Simulation and Prediction of Earthquake
Ground Motion and Structural Performance

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Figure 1. Research subjects of Structural Performance Team, EDM / RIKEN.
Asperity model of the 2001 Tottori-ken Seibu earthquake.

(Left) Microtremor observation sites in Yonago city.

(Lower) H/V spectra of microtremors along the line passing Sites A01-A26.

Figure 2. Asperity model of the 2001 Tottori-ken Seibu earthquake and H/V spectra of microtremors along the observation sites in Yonago city (see details inside this report).
The Earthquake Disaster Mitigation Research Center (EDM) was established on the 1st of January 1998 under the framework of the Strategic Frontier Research Program under the Institute of Physical and Chemical Research (RIKEN). The activities of the EDM were initiated by the following three research teams: (1) Disaster Process Simulation Team, (2) Disaster Information System Team and (3) Structural Performance Team. The research projects are essentially based on the lessons learned from the disasters during the 1995 Hyogo-ken Nanbu Earthquake, and are aimed to carry out multi-disciplinary research on earthquake disaster mitigation, encompassing seismology, earthquake engineering, information science and social science. Through the EDM research project, evaluation methods are developed for comprehensive understanding of disaster processes with the keywords of "social", "information" and "physical" phenomena, and are devised for the visual presentation of research results using advanced technologies to enable dissemination and promotion of the work of the EDM.

The Structural Performance Team has placed its major research subjects on (1) rupture process simulation and earthquake source evaluation; (2) strong ground motion evaluation in consequence of wave generation, wave propagation and soil amplification within the wave propagation path; (3) soil liquefaction and foundation responses; and (4) responses of superstructure and damage mechanism subjected to an intense ground motion, the research result of which can be extended to reliable damage and vulnerability assessment of urban structures. The members of the team have been coming from the areas of seismology, geotechnical engineering, civil engineering and structural engineering. The maximum number of members has been five at the moment in the fiscal year of 2000 (April 2000 to March 2001), carrying their research works together with the team leader. The research activities carried out within the team can be summarized in the following items.

Applying an integrated analytical model together with a sophisticated analytical method to the building located in the heavily damaged area in Kobe, a case study on the evaluation of the ground motion and simulation of the earthquake responses of soil-foundation and superstructure has been carried out. The building studied herein has been affected by the 1995 Hyogo-ken Nanbu Earthquake. The reliability and performance of the analytical model and method developed and used herein the study has been examined and verified by comparison of the evaluated ground motion and simulated soil-foundation-structure responses with the observed responses obtained during the earthquake.

Another case study on simulation of the ground motion and building response has been carried out for the case during the 1999 Kocaeli, Turkey Earthquake. The study hereby has been carried out based on the reported data on the earthquake fault, strong ground motion records, ground structures and the general structural properties of buildings in Turkey. The simulated results obtained from the study have been compared with the observed ground motions and building damage distribution observed during the earthquake, and a possible cause of the intense ground motion generation and severe disaster distribution has been discussed.

In the third attempt, the prediction analysis on the ground motion and building response for a scenario earthquake has been carried out. The analytical models and methodologies have been verified through the study. In Japan, it is becoming a relevant approach to the disaster prevention to determine the seismic hazard for a region from the physical model of active faults surveyed to be located nearby the area. The study herein is positioned as an
exclusively prediction attempt. Strong ground motion simulation, nonlinear soil response analysis and inelastic building response analysis have been performed for the scenario earthquake in the western region of the Tottori Prefecture including the cities of Yonago and Sakaiminato. Using an identical settlement of fault line and mechanisms among the case study, the evaluated results have been discussed from the viewpoint of the significance of the location of the asperities in the fault line during the 2000 Tottori-ken Seibu earthquake, Japan.

In the further studies, analytical models and methodologies on the simulation of strong motion and responses of soil and building structures have been developed and verified individually by the members of the Team. These models and methodologies have been integrated and applied to (1) the prediction of a broadband strong ground motion from a characterized asperity model; (2) the inverse analysis of microtremor H/V spectrum for estimating the layer thickness of subsurface soil and wave propagation in irregular soil deposits; (3) the one- and three-dimensional effective stress analysis for simulating the soil responses and liquefaction; and (4) the modeling of superstructures for three-dimensional nonlinear structural analysis for earthquake responses simulation.

The integrated research theme of the Team on the development of simulation, prediction, evaluation of strong ground motion, soil structure response and urban structure response leading to earthquake damage assessment for urban structural systems will be extended in the coming years to cover the research subjects located between each individual research themes. The research activities intended herein would make the interactive theme between the individual topics clear as well. The research activities and achievements of the Structural Performance Team are summarized in this Technical Report compiled and published hereby.

The Structural Performance Team would like to express sincere thanks and appreciation to the Hyogo-ken Prefectural Government for their firm support for providing the research facilities and the research data utilized in this research; to the Association of Earthquake Disaster Prevention for the records of the strong ground motions during the Hyogo-ken Nanbu Earthquake; to the local correspondences who gave much support in the reconnaissance investigation to the 1999 Turkey Kocaeli Earthquake; and to the staff who worked with the team at EDM to process the data and graphics.

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Table of Contents

Chapter 1. Introduction

Chapter 2. Theoretical Background and Numerical Methods

2.1 Earthquake Source Parameters and Strong Ground Motion Modeling
2.2 $V_s$ Profiling Using Microtremor H/V Spectra
2.3 Ground Response Analysis
2.4 One-Dimensional Soil Liquefaction Analysis
2.5 Three-Dimensional Soil Liquefaction and Foundation Response Analysis
2.6 Models and Methods for Three-Dimensional Building Structural Analysis

Chapter 3. Case Study on Ground and Building Responses in the 1995 Hyogo-ken Nanbu Earthquake

3.1 Introduction
3.2 Simulation of Ground Response and Ground Motion Distribution
3.3 Simulation of Subsurface Soil Response and Liquefaction
3.4 Simulation of Pile Foundation Response and Damage
3.5 Simulation of Building Response and Damage
3.6 Summary

Chapter 4. Case Study on Ground and Building Responses in the 1999 Kocaeli Turkey Earthquake

4.1 Introduction
4.2 Fault Rupture Process and Strong Motion Modeling
4.3 Estimation of $V_s$ Structure
4.4 Simulation of Ground Response and Ground Motion Distribution
4.5 Simulation of Building Damage Distribution
4.6 Summary

Chapter 5. Prediction of Ground Motion and Building Damage in a Scenario Earthquake for Tottori Region

5.1 Introduction
5.2 Earthquake Source Characterization and Ground Motion Modeling
5.3 Estimation of $V_s$ Structure
5.4 Simulation of Ground Response and Ground Motion Distribution
5.5 Soil Liquefaction and Ground Motion
5.6 Estimation of Building Response and Damage
5.7 Summary

Chapter 6. Concluding Remarks
Chapter 1. Introduction

Seismic disasters are produced by a series of phenomena of the generation of seismic wave, propagation and amplification of wave, interaction among soil-foundation and structure, and response of structural systems subjected to the seismic action. A large number of research studies have been carried out on the seismic disaster mitigation issues. These studies have, in general, individually focused their attention on the specific topics.

The Structural Performance Team of EDM would place the major emphasis to reach comprehensive understanding of structural earthquake response and damage mechanism, and to make accurate assessment of the vulnerability and damage of urban structures subjected to a strong seismic motion. The research themes are itemized as described in the following:

1. Fault rupture process and strong motion generation mechanisms;
2. Propagation and amplification of seismic motion within the wave propagation path;
3. Soil liquefaction and foundation responses;
4. Super-structural responses and damage mechanism; and
5. Damage and vulnerability assessment of urban structures.

Through the items (1) through (4) above, we will evaluate the seismic disaster of structural systems using the common data from the initiation of seismic wave to the response of structural system. The advanced and sophisticated analytical technologies individually developed in seismology, civil and structural engineering field are to be integrated in analysis.

Figure 1.1. Correlation of the research themes within the study.
The developed technologies should be verified and further improved. Data employed in the analytical studies are shared on common basis from one analysis to the other analysis. Integrating the research results obtained herein, we can extend our research works to assessment of vulnerability and damage of urban structures when subjected to the intense seismic motion. Figure 1.1 illustrates the correlation of the research themes within the study focusing our emphasis to develop the damage and vulnerability assessment of urban structure technology. The application of the technologies in damage assessment, restoration and recovery, and in earthquake strengthening and retrofitting is to be studied and developed as well. The final goal of the research is to provide design engineers with reliable data and proper measures to ensure the earthquake-resistant capacity of structures against severe damage and collapse, placing the goal to reduce the preventable human lives and urban socio-economic system from earthquake disasters.

Chapter 2 summaries the analytical models for the items (1) through (4) above described. In the section 2.1, methodology for prediction of a broadband strong ground motion from a characterized asperity model of source model is summarized. In the section 2.2, we review an inverse analysis of microtremor H/V spectrum for estimating the layer thickness of subsurface soil, when data of Vs values at a site is given in advance. In the section 2.3, the numerical method for the two-dimensional wave propagation analysis in irregular soil deposits is described. The proposed method is an equivalent linear response analysis in the frequency domain. In the section 2.4, a one-dimensional effective stress analysis, so-called liquefaction analysis herein, is shortly reviewed. In the section 2.5, the brief description of both the equations of motion and constitutive models for both sand and clay in the three-dimensional liquefaction analysis is given. In the sections 2.4 and 2.5, the verification of the analytical study has been presented. Within the verification study, ground motions obtained by the vertical array system during 1995 Hyogo-ken Nanbu earthquake are employed. In the section 2.6, modeling method of building structure and its constituent members for a three-dimensional nonlinear structural analysis is presented. The numerical method is described as well. And the reliability of both the modeling and method is examined through the verification analysis.

In the first step of integrating the analytical methods, the reliability and performance of the analysis models developed and utilized in the individual areas are examined by comparing the evaluated ground motion and simulated soil-foundation-structure responses with the observed data obtained during the real earthquake. For this purpose, we carry out two sets of the case studies for the 1995 Hyogo-ken Nanbu and the 1999 Kocaeli, Turkey earthquakes. The examined results are summarized in the Chapters 3 and 4 for the Hyogo-ken Nanbu, Japan and Kocaeli, Turkey earthquakes, respectively.

