigation in Urban Areas – Significant Improvement of the Seismic Performance of Structures

3.1 OVERVIEW

DaiDaiToku is a very large research project sponsored by the Japanese Ministry for Science, Education and Technology (Monbusho) with the goal of mitigating earthquake disasters in urban areas. The project began in September 2002, and is planned to finish in March 2007. The research is divided into four main tasks:

1. Regional characterisation of the crust in metropolitan areas for prediction of strong ground motion
2. Significant improvement of the seismic performance of structures
3. Advanced disaster management systems
4. Integration of earthquake disaster mitigation research results

The research discussed in this report is part of the second project, the significant improvement of seismic performance of structures.

The research in the second task has a long term goal of mitigating earthquake disasters in urban areas by improving the seismic performance of structures. Its scope includes wooden residential buildings, reinforced concrete buildings and soil-foundation-structure systems. There are a large number of research organisations working on many different projects under these broad goals, and only a few utilise the E-Defense facility.

Dealing more specifically with the soils and foundations group, the research was divided into two phases. The first phase of the project was an initial phase, which involved many experimental tests performed from 2003 – 2005 on medium scale shake tables across Japan. It also served as a preparation phase for the E-Defense experiments, to develop the skills and experience needed to make full use of the large scale experiments. The second phase of the project is concentrated around the E-Defense experiments, which involves the testing of large scale models at the E-Defense facility and advanced numerical analyses.

3.2 PROJECTS

As previously mentioned, the research into the improvement of seismic performance of structures within the DaiDaiToku project is focussed on three types of structural systems: wooden residential buildings, reinforced concrete buildings and soil-pile-structure systems.

3.2.1 Wooden Residential Buildings

Traditional Japanese houses are wooden and often have heavy roof systems to protect against strong winds from typhoons. These top heavy structures are quite vulnerable to earthquakes and the spread of fire, as observed in the 1995 Kobe earthquake. For these reasons wooden houses are one of the principal subjects of research in Japan.

The research performed on wooden residential buildings has two goals. The first is to investigate the three dimensional dynamic behaviour and earthquake capacity of existing conventional wooden houses. The other is to develop both new earthquake resistant designs and retrofitting techniques for existing buildings. Figure 14 shows a schematic view of the houses tested at E-Defense.

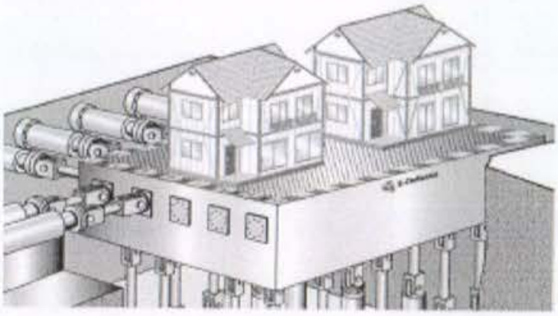


Figure 14. Schematic view of Japanese-style wooden houses on the shake table (Sato and Inoue 2004)

3.2.2 Reinforced Concrete Buildings

The full scale experiments and subsequent research on reinforced concrete (RC) buildings conducted at E-Defense have many goals, including:

- Investigation of three dimensional earthquake response
- Investigation of failure mechanisms of real structures
- To obtain data for the establishment of 3D numerical simulation techniques
- Development of advanced methods for evaluation of the earthquake resisting capacity of a reinforced concrete structure
- Verification of these techniques in order to enable sufficiently accurate evaluation and prediction of structural behaviour
- Development of advanced earthquake resistant structures
- Development of seismic examination and retrofit techniques for existing structures

Figure 15 shows a schematic view of the reinforced concrete frame building tested at E-Defense.

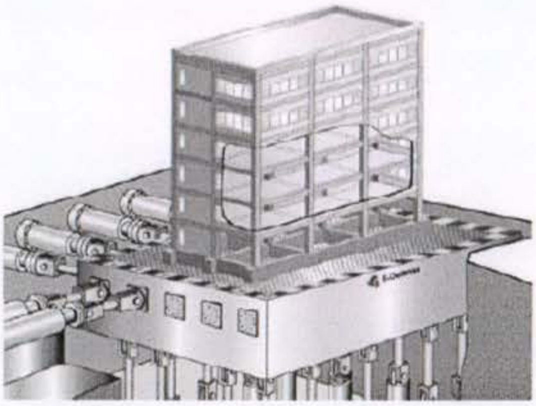


Figure 15. Schematic view of the reinforced concrete building on the shake table (Sato and Inoue 2004)

3.2.3 Soil-Pile-Structure Systems

The soils and foundation structures experiments focus on deep foundations embedded in soil. Two key features of the behaviour, i.e. response to cyclic shaking and response to lateral spreading of the ground, have been investigated with particular emphasis on the effects of liquefaction on foundations. During the Kobe earthquake the piles of many modern buildings, bridges and storage tanks suffered large amounts of damage. This damage can be divided into two groups; piles near the waterfront and piles on level ground. Many studies were conducted investigating this damage, and issues were raised regarding the seismic performance of pile foundations.

The two groups of damage were both modelled as part of the DaiDaiToku project in two separate sets of experiments; level ground experiments and lateral spreading experiments. The level ground experiments were designed to quantify the following aspects of pile behaviour and model them accurately using advanced numerical analysis:

- Cyclic phase of the response
- Performance of the piles
- Effects of 3D input motion
- Effects of soil softening including liquefaction
- Process of pore-water pressure development
- Combination of inertial and kinematic effects
- Mechanism of damage to piles

The lateral spreading experiments were designed to address the following

- Post liquefaction behaviour
- Large ground displacements
- 3D effects
- Combination of inertial and kinematic effects
- Lateral loads from crust layer
- Damage mechanism

Figure 16 schematically shows the two typical experiments performed.

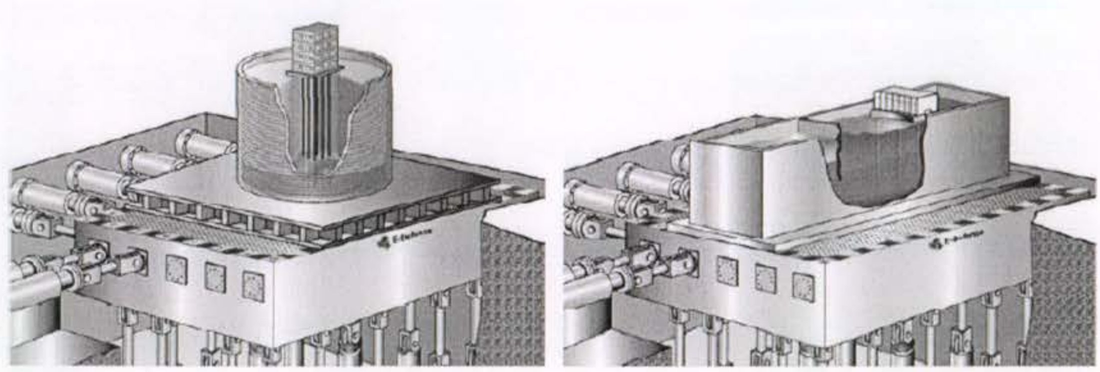


Figure 16. Schematic showing experiments of piles in soil undergoing (a) cyclic shaking and (b) lateral spreading (Sato and Inoue 2004)

3.3 RESEARCHERS

The research is being performed by the close collaboration of a large group of Japanese research centres, universities, independent administrative institutions and private companies. Table 3 shows the organisations involved in the three main research topics.

Table 3. Research topics and related organisations (Sato and Inoue 2004)

Research Topic	Research Organisation
Reinforced concrete structures	NIED, University of Tokyo, Toyohashi University of Technology, Building Research Institute, Kyoto University, Kajima*, Shimuzu*
Soil-pile-structure systems	NIED, Tokyo Institute of Technology, National Institute for Rural Engineering, Public Works Research Institute, Kajima*, Taisei*, Takenkaka*, Tohuku University, Kiso-Jiban Consultants (2002-2005*, Japanese Geotechnical Society, Science University of Tokyo (University of Canterbury)
Wooden residential buildings	NIED, University of Tokyo, Building Research Institute, Kyoto University, Forestry and Forest Products Research Institute, Nihon System Sekkei*
	* denotes private Japanese company

For each research topic, several teams of researchers investigating different subtopics were organised into experimental and analytical groups. The experimental groups conducted a series of large-scale shake tables while the analytical groups predicted the results of these experiments by means of advanced numerical analyses. The research teams were selected in national competitions in 2002 and 2005.

3.4 FUNDING

The DaiDaiToku project is funded by Japan's Ministry of Education, Culture, Sports and Technology (MEXT). MEXT has funded the project since its inception in September 2002, and will continue to fund it until March 2007.

3.5 TIMELINE OF E-DEFENSE EXPERIMENTS

The wooden residential building tests occurred on the 11th and 21st of November, 2005. The reinforced concrete building test occurred on the 13th of January, 2006. The cyclic shaking experiment occurred on 24th of February 2006 and the lateral spreading experiment occurred on the 23rd March 2006. A further cyclic shaking experiment will occur in August 2006, and another lateral spreading experiment in December 2006.

3.6 DAIDAITOKU PROJECT INITIAL PHASE (2003 – 2005)

3.6.1 General

In the first three years of the DaiDaiToku project research was carried out using tests on medium-size shake tables. By and large, for the following research topics:

1. Am experimental study on the 3D earthquake response and failure mechanisms was conducted

2. Advanced 3D numerical analysis and procedures for the experiments were developed
3. The development of the project served as a preparation phase for the E-Defense experiments and this helped decide the optimal experimental programme for the E-Defense facility

In the preliminary study for the wooden house, three models tested on in medium scale experiments:

- A wall element unit extracted from an existing conventional wooden structure
- Frame houses designed to the standard before the revised earthquake-resistant standard was introduced in 1981
- Frame houses designed by traditional methods

The results from these studies served as base data that was used to establish and verify the numerical analysis methodology, and to plan the direction of the E-Defense experiments.

A similar background preliminary study was also conducted for the RC project; with one to three scaled down models tested on existing 1D and 3D shake tables.

For both soil-pile-structure system experiments, the same background research was performed. This involved medium scale experiments on scaled down models and simulation of the behaviour of the piles and soil by means of 3D effective stress.

Section 3.6.2 describes the preliminary studies conducted for the lateral spreading experiment more specifically as an example.

3.6.2 Lateral Spreading Preparation Phase

To fully utilise the large-scale shake table experiments at E-Defense, a three year research programme was conducted to identify important physical and numerical modelling issues. The accuracy of 3-D numerical effective stress analysis in simulating the behaviour of piles undergoing lateral spreading of liquefied soils was determined using observations from medium scale shake table experiments.

From a series of lateral spreading experiments conducted by Japan's Public Works Research Institute (PWRI) in 2003, 2004 and 2005, four medium-scale shake table tests were simulated (NIED 2004). This section describes these tests and simulations in detail.

The experimental models were all similar to the one shown in Figure 17. The pile foundation was embedded in a three layer sand deposit, located 650mm from a sheet pile wall representing the waterfront. The soil deposit was 1.8m thick, with a coarse sand layer above the water table, a loose saturated layer of Toyoura sand (D_r =35%) and a dense saturated layer of Toyoura sand at the base. The submerged sand in front of the sheet pile was also loose Toyoura sand.

The piles were stainless steel, with a diameter of 50.8mm, thickness of 1.5mm and flexural rigidity of EI =12.8kN-m^2. The piles were fixed at the based and rigidly connected to the footing at the top. The pile spacing was equal to 2.5 diameters, i.e. d =128mm. The sheet pile was a 6mm thick steel plate, which was set to free to move and rotate at its base.

The model was built in a rigid container a subjected to a horizontal base excitation in the direction perpendicular to the sheet pile. The base input motion consisted of 20 relatively uniform cycles with a frequency of 5 Hz.

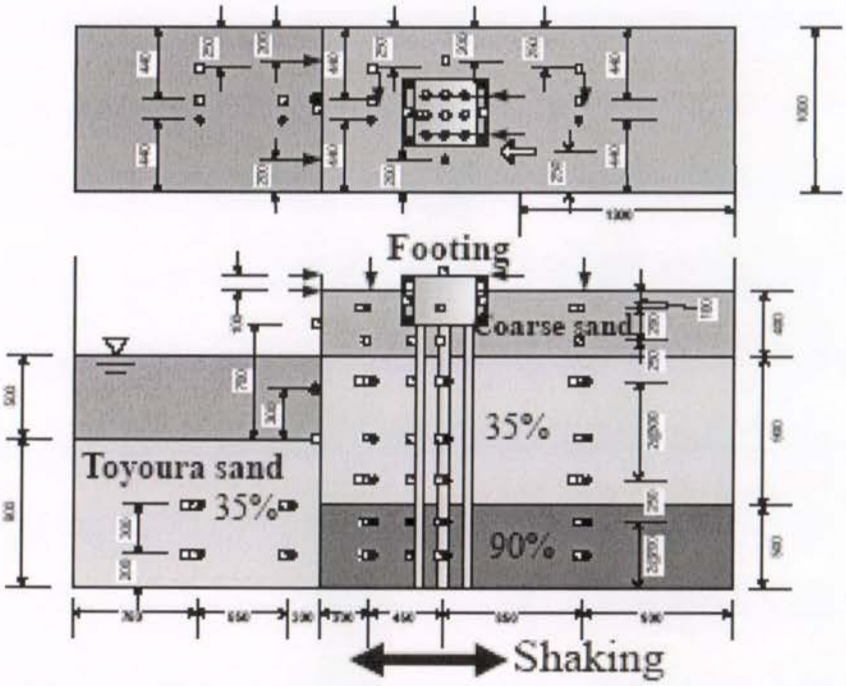


Figure 17. Generic physical model (NIED 2004)

The mass at the pile top, foundation layout and input motion were varied to identify important pile behaviour parameters under different conditions, and whether these parameters could be captured in the analysis. A summary of the conditions used in the experiments are shown in Table 4 and Figure 18.

Experiments H14-1, H15-3 and H16-2 all had a foundation model of 9 piles in a 3x3 group. Each test had a different mass at the pile top, but similar base excitation. Therefore these tests aimed to investigate the behaviour of piles under different inertial loads. However the experiment H16-3 had 4 piles arranged in a 2x2 group subjected to a very large base excitation, thus investigating effects of extreme ground distortion and inelastic pile deformation.

Table 4. Simulated shake table experiments of PWRI (NIED 2004)

Test	Foundation	Mass at pile top (kg)	Peak base acc. (gal)
H14-1	3x3 pile group	**22**	470
H15-3	-- " --	**170** (185)	470
H16-2	-- " --	**320** (358)	500
H16-3	2x2 pile group	140 (145)	**1200**

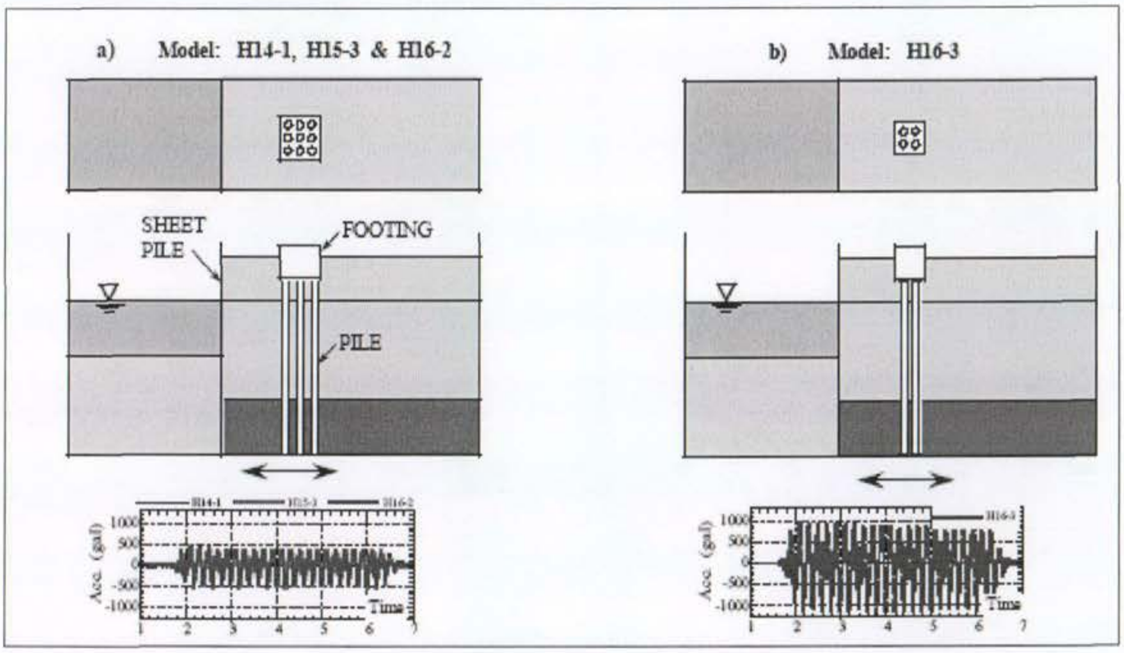


Figure 18. Physical models and base input motions used in the tests (NIED 2004)

The Toyoura sand used in the experiments was modelled using the Stress-Density Model (Cubrinovski and Ishihara 1998a); (Cubrinovski and Ishihara 1998b). The model parameters were determined from a large number of torsional tests (Cubrinovski and Ishihara 1998a); (Cubrinovski and Ishihara 1998b). Particular attention was given to modelling the sand at low confining stress, similar to that experienced in the ground model. The model simulation used for the liquefaction strength of the Toyoura sand is shown in Figure 19(a). The values of γ=3% (dashed lines) and 7.5% (solid lines) represent the value of shear strain achieved.