In Chapter 3, a condominium building located in Kobe is examined for a verification case study. The building is located in the so-named heavily damaged belt area, while it suffered slight damage. The North-South ground section across the building location from the Rokko mountainside to the seaside are selected and modeled. The response analyses of deep basin soil structure, subsurface shallow soil structure, structure-foundation, and superstructure are carried out separately at the first verification attempt to perform an integrated evaluation and simulation of a series of phenomena from earthquake wave generation to structural damage evaluation. The results are compared with the accelerometer records and post-earthquake onsite observation on the building, and summarized is examination of the reliability and appropriateness of the analytical model and numerical method employed in the study.

In Chapter 4, we simulate the ground motions at Golecek, Turkey, the severely damaged site during the quake, and examine the building damage distribution during 1999 Kocaeli, Turkey earthquake using the numerical methodologies presented previously in Chapter 2.
The information on the earthquake fault, ground structures, and structural properties of the Turkish buildings is taken into analysis. Throughout the analyses, the simulated analytical results are compared with the obtained ground motions and observed building damage distribution, and the possible/probable causes of the intense ground motions and serious disaster have been discussed.

In Chapter 5, summarized are our endeavors to try to predict the ground motion and building response for a so-called scenario earthquake. The research work described herein is obtained from a viewpoint of prediction of seismic disaster evaluation. In these days, it would be a relevant issue and technology in Japan for the disaster prevention to determine the seismic hazard for the specific site from estimation based on a physical model of the active faults located near around the site. In this chapter, we perform a strong ground motion simulation, nonlinear soil analysis and inelastic building response analysis for the earthquake motion generated by the scenario earthquake. The model site locations in the analysis is determined to be the cities of Yonago and Sakaiminato based on the observed damages during the 2000 Tottori-ken Seibu, Japan earthquake. Herein, we discuss the significance of location modeling of asperities in fault mechanism model generating seismic waves considering variation of asperity placement within the fault plane.

In Chapter 6, we summarize the conclusive remarks obtained in each study presented in Chapters 2 through 5.
Chapter 2. Theoretical Background and Numerical Methods

2.1 Earthquake Source Parameters and Strong Ground Motion Modeling

2.1.1 Earthquake Source Characterization

The great disasters from recent large earthquakes like the 1999 Kocaeli (Turkey), 1999 Chi-Chi (Taiwan), 2001 El Salvador and 2001 Gujarat (India) earthquakes, put in evidence the importance of making detail hazard analysis studies that take into account the active faults surrounding the urban areas. The first very important step in the prediction of strong ground motion for future earthquakes is the one related with the determination of the fault parameters for a particular tectonic region.

Global Fault Parameters

The first set of parameters that should be considered are the “Global fault Parameters”, namely the fault gross properties like fault width, length and geometry (Irikura 2000). The fault length is determined from geological observations on the active faults morphology. Features like fault segmentation are very important in determining the possible total length that an earthquake can develop. Several earthquakes like the Landers 1992 (Hauksson 1993, Wald and Heaton 1994, Cotton 1995) and the Hyogo-ken Nanbu (Sekiguchi 1999) earthquakes have shown a very complex rupture process of faults with multiple segments. The fault geometry is of great importance, since the orientation of the fault segments relative to the tectonic stress, determines the condition under which an earthquake would stop in an early stage or develop into a larger event where the rupture is transferred from one fault segment to the neighbor segment (Aochi 2000). The fault width is determined by the thickness of the seismogenic zone for earthquakes with magnitude larger than 6.8. For smaller events the width might be determined from empirical relationships between the fault width and length.

Local Fault Parameters

The second set of parameters are the “Local fault parameters”, namely the parameters related with intrinsic properties of the fault. Among the most important we should include, fault asperity size, number, location and average slip, rise time, stress drop and rupture velocity (Irikura 2000). Earthquake slip models from large crustal earthquakes (Mw = 5.6 to 7.2) for the past decade have allowed to statistically studying the scaling relationships between the seismic moment and the asperity parameters (Somerville et al. 1999). The result of such empirical relationships obtained from slip models of Californian earthquakes is shown in Figures 2.1.1, 2.1.2 and 2.1.3.

Summary of the Fault Parameters Estimation

The procedure for estimating the fault parameters can be summarized as follows. First of all we should determine the fault length and width and possible fault segmentation of the scenario earthquake we want to simulate, based on the active fault information above described. The second step is to calculate the total seismic moment using the empirical relationship between the fault rupture area and seismic moment as shown in Figure 2.1.1. With the seismic moment we can then read the asperity parameters like the largest asperity area (Figure 2.1.2) and the average fault rise time (Figure 2.1.3). There is a similar relationship for calculating the combined area of asperities, which account for the total area
of asperities, and the number of asperities against the seismic moment (Somerville 1999). Those relationships can be used together to determine the total number of asperities and their size.

![Graph showing the relationship between rupture area and seismic moment](image1)

Figure 2.1.1. Scaling relationship between the rupture area and the earthquake seismic moment (Somerville 1999).

![Graph showing the relationship between asperity area and seismic moment](image2)

Figure 2.1.2. Scaling relationship between the largest asperity area and the earthquake seismic moment (Somerville 1999)

The estimation of the static stress drop of the asperities can be made from theoretical relationships linking the seismic moment and a fault characteristic dimension. For the case of a strike-slip fault embedded in a half space the expression is as follows:

$$\Delta \sigma = \frac{2 M_0}{\pi W^2 L}$$  

(2.1.1)
For a dip-slip fault the expression is as follows:

\[
\Delta \sigma = \frac{4(\lambda + \mu)}{\pi(\lambda + \mu)} \frac{M_0}{W^2 L}
\]  
(2.1.2)

For a circular fault:

\[
\Delta \sigma = \frac{7 M_0}{16 r^3}
\]  
(2.1.3)

Where \( \Delta \sigma \) is the stress drop, \( \lambda \) and \( \mu \) are the Lamé constants, \( L \) and \( W \) fault length and width and \( r \) is the fault radius.

Concerning the rupture velocity this is a very important parameter related with the dynamic rupture process of the fault that has a large influence on the ground motion characteristics (Fukuyama and Madariaga 1998). A common assumption is to consider a value between 0.7 to 0.9 times the average S-wave velocity in the rupture area. This parameter could have a large heterogeneity across the fault plane and thus more research is required to clarify its spatial variation across the fault.

![Scaling relationship between the average fault rise time and the earthquake seismic moment (Somerville 1999)](image)

Figure 2.1.3. Scaling relationship between the average fault rise time and the earthquake seismic moment (Somerville 1999)

2.1.2 Ground Motion Estimation Methodology

The basic idea of the simulation methodology is to evaluate the strong ground motion radiated from a finite source model composed of asperities or regions in the fault plane with a large slip, embedded in a layered velocity structure. The ground motion at a particular target station is obtained from the contribution of the radiation of all the asperities in the fault plane that are assumed to have a finite area. For the case of calculating the strong ground motion from a scenario earthquake (ground motion simulation for future earthquakes) the fault and asperity parameters are calculated as shown in the above sections. A strong ground motion simulation for a scenario earthquake for the Tottori prefecture (Japan) will be shown in chapter 5.
Figure 2.1.4. Total Slip distribution of 1992 Landers earthquake (Wald and Heaton 1994). The asperities and rupture area determined based on the methodology by Somerville et al. (1999) are shown by a thick line.

For the purpose of calculating the strong ground motion for past earthquakes as in the case of our strong ground motion simulation of the 1999 Kocaeli (Turkey) earthquake (discussed in chapter 4), we will use the definition of an asperity as the region in the fault plane where the slip distribution is 1.5 or more times larger than the total fault plane average slip (Somerville et al. 1999). In a similar way the rupture area is obtained from trimming an edge row or column of the slip model if its average slip is less or equal than 0.3 time the average slip over the total area (Somerville 1999). An illustration of these calculations for the 1992 Landers earthquake from the slip model of Wald and Heaton (1994) is shown in Figure 2.1.4 by thick lines.

The ground motion estimation methodology aims to produce ground motions in a broadband frequency range (0.1Hz to 10 Hz) in order to be able to compare the simulated ground motions with the observed damage distribution. The procedure to be applied is a hybrid ground motion simulation technique, which consists in the generation of ground motions in a low frequency (<1Hz) and high frequency (>1Hz) bands as illustrated in Figure 2.1.5 (Kamae and Irikura 1998).

The low frequency part of the ground motion is calculated from the 3D radiation of an asperity model like the one shown in Figure 2.1.4 for the Landers earthquake, propagating in a flat-layered velocity structure. For this purpose a Discrete Wave Number method for a 3D elastic wave propagation in a layered media is applied (Bouchon 1981). An extended source discretized into several sub-faults is used to calculate the ground motion from each asperity. The contribution from each sub-fault inside the asperities is time delayed according to an assumed rupture velocity.

The high frequency motion generation uses the idea of the empirical Green's function technique (Irikura 1983), which consists in using recordings from small events (aftershocks) in order to reproduce the ground motion from a large event (mainshock). For that purpose the scaling relation of the source spectra and the source parameters together with an appropriate selection of the small event is considered. For regions, where no appropriate recording of aftershocks is available, the seismograms of the small event are generated stochastically in such a way that they follow an omega square model and a regional attenuation relationship (Boore 1983). Then the empirical Green's function method is applied using the synthetic aftershock waveform obtained previously.
Finally the amplification of the seismic waves and the nonlinearity effect of surficial layers should be included to get the ground motion at a specific site. The methodology for performing the non-linear calculations of the ground motion is described in sections 2.4 and 2.5 in this volume. The final motion is obtained from the summation of the low and high frequency parts obtained before.

**High Frequency Ground Motion**

The stochastic Green's functions are calculated according to Boore (1983). The waveforms are generated to meet an acceleration Fourier spectra that follows an omega square model and a regional attenuation relationship. We modified the original equation of Boore in order to include a frequency dependent site effect $F(w)$ into the acceleration Fourier spectra as follows:

$$A(w) = f(R_{\theta\phi})M_0F(w)S(w, w_c)A_r(Q, w) \quad (2.1.4)$$

where: $w$ is the frequency, $A(w)$ is the acceleration Fourier spectra, $F(R_{\theta\phi})$ the radiation pattern, $M_0$ Seismic Moment and $F(w)$ is a frequency dependent site effect.

The source omega square model is defined as follows:

$$S(w, w_c) = \frac{w^2}{1 + \left(\frac{w}{w_c}\right)^2} \quad (2.1.5)$$

where $w, w_c$ are frequency and corner frequency respectively.