The stress-strain characteristics of the coarse sand in the surface layer were found using results from drained triaxial compression tests. The stress-strain curve observed in tests with a confining stress of 20kPa was used as a target, and the stress-strain parameters used in the model were set to give a good simulation, as shown in Figure 19(b).

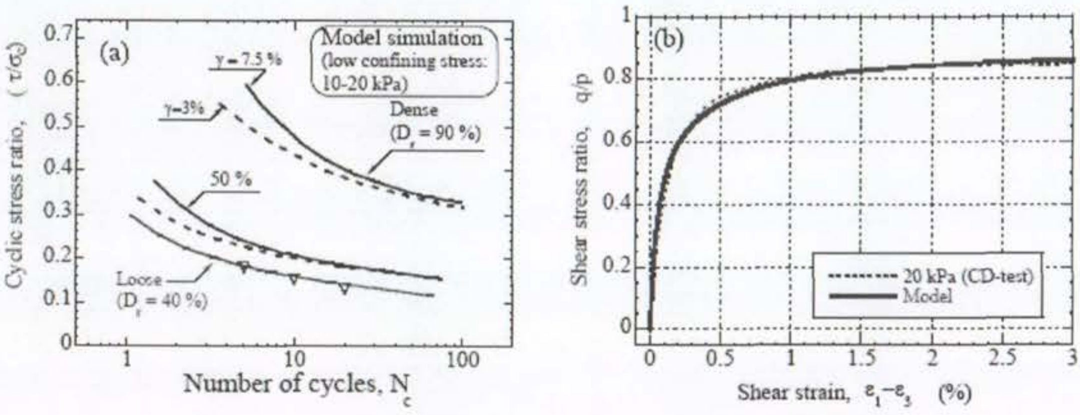


Figure 19. Model simulation of test soils: (a) Liquefaction resistance of Toyoura sand at Dr = 40%, 50% and 90%; (b) Monotonic stress-strain curve of coarse sand crust layer (NIED 2004)

The 3-D effective stress analyses were conducted using the Diana-J3 finite element code with the Stress-Density Model described above. The FEM model consisted of eight-node solid elements and beam elements representing the soil and piles respectively. Solid elements were also used for the footing and sheet pile wall. A half model was used in analysis to reduce computational demands. The numerical model used is shown in Figure 20.

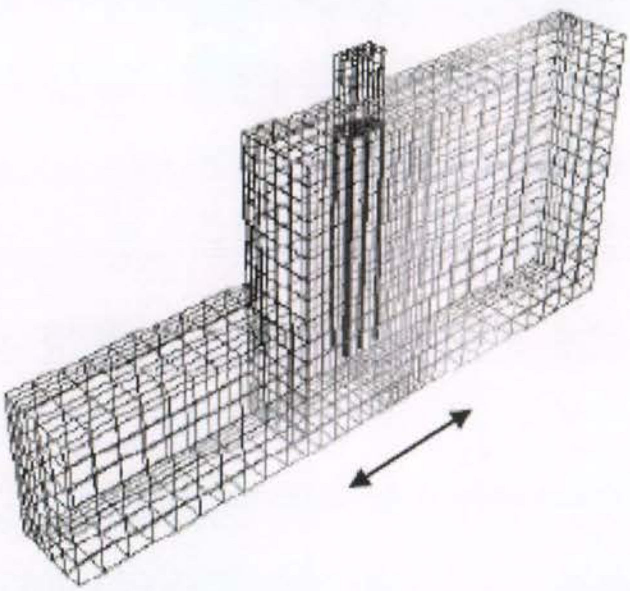


Figure 20. Numerical model used in the 3D analyses (NIED 2004)

Before the analysis, care was needed to correctly evaluate the initial stress state (i.e. before the shaking) caused by the preparation of the model. The analysis was conducted over a period of 6s, with a time step of 0.0004s. Drained conditions were assumed in the analysis.

The analysis was able to predict the excess pore water pressure, displacement of the soil, piles and sheet pile wall, the bending moments in the piles and ground acceleration. Figure 21 shows the computed horizontal displacement, where a blue colour denotes zero movement and a red colour denotes maximum displacement. Figure 22 shows the development of excess pore water pressure calculated for experiment 15-1 (NIED 2004). A red colour indicates an excess pore water pressure of 1, which indicates complete liquefaction of the soil.

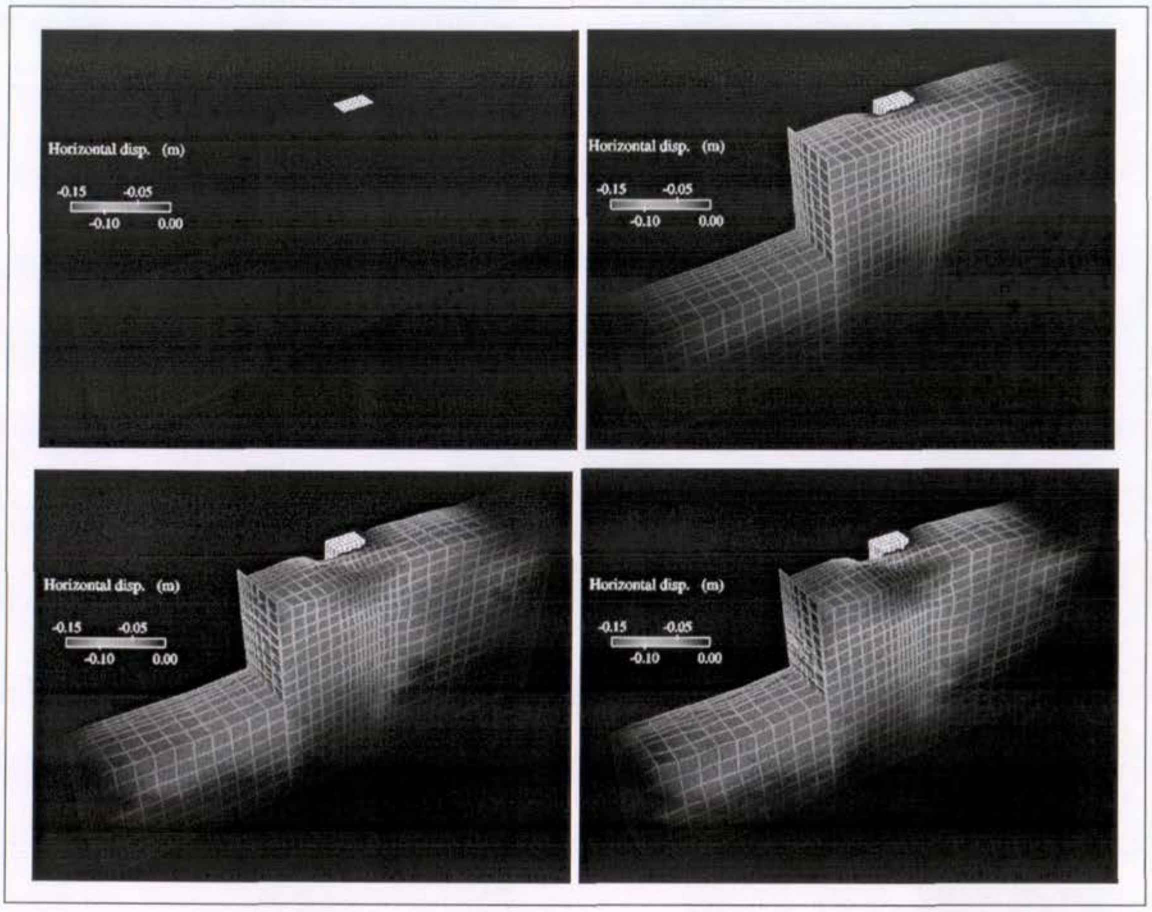


Figure 21. Computed horizontal ground displacement (NIED 2004)

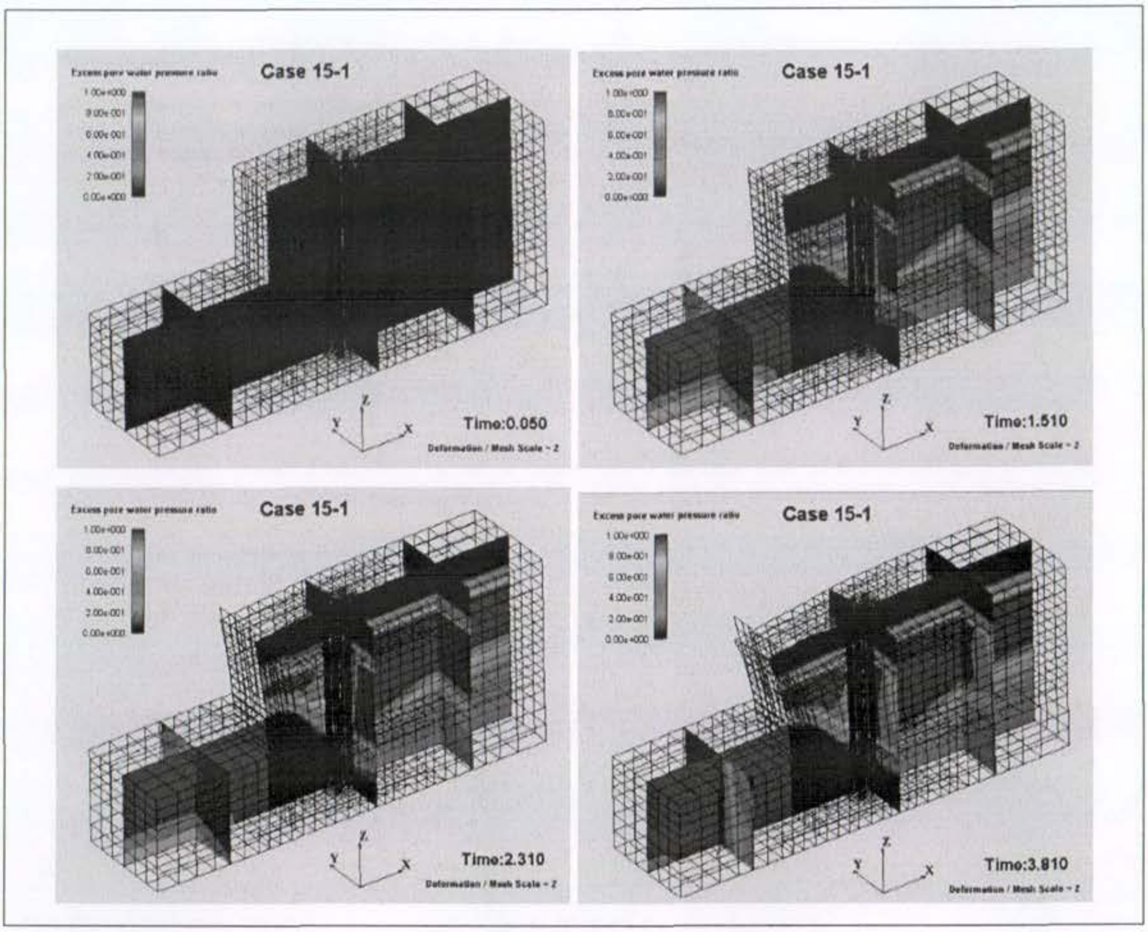


Figure 22. Development of excess pore water pressure (NIED 2004)

The experiments were characterised by a sudden and large lateral movement of the sheet pile wall at the start of the intensive shaking, as shown in Figure 23. In all the test cases, the Toyoura sand liquefied after one or two cycles of shaking. The development of excess pore water pressure and extent of liquefaction were predicted very well. The phenomenon of lateral spreading and settlement of the ground were predicted well also, including 3D effects and the flow-like movement of soil around the foundation. The ground accelerations were also well predicted.

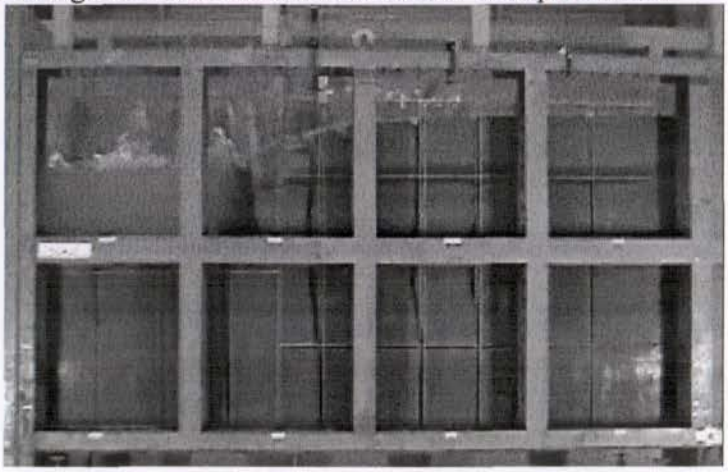


Figure 23. Lateral spreading of the sheet pile wall (NIED (website) 2006)

The displacement of the sheet pile wall was underestimated in all cases, as shown in Figure 24(a). It is thought that the coarse mesh and the use of eight Gauss points limited the movement and development of large displacements of the sheet pile wall. This reasoning is supported by the more accurate results of the 2D analysis, conducted with a single Gauss point only.

The prediction of footing displacement was excellent in all the analyses, especially regarding the peak displacement. The results are summarised in Figure 24(b). Effective stress analysis also enables detailed time history predictions, as shown by the comparison of experimental and analysis footing displacement in Experiment 16-3, shown in Figure 25.

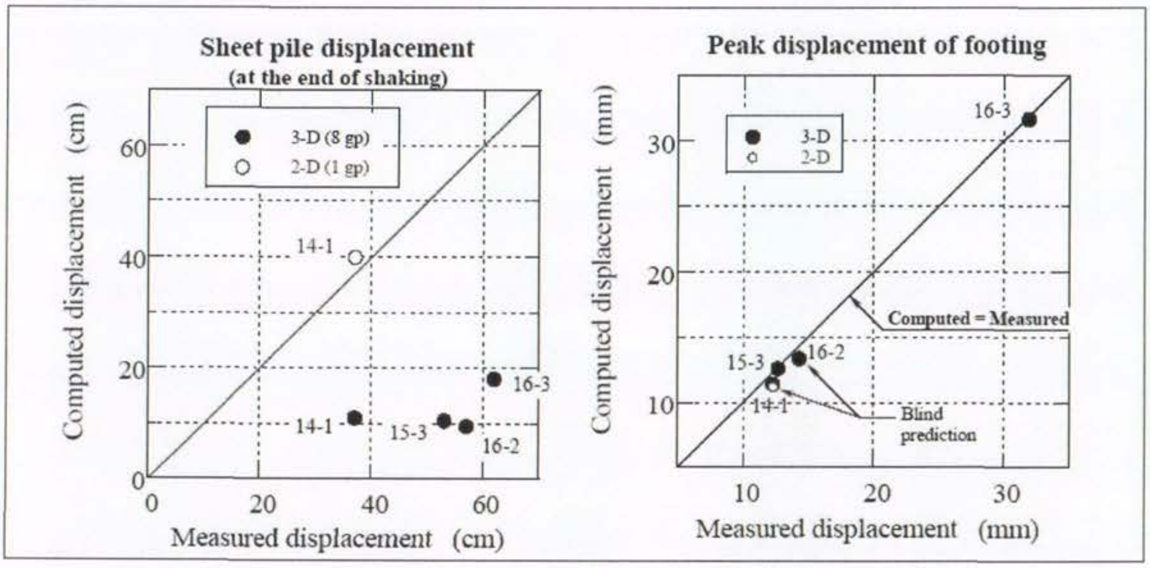


Figure 24. Comparison of computed and measured: (a) final sheet pile displacements; (b) peak footing displacements (NIED 2004)

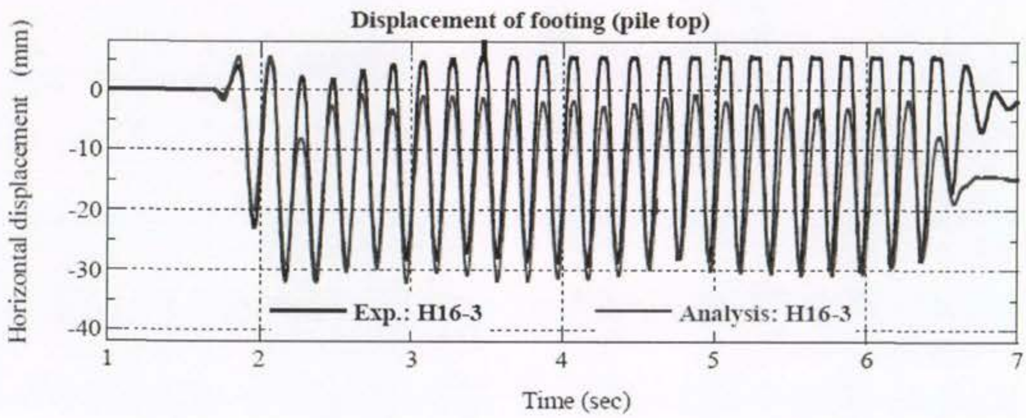


Figure 25. Comparison of computed and recorded displacement of the footing in Experiment 16-3 (NIED 2004)

Good agreement was also obtained for the distribution of bending moments along the pile length, as shown in Figure 26. The bending moments shown correspond to the time of the peak lateral displacement of the footing. After the peak response had been achieved in the tests, the pile foundation showed a gradual return to the initial undeformed state, whereas the computed displacements typically showed more residual displacement.

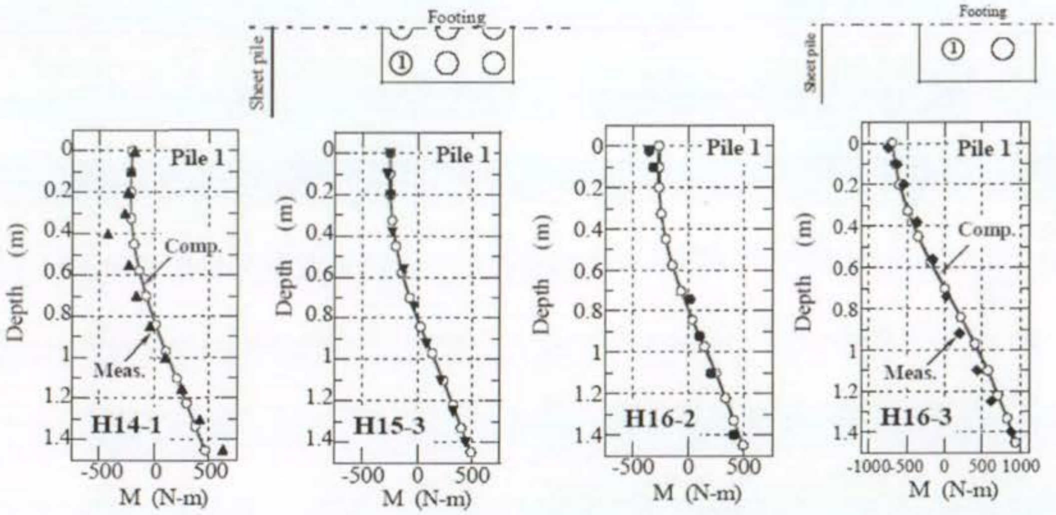


Figure 26. Comparison of computed and recorded bending moments along Pile 1 (NIED 2004)

From the preliminary phase of the lateral spreading research, it was concluded that the 3D effective stress analysis provides a very reasonable simulation of pile behaviour undergoing lateral spreading. Note that many of the above predictions were A-class behind predictions conducted and reported before the execution of the experiments. The method of analysis was strongly recommended for use as the principal tool in the E-Defense experiments, to enable model design, interpretation of the tests and parametric studies.

3.7 E-DEFENSE EXPERIMENTS (2005 – 2007)

3.7.1 General

The large scale experiments at E-Defense attract a large audience of researchers, television crews and the general public. Visitors attend a presentation in the conference room, describing the background, aims and set-up of the experiments. A presentation is then given by researchers who may present a blind prediction of the experiment. The experiments are viewed from a balcony around the perimeter of the experiment building, looking down on the model and shake table. After the conclusion of the test, closer inspection of the model may be allowed either through the crane gondola or by climbing onto the model itself.

The Hyogo Earthquake Engineering Research Center collects and organises the data recorded during the experiment, then distributes the results to the relevant research organisations.

3.7.2 Wooden Residential Buildings

On the 11th and the 21st of November 2005 full scale tests were conducted on wooden residential buildings. Figure 27 shows a comparison between typical buildings in Japan and what was tested on the shake table at E-Defense. Figure 27 (b) shows two identical 30 year old houses transported from their original location to E-Defense. One was reinforced and the other was not.

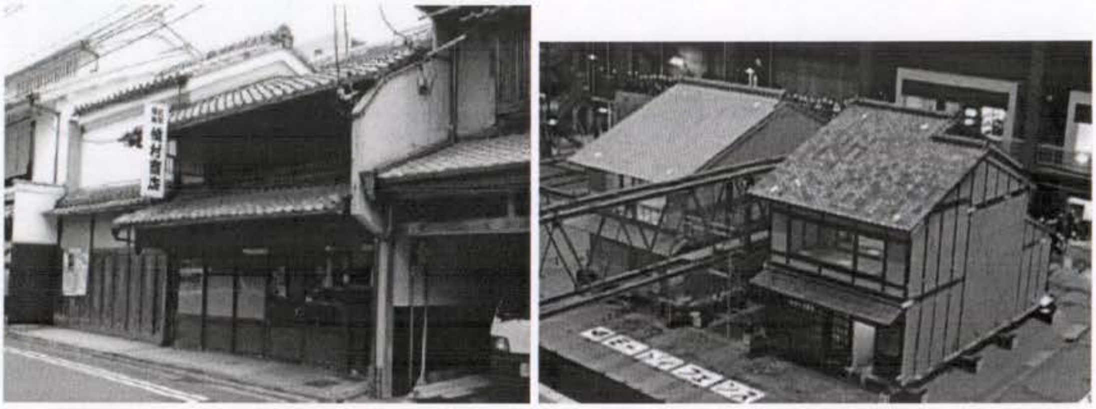


Figure 27. Conventional wooden residential buildings; (a) photo of actual building, (b) photo of building on the shake table (NIED (website) 2006)

Due to the large size of the table it was possible to test the existing and improved designs simultaneously, side-by-side on the shake table. This showed a direct visual comparison of the 3D behaviour and failure modes. Figure 28 shows the behaviour and failure modes observed in the tests. The retrofitted houses performed well and avoided collapse, whereas the traditional construction houses failed on the ground floor, causing the second storey to fall on top of it.

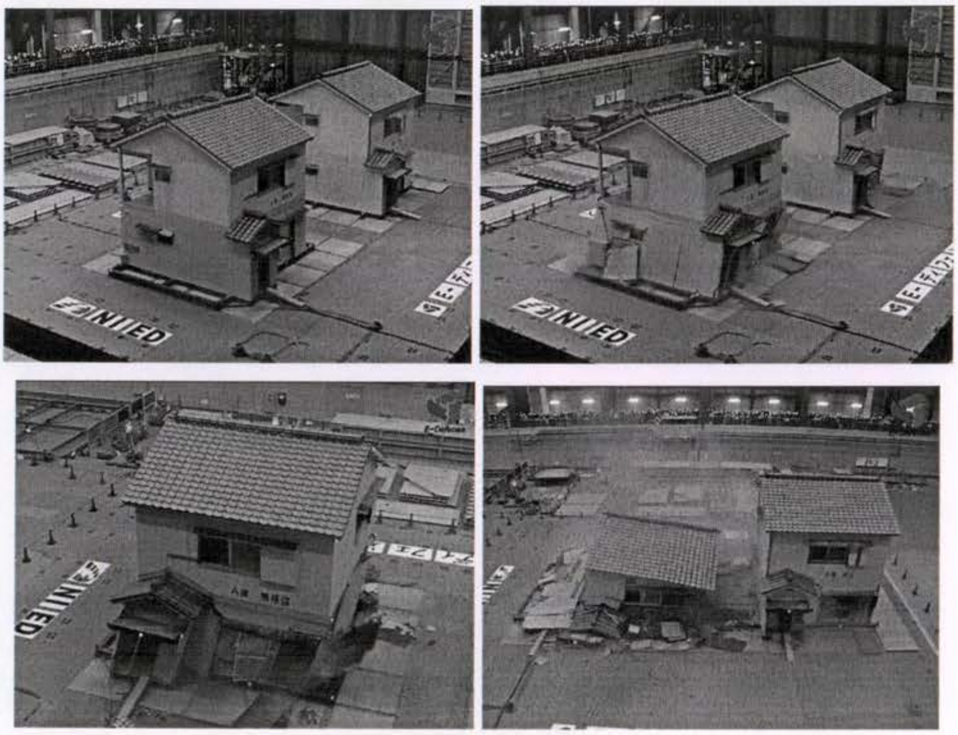


Figure 28. Performance and failure mode of large scale model of wooden residential houses on the shake table (NIED (website) 2006)

3.7.3 Reinforced Concrete Buildings

The model used in the E-Defense experiment is shown in Figure 29. The frame consisted of 2x3 spans and the column spacing was 5m. It was six stories, with a total height of 18m and a mass of 800 tonnes.

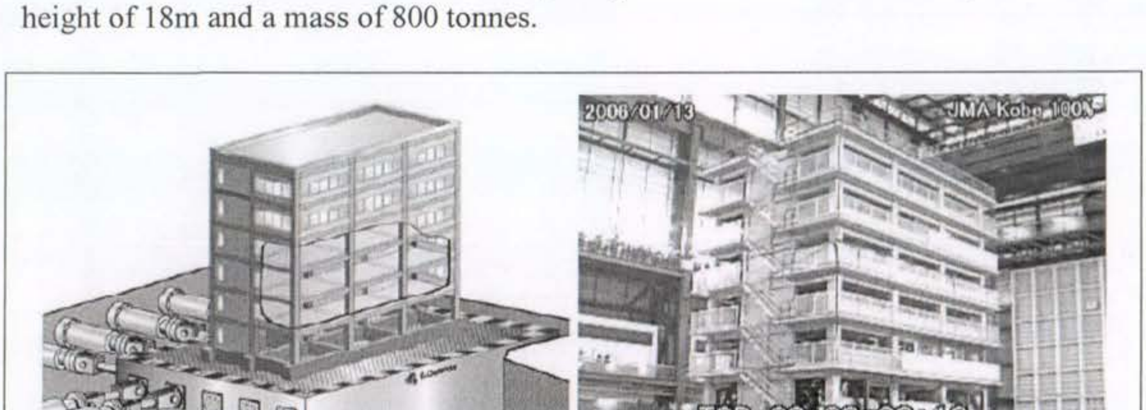


Figure 29. Overall view of the full scale RC structure used in E-Defense, (a) schematic view, (b) actual model (NIED (website) 2006)

The E-Defense experiment was conducted on the 13th of January 2006. Figure 30 and Figure 31 show the behaviour and failure modes observed in the ground floor columns.

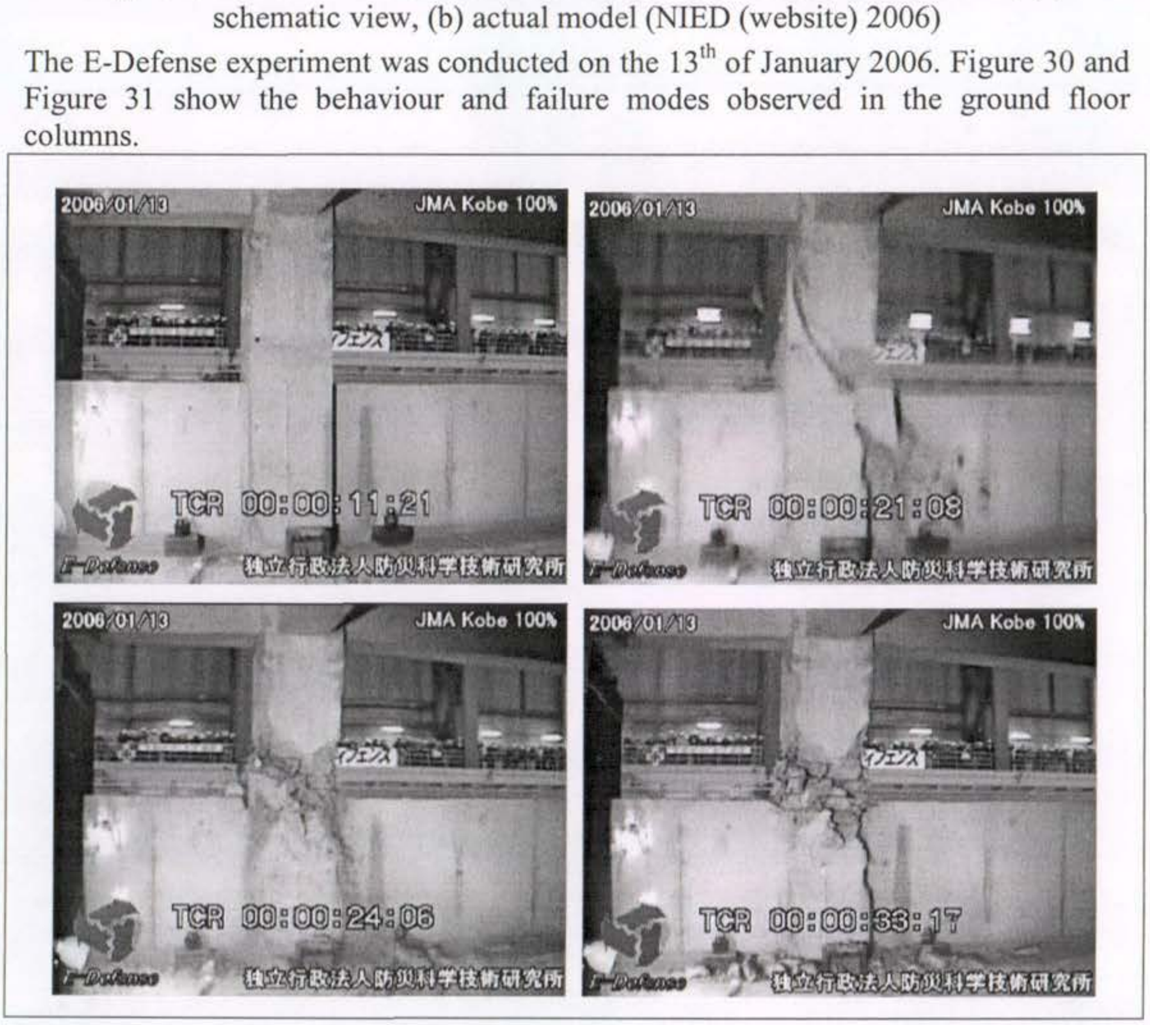


Figure 30. Failure of ground floor columns of RC building (NIED (website) 2006)

Figure 31. Failure of ground floor columns of RC building (NIED (website) 2006)

3.7.3 Soil – Foundation Systems

The soil – foundation systems research focuses on pile foundations in two situations, as shown in Figure 32. The first experiment on soil-structure interaction was conducted in February 2006, and I attended the lateral spreading experiment in March 2006.

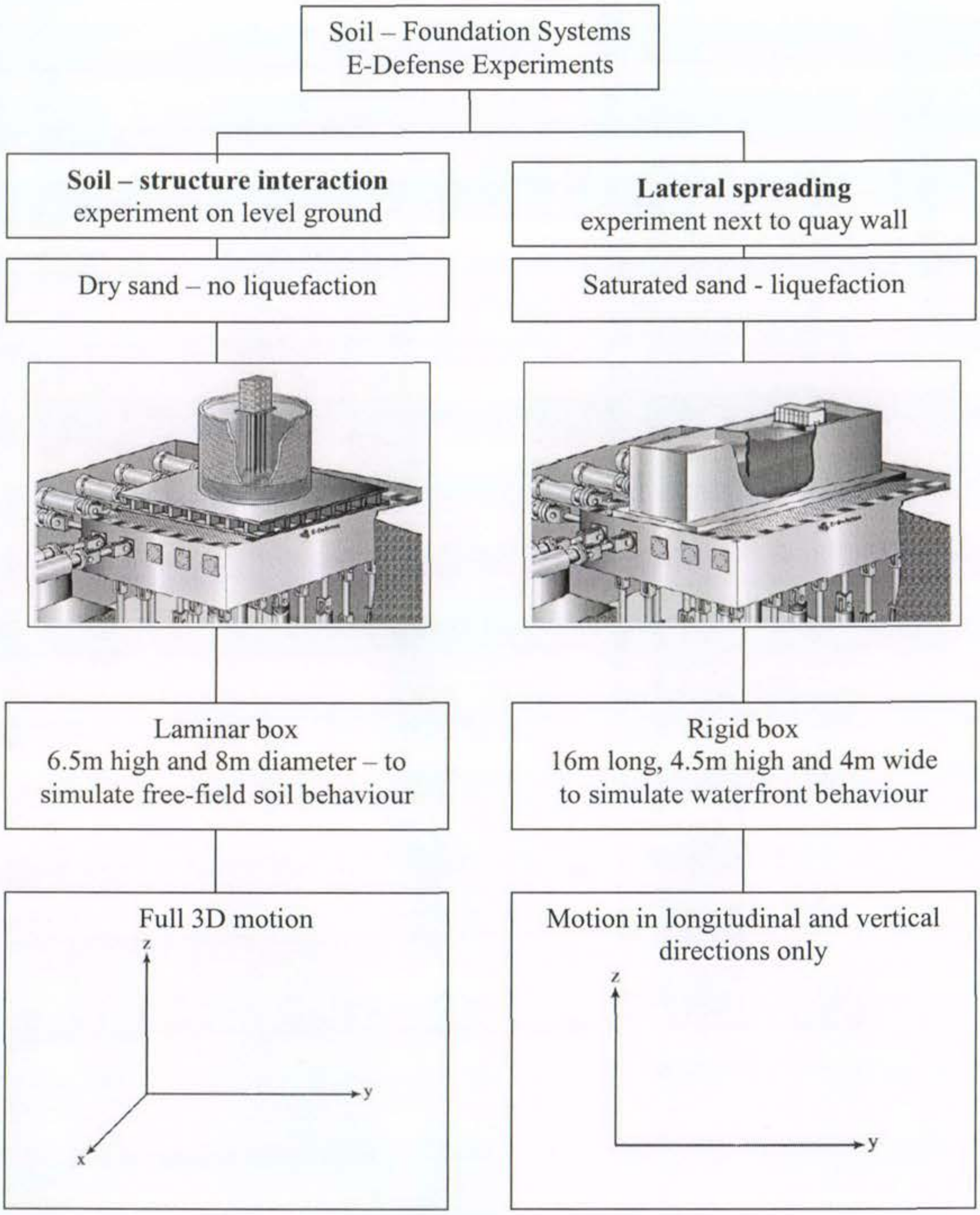


Figure 32. E-Defense experiments on soils and foundations

The test on pile response to cyclic shaking was performed on 24th February 2006. The model was built in a cylindrical laminar box 6.5m high and 8m in diameter. The pile foundation consisted of nine hollow cylindrical steel piles, with a diameter of 150mm and a thickness of 2mm. This test was conducted with dry sand, hence examining purely the behaviour during cyclic shaking of soil, not affected by liquefaction. The experimental set up is shown in Figure 33. Another test with saturated sand will be conducted in August next year.