The regional attenuation is calculated in the following way:

$$A_r(Q, w) = e^{-\frac{wR}{2Q\beta}} \quad (2.1.6)$$

where $Q$ is the quality factor, $R$ is the epicentral distance, $\beta$ is the S wave velocity.
The empirical Green’s Function Method (Irikura 1986) is then used to calculate the ground motion from the same asperity model as in the low frequency case. Each asperity is discretized into several sub-faults as in Figure 2.1.6, and the Green function from them is calculated from the stochastic methodology above described. The total ground motion from the asperity is obtained by the convolution of the ground motion from each sub-fault \( u(t) \) with a scaling function between the slip of the large event and small event:

\[
U(t) = \sum_{i=1}^{N} \sum_{j=1}^{N} \frac{r}{r_{ij}} F(t-t_{ij}) * u(t)
\]  

(2.1.7)

where: \( U(t) \) is ground motion of main (target) event and \( u(t) \) is ground motion small event.

The rupture delay time \( t_{ij} \), slip scaling between the large and small event \( F(t) \) and scaling source parameter \( N \) are shown as follows:

\[
t_{ij} = \frac{r_{ij} - r_0}{\beta} + \frac{\xi_{ij}}{V_r}
\]  

(2.1.8)

\[
F(t) = \delta(t) + \frac{1}{n} \sum_{k=1}^{(N-1)n'} \delta(t-(k-1)\frac{\tau}{(N-1)n'})
\]  

(2.1.9)

\[
N = \left( \frac{M_{0r}}{cM_{0s}} \right)^{1/3}
\]  

(2.1.10)

where: \( r_0, r_{ij} \) are the distance between the target station and the hypocenter and \( ij \) subfault respectively (Figure 2.1.6), \( \xi_{ij} \) is the distance between the hypocenter and the \( ij \) subfault, \( \beta \) is the S wave velocity, \( \tau \) is the rise time of the target event, \( n' \) is an integer to reduce spurious periodicity related with the summation procedure (Irikura 1983), \( V_r \) is the rupture velocity, \( M_{0r}, M_{0s} \) are the target and small event seismic moments and \( c \) is the stress drop ratio between the large and small event.

Figure 2.1.6. Empirical Green’s function. Fault discretization and sub-fault summation (modified from Miyake et al. 1999).
2.1.3 Kinematic vs. Dynamic Models of the Source

The model described so far is a Kinematic model of the source in which no consideration about the tectonic stress surrounding the active faults, and the earthquake stress release process have been made. More complex analysis of the rupture requires a fault dynamic model in which the rupture velocity is not constrained, and an assumption of the tectonic stress surrounding the fault is made. Dynamic models of the source are based on parameters whose estimation is less constrained than the ones for kinematic models. For estimating the dynamic parameters of the rupture a methodology has been developed based on the spatio-temporal distribution of the earthquake radiated energy and apparent stress over the fault plane (Pulido and Irikura 2000).

The dynamic models of the source will allow in the future to make a strong ground motion simulation that fully includes the earthquake dynamic rupture process from the nucleation to the arrest of the rupture, the tectonic stress surrounding the faults and the complex stress interaction in the non-planar geometry of segmented faults.

References


2.2 \( V_S \) Profiling Using Microtremor H/V Spectra

2.2.1 Theoretical Background

After the pioneer work by Kanai and Tanaka (1961), microtremor measurement using one station has been used for estimating dynamic characteristics at a site, specifically natural site period. They assumed that microtremor horizontal motions at periods less than 1 s consist mainly of shear waves, and that Fourier spectra of observed horizontal motions reflect transfer function of ground at a site. However, many researchers, e.g., Udwadia and Trifunac (1978), indicated that microtremor spectra often pointed the exciting function rather than transfer function of a site.

Nakamura and Ueno (1986), and Nakamura (1989) proposed a revised method in which the effect of source function might be minimized by normalizing horizontal spectral amplitude in terms of vertical one. Assuming that S-waves dominate microtremors, they indicated that horizontal-to-vertical (H/V) spectral ratio of microtremors at a site roughly equals S-wave transfer function between ground surface and bedrock at a site. This means that H/V peak period and peak value itself respectively correspond to natural site period and amplification factor. This method has potential to make site periods more reliable, however, it rests on tenuous assumptions (e.g., Finn, 1991; Horike, 1993). Recent studies have, in fact, shown that Frequency-wave number (F-k) spectral analysis (Capon, 1969) and Spatial Auto-Correlation (SAC) analysis (Aki, 1957; Matsushima and Okada, 1990) of microtremor records measured with arrays at a site can yield dispersion characteristics of Rayleigh waves (and Love waves under a certain condition), and the inverse analysis of dispersion data successfully results in \( V_S \) profile at a site (e.g., Horike, 1985; Okada and Matsushima, 1986; Tokimatsu et al., 1992). These present that microtremors mainly consist of surface (Rayleigh and Love) waves.

Based on the above findings, Tokimatsu and Miyadera (1992) pointed out that the variation with frequency of H/V ratio of microtremors corresponds to that of fundamental Rayleigh mode at a site. However, H/V values of microtremors are not comparable to those of fundamental Rayleigh wave. It is considered that this disagreement could be caused by the influence of other surface waves in microtremors, i.e., higher Rayleigh modes and Love waves (e.g., Tokimatsu and Tamura, 1994; Lachet and Bard, 1994). In order to solve this problem, Tokimatsu and Arai (1998) presented theoretical formulas for simulating microtremor H/V spectra with the use of surface (both Rayleigh and Love) waves propagating on a layered half-space, considering the effects of fundamental and higher modes. Using these formulas, Arai and Tokimatsu (1998) also presented an inverse analysis of microtremor H/V spectrum for estimating the layer thickness of subsurface soil, when a prior information of \( V_S \) values at a site is given. In this chapter, these microtremor H/V methods revised by Tokimatsu and Arai (1998) and Arai and Tokimatsu (1998) are briefly reviewed.
2.2.2 H/V Spectra of Rayleigh and Surface Waves

Soil model is assumed to be a semi-infinite elastic medium made up of N parallel, solid, homogeneous, isotropic layers (Figure 2.2.1(a)). Each layer is characterized by thickness, H, mass density, \( \rho \), P-wave velocity, \( V_p \), and S-wave velocity, \( V_S \). The origin of orthogonal coordinate system is placed on free surface (Figure 2.2.1(b)). To model loading sources of microtremors, it is assumed that Fourier time transformed vertical and horizontal point forces, \( L_V(t) \) and \( L_H(t) \), are randomly distributed on free surface (e.g., Lachet and Bard, 1994). At the origin, vertical and two orthogonal horizontal displacements induced by point sources are observed. Amplitude of horizontal motions at the origin is defined as square root of sum of squares of two orthogonal horizontal displacements.

Surface and body waves are generated from each source and propagate on the medium. Based on the studies by Harvey (1981) and Tokimatsu and Tamura (1995), displacements at the origin could be expressed only by surface waves under the following conditions: (1) each distance between the origin and point source is longer than unity wavelength of Rayleigh or Love wave (\( \lambda_{Rj} \) or \( \lambda_{Lj} \); see Figure 2.2.1(b)), and (2) effective periods are less than natural site period. Under these conditions, vertical and horizontal power of j th Rayleigh mode from the i th vertical point source may be expressed by Harkrider (1964):

\[
P_{VRij}^V = L_V^2 A_{Rj}^2 |H_0^{(2)}(k_{Rij}r)|^2 \exp(-2hk_{Rij}r) \tag{2.2.1}
\]

\[
P_{HRij}^V = L_V^2 A_{Rj}^2 (u/w)^2 |H_1^{(2)}(k_{Rij}r)|^2 \exp(-2hk_{Rij}r) \tag{2.2.2}
\]

in which \( A \) is a medium response factor (Harkrider, 1964), \( k \) is wave number, \( u/w \) is H/V ratio of Rayleigh mode at free surface, subscript \( R \) presents Rayleigh wave, \( r \) is distance between the origin and the point source, \( H_n^{(2)}(\cdot) \) is Hankel function of the second kind of order \( n \), and \( h \) is scattering damping ratio of soil. Similarly, vertical and horizontal power of the j th Love mode from the i th horizontal point source may be expressed by

\[
P_{VRij}^I = (L_H^2/2) A_{Rj}^2 (u/w)^2 |H_1^{(2)}(k_{Rij}r)|^2 \exp(-2hk_{Rij}r) \tag{2.2.3}
\]

\[
P_{HRij}^I = (L_H^2/2) A_{Rj}^2 (u/w)^4 |H_0^{(2)}(k_{Rij}r)|^2 \exp(-2hk_{Rij}r) \tag{2.2.4}
\]

Horizontal power of the j th Love mode from the i th horizontal point source may be

\[
P_{HLij}^I = (L_H^2/2) A_{Lj}^2 |H_0^{(2)}(k_{Lij}r)|^2 \exp(-2hk_{Lij}r) \tag{2.2.5}
\]

in which subscript \( L \) presents Love wave. Assuming the statistical independence among the loading phases of all sources, vertical and horizontal relative powers of all waves observed at the origin are given by integrating Eqs. (2.2.1)-(2.2.5) for all point sources and Rayleigh- or Love- modes (i.e., for all subscript \( i \) and \( j \) existed). The resulted vertical and horizontal relative powers, \( P_{VS}, P_{HS} \), are presented as follows:

\[
P_{VS} = P_{VR} = \sum_j (A_{Rj}/k_{Rj})^2 \{1+(\alpha^2/2)(u/w)_j^2\} \tag{2.2.6}
\]

\[
P_{HS} = P_{HR} + P_{HL} \tag{2.2.7}
\]

\[
P_{HR} = \sum_j (A_{Rj}/k_{Rj})^2 (u/w)_j^2 \{1+(\alpha^2/2)(u/w)_j^2\}, \quad P_{HL} = \sum_j (\alpha^2/2)(A_{Lj}/k_{Lj})^2
\]

where \( \alpha \) is H/V ratio of loading force, \( L_H/L_V \). Using Eqs. (2.2.6) and (2.2.7), H/V ratio of
Rayleigh waves, \( (H/V)_R \), that of surface waves, \( (H/V)_S \), and Rayleigh to Love wave amplitude ratio in horizontal motions, \( R/L \), are expressed as

\[
(H/V)_R = \left( \frac{P_{HR}}{P_{VR}} \right)^{1/2} \tag{2.2.8}
\]

\[
(H/V)_S = \left( \frac{P_{HS}}{P_{VS}} \right)^{1/2} \tag{2.2.9}
\]

\[
R/L = \left( \frac{P_{HR}}{P_{III}} \right)^{1/2} \tag{2.2.10}
\]

Eqs. (2.2.8)-(2.2.10) are formulated in three-dimensional wave propagating field. In two-dimensional wave field, \((H/V)_R\), \((H/V)_S\), and \(R/L\) can also be formulated by Eqs. (2.2.8)-(2.2.10), because the relative amplitudes of the \(j\) th Rayleigh and Love modes in Eqs. (2.2.1)-(2.2.5) are only replaced by \(A_{Rj}/k_{Rj}\) and \(A_{Lj}/k_{Lj}\), respectively (Regan and Harkrider, 1980; Hisada et al., 1991).