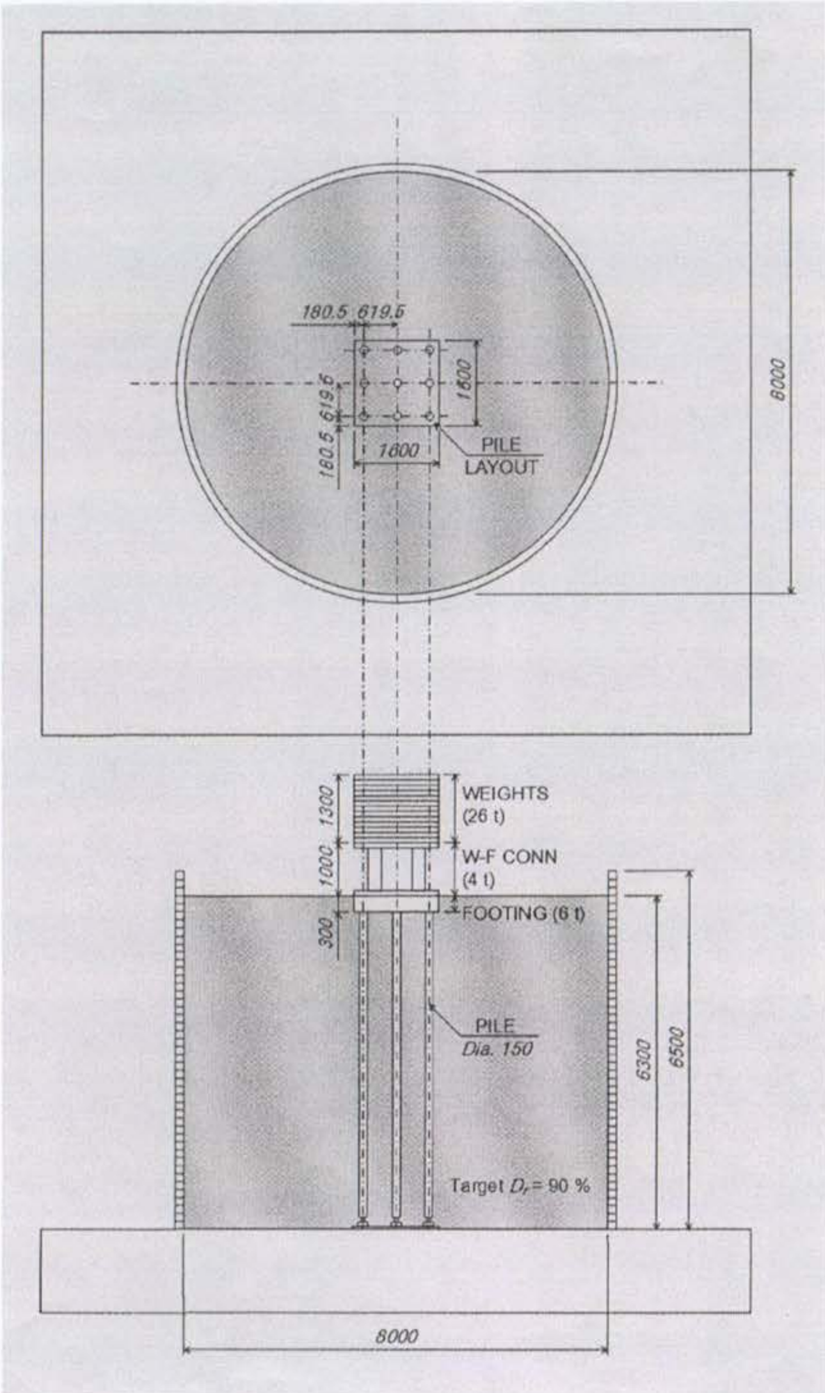


Figure 33. Experimental set-up of E-Defense soil-structure interaction model

Figure 34 shows the model before, during and after shaking, as well as a view after the soil has been removed. The piles were hollow circular steel piles, pinned at the bottom. There was a weight on top of the piles to simulate inertial forces from a building. The inertial forces from the weight above caused the piles to fail 1-2m below the footing.

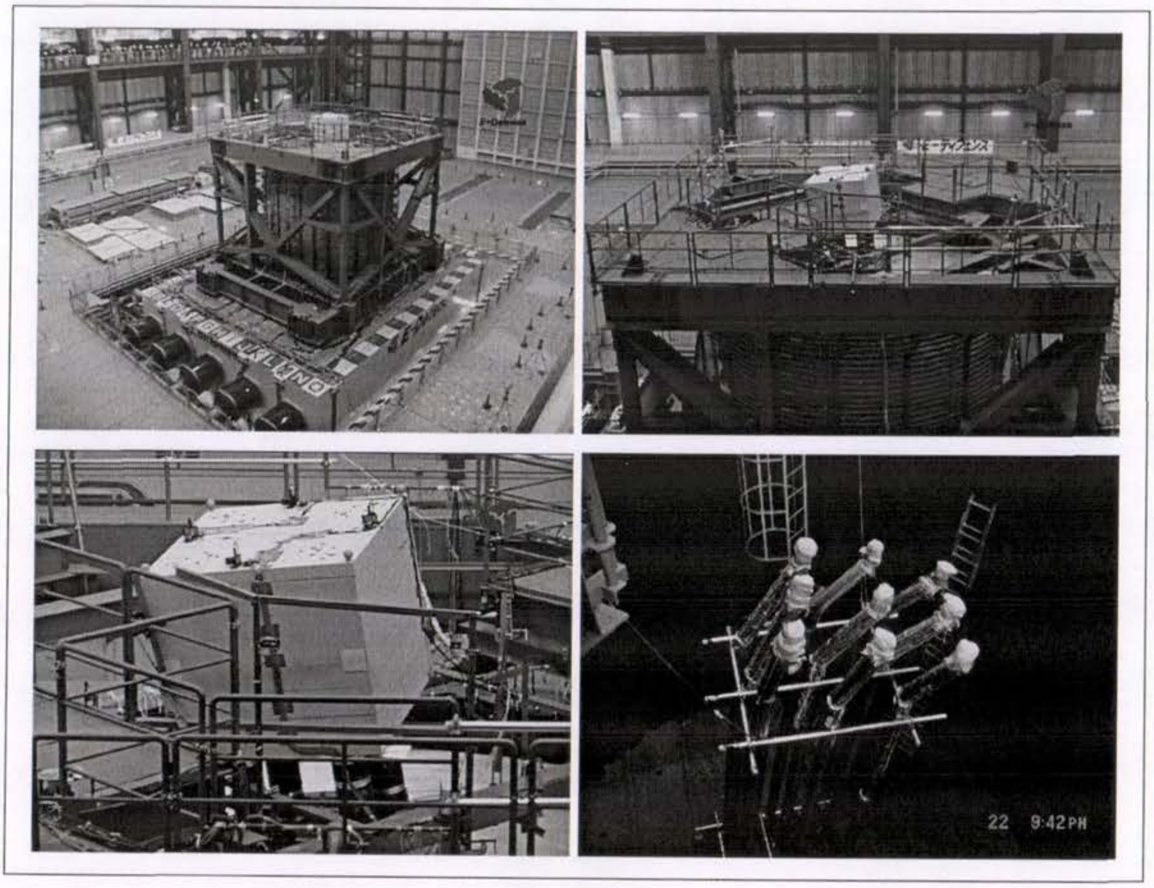


Figure 34. Model of piles undergoing cyclic shaking, (a) before shaking, (b) during shaking, (c) after shaking, (d) after soil has been removed (NIED (website) 2006)

The experiment on piles embedded in liquefiable soil next to a quay wall was observed on March 23rd, 2006, and is described in more detail in section 5.

4 Lateral Spreading Experiment

4.1 BACKGROUND

Lateral spreading of liquefied soil has been a major cause of damage to pile foundations in many strong earthquakes. A number of case histories from the 1995 Kobe and 1964 Niigata earthquakes have shown the vulnerabilities of pile foundations to large ground deformation. These events have lead to intensive research in order to better understand the problems of soil-pile-structure interaction, and to improve the seismic performance of pile foundations.

Large scale shake-table experiments provide a valuable tool for understanding and modelling soil and foundation behaviour. The large model size and ground excitation that E-Defense can provide allows the most realistic experiments to be performed. The dense instrumentation networks and advanced data acquisition facilities can capture all the facets of the experiments, thus providing ultimate verification for the effective stress analysis tools.

4.2 DESCRIPTION OF THE EXPERIMENT

The experimental model is a pile foundation is embedded in backfill soil behind a sheet pile, and the model is shaken in the longitudinal direction. The backfill soil consists of three layers. The surface layer is dry sand above the water table. Beneath that there is a layer of loose, saturated sand, susceptible to liquefaction, which overlies a layer of dense sand. The opposite side of the sheet pile contains water above saturated loose sand.

The experimental model simulates specifically the situation in practice where a building on pile foundations is located on loose, possibly reclaimed soil near the waterfront. It also simulates the general case of lateral spreading, which affects the foundations of buildings and bridges near riverbanks and on gentle slopes. A large weight was placed above the pile cap to simulate inertial forces from a building above.

4.3 PHYSICAL MODEL

4.3.1 Experimental Setup

The model itself is a full-size model of a pile foundation embedded in liquefiable soil. It was prepared in a large rigid box with a plan area of 4 x 16m and a height of 4.5m as shown in Figure 35. The pile foundation is located behind a sheet pile wall representing the waterfront, where lateral spreading is expected. The model was designed to induce a large lateral movement of the sheet pile wall, which would in turn result in spreading of the soil. The piles would then be subjected to both large shaking and lateral ground displacements.

The pile foundation consists of six piles arranged in a 2x3 group. They are steel, 4m long and are rigidly connected to a footing at the pile head and pinned at the base. The properties of the piles are as follows; they have a diameter of 152.4mm, thickness of 2mm and flexural rigidity of EI = 550.4 kN-m^2. The footing has dimensions of 1.6 x

1.6 x 0.5m and a mass of M_F = 10t. To simulate the effects of a building superstructure above, a 12t mass was add to the top of the foundation.

The foundation was embedded in a three layer model of Albany sand. The three layers consisted of (a) a 0.5m thick crust layer of unsaturated sand with D_r = 60%, (b) a 3.3m thick middle layer of saturated sand with D_r = 60%, and (c) a 0.7m thick base layer of slightly denser saturated sand with D_r = 65-70%. On the opposite side of the sheet pile wall is a 3.2m layer of saturated sand with D_r = 60%, underneath 0.8m of water.

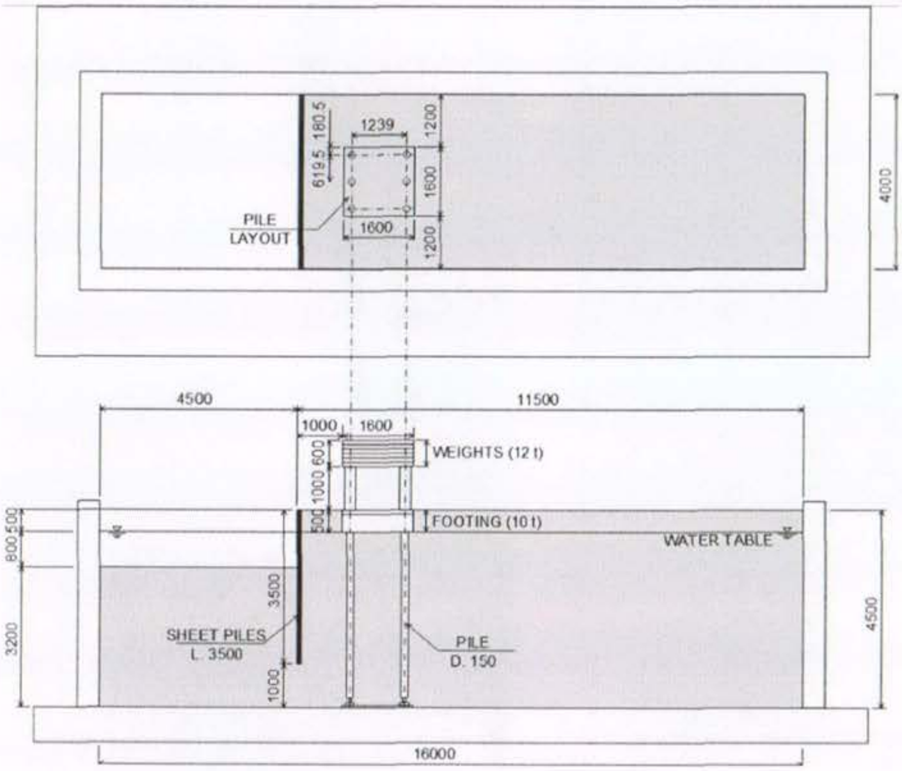


Figure 35. Experimental set-up of large scale shake table test on piles undergoing lateral spreading

4.3.2 Instrumentation

It took three weeks to set up the instrumentation. Over 1000 channels of data were used in measuring the response, which resulted in vast amounts of cables, as shown in Figure 36. The measurement instruments include strain gauges, accelerometers, pore water pressure transducers, displacement gauges, pressure gauges, shape tape and bender elements.