In computing \((H/V)_R\) and \((H/V)_S\) using Eqs. (2.2.8)-(2.2.10), the values of \(\alpha\) or \(R/L\) are required in addition to soil model. According to the studies by Matsushima and Okada (1990) and Tokimatsu and Arai (1998) values of \(R/L\) estimated from F-k and SAC analyses of microtremor array data could be stable over a day, and be 0.4-1 in a period range of 0.1-5 s. Thus, it could be assumed that the \(R/L\) value for computing \((H/V)_R\) and \((H/V)_S\) is 0.7 at any frequency.

Details of all the theoretical \(H/V\) formulation and microtremor \(R/L\) estimation were described in the studies by Tokimatsu and Arai (1998) and Arai and Tokimatsu (2000).

\[\text{Figure 2.2.1. Geometry of (a) soil layer and (b) microtremor source models (Tokimatsu and Arai, 1998).}\]
2.2.3 H/V Spectra of Microtremors

In the past studies by Tokimatsu and Miyadera (1992), Tokimatsu (1995), and so on, microtremor H/V spectral ratio, \((H/V)_{mR}\), was defined as

\[
(H/V)_{mR} = \frac{(S_{NS} S_{EW})^{1/2}}{S_{UD}}
\]  

(2.2.11)

where \(S_{UD}\) is Fourier amplitude of vertical motion, and \(S_{NS}\) and \(S_{EW}\) are that of two orthogonal horizontal motions. In this definition, \((S_{NS} S_{EW})^{1/2}\) was assumed to correspond to amplitude of Rayleigh waves. In some cases, however, this definition might not be equivalent to theoretical \((H/V)_R\) in Eq. (2.2.8). Thus, microtremor H/V spectral ratio, \((H/V)_{mS}\), which could be equivalent to \((H/V)_S\) in Eq. (2.2.9), was defined as

\[
(H/V)_{mS} = \frac{(S_{NS}^2 + S_{EW}^2)^{1/2}}{S_{UD}}
\]  

(2.2.12)

In order to examine the applicability and limitation of Eqs. (2.2.8)-(2.2.10) to microtremor H/V spectra defined by Eqs. (2.2.11) and (2.2.12), three-component microtremor data observed at four sites nearby Tokyo, hereinafter called Sites A, E, F, and G, were used in the studies by Tokimatsu and Arai (1998) and Arai and Tokimatsu (2000). Details of the observations can be found elsewhere (e.g., Tokimatsu, 1995). Shallow \(V_S\) profiles from down-hole method at the sites are shown in Figure 2.2.2 (Ishihara et al., 1989; Tokimatsu, 1995). Tables 2.2.1 and 2.2.2 show deep soil profiles at the sites modeled by the results of other geophysical investigations (Shima et al., 1976; Higashi and Kudo, 1992).

![Figure 2.2.2. Shallow V_S profiles at Sites A, E-G (Ishihara et al., 1989; Tokimatsu, 1995).](image)

Table 2.2.1. Deep soil layer models at Sites A, E, and F (Shima et al., 1976).

<table>
<thead>
<tr>
<th>Depth (km)</th>
<th>(\rho) (t/m³)</th>
<th>(V_p) (km/s)</th>
<th>(V_s) (km/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1.5</td>
<td>1.9</td>
<td>1.8</td>
<td>0.7</td>
</tr>
<tr>
<td>1.5 - 2.3</td>
<td>2.2</td>
<td>2.8</td>
<td>1.5</td>
</tr>
<tr>
<td>2.3 -</td>
<td>2.5</td>
<td>5.6</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Table 2.2.2. Deep soil layer models at Sites G (Higashi and Kudo, 1992).

<table>
<thead>
<tr>
<th>Depth (km)</th>
<th>(\rho) (t/m³)</th>
<th>(V_p) (km/s)</th>
<th>(V_s) (km/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.3</td>
<td>2.0</td>
<td>2.3</td>
<td>0.8</td>
</tr>
<tr>
<td>0.3 - 2.0</td>
<td>2.3</td>
<td>3.0</td>
<td>1.5</td>
</tr>
<tr>
<td>2.0 - 3.2</td>
<td>2.5</td>
<td>4.2</td>
<td>2.4</td>
</tr>
<tr>
<td>3.2 -</td>
<td>2.8</td>
<td>5.5</td>
<td>2.8</td>
</tr>
</tbody>
</table>
Effects of Rayleigh Waves on Microtremor H/V Spectra

Microtremor H/V spectra \((H/V)_{mR}\) determined by Eq. (2.2.11) at Sites A and G are shown in Figure 2.2.3 in crosses. Broken and solid lines in the figure are H/V spectra of fundamental and superposed Rayleigh modes, \((H/V)_{R0}\) and \((H/V)_R\), respectively, computed for the soil profiles at the sites. At Site A, variation with the period of computed H/V spectrum \((H/V)_{R0}\) is in good agreement with the observed one, \((H/V)_{mR}\). At Site G, however, the computed spectrum \((H/V)_{R0}\) is inconsistent with the observed \((H/V)_{mR}\) in a period range of 0.5-3 s. Besides, the values of \((H/V)_{R0}\) are not comparable to those of the observed \((H/V)_{mR}\). On the other hand, the amplitude of \((H/V)_R\), considering the effects of higher modes, is in good agreement with the observed \((H/V)_{mR}\) at each site.

Vertical and horizontal response functions, \(A_R/k_R\) and \((A_R/k_R)(u/w)\), up to the 3rd higher Rayleigh-mode, are shown in Figure 2.2.3 in lines to investigate why the good agreement exists between \((H/V)_R\) and \((H/V)_{mR}\). The good agreement between \((H/V)_R\) and \((H/V)_{mR}\) is found mainly in a period range of 1.2 s and over 0.5 s at Sites A and G, respectively. In these period ranges, the values of both vertical and horizontal response functions of higher Rayleigh-modes are larger than those of fundamental mode. This shows that higher modes of Rayleigh waves have significant effects on H/V spectra of microtremors in the frequency range where response functions of higher modes are predominant.

![Figure 2.2.3](image)

Figure 2.2.3. H/V spectra of microtremors compared with those of Rayleigh waves at Sites A and G (Tokimatsu and Arai, 1998).
Effects of Love Waves on Microtremor $H/V$ Spectra

In Figure 2.2.3, the values of $(H/V)_R$ are less than those of the observed $(H/V)_{mR}$. The misfit is considered to be resulted from the definition of $H/V$ spectra of microtremors, $(H/V)_{mR}$, in Eq. (2.2.11). In Figure 2.2.4, therefore, $H/V$ spectra of microtremors, $(H/V)_{mS}$, determined by Eq. (2.2.12) at Sites A and E-G are shown in open circles. Solid lines in the figure are $H/V$ spectra of surface waves, $(H/V)_S$, computed for the soil profiles at the sites, assuming that the values of horizontal Rayleigh to Love wave amplitude ratio, $R/L$, are 0.7 at given frequencies. At each site, both the values and variation with frequency of theoretical $(H/V)_S$ show fairly good agreement with those of observed $(H/V)_{mS}$, and fitness between theoretical and observed $H/V$ spectra is better than that shown in Figure 2.2.3. This confirms that theoretical $H/V$ spectrum of surface waves, $(H/V)_S$, determined by Eq. (2.2.9) well simulate that of microtremors, $(H/V)_{mS}$, defined by Eq. (2.2.12). Besides, a comparison of Figure 2.2.3 with Figure 2.2.4 can point out that $(H/V)_S$ is almost parallel to $(H/V)_R$ in log-log scale at each site, indicating that variation with frequency of $H/V$ ratio of microtremors is mainly controlled by that of Rayleigh waves, and that the value of $H/V$ ratio of microtremors is controlled by Love waves.

Details of comparison between theoretical and observed microtremor $H/V$ spectra were discussed in the studies by Tokimatsu and Arai (1998) and Arai and Tokimatsu (2000).

Figure 2.2.4. $H/V$ spectra of microtremors compared with those of surface waves at Sites A and E-G (Tokimatsu and Arai, 1998).
2.2.4 Possibility of Soil Profiling Using Microtremor H/V Spectra

Sensitivity analyses of H/V ratios of Rayleigh and surface waves for soil models were performed to investigate a possibility of soil profiling using microtremor H/V spectra based on the theoretical formulas expressed previously. Sensitivity, \( D \), of H/V ratios of surface waves, \((H/V)_S\), for parameter \( p \) (thickness, mass density, P-wave velocity, and S-wave velocity) at a layer of soil model could be expressed as

\[
D = |(p / (H/V)_S) (\partial (H/V)_S / \partial p)|
\]  
(2.2.13)

In Eq. (2.2.13), the larger the value of \( D \) is, the higher the sensitivity of \((H/V)_S\) for the parameter \( p \) becomes. Figure 2.2.5(a) shows one of the soil models used in the sensitivity analyses. In the analyses, deep soil layers were assumed to be those shown in Table 2.2.1. Figures 2.2.5(b)-(e) show the results of the sensitivity analyses for thickness, mass density, P-wave velocity, and S-wave velocity at each layer of the test model shown in Figure 2.2.5(a). The values of sensitivities of \((H/V)_S\) for thickness and S-wave velocity (Figures 2.2.5(b) and (e)) are larger than those for mass density and P-wave velocity (Figures 2.2.5(c) and (d)). The same trend was confirmed for several soil models and for H/V ratios of Rayleigh waves, \((H/V)_R\). This indicates that either thickness or S-wave velocity at each layer of a soil model or both of them, i.e., \(V_S\) profile, could be estimated from H/V ratio of microtremors at a site using the theoretical formulas stated previously.