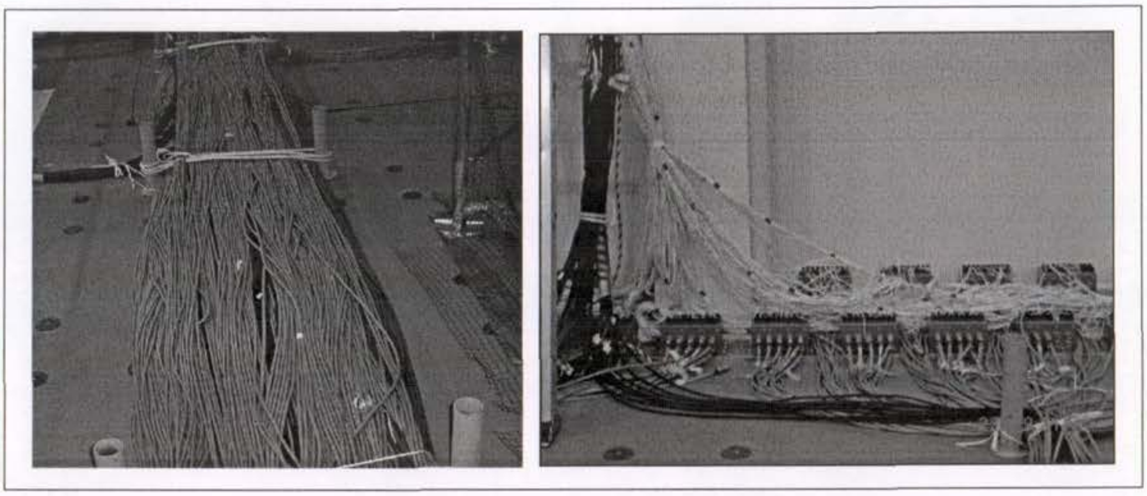


Figure 36. Instrumentation cables

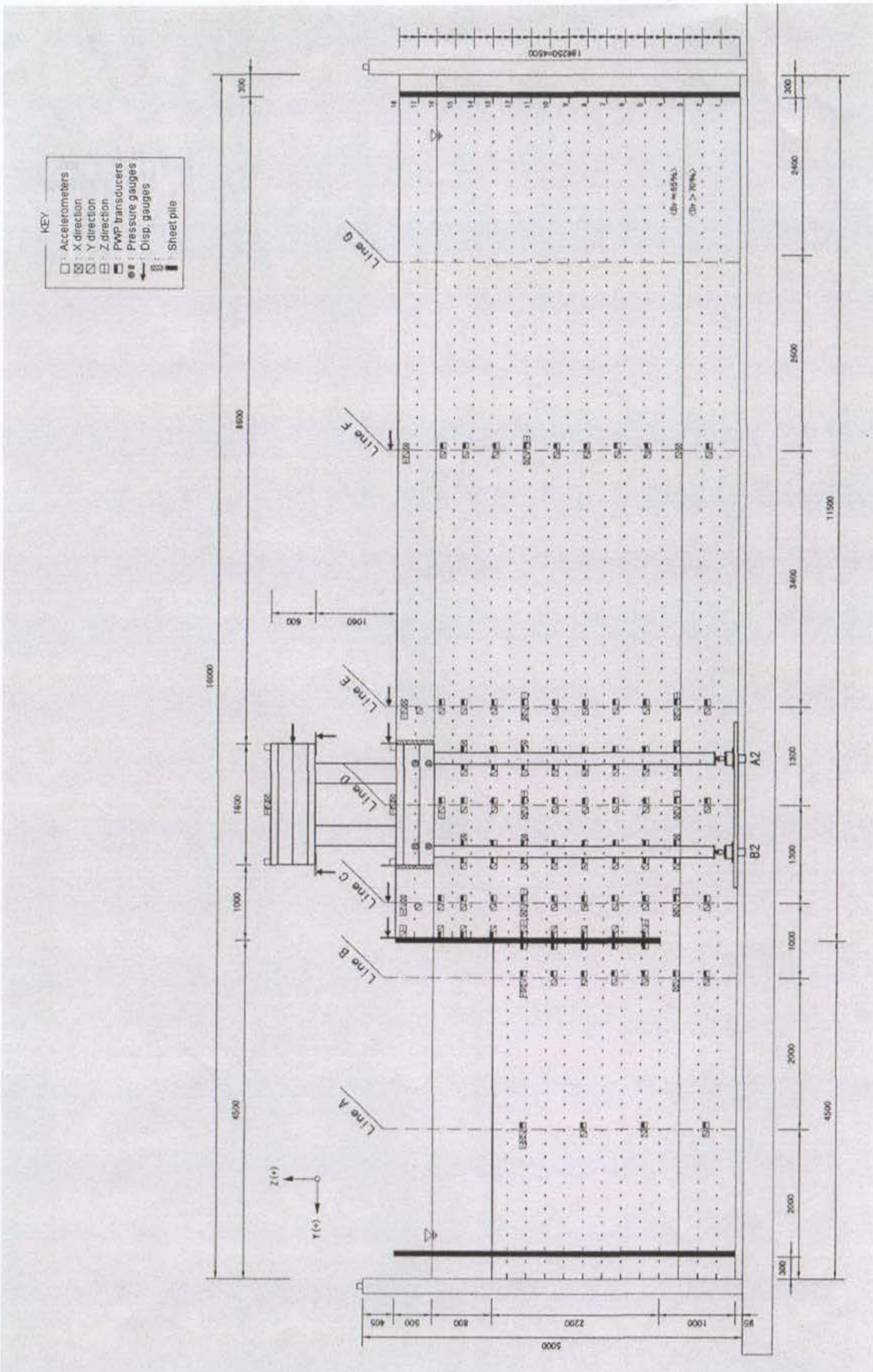


Figure 37. Instrumentation layout for the lateral spreading experiment

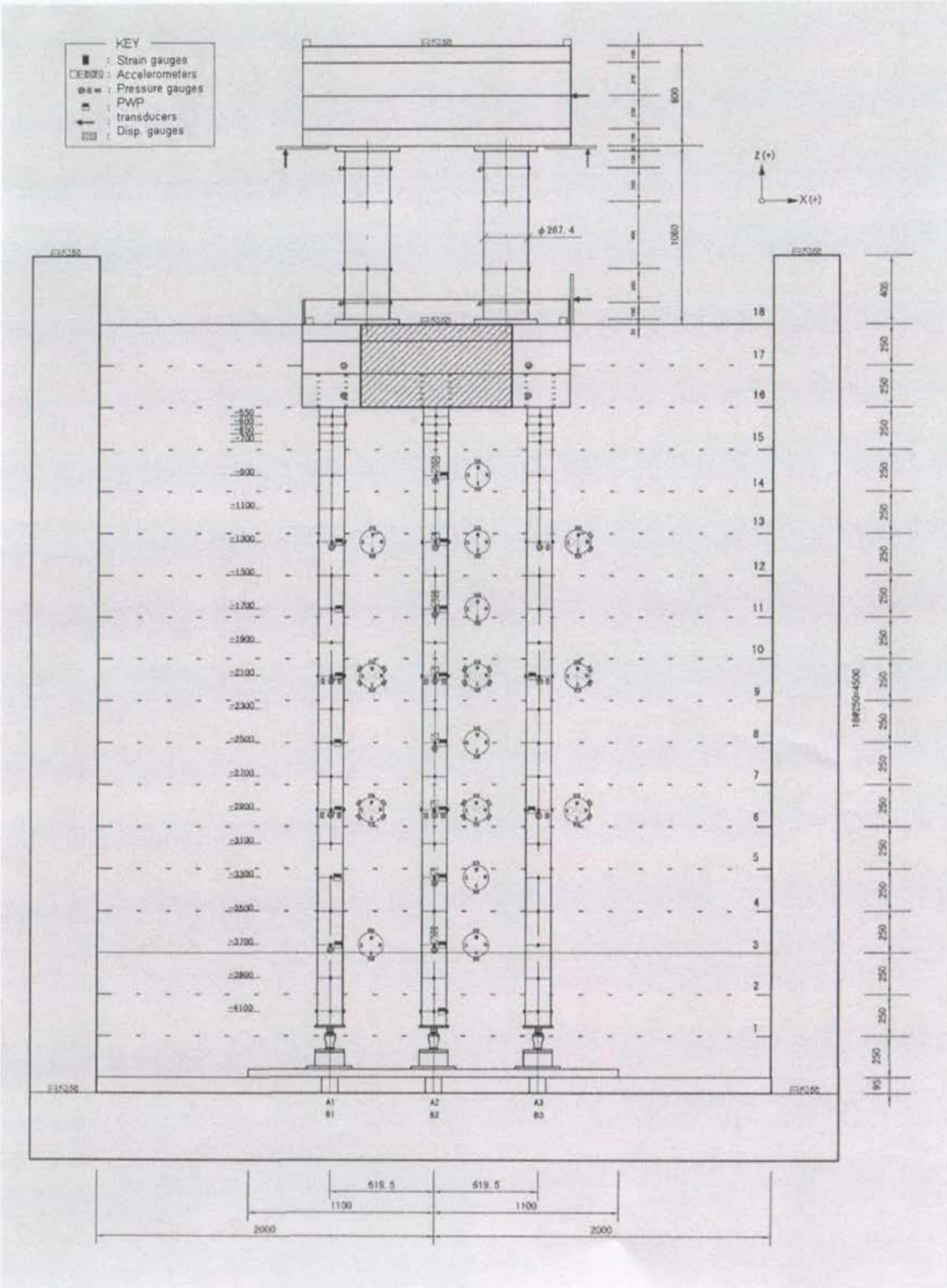


Figure 38. Instrumentation layout on the piles

4.3.3 Photographs

Photographs of the completed model are shown in Figure 39. The model was available for viewing the day prior to the test. Two sets of stairs were erected for instrumentation and inspection purposes, and these were put back up after the test to allow for inspection.

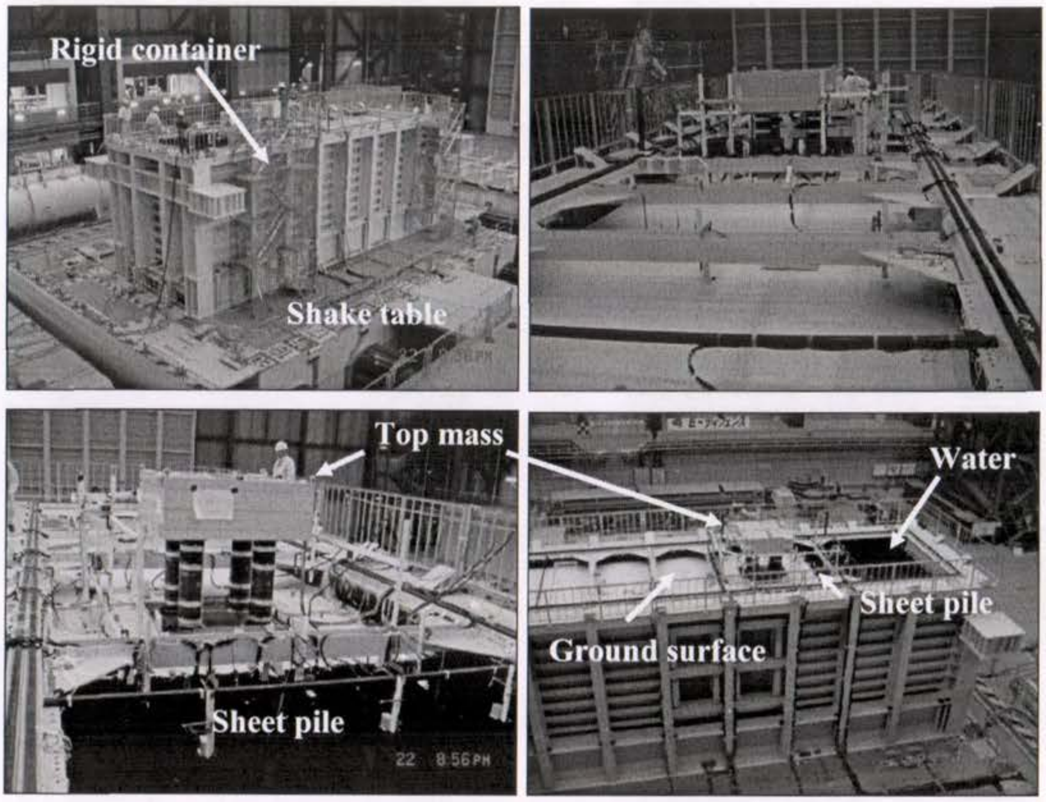


Figure 39. Large scale model before the test

4.4 ALBANY SAND

The sand used in the experiment was sourced from Albany, Western Australia. Albany sand is a uniform sand with very few fines and a grain size distribution very similar to that of Toyoura sand. Albany sand differs from Toyoura sand in that it has predominantly sub-round and round particles, as shown in the microphoto in Figure 40. This is shown in the minimum and maximum densities, with rather low limiting void ratios of $e_{max} = 0.759$ and $e_{min} = 0.469$ respectively.

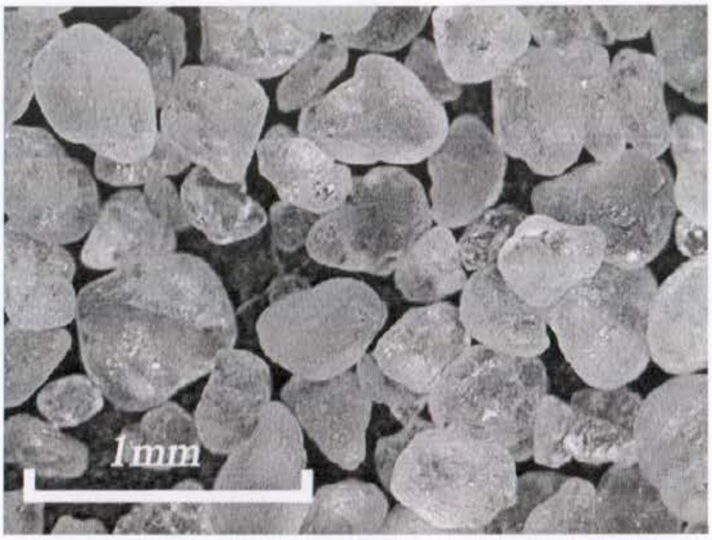


Figure 40. Microphoto of Albany sand (NIED 2006)

4.5 MODEL PREPARATION

4.5.1 Method

The model was prepared in the preparation building at E-Defense. After the construction of the footing and pile foundation the sand was poured into the rigid box used for the experiment in a series of layers 250mm deep. After each layer was poured it was compacted using the vibration compaction technique. The base layer was compacted until a relative density of D_r = 65-70% was reached, whereas the layers above were compacted to a relative density of approximately D_r = 60%.

4.5.2 Transportation

After the sand was deposited the model was transported to the experiment building on large steel frame with supported by a series of rollers, shown in Figure 41. Once inside the experiment building the model was lifted up and set in place on the shake-table using the two 400-tonne overhead cranes.

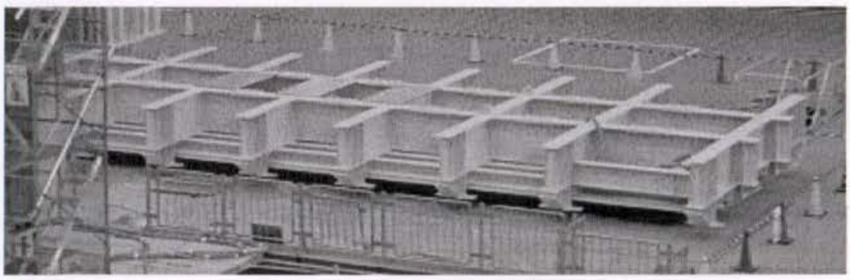


Figure 41. Steel frame on rollers used to transport experimental model

4.5.3 Saturation

Saturation of the model took three days. A large steel cover was placed over the top of the model and a vacuum was applied. Two large water tanks pumped in water from the bottom of the rigid box. The water table was raised until the desired level by correlating the volume of voids in the soil (which was known as the relative densities were known) with the volume of water being pumped into the sample. Instruments inside the box were also used as a check.

4.6 INPUT MOTION

The input motion used in the experiment is known as Takatori, a motion recorded during the 1995 Kobe earthquake. Full size Takatori motion was used in the longitudinal direction (N-S component) and in the vertical direction (U-D component). The first 20 seconds of the Takatori N-S accelerogram used in the analysis is shown in Figure 42. It has a peak ground acceleration of 0.62g. The Takatori motion that was used in the experiment had a further 10-15 seconds of shaking.

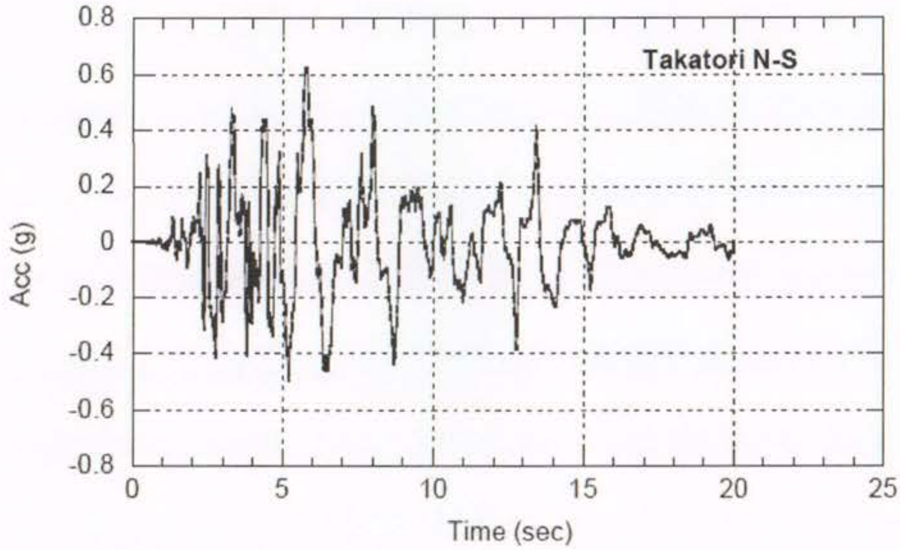


Figure 42. Takatori N-S accelerogram used a input motion in the analysis (NIED 2006)

4.7 EXPERIMENTAL OBSERVATIONS

The shake table and model shook violently for approximately fifteen seconds. It was evident that liquefaction had occurred within the first few seconds of shaking. As the shaking continued the water behind the sheet pile wall sloshed forcefully, eventually spilling over the wall and flooding the sand.

A very large lateral movement of the sheet-pile wall occurred towards the water, with a permanent displacement in the order of one metre. Both the soil and foundation piles moved a similar amount. The piles failed, which was apparent due to the tilting of the superstructure above and the large permanent displacements of the footing. However the footing and superstructure tilted backwards, away from the water, which was unexpected. Figure 43 shows the behaviour of the model during the test.

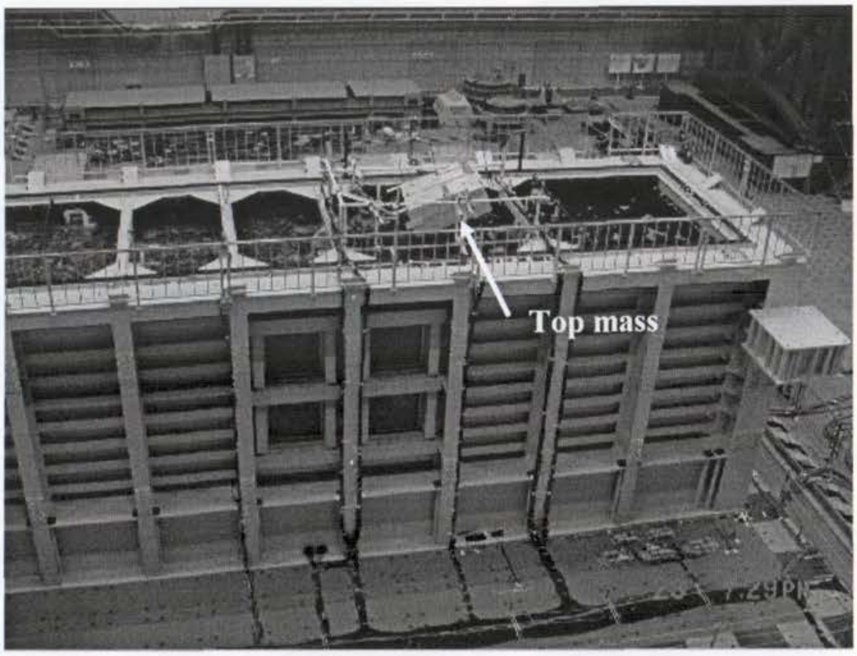


Figure 43. Full-scale experimental model during the shaking

After the test, viewing was allowed via a gondola suspended from the overhead cranes and later by climbing the stairs to the top of the model. The water had drained away and the deformation to the piles, sheet pile wall and soil were able to be seen. Figure 44 shows the model after the shaking.

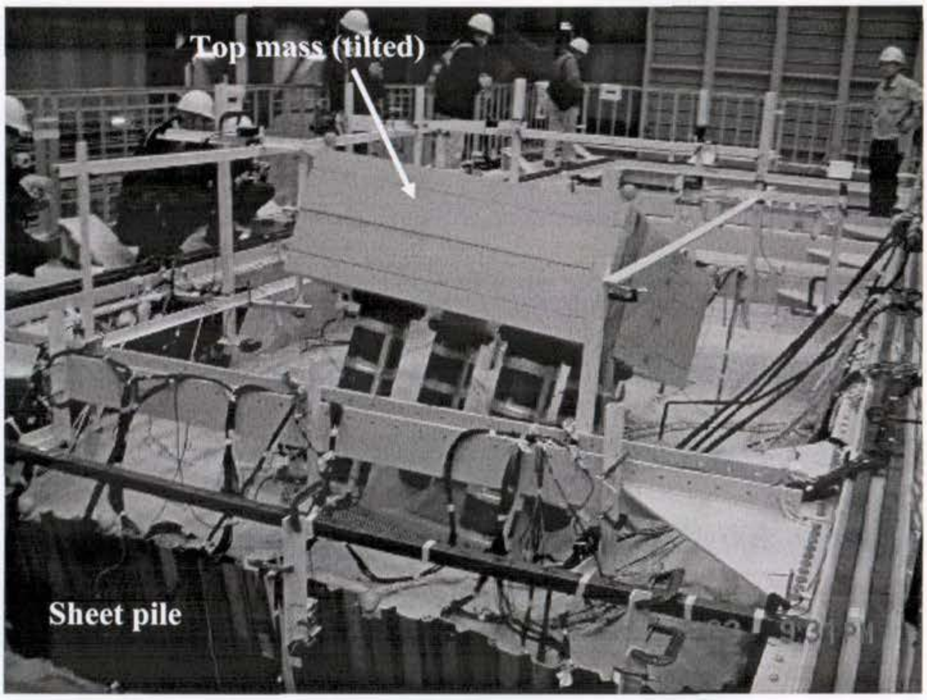


Figure 44. Full-scale experimental model after the shaking

4.8 PREDICTIONS AND MODELLING

4.8.1 General

The primary objective of the numerical modelling was to assess the accuracy of 3-D effective stress analysis in predicting the response of piles subjected to strong earthquake motion and large ground deformation. The intention was to submit a blind prediction of the pile and soil response before the test, however many difficulties arose, most notably that key details of the test preparation and parameters were not known to the predictors during the analysis. As such, the analysis described in this section can be described as a preliminary analysis only.