Details of the sensitivity analyses of H/V spectra were described in Arai and Tokimatsu (2000).

![Figure 2.2.5](image)

Figure 2.2.5. An example of sensitivity analyses of surface wave H/V ratios (Arai and Tokimatsu, 2000).
2.2.5 Inversion of Microtremor H/V Spectra

Soil model is assumed to be a semi-infinite elastic medium made up of $N$ parallel, solid, homogeneous, and isotropic layers (Figure 2.2.6). Each layer is characterized by thickness, $H$, mass density, $\rho$, P-wave velocity, $V_p$, and S-wave velocity, $V_S$. These soil layer modeling are completely same as those in the previous section 2.2.2. The misfit function $F$ to be minimized in the inversion could be defined as follows:

$$F = \sum_j (H/V)_{mR}(f_j) - (H/V)_{R}(f_j))^2 W_j$$  \hspace{1cm} (2.2.14)

or

$$F = \sum_j (H/V)_{mS}(f_j) - (H/V)_{3}(f_j))^2 W_j$$  \hspace{1cm} (2.2.15)

where $W_j$ is a weighting factor defined by adaptive biweight method (Tukey, 1974) at a given frequency of $f_j$. Adaptive biweight method is a kind of maximum likelihood estimation, and is also of robust estimation.

In this inversion, both genetic algorithm (GA) and non-linear least-squares method are used. First, soil model is searched by GA method using elite selection and dynamic mutation techniques (Yamanaka and Ishida, 1995). Since GA method is a kind of probabilistic approach, the GA solution is determined by averaging the models from 10 GA inversions, where each inversion uses different initial model. Details of the GA inversion methodology can be found elsewhere (e.g., Yamanaka and Ishida, 1995). With the GA solution, non-linear least-squares inversion is performed, in which modified Marquardt technique (Marquardt, 1963) and singular value decomposition method (e.g., Matsu'ura and Hirata, 1982; Horike, 1985) are used. Iteration procedure is terminated when the norm of modification vector is converged into a sufficiently small value, and then the final solution is determined (see Figure 2.2.6). Details of the non-linear least-squares inversion can be found elsewhere (e.g., Wiggins, 1972; Matsu'ura and Hirata, 1982; Horike, 1985; Okada and Matsushima, 1986; Tokimatsu et al., 1992; Yuan and Nazarian, 1993; Arai and Tokimatsu, 1998).

![Figure 2.2.6. Inversion procedure of H/V spectrum (Arai and Tokimatsu, 1998).](image)
In order to examine the effectiveness of the inversion methodology expressed above, H/V spectra of microtremors observed at Sites A, G, and other two sites in Japan, hereby called Sites H and I, were used in the study by Arai and Tokimatsu (1998). Sites H and I are Kushiro Japan Meteorological Agency and Kushiro Harbor in Hokkaido. Details of the observation can be found elsewhere (e.g., Tokimatsu, 1995).

The observed microtremor H/V spectra, \( (H/V)_{\text{obs}} \), defined by Eq. (2.2.12) at the sites are shown in Figures 2.2.7(a)-(d) in open circles. Using these data, the inverse analyses were conducted on the presumption that soil profile at each site consists of 5-8 layered half-space, deep soil layers with \( V_s \) over 700 m/s are those shown in Tables 2.2.1-2.2.4, modeled from the results of other field investigations (e.g., Shima et al., 1976; Higashi and Kudo, 1992; Tokimatsu, 1995), and \( V_s \) values of shallow soil layers are from down-hole method. The above assumptions leave only the thickness of shallow soil layers unknown. Thus, the thickness of the layers with \( V_s \) less than 700 m/s are sought so that misfits between H/V ratios of observed microtremors and theoretical surface waves can be minimized.

Figures 2.2.8(a)-(d) show the inverted \( V_s \) profiles with standard errors (e.g., Matu'ura and Hirata, 1982) of layer thickness. Solid lines in Figures 2.2.7(a)-(d) are the theoretical H/V spectra, \( (H/V)_S \), determined by Eq. (2.2.9) for the inverted profiles at the sites. The computed H/V spectra are in fairly good agreement with the observed ones for all the sites. In Figures 2.2.8(a)-(d), the soil profiles estimated from microtremors are consistent with those from down-hole method shown in broken lines. Furthermore, evaluated standard errors of layer thickness at each site are quite small. These suggest that the inversion of microtremor H/V spectrum is effective for estimating layer thickness of subsurface soil, when a prior information of \( V_s \) values at a site is given.

<table>
<thead>
<tr>
<th>Depth(km)</th>
<th>( \rho ) (t/m³)</th>
<th>( V_p )(km/s)</th>
<th>( V_s )(km/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>- 0.1</td>
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<td>0.7</td>
</tr>
<tr>
<td>0.1 -</td>
<td>2.0</td>
<td>4.0</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Table 2.2.3. Deep soil layer models at Sites H (Tokimatsu, 1995).

<table>
<thead>
<tr>
<th>Depth(km)</th>
<th>( \rho ) (t/m³)</th>
<th>( V_p )(km/s)</th>
<th>( V_s )(km/s)</th>
</tr>
</thead>
<tbody>
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<td>- 0.2</td>
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<td>0.7</td>
</tr>
<tr>
<td>0.2 -</td>
<td>2.0</td>
<td>4.0</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Table 2.2.4. Deep soil layer models at Sites I (Tokimatsu, 1995).

Figure 2.2.7. H/V spectra of microtremors compared with those of surface waves theoretically computed for the inverted soil layer models at Sites A and G-I (Arai and Tokimatsu, 1998).
Figure 2.2.8. Comparison of $V_s$ profiles estimated from microtremors with those from down-hole methods at Sites A and G-I (Arai and Tokimitsu, 1998).

References


Ishihara, K. et al. (1989). “In-situ pore water pressures and ground motions during the 1987


2.3 Ground Response Analysis

2.3.1 Outline of Numerical Method

The Finite Element Method (FEM) is a numerical analysis theory that is widely used for various situations in engineering. Analyses on dynamic soil-structure interaction (SSI) problem with the FEM can be dated from 1960s. Most of the case, such a problems are performed by 2-dimentional models with plain strain elements.

One of the major issues for such analyses is how to treat the infinity of ground stiffness by the finite model. The boundary should be set sufficiently far from the site in general, so as to attenuate the energy of outwardly propagating wave, it will be quite wasteful of time and money, however. In the appropriate size of model, reflections of propagating waves on artificial boundary must be avoided, consequently. For the purpose, Lysmer and Kuhlemeyer (1969) had proposed the transmitting boundary with frequency independent viscous dashpot.

Another issue is the method for considering the non-linearity of soil. It can be computed by time domain analysis considering non-linear hysteresis. However, it is inferior in the point of practical use. For this reason, an equivalent linear method is commonly used for approximate approach to consider non-linearity of the soil.

In the following case studies, analyses were performed by the finite element method in frequency domain considering the above transmitting boundary and treatment of non-linearity. The most popular program based on the method is FLUSH (Lysmer et al., 1975), which had been used for dynamic response analysis of SSI and ground.

2.3.2 Analysis Models

In all the case studies, square shaped element was applied for the plain strain element in each layer of the ground model. Transmitting and viscous boundaries were used on both sides and at the bottom of the soil model, respectively. For the case subsurface soil is involved in the analysis model; i.e. Golcuk (Chapter 4.4) and Yonago (Chapter 5.4), equivalent linear method was used for the strong motion simulation in order to take the nonlinear behavior into account. Hardin-Drnevich (H-D) model was inferred for strain-stiffness and strain-damping relationships of soils.

Maximum element size of each layer $H_{\text{max}}$ was determined as following:

$$H_{\text{max}} = \frac{1}{5} \cdot \frac{V_s}{F_{\text{max}}}$$

(2.3.1)

where $V_s$ is shear velocity of the layer, $F_{\text{max}}$ is effective frequency range for calculation, and the value $1/5$ is the reciprocal of the partition number in a element.

As the input motion, outcrop motions assumed at the bottom of the model were applied. Inverted bedrock motions from observed record at ground surface were used in the Kobe case (Chapter 3.2) and Golcuk case (Chapter 4.4). Meanwhile, the estimated bedrock motions from the methodology of broadband frequency estimation were used in Yonago case (Chapter 5.4). In all cases, the input motion to the ground model is assumed to be composed of vertically incident S-waves with in-plain particle motions. Hence, in-plain horizontal motions were generated from two horizontal components for the input.

References

2.4 One-Dimensional Soil Liquefaction Analysis

One-dimensional effective stress analysis, so-called “liquefaction analysis,” used in this report is based on those proposed by Shamoto et al. (1992, 1997) and Zhang et al. (1997). In this section, the analytical methodology is briefly reviewed. Details of the methodology can be found in the studies by Shamoto et al. (1992, 1997) and Zhang et al. (1997).

2.4.1 Equation of Motion

In order to solve one-dimensional dynamic response of a multi-layered soil deposit shown in Figure 2.4.1(a), soil deposit is modeled into a discrete mass-spring-damper system in Figure 2.4.1(b). Subject to a uniform soil layer of thickness $H$ with unit weight $\rho$ and rigidity (shear modulus) $G$, and a linear distribution of displacements within each layer, the equation of motion for the discrete system can be presented by a matrix form:

$$[M] \frac{d^2 \{x\}}{dt^2} + [C] \frac{d\{x\}}{dt} + [K]\{x\} = -\frac{d^2 y}{dt^2} [M]\{I\}$$  \hspace{1cm} (2.4.1)

where $[M]$ is lumped mass matrix determined from unit weight and thickness of each layer, $[K]$ is stiffness matrix from shear modulus and thickness of the layers, $[C]$ is Rayleigh viscosity matrix computed from $[M]$ and $[K]$ matrixes, $\{I\}$ is unit vector, $\{x\}$ is displacement vector of mass points relative to base layer, $y$ is displacement of base layer, and $t$ denotes time.

![Diagram of horizontally layered soil deposit](image)

Figure 2.4.1. Discrete model of horizontally layered soil deposit.
Increasing and decreasing process of pore water pressure, \( u \), could be expressed as follows:

\[
\frac{\partial u}{\partial t} - \frac{\partial u_g}{\partial t} = \left( \frac{k_w}{(m_v \gamma_w)} \right) \frac{\partial^2 u}{\partial z^2}
\]  

(2.4.2)

where \( k_w \) is coefficient of permeability, \( m_v \) is bulk modulus, \( \gamma_w \) is unit weight of water, \( z \) is depth, and \( u_g \) is pore water pressure generated by cyclic shear deformation of soil. Converting Eqs. (2.4.1) and (2.4.2) into incremental and differential expressions, the equations can be alternately solved on time series following a step-by-step time integration with an interval of \( \Delta t \), in which Newmark's \( \beta \) method (\( \beta = 0.25 \)) is used.

### 2.4.2 Constitutive Model of Soils

Constitutive model of soil employed in the analysis consists of 1) shear stress-strain relations of soils and 2) generation of pore water pressure, \( u_g \), due to cyclic shear loadings. In this study, initial and degraded skeleton and hysteresis curves for stress-strain relations are determined by modified Ramberg-Osgood model (Jennings, 1946) considering effective stress and by Masing's Law (Masing, 1926). Pore water pressure is evaluated by extending empirical equations originally proposed by Seed et al. (1976). The extended equations considering cyclic mobility, moving critical stress state and phase-transformation lines, and post-liquefaction were shown in the studies by Shamoto et al. (1992, 1997) and Zhang et al. (1997). Finally, constitutive model for determining cyclic stress-strain behavior is composed of following Eqs. (2.4.3)-(2.4.25):

1) Stress-strain relations

\[
\gamma = \gamma_0 + \gamma_d
\]  

(2.4.3)

\[
\gamma_0 = \left( 14.08 / M_f \right) \left( e_0 - e_{\text{min}} \right) / \left( 1 + e_0 \right) \left( \gamma_{\text{max}} - \gamma_{\text{entry}} \right)^{0.76}
\]  

(2.4.4)

\[
\dot{\gamma}_d = (d\gamma / G) \left( 1 + \alpha \beta \left| \Delta \tau / G \right|^{\beta - 1} \right) \quad \text{(on skeleton curve)}
\]  

(2.4.5)

\[
\dot{\gamma}_d = (d\gamma / G) \left( 1 + \alpha \beta \left| \Delta \tau / 2G \right|^{\beta - 1} \right) \quad \text{(on hysteresis curve)}
\]  

(2.4.6)

\[
\alpha = \left( 2 / \gamma_{\text{ef}} \right)^{\beta - 1}
\]  

(2.4.7)

\[
\beta = \left( 2 + \pi h_{\text{max}} \right) / \left( 2 - \pi h_{\text{max}} \right)
\]  

(2.4.8)

\[
\gamma_d = \gamma_{\theta, 0} \left( \sigma' / \sigma_0' \right)^{0.5}
\]  

(2.4.9)

\[
G = G_0 \left( \sigma' / \sigma_0' \right)^{0.5}
\]  

(2.4.10)

\[
\sigma' = \sigma - u
\]  

(2.4.11)

\[
\sigma = \sigma_v (1 + 2K_0) / 3
\]  

(2.4.12)

\[
K_0 = 1 - \sin \phi_d
\]  

(2.4.13)

2) Generation of pore water pressure

a) Before effective stress state point attains moving phase-transformation line,

\[
u_g = \sigma_0' (2 / \pi) \sin^{-1} \left( R_N^{1.14} \right)
\]  

(2.4.14)

\[
R_N = \Sigma (n_i - n_{i-1})
\]  

(2.4.15)

\[
n_i = 2 \left( \Delta \tau / (2 \sigma_0' C_1) \right)^{1.25}
\]  

(2.4.16)

\[
C_1 = (1/20)^{-0.25} R_{20}
\]  

(2.4.17)

(Negative dilatancy)
b-1) Loading phases once after effective stress state point hits moving phase-transformation line,

\[
\Delta u_e = -\theta_{\text{max}} \left( |\tau| / (\sigma_0' - \sigma_t) |- M_0 | / \left( M_f - M_0 \right) \right) |\Delta \tau|
\]

(2.4.18)

\[M_0 = 0.9 \, M_f\]  

(2.4.19)

\[M_f = 1 / \theta_{\text{max}}\]  

(2.4.20)

\[\theta_{\text{max}} = 10^{-1.27 + 1.66 \, Dr}\]  

(2.4.21)

\[
\sigma_t / \sigma_0' = 1 - (\Delta \gamma_\mu / (\Delta \gamma_\mu_{\text{cr}})) / (c_0 + (1 - c_0) / \Delta \gamma_\mu / (\Delta \gamma_\mu_{\text{cr}}))
\]  

(2.4.22)

(Positive dilatancy)

b-2) Unloading phases once after effective stress state point hits moving phase-transformation line,

\[
\Delta u_e = \sigma_{\text{cr}}' \left( 2 / \pi \right) \sin^{-1} \left( R_{\text{nc}}^{1/1.4} \right)
\]

(2.4.23)

\[R_{\text{nc}} = (\Delta \tau_c / \tau_{\text{cr}})^3\]  

(2.4.24)

\[
\sigma_{\text{cr}}' = \sigma' \left( 1 + 2 / \left( 3 + (\Sigma (\Delta \tau_c / \sigma_0'))^4 \right) \right)^{-1}
\]  

(2.4.25)

(Negative dilatancy)

where

- \(\gamma\): shear strain
- \(\gamma_0\) and \(\gamma_4\): shear strains occurring in zero and non-zero effective confining stress states, respectively (see Figure 2.4.2)
- \(e_0\) and \(e_{\text{min}}\): current and minimum void ratios, respectively
- \(\gamma_{\text{max}}\): maximum double amplitude shear strain induced by preceding cyclic undrained loadings
- \(\gamma_{\text{entry}}\): minimum double amplitude shear strain required to cause initial liquefaction
- \(\tau\): shear stress
- \(\Delta \tau\): incremental shear stress from latest reversal point of cyclic stress
- \(\gamma_r\): reference shear strain
- \(h_{\text{max}}\): maximum damping ratio
- \(\sigma'\) and \(\sigma_0'\): current and initial mean effective stress, respectively
- \(\sigma\) and \(\sigma_r\): mean total stress and overburden stress, respectively
- \(K_0\): coefficient of earth pressure at rest
- \(\phi_0\): internal friction angle
- \(R_N\): cyclic ratio defined by a ratio between number of applied stress cycles \(n_i\) and accumulative number of cycles required to cause initial liquefaction
- \(R_{20}\): shear stress ratio causing initial liquefaction at 20th cycle
- \(\sigma_r\): reference stress to determine moving critical stress state and phase-transformation lines (see Figure 2.4.3)
- \(Dr\): relative density
- \(\Delta \gamma_\mu\) and \((\Delta \gamma_\mu)_{\text{cr}}\): accountable shear strain increment and it’s critical value, respectively
- \(c_0\): a function of relative density to determine \(\sigma_r\)
- \(\sigma_{\text{cr}}', \tau_{\text{cr}}, \) and \(\Delta \tau_c\): critical mean effective, shear, and incremental shear stress to determine effective stress path in unloading phases (see Figure 2.4.3)
Figure 2.4.2. Two post-liquefaction shear strain components (Shamoto et al., 1997).

Figure 2.4.3. Formulation of the effective stress path related with negative and positive dilatancy (Shamoto et al., 1992; Zhang et al. 1997).
2.4.3 Determination of Model Parameters

Some values of parameters to determine the constitutive model can be predicted from past studies, results from laboratory tests, and so on. In the effective stress analysis using Eqs. (2.4.1)-(2.4.25), the values of initial shear modulus, \( G_0 \), reference shear strain, \( \gamma_{rl} \), maximum damping ratio, \( h_{\text{max}} \); internal friction angle, \( \phi_0 \), liquefaction resistance, \( R_{20} \), and relative density, \( D_r \), are required at each layer. \( G_0 \) can be computed by the following relation:

\[
G_0 = \rho V_S^2
\]  
\[ \text{(2.4.26)} \]

where \( V_S \) is shear wave velocity. The values of \( \rho \) and \( V_S \) can be estimated from density or S-wave logging data, soil type (e.g., AIJ, 1996), and/or SPT N-values. The values of \( \phi_0 \), \( R_{20} \), and \( D_r \) of sandy layers can be assumed from SPT N-values and fines contents (e.g., Hatanaka et al., 1998; Tokimatsu and Yoshimi, 1983). The values of \( \gamma_{rl} \) and \( h_{\text{max}} \) can be inferred from soil type and initial mean effective stress, i.e., mass density of soils and level of water table (e.g., AIJ, 1996). Thus, all the parameters required in the one-dimensional effective stress analysis used in this report can be determined from just the three factors; soil type, SPT N-value, and water table only, which can be easily derived from boring data.

2.4.4 Application to Existing Soil Deposit

In order to examine the effectiveness of the one-dimensional effective stress analysis expressed previously, strong motion data observed at Yume-no-shima in Tokyo and at East Kobe Bridge in Kobe, hereby called Sites YME and EKB, are used in the section. Soil profiles from boring and PS logging data at Sites YME and EKB are shown in Figures 2.4.4 and 2.4.5 (Ishihara et al., 1989; Public Works Research Institute, Ministry of Construction, 1995). Both sites were instrumented with arrays of down-hole and subsurface accelerograms. The down-hole accelerograms were installed at a depth of 10, 18.3, 89.5 m in Site YME and at a depth of 33 m in Site EKB (see Figures 2.4.4 and 2.4.5). At Sites YME and EKB, digitized strong motion records during the 1987 Chiba-ken Toho-oki earthquake and the 1995 Hyogo-ken Nambu earthquake are available (Ishihara et al., 1989; Public Works Research Institute, Ministry of Construction, 1995). Hereafter, the earthquake horizontal component with maximum peak ground acceleration, E-W and N-S components at Sites YME and EKB are used.

One-dimensional effective stress analyses by Eqs. (2.4.1)-(2.4.26) are then conducted for the soil profiles shown in Figures 2.4.4 and 2.4.5 using the down-hole records as within motions during either the 1987 or 1995 event at the sites, which acceleration time series are shown in the bottoms of Figures 2.4.6 and 2.4.7. Soil parameters required in the analyses are inferred from boring data, e.g., soil types, SPT N-values, and water tables, at the sites, and are shown in Tables 2.4.1 and 2.4.2. In the tables, \( n_e \) denotes number of elements in each layer, and the values of \( \gamma'_{rl} \) and \( m_{\text{v}} \) are those under the condition \( \sigma_0' = 1 \text{kgf/cm}^2 \).