4.8.2 FEM code and constitutive model

Three-dimensional effective stress analyses were performed to simulate the response of the model. The finite element code Diana-J3 was used for the numerical model, and the Stress-Density Model (Cubrinovski and Ishihara, 1998a; 1998b) was used a constitutive model for the sand. The Stress-Density Model is the same as was used in the preparation phase described previously in section 3.6.2.

The constitutive model uses the state index parameter, I_s, which characterises sand with respect to its density and initial stress state. Hence the model offers an integral representation of sand behaviour over a range of densities and stress states. A modified elastoplasticity rule is employed to develop an incremental formulation that captures the characteristic features of sand behaviour.

4.8.3 Sand modelling

The Stress-Density Model requires four groups of parameters to be determined: elastic parameters, reference lines, stress-strain parameters and dilatancy parameters. Three series of tests must be performed, monotonic undrained tests, monotonic drained tests and cyclic undrained (liquefaction) tests. Each series needs 4 – 5 tests, resulting in a total of 12 – 15 tests to identify these parameters.

Monotonic undrained tests are needed to determine the reference UR and QSS lines on the *e-p* plot and also the dilatancy parameters μ_0 and M. Monotonic drained tests give the stress-strain parameters and cyclic undrained tests give the dilatancy strain parameter S_c. These tests were performed on Albany sand at the Science University of Tokyo and E-Defense. Table 5 shows the model parameters used in the analysis.

Table 5. Model parameters of Albany sand (NIED 2006)

Type	Parameter		Value
Elastic	Shear constant	A	450
	Poisson's ratio	ν	0.20
	Exponent	n	0.90
Reference state	Quasi steady state line:		(e, p')-values
Stress-strain	Peak stress ratio coef.	a_1, b_1	0.66, 0.015
	Max. shear modulus coef.	a_2, b_2	317, 86
	Min. shear modulus coef.	a_3, b_3	255, 28
	Degradation constant	f	1.5
Dilatancy	Dilatancy coef. (small strains)	μ_o	0.20
	Critical state stress ratio	M	0.72
	Dilatancy strain	S_c	0.0055

The results of the undrained conventional triaxial compression tests and torsional tests are shown in the *e-p* plot of Figure 45. The initial states and end states of the tests are shown, producing the red steady state curve. The majority of tests fitted well onto a well defined curve; however the end states of some of the tests did not reach steady state deformation. This was because of either not reaching large enough strain levels or insufficient back pressure employed in the test.

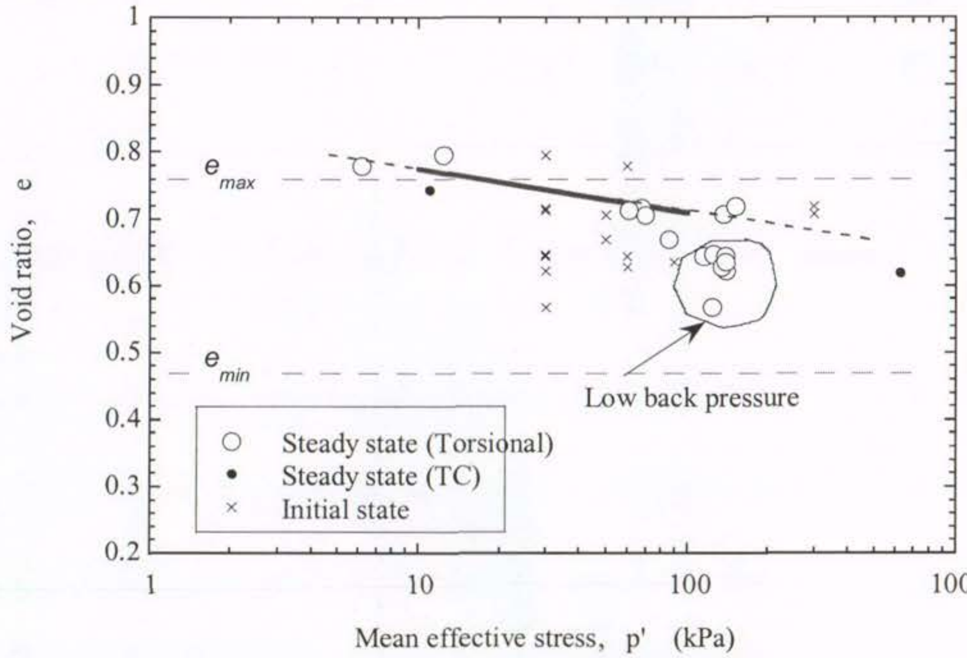


Figure 45. Steady state line of Albany sand (NIED 2006)

Two series of cyclic triaxial tests were performed to determine the liquefaction resistance of Albany sand. Seven tests were performed by Shimizu on samples prepared by air pluviation, with relative densities of D_r = 74-77%. Two additional tests were performed, this time using wet tamping as the sample preparation method, with relative densities of D_r = 64% and 73%, and the results are plotted in Figure 46. As shown in Figure 46, liquefaction resistance varies widely with the fabric and density of the soil. As the ground model in the experiment was prepared by the vibration compaction technique, and the relative density was between 60 - 70%, none of the triaxial tests are representative of the liquefaction resistance of the sand in the model. The difficulties arose because the method of soil preparation used in the tests was not known by the researchers at the time of the element tests. To overcome this, the liquefaction resistance curve of Figure 47, an average of the test data in Figure 46, was used in the simulations. Additional tests will be performed on Albany sand prepared using the dry compaction method at a later date, to enable more precise evaluation of the liquefaction strength of the model sand.

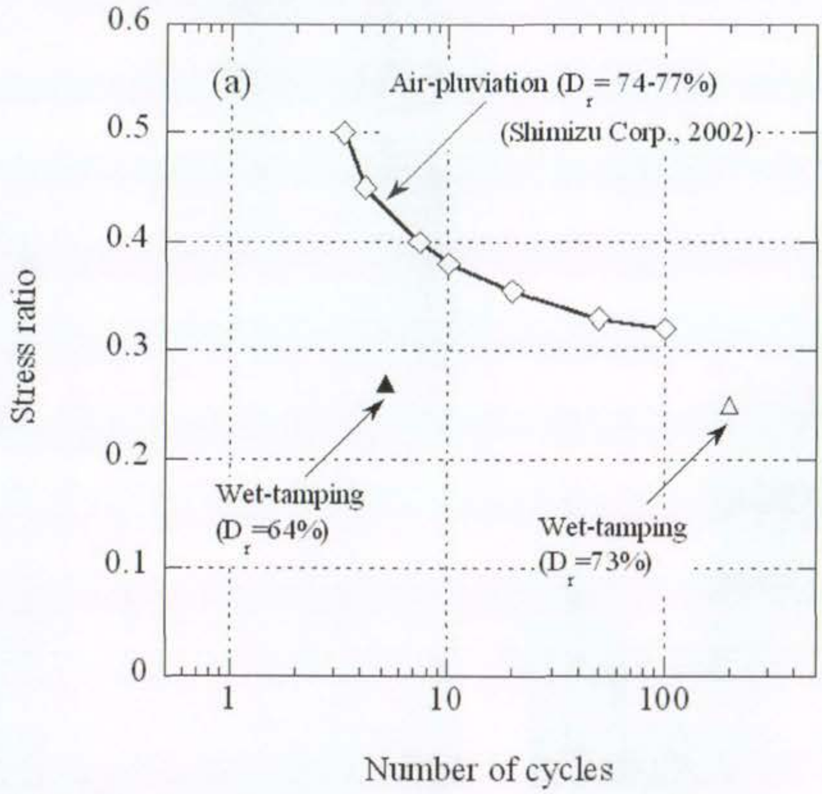


Figure 46. Liquefaction resistance of Albany sand from results of cyclic triaxial tests (NIED 2006)

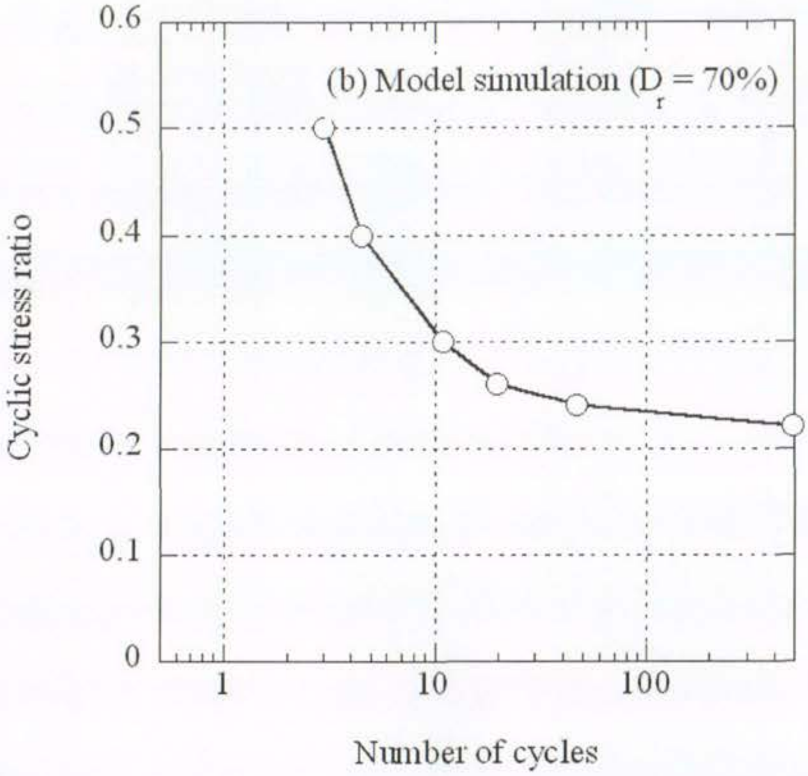


Figure 47. Liquefaction resistance of Albany sand adopted in the simulations (NIED 2006)

Effective stress paths and stress-strain curves that were obtained in simulations are shown in Figure 48. These plots show the modelled cyclic behaviour of Albany sand at different stress ratios. The simulations were conducted for a relative density of 70% and initial mean effective stress of p' = 50kPa, as they are taken to be representative of the density and confining stress for the model.

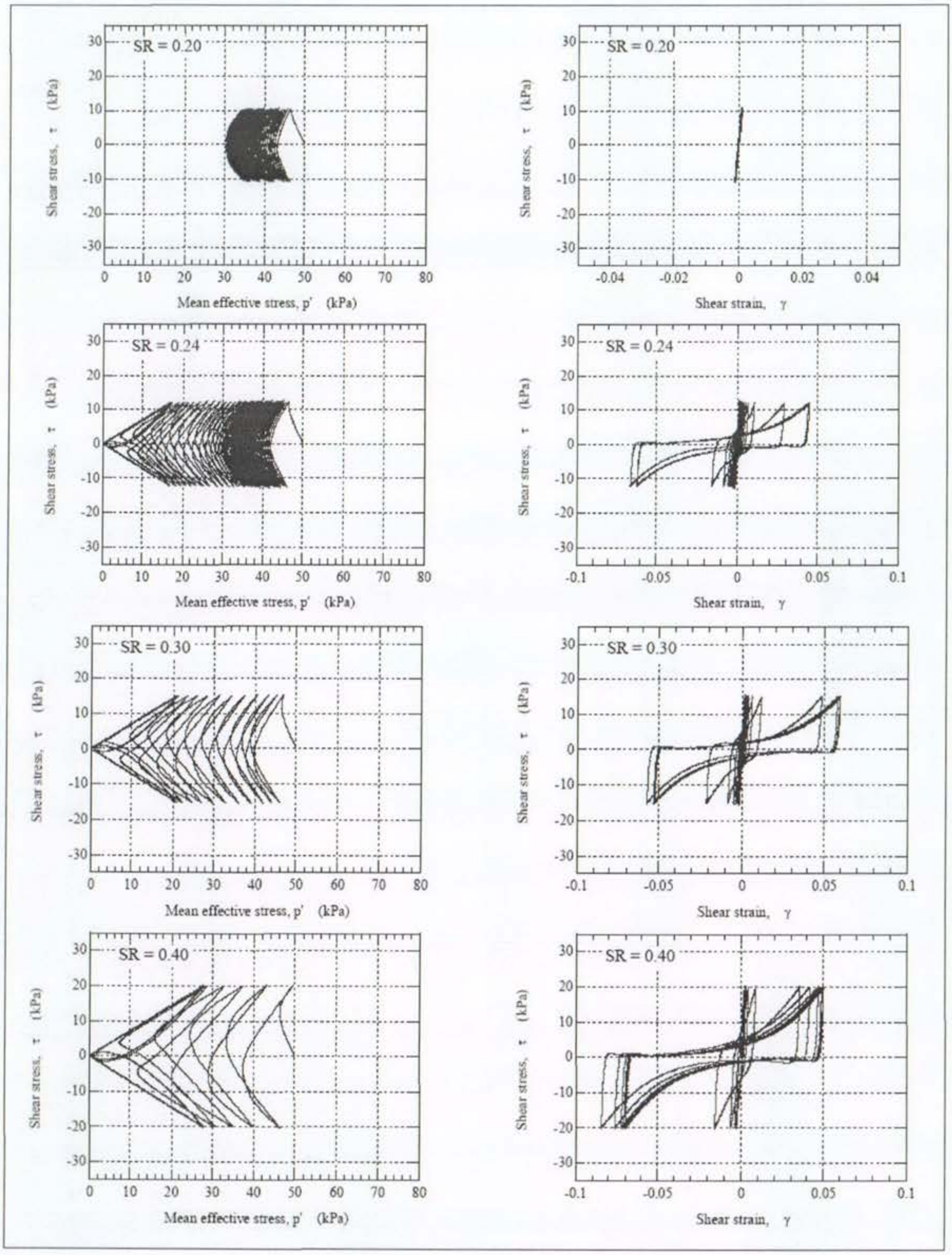


Figure 48. Cyclic behaviour of Albany sand simulated with the model (NIED 2006)

The stress-strain parameters of the constitutive model are commonly determined from drained p' constant torsional tests or drained triaxial compression tests. A number of tests are needed to be conducted at different initial densities and confining stresses. A limited series of tests were carried out using the wet tamping technique as this allowed samples to be prepared at a range of densities. These tests were conducted before the model soil preparation technique (vibration-compaction) was known, and since the soil fabric would be remarkably different than the *in situ* soil fabric in the model, the wet tamping tests were not used. Rather, the parameters were determined from two cyclic drained tests conducted by Uzuoka (NIED 2006) which use samples prepared by air pluviation then compaction, providing a better resemblance to the model sand fabric. The cyclic tests defined the stress-strain behaviour well at small strains, as shown in Figure 49, but however the behaviour at large strains and for different densities and confining stresses were approximated based on the available data. The stress-strain parameters shown in Table 5 are representative of Albany sand, however the features associated with the particular method of preparation of the model and it effects on the stress-strain behaviour and liquefaction resistance were not fully accounted for.

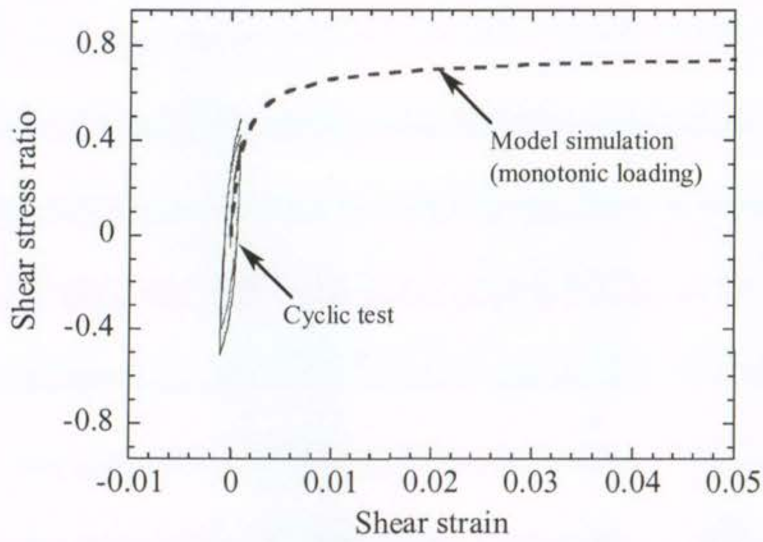


Figure 49. Drained cyclic tests on Albany sand (Uzuoka et al., 2006) and model simulation of monotonic loading behaviour (NIED 2006)

4.8.4 Pile modelling

Three different methods can be used to model the piles: (a) elastic piles, (b) equivalent linear piles and (c) non-linear piles. It is intended to perform simulations with all three methods, however at the time of this report only the simulation with elastic piles was fully completed. Thus the results from this simulation were obtained using linear elastic piles with a flexural rigidity of $EI = 550.4$ kN-m^2.

4.8.5 Numerical Model

The finite element model used in the 3-D analysis is made up of eight-node solid elements and beam elements representing the soil and the piles respectively. Solid elements were also used to represent the footing, added mass and sheet pile wall while beam elements were used to model the columns connecting the added mass to the footing. To reduce computational demands the model was cut in half along its axis of symmetry.

The boundaries on the left and right hand sides of the model were fixed in the x-direction (longitudinal direction), to simulate the conditions imposed by the rigid container. Along the symmetry boundary the displacements were fixed in the y-direction (the direction perpendicular to the vertical plane of symmetry). Along all the soil-sheet pile and soil-pile interfaces, condition was imposed that forces the soil and pile to share identical displacements in the horizontal direction but allows different vertical displacements. This permits unconstrained settlement of the ground. In modelling the soil-footing interface, it was assumed that the soil and footing move together on the backfill side while along the side and front interfaces it was assumed they move separately. Figure 50 shows the numerical model used in the analyses.

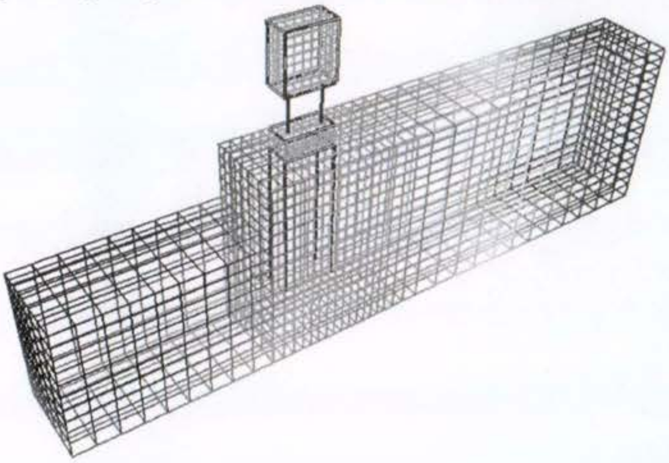


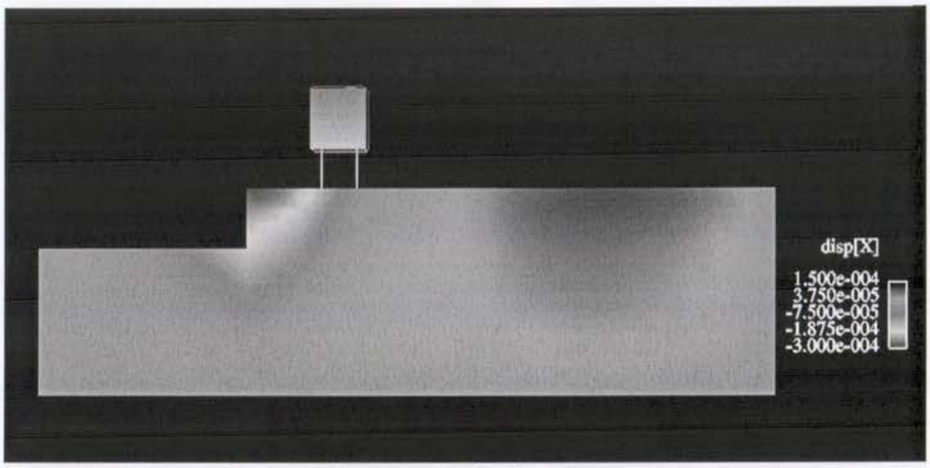
Figure 50. Numerical model used in the 3D analyses (NIED 2006)

4.8.6 Initial Stress State

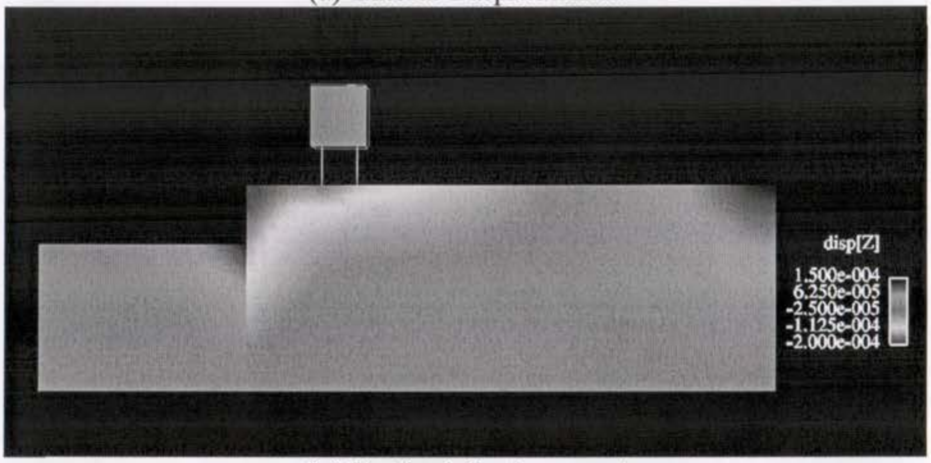
The determination of the initial stress state of the soil is very important. The seismic response and permanent ground displacements caused by lateral spreading are both significantly affected by the presence of initial shear stresses. To evaluate the initial stress state a 2-D static analysis was performed considering the preparation stages of the model.

Two stages in the development of the initial stress state were distinguished. First, the stress-state was evaluated after the deposition, compaction, saturation and consolidation of the sand. During this stage, the sheet pile wall was supported in the horizontal direction, effectively fixing the sheet pile in the longitudinal direction. Thus it was assumed that the sand deposit underwent K_0-consoildation, i.e. it was assumed that $\sigma'_v = \gamma h$ and $\sigma'_h = K_0 h$. K_0 was taken to be 0.5.

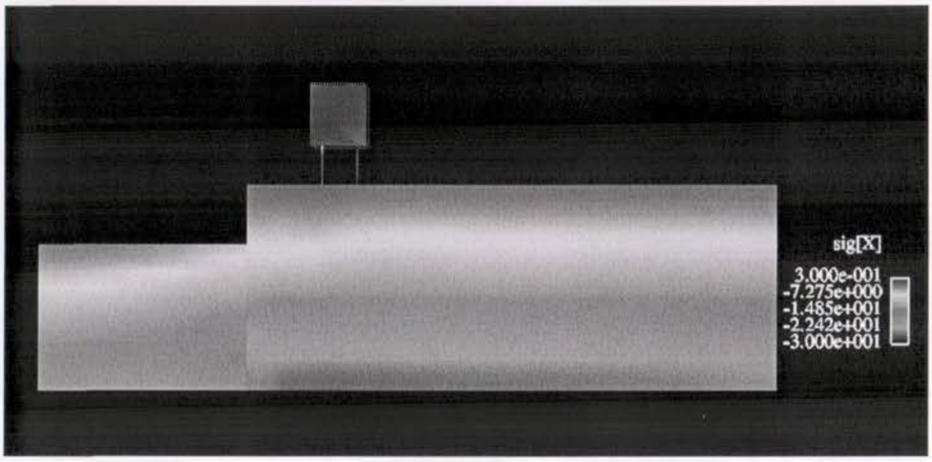
The second stage of the initial stress development was the removal of the horizontal supports. This created an unbalanced earth pressure which created a lateral displacement towards the water. The stress state was then evaluated by applying the unbalanced earth pressure to the post consolidation stress state. The horizontal and vertical displacements and the lateral and shear stresses that were computed are shown in Figure 51.



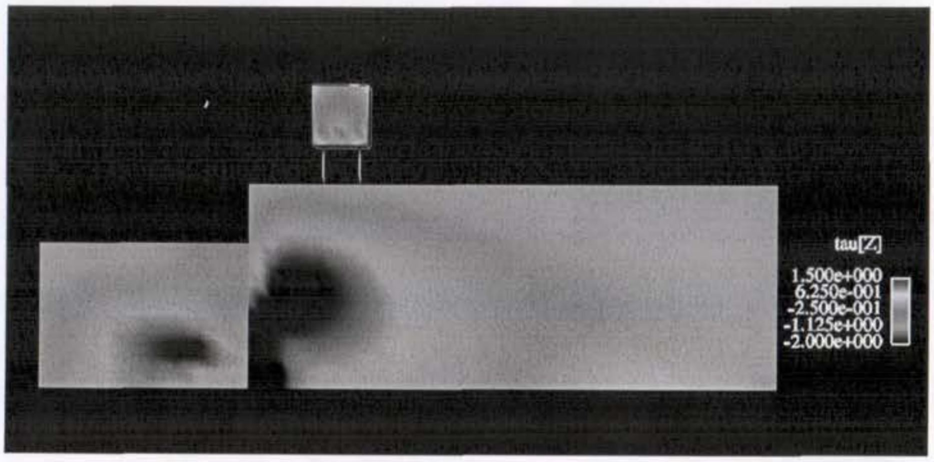
(a) Lateral Displacement



(b) Vertical displacement



(c) Horizontal stress



(d) Shear stress

Figure 51. Initial stress analysis results (NIED 2006)

4.8.7 Dynamic analyses

Step-by-step dynamic analyses were conducted using a time step of either $\Delta t = 0.002$ sec or $\Delta t = 0.0012$ sec and Rayleigh damping with parameters of $\alpha = 0$ and $\beta = 0.003$ were adopted. The analysis was only performed during the period of intense shaking, with a computational time of 20 sec. Hence the effects of pore pressure dissipation and post-shaking deformation were not evaluated, which implies the computed permanent displacements are expected to be smaller than observed in the experiment. The effects of the vertical component of motion were also ignored in the analysis, as it was reasoned that first the horizontal motion effects must be understood in this preliminary analysis. If the vertical components are found to be significant they can be superimposed later as a secondary effect on the response.

4.9 RESULTS AND DISCUSSIONS

4.9.1 General

The recorded data from the experiment was not available at the time of submission of this report; hence comparisons cannot yet be made between the computed and recorded data. The results from the preliminary analysis are fully presented and compared to the visual observations from the experiments.

4.9.2 Overall ground response

Figure 52 shows the computed ground displacements in the longitudinal direction for the first 20 seconds. A full analysis of at least 30 seconds was not performed due to computation time restraints. The analysis gave a similar pattern of deformation to that observed in the experiment; however the magnitudes were much smaller. Despite the discrepancy in the magnitudes of the displacement, the analysis showed 3D effects with a flow type movement of soil around the pile foundation. This can be seen in Figure 52 by noticing the smaller deformations observed around the pile foundation.

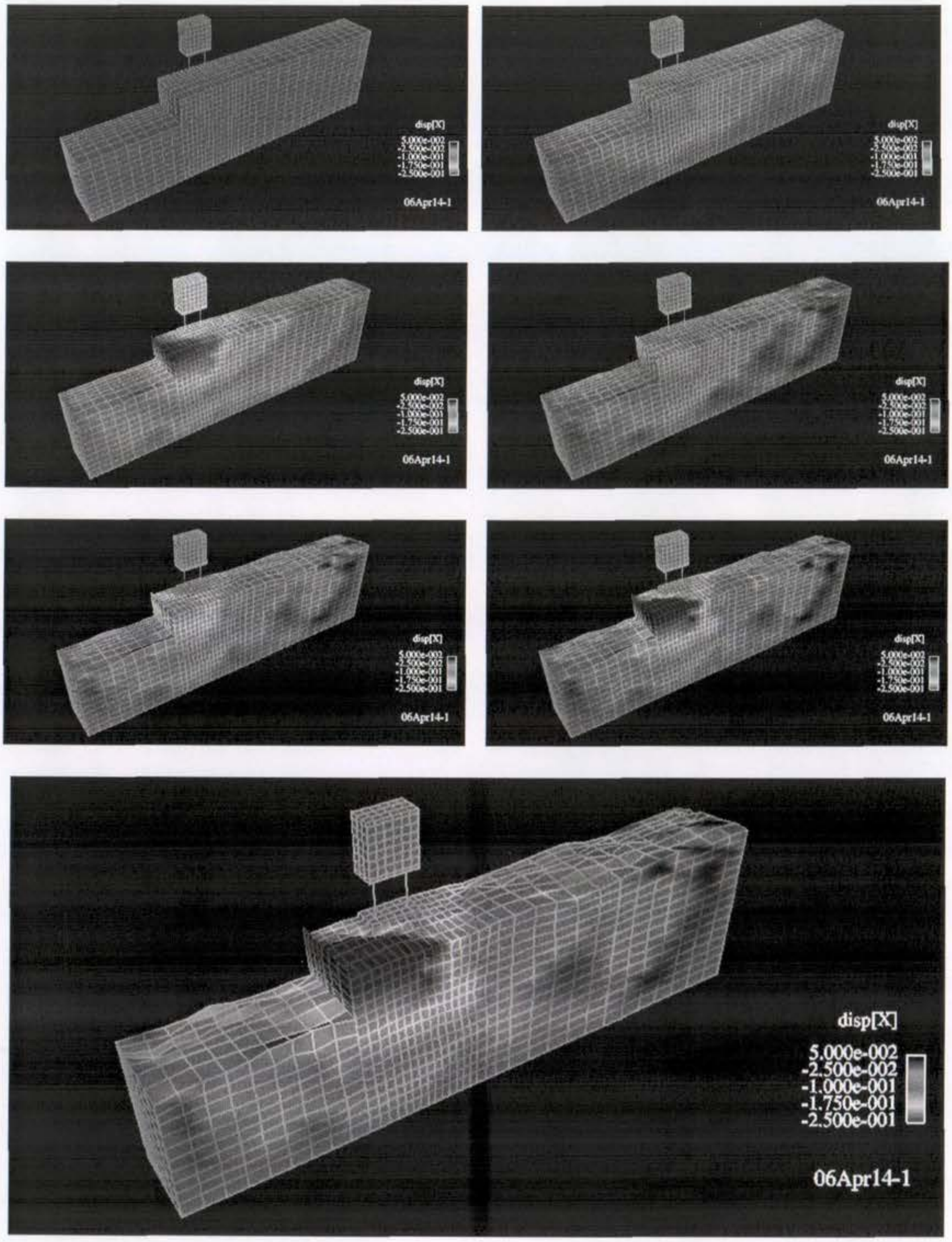


Figure 52. Computed lateral displacements in the longitudinal direction (x-displacements) at 1, 5, 6, 7, 10, 15 and 20 sec (NIED 2006)

The development of excess pore water pressure is shown in Figure 53, Figure 54 and Figure 55 at locations 4m away from the pile foundation, in the vicinity of the piles and behind the sheet pile wall respectively. The excess pore pressures are computed at four different depths in the range between 1.9 - 3.14m below the surface, shown as (a) – (d) in Figure 53, Figure 54 and Figure 55.

Complete liquefaction occurred in the soil away from the pile foundation, as the excess pore water pressure reached the effective overburden stress. At around 5-6 seconds the excess pore water pressure drops and becomes negative, indicating strong dilative behaviour. This is typical of medium and dense sands. In the vicinity of the piles and next to the sheet pile wall the dilatancy is more pronounced, with a clear decrease in the excess pore water pressure as the shaking continued. The excess pore pressure at the end of the analysis increases with the distance from the sheet pile wall. This indicates that the large lateral movement and the relative displacement between (a) the soil and sheet pile wall, and (b) the soil and piles affect the pore water pressure in the analysis and make the soil response to be more dilative. These effects may be somewhat exaggerated and different from observed in the experiment, due to the limitations of the numerical model.

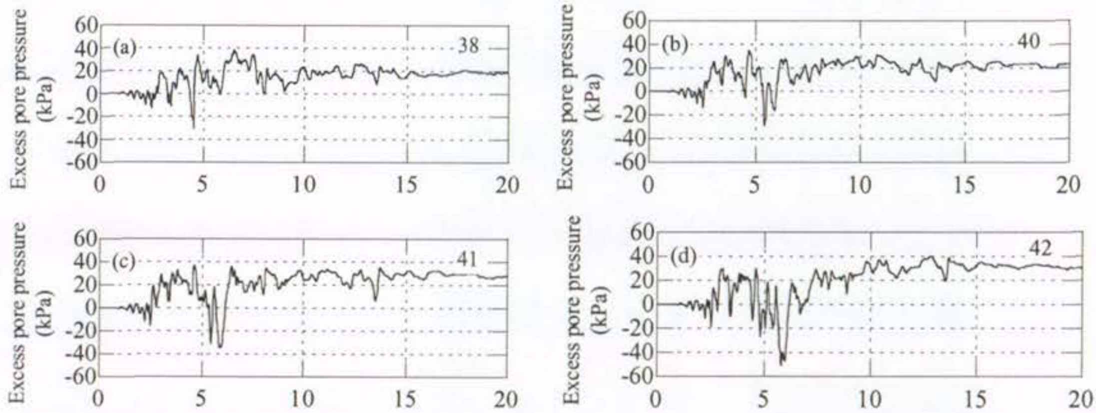


Figure 53. Computed excess pore water pressures in the backfill at a distance approximately 4m from the pile foundation (NIED 2006)

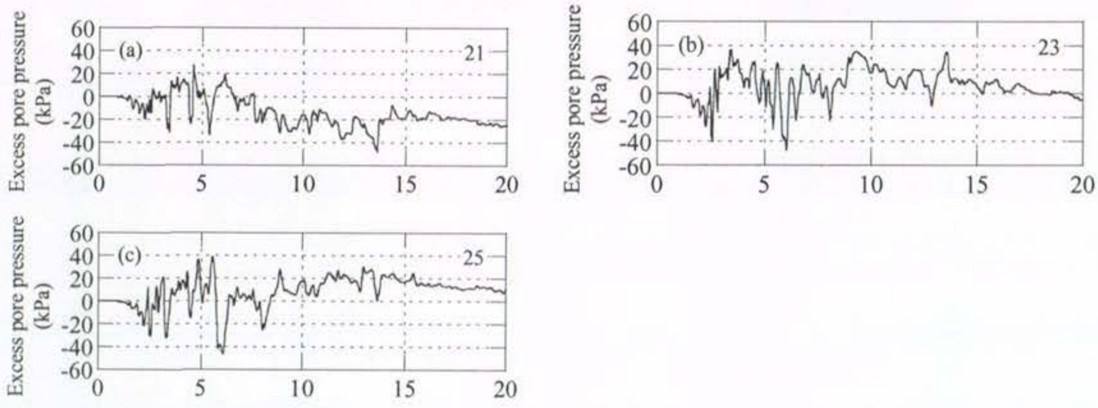


Figure 54. Computed excess pore water pressures in the soil in the vicinity of the piles (NIED 2006)

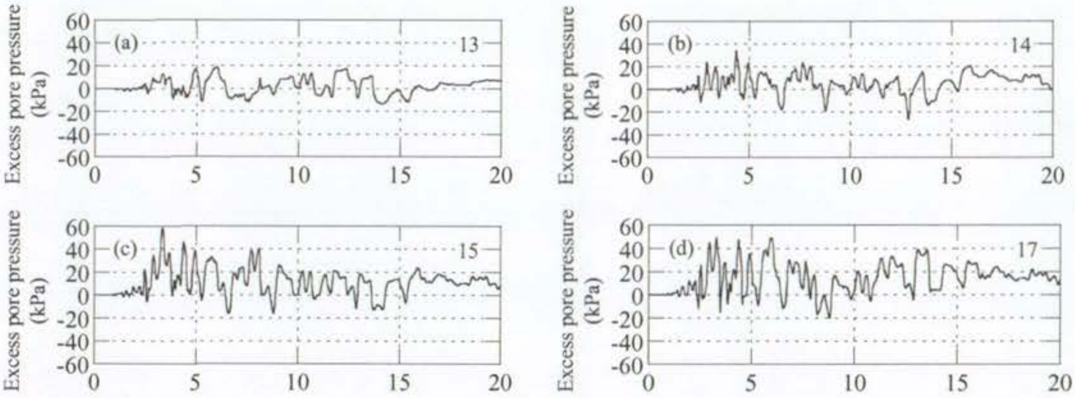


Figure 55. Computed excess pore water pressures in the soil behind the sheet-pile wall (NIED 2006)

Figure 56 shows computed acceleration time histories at four locations. The response at the ground surface of the submerged layer is shown in (a) and (b), near the sheet pile wall is shown in (c) and approximately 4m behind the pile foundation in (d). The record 4m behind the foundation can be considered as the free field response.