Broken lines in Figures 2.4.6 and 2.4.7 are (a) acceleration and (b) velocity time series of the computed ground surface motions at the sites. Solid lines in the figures show those of the recorded motions at the sites. At each site, the computed and observed ground motions show fairly good agreements in both acceleration and velocity waveforms.

Figures 2.4.8 and 2.4.9 show variations with a depth of (a) maximum acceleration, (b) shear strains, and (c) excess pore water pressure ratios from the results of analyses for the sites. At both sites, the computed peak acceleration shows fairly good agreement with the observed one at a depth where seismometers were installed. In Figure 2.4.8, maximum excess pore water pressure ratio is almost zero at any depth, indicating that no soil layer
liquefied at Site YME during the 1987 Chiba-ken Toho-oki earthquake. At Site EKB, on the other hand, maximum excess pore water pressure ratios are unity with a depth range of 2-15 m, suggesting that sandy layers in the depths liquefied during the 1995 Hyogo-ken Nambu earthquake (comparing Figures 2.4.5 with 2.4.9).

![Soil profile at Site YME](image1)

**Figure 2.4.4.** Soil profile at Site YME (Ishi hara et al., 1989).

![Soil profile at Site EKB](image2)

**Figure 2.4.5.** Soil profile at Site EKB (Public Works Research Institute, 1995).

### Table 2.4.1. Soil parameters in 1-D effective stress analysis at Site YME.

<table>
<thead>
<tr>
<th>$H$ (m)</th>
<th>$n_e$</th>
<th>$\rho$ (t/m³)</th>
<th>$V_s$ (m/s)</th>
<th>$\phi_d$ (deg.)</th>
<th>$\gamma_{ri}$</th>
<th>$h_{max}$</th>
<th>$R_{20}$</th>
<th>$D_r$</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>$m_v$</th>
<th>$k_w$ (cm²/kgf) (cm/s)</th>
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<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
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### Table 2.4.2. Soil parameters in 1-D effective stress analysis at Site EKB.

<table>
<thead>
<tr>
<th>$H$ (m)</th>
<th>$n_e$</th>
<th>$\rho$ (t/m³)</th>
<th>$V_s$ (m/s)</th>
<th>$\phi_d$ (deg.)</th>
<th>$\gamma_{ri}$</th>
<th>$h_{max}$</th>
<th>$R_{20}$</th>
<th>$D_r$</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
<th>$m_v$</th>
<th>$k_w$ (cm²/kgf) (cm/s)</th>
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</table>
Figure 2.4.6. Observed and computed (a) acceleration and (b) velocity waveforms at Site YME (E-W component).

Figure 2.4.7. Observed and computed (a) acceleration and (b) velocity waveforms at Site EKB (N-S component).
Figure 2.4.8. Variation with depth of (a) maximum accelerations, (b) shear strains, and (c) excess pore water pressure ratios from effective stress analyses for Site YME (E-W component).

Figure 2.4.9. Variation with depth of (a) maximum accelerations, (b) shear strains, and (c) excess pore water pressure ratios from effective stress analyses for Site EKB (N-S component).
Figure 2.4.10. (a) Stress-strain relation, (b) effective stress path, and (c) excess pore water pressure ratio in a depth range of 5 – 6.75 m from effective stress analyses for Site EKB (N-S component).

Figures 2.4.10(a)-(c) show shear stress-strain relation, effective stress path, and excess pore water pressure ratio at a depth of 5-6.75 m in the liquefied layers estimated from the analysis for Site EKB. Excess pore water pressure ratio starts increasing at 15 s and reaches unity at 21-22 s after the initiation of shaking (comparing Figures 2.4.7 with 2.4.10(c)), and both stress-strain relation and effective stress path show unique characteristics of liquefied soil. In Figure 2.4.9(a), the computed and observed peak acceleration is deamplified in the liquefied layers by a factor of 0.7, while the acceleration of Site YME is amplified in subsurface soil by the factor of 2 (see Figure 2.4.8(a)), thus, it presents that the deamplification trend at Site EKB could be caused by the soil liquefaction. These results are consistent with the characteristics of recorded acceleration waveforms at Sites YME and EKB during the 1987 and 1995 events, and field observation, e.g., existence of sand boils (e.g., Hamada et al., 1995).

The above findings confirm that the one-dimensional effective stress analysis employed in the report, proposed by Shamoto et al. (1992, 1997) and Zhang et al. (1997), can be a reliable means for evaluating or predicting ground responses during earthquakes considering soil liquefaction.

References


2.5 Three-Dimensional Soil Liquefaction and Foundation Response Analysis

The effective stress analysis technologies may be one of the promising tools that can predict the behavior of soil liquefaction. The author has examined the 2/3-dimensional dynamic response FE analysis that incorporates a cyclic elasto-plastic model for sand, cyclic elasto-viscoplastic model for clay and Biot's two phase mixture theory. The numerical method has been applied to the vertical array records in the liquefied ground, the embankments on the liquefied ground (Matsuo et al, 2000), the damaged quay walls in the reclaimed island (Uzuoka and Mihara, 1998), the damaged buildings and the pile foundation on the liquefied ground (Uzuoka et al, 2001). The numerical method quantitatively reproduced the measured records and damage extent in these case studies. Firstly, the brief description of the governing equations and constitutive models for sand/clay is given below. The models for the structural members such as piles and buildings were described in the case studies in section 3.4. Finally, the verification example for vertical array records observed in Port Island during 1995 Hyogo-ken Nanbu earthquake.

2.5.1 Governing Equations

In this study, the governing equations of for the coupling problems between soil skeleton and pore water were obtained with the two phase mixture theory (Biot, 1962). Using a u-p (displacement of the solid phase - pore water pressure) formulation (Zienkiewicz and Bettess, 1982), a simple and practical numerical method for the two-dimensional liquefaction analysis was formulated. The finite element method (FEM) has been usually used for the spatial discretization of the governing equations. In this study, however, the finite element method (FEM) was used for the spatial discretization of the equilibrium equation, while the finite difference method (FDM) was used for the spatial discretization of the pore water pressure in the continuity equation (Akai and Tamura, 1978). The Newmark's β method was used for time integration scheme. The accuracy of the proposed numerical method was verified by Oka et al. (1994) through a comparison of numerical results and analytical solutions for transient response of saturated porous solids. As details of this method were given in Oka et al. (1994), only a brief description of the method is given below. The governing equations are formulated by the following assumptions:

1) The infinitesimal strain;
2) The smooth distribution of porosity in the soil;
3) The small relative acceleration of the fluid phase to that of the solid phase compared with the acceleration of the solid phase;
4) Incompressible grain particles in the soil.

The equilibrium equation for the mixture is derived as follows:

\[ \rho \ddot{u}_j = \sigma_{ij,j} + \rho \ddot{b}_j \]  \hspace{1cm} (2.5.1)

where \( \rho \) is the overall density, \( \ddot{u}_j \) is the acceleration of the solid, \( \sigma_{ij} \) is the total stress tensor and \( \ddot{b}_j \) is the body force. The continuity equation is derived as follows:

\[ \rho' \ddot{e}_{\nu} - p' - \frac{\gamma_s}{k} \ddot{e}_{\nu} + \frac{m_y}{kK'} \ddot{p} = 0 \]  \hspace{1cm} (2.5.2)

where \( \rho' \) is the density of the fluid, \( p \) is the pore water pressure, \( \gamma_s \) is the unit weight of the fluid, \( k \) is the coefficient of permeability, \( \ddot{e}_{\nu} \) is the volumetric strain of the solid, \( n \) is porosity and \( K' \) is the bulk modulus of the fluid.

2.5.2 Constitutive Models for Sand

The constitutive equation used for sand is a cyclic elasto-plastic model (Oka et al., 1999). The performance of the constitutive model was verified by Oka et al. (1999) and Matsuo et al
The model succeeded in reproducing the experimental results well under various stress conditions, such as isotropic and anisotropic consolidated conditions, with and without the initial shear stress conditions, principal stress axis rotation, etc. The elasto-plastic model for sand is briefly described. This constitutive model is formulated on the following assumptions:

1) The infinitesimal strain is used;
2) The elasto-plastic theory;
3) The non-associated flow rule;
4) The concept of the overconsolidated boundary surface;
5) The non-linear kinematic hardening rule.

**Overconsolidated boundary surface**

The boundary surface between the normally consolidated region \((f_n \geq 0)\) and the overconsolidated region \((f_n < 0)\) is given by

\[
f_n = \frac{1}{2} M_n \ln \left( \frac{\sigma_{o}'}{\sigma_{o0}} \right) = 0 \tag{2.5.3}
\]

\[
\bar{\eta}_{o0} = \left\{ \left( \eta_y - \eta_{y(0)} \right) \left( \eta_y - \eta_{y(0)} \right) \right\}^{1/2} \tag{2.5.4}
\]

\[
\eta_y = \frac{s_y}{\sigma_m'} \tag{2.5.5}
\]

where \(\sigma_{o}'\) is the mean effective stress, \(s_y\) is the deviatoric stress tensor, \(M_n\) is the value of the stress ratio when the maximum compression of the material takes place (called phase transformation stress ratio), and \(\eta_{y(0)}\) denotes the value of \(\eta_y\) at the end of anisotropic consolidation. Generally \(\sigma_{o0}'\) is given by the following rule in the overconsolidated region:

\[
\sigma_{o0}' = \sigma_{o0}' + \exp \left\{ \left( 1 + e_0 \right) / (\lambda - \kappa) v^p \right\} \tag{2.5.6}
\]

where \(\sigma_{o0}'\) is the initial value of \(\sigma_{o0}'\), \(e_0\) is the initial void ratio, \(\lambda\) is the compression index, \(\kappa\) is the swelling index and \(v^p\) is the volumetric plastic strain.