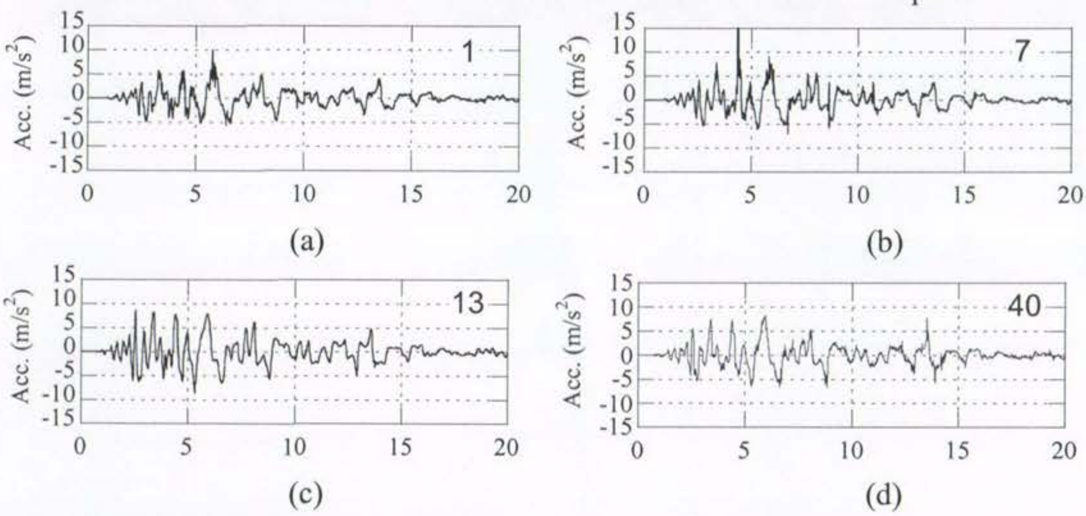


Figure 56. Computed horizontal accelerations at the top of the ground model; (a,b) submerged soil; (c) near the sheet pile wall, and (d) free field response (NIED 2006)

4.9.3 Horizontal displacements of the piles and sheet-pile wall

The computed lateral displacement of the sheet pile wall is shown in Figure 54. The displacement gradually increases to reach a value of nearly 0.3m in 20 seconds. The most rapid increase in displacement coincides with the period of most intense shaking at 5.9 seconds. The peak displacement is much less than observed visually during the experiment, which was in the order of one metre. The full reasons for this will not be known until a more complete analysis is performed and the recorded observations are made available. However there are a few clear reasons affecting the results: (a) firstly, elastic piles were used, which significantly constrained the deformations of the sheet pile and the piles in the analysis; (b) the intensive part of the adopted shaking is at about 30 seconds, whereas only the first 20 seconds were used in this analysis; and (c) it was indicated in the previous study (Cubrinovski *et al.*, 2005) that the use of the 2-

by-2-by-2 integration rule (eight Gauss points) adopted in the analysis may stiffen the model and reduce the lateral displacements of the sheet pile wall. Another analysis will be performed later this year with the full duration of shaking and non-linear piles, and it is expected that this will significantly increase the displacement of the sheet pile wall.

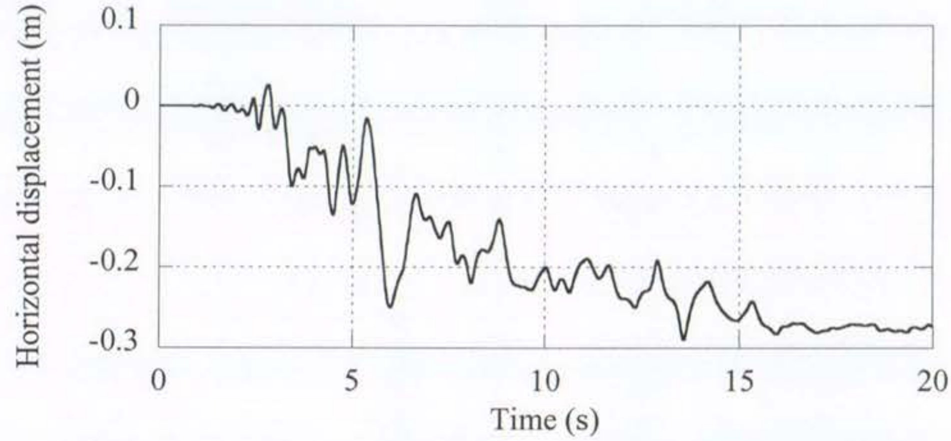


Figure 57. Computed horizontal displacement of the sheet-pile wall in the analysis with elastic piles (NIED 2006)

Similarly, the displacement of the footing, shown in Figure 58, was much less than observed. Similar reasoning as above can explain the discrepancy.

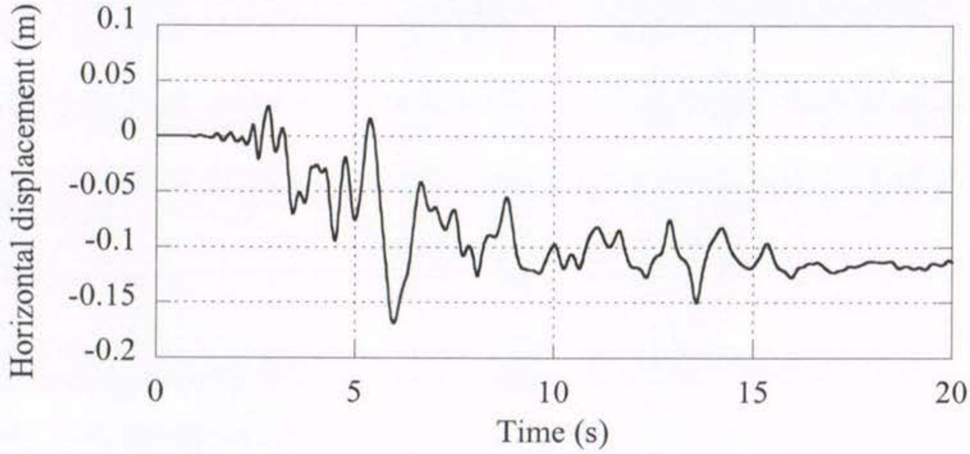


Figure 58. Computed horizontal displacement of the footing (pile cap) in the analysis with elastic piles (NIED 2006)

4.9.4 Bending moments of piles

Bending moment time histories of the piles were computed at 13 different depths along the length of the pile. Six time history plots are shown in Figure 59 for pile No. 1. The peak bending moment reached in the analysis was 30kN-m, which is three times greater than the yield moment of the pile. This is a result of the use of elastic piles, and clearly predicts the failure of the piles.

In the complete analysis, with non-linear piles, it is expected that the displacements of the piles will increase significantly because of the reduced stiffness due to yielding, and also the peak bending moments will be reduced due to the non-linear M-ϕ curve.

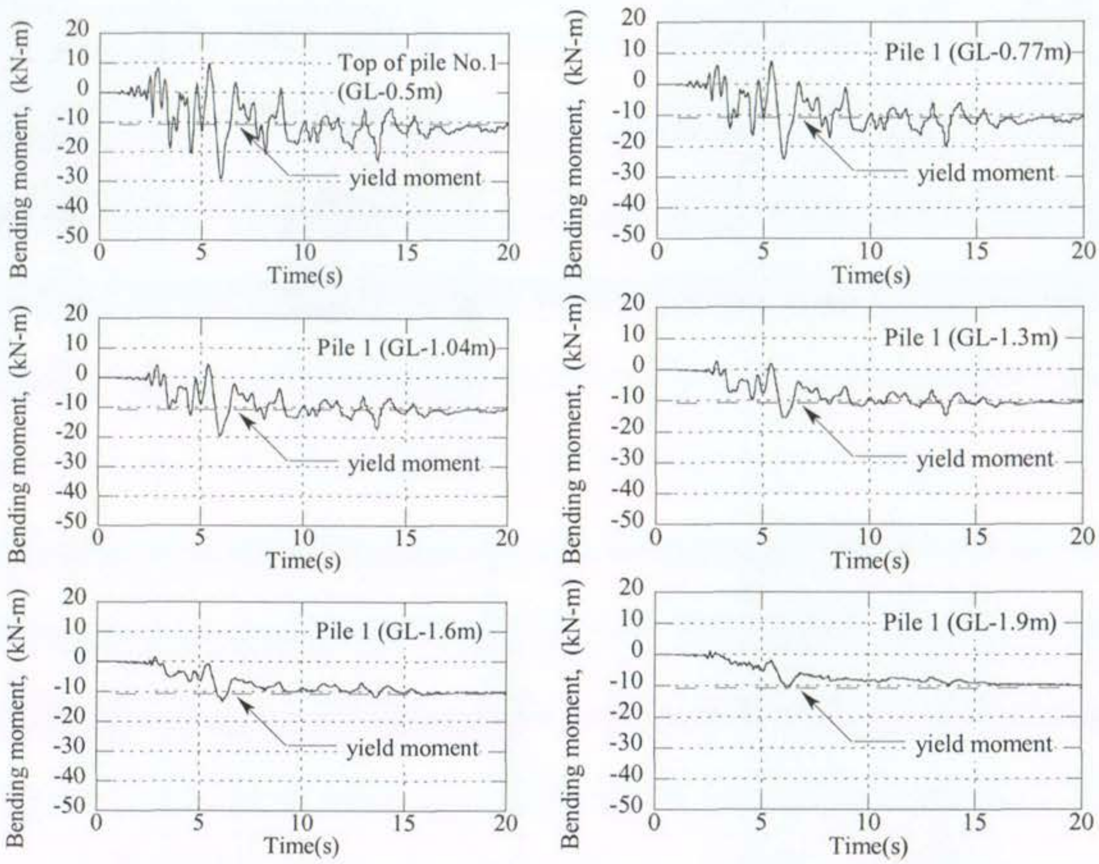


Figure 59. Computed bending moments for pile 1 in the analysis using elastic piles (NIED 2006)

The bending moment distributions throughout the length of the piles 1 – 4 computed at t = 5.76 seconds is shown in Figure 60. The peak bending moments were observed at the top of the piles, again with values much larger than the yield moment. The back piles show an increase in bending moment approximately 0.6 – 0.7m above the base of the pile. As the differences in relative density between the middle layer and base layer were so slight, the effects of lateral stiffening from the base layer were not observed.

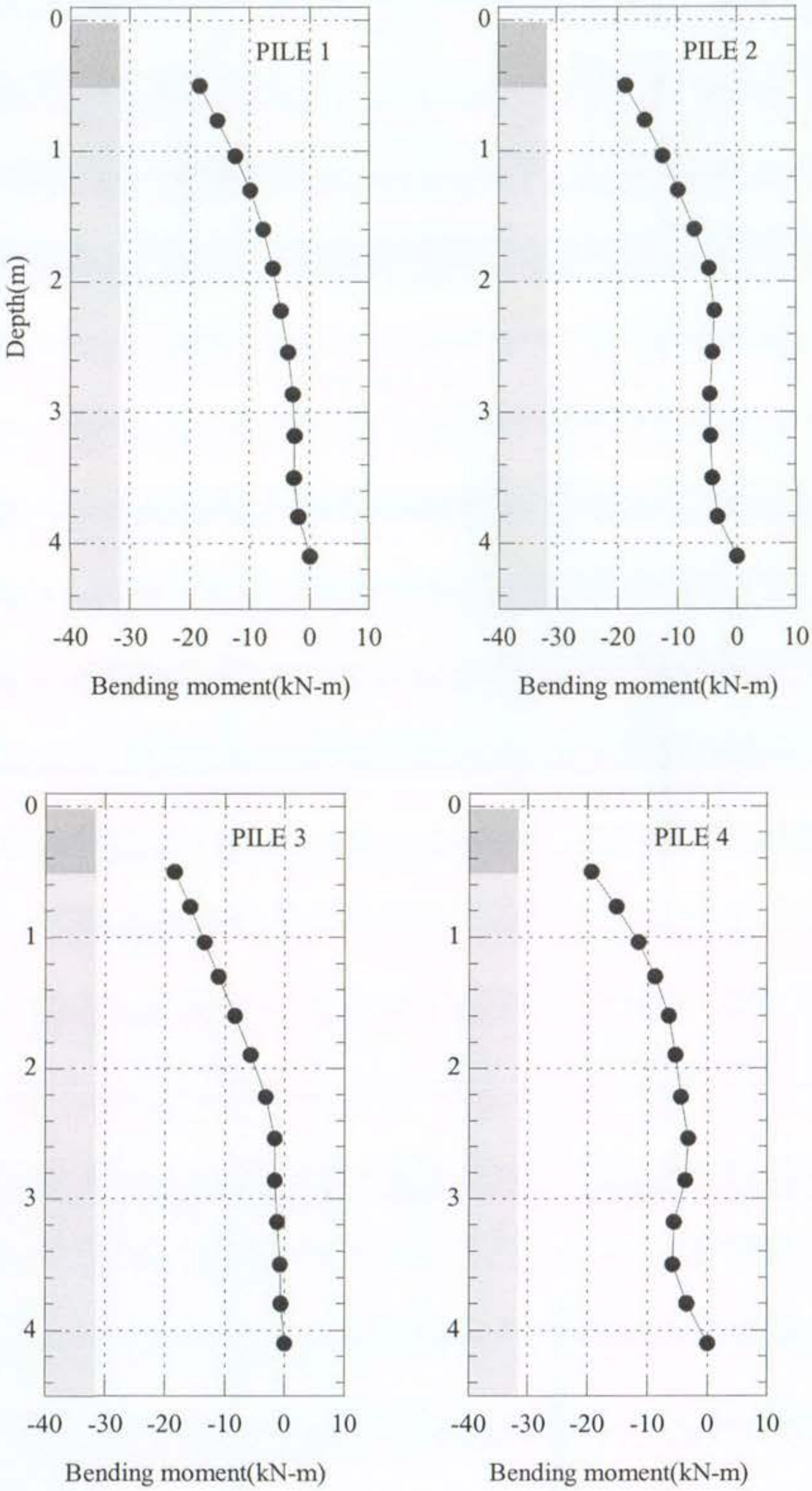


Figure 60. Computed distribution of bending moments for piles 1-4 in the analysis using elastic piles (t = 5.76 sec) (NIED 2006)

5 Conclusions

E-Defense is a large-scale shake table near Kobe, Japan. The combination of its large size that enables full-scale testing, its ability to reproduce strong ground motions, and the advanced data acquisition facilities contained within make E-Defense the largest and most technologically advanced test facility in the world. Consequently it can be regarded as the ultimate tool for verification of the seismic capacity of structures.

The experiments currently being performed as part of larger research project, DaiDaiToku, which has the significant improvement of the seismic performance of structures as one of its many goals. Within this specific goal many experiments have been carried out, modelling wooden houses, reinforced concrete buildings and soil-pile-structure systems. The E-Defense experiments are a final stage of an experimental programme that has included many full-scale experiments designed to observe 3D earthquake response and failure mechanisms. These experiments have been modelled using advanced numerical analyses.

The experimental tests on soil-pile-structure systems modelled the behaviour of piles in two situations, on level ground and next to quay walls. The effects of 3D input motion, pile group effects, pore-water pressure development, the combination of inertial and kinematic forces on piles, post liquefaction behaviour, large ground displacements, lateral loads from crust layers and damage mechanisms were all studied. The effective stress method of analysis was applied to numerically model the experiments, with accurate results.

The future of E-Defense is uncertain at this stage. The facility was designed to have a service life of 20 years; however the high costs of running the facility might be too high for academic institutions without government support. A possibility is for large companies in Japan to use the facility for private use, however many of these facilities own similar (albeit smaller) private facilities. International use of the facility is also a possibility. The current cooperation between E-Defense and the University of Canterbury will continue in the immediate future.

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