**Yield surface**

The yield function is given by

\[
f = \left( \eta_y - \chi_y \right) \left( \eta_y - \chi_y \right) - Rd = 0 \tag{2.5.7}
\]

where \(Rd\) is the numerical parameter which defines the elastic region, and \(\chi_y\) is the kinematic hardening tensor. By introducing the non-linearity of the kinematic hardening, \(\chi_y\) can be written as

\[
d\chi_y = B \left( M_f d\varepsilon_{pl}' - \chi_y \varepsilon^p \right) \tag{2.5.8}
\]

in which \(B\) is the hardening parameter, \(M_f\) is failure stress ratio, and \(d\varepsilon_{pl}'\) is the increment of the plastic deviatoric strain. The key to this simple model is the second term on the right hand side, which is proportional to \(\chi_y\) and the second invariant of the increment of the plastic deviatoric strain,

\[
d\gamma^p = \left( d\varepsilon_{pl}' d\varepsilon_{pl}' \right)^{1/2} \tag{2.5.9}
\]

**Plastic Potential Function**

Based on the relationship between the stress ratio and the increment of plastic strain, the plastic potential is expressed as follows:

\[
g = \left\{ \left( \eta_y - X_y \right) \left( \eta_y - X_y \right) \right\}^{1/2} + M \ln \left( \frac{\sigma_{o0}'}{\sigma_{o0}'} \right) = 0 \tag{2.5.10}
\]

where \(\sigma_{o0}'\) is a constant and \(M\) is denoted by
\[
\dot{M} = \begin{cases} 
M_n & f_b \geq 0 \text{(Normally consolidated region)} \\
\eta & f_b < 0 \text{(Overconsolidated region)} 
\end{cases}
\]

where the current stress ratio \( \eta \) is defined as
\[
\eta = (\eta_0 \eta_0^{-1/2})^{1/3}
\]
and
\[
\sigma_{\text{mc}}' = \sigma_{\text{mc}} \exp(\eta_0 / M_n)
\]
in which the initial stress ratio, \( \eta_0 \) is defined as
\[
\eta_0 = \left(\eta_{00} \eta_{00}^{-1/2}\right)^{1/2}
\]

Taking an elastic component of strain increment into account, a constitutive equation is expressed as
\[
d\varepsilon_y = \frac{1}{2G} d\sigma_{yy} + \frac{\kappa}{3(1+\epsilon_y)} \sigma_{\text{mc}}' \delta_y + d\varepsilon_y
\]

Furthermore, this constitutive model was modified in order to describe the sand behavior under various stress conditions (Tateishi et al., 1995) as follows.

**Generalized flow rule**

In order to adjust the slope of the liquefaction strength curve, the stress-dilatancy characteristics is expressed by using the generalized flow rule as
\[
\frac{d\varepsilon}{d\gamma} = D \left( \dot{M} - \eta_0 \right)
\]
where
\[
D = D_0 \left( \frac{\dot{M}}{M_n} \right)^n
\]
\[
\eta_0 = \frac{\eta_{00} - \chi_{00}}{\left(\eta_{00} - \chi_{00}\right) - \chi_{00}}
\]

\( D \) is so-called coefficient of dilatancy, and \( D_0 \) and \( n \) are the dilatancy parameters. When the value of \( n \) is larger than one, the coefficient of dilatancy changes drastically due to the change in the stress ratio in the overconsolidated region.

**Plastic strain dependence of the plastic shear modulus**

In order to reproduce the continuous increase in the shear strain after the stress path passes the phase transformation line, the strain dependence of the plastic shear modulus is taken into account. In this study, the hardening parameter related to the plastic shear modulus is expressed by
\[
B = \frac{B_0}{1 + \gamma^p / \gamma^p}
\]
where \( B_0 \) is the initial value of \( B \) and \( \gamma^p \) is the reference strain parameter. The lower limit of \( B \) is also parameter \( B_1 \), which is adjusted to describe the strain accumulation. The experiential value of \( B_1 \) is about 1/100 of \( B_0 \). The same equation is also applied to modeling the reduction in the elastic modulus in which \( \gamma^p \) is used as a reference value in place of \( \gamma^p \).

**Disappearance of initial anisotropy**

In order to reproduce the disappearance of initial anisotropy during cyclic loading in soils, the overconsolidated boundary surface is modified. The intersection of the
overconsolidated boundary surface and the effective mean stress axis as shown in equation (2.5.13) is expressed by

\[ \sigma_{nc}' = \sigma_{nc}' \exp \left\{ \exp \left( -C_d \gamma^p \right) / M_n \right\} \]  \hspace{1cm} (2.5.19)

where \( C_d \) is the constant which controls the rate of disappearance of anisotropy.

### 2.4.3 Constitutive Models for Clay

The constitutive equation used for clay is a cyclic elasto-viscoplastic model (Oka, 1992). The model is based on almost the same assumption of the elasto-plastic model for sand, except the flow rule of the model that is different from the model for sand. The model adopts the viscoplastic flow rule that can take the rate dependency of cohesive soil into account. As details of this model are given in Oka (1992), a brief description of the model is given below. This constitutive model is formulated on the following assumptions:

1) The infinitesimal strain is used;
2) The over-stress type of viscoplasticity;
3) The non-associated flow rule;
4) The concept of the overconsolidated boundary surface;
5) The non-linear kinematic hardening rule.

The overconsolidation boundary surface, the yield surface and the plastic potential function are almost same as those of elasto-plastic model for sand except for using viscoplastic strain in replace of plastic strain. For the overconsolidation boundary surface, \( \sigma'_{nc} \) is given by the following rule in replace of equation (2.5.6):

\[ \sigma'_{nc} = \sigma_{nc}' \exp \left\{ \left( 1 + e / (\lambda - \kappa) \right) \nu^p \right\} \]  \hspace{1cm} (2.5.20)

where \( \nu^p \) is the volumetric viscoplastic strain. For the yield function, the kinematic hardening tensor \( \chi_\nu \) is given by the following rule in replace of equation (2.4.8):

\[ d\chi_\nu = B \left( M_\nu d\gamma^p - \chi_\nu d\gamma^p \right) \]  \hspace{1cm} (2.5.21)

where \( d\gamma^p \) is the increment of the viscoplastic deviatoric strain and \( d\gamma^p \) is the second invariant of the viscoplastic deviatoric strain increment tensor, namely,

\[ d\gamma^p = \sqrt{d\sigma^p \cdot d\sigma^p} \]  \hspace{1cm} (2.5.22)

In the viscoplastic model, the over-stress type flow rule is used based on Perzyna’s viscoplastic theory. The viscoplastic strain rate tensor \( \dot{\varepsilon}_{ij}^p \) is assumed to be given by

\[ \dot{\varepsilon}_{ij}^p = \left\{ \Phi_{ij\mu} \left( F \right) \Phi_{2ij\mu} \left( \xi \right) \right\} \frac{\partial f}{\partial \sigma_{ij}^p} \]  \hspace{1cm} (2.5.23)

where

\[ \left\{ \Phi_{ij\mu} \left( F \right) \right\} = \begin{cases} \Phi_{ij\mu} \left( F \right) & F > 0 \\ 0 & F \leq 0 \end{cases} \]  \hspace{1cm} (2.5.24)

\[ F = f = \left( \left( \eta_{ij} - \chi_{ij} \right) \left( \eta_{ij} - \chi_{ij} \right) \right)^{1/2} - Rd \]  \hspace{1cm} (2.5.25)

where \( \Phi_{ij\mu} \left( F \right) \) is the first material function and \( \Phi_{2ij\mu} \left( \xi \right) \) is the second material function. The first material function is the functional of \( F \) in which \( F=0 \) denotes the yield function, and it shows rate sensitivity. The first material function is assumed to be the fourth order isotropic tensor function. The complete shape of the first material function is experimentally determined.

\[ \Phi_{ij\mu} \left( F \right) = C_{ij\mu} \Phi' \left( F \right) \]  \hspace{1cm} (2.5.26)

where

\[ C_{ij\mu} = a \delta_{ij} \delta_{\mu} + b \left( \delta_{ij} \delta_{\mu} + \delta_{i\mu} \delta_{j\mu} \right) \]  \hspace{1cm} (2.5.27)
\[ \frac{\Phi'(F)}{\sigma'_w} = \exp \left[ m'_b \left( \frac{(\eta_y - \chi_y)(\eta'_y - \chi'_y)}{(\eta_y - \chi_y)(\eta'_y - \chi'_y)} \right)^{1/2} \right] \]  

(2.5.28)

where \( a \) and \( b \) are material constants and \( m'_b \) is the viscoplastic parameter. For the convenience of the numerical application, the equations are set as follows:

\[ C_{\mu} = 2b, \quad C_{\alpha} = 3a + 2b \]  

(2.5.29)

The second material function was introduced in order to take the rate dependency into account at the failure state. In this study, it was assumed to be 1.0 for simplicity. Taking an elastic component of strain rate into account, the total strain rate \( \dot{\varepsilon}_y \) is expressed as

\[ \dot{\varepsilon}_y = \frac{1}{2G} \dot{\gamma} + \frac{\kappa}{3(1+e_y)} \sigma'_w \delta_y + \dot{\varepsilon}_y^{\text{el}} \]  

(2.5.30)
2.5.4 Verification Example

The liquefaction analysis method mentioned above was applied to reproduce the vertical array records observed in Port Island during 1995 Hyogo-ken Nanbu earthquake.

**Vertical Array Records in Port Island**

The borehole strong motion records located in Port Island as shown in Figure 2.5.1 were measured by the Development Bureau of Kobe City during 1995 Hyogo-ken Nanbu earthquake. The soil profile at the borehole is shown in Figure 2.5.2. The three acceleration components in NS, EW and UD directions were recorded at four levels of GL-83m, GL-32m, GL-16m and GL-0m. The Development Bureau of Kobe City (1995) and Gifu University with EDM, RIKEN conducted the in-situ soil investigations in 1999. The N values by standard penetration test and shear velocity by PS logging in 1995 and 1999 are shown in Figure 2.5.2. Figure 2.5.3 and Figure 2.5.4 show the time histories of acceleration and velocity respectively in NS and EW directions at four levels. These observed acceleration records were corrected of the orientation error by Sugito et al (1996). The peak acceleration at the ground surface was smaller than that at deeper ground, and its predominant period became longer due to liquefaction of reclaimed soil layer.

![Figure 2.5.1. Location of vertical array station at Port Island](image-url)
**Numerical Conditions**

The 3-dimensional soil column at the borehole site was modeled by the 31 rectangular solid finite elements. The bottom of the model was fixed, and the four nodes at the same depth were constrained to yield the same displacement for all components. The underground water surface was drained boundary, and the other boundaries were impermeable. The recorded accelerations for three components at GL-83m were input from the bottom of the FE model. Rayleigh damping proportional to initial stiffness, which was determined by assuming that the damping factor is 1%, was used in order to obtain numerical stability in the high frequency domain. The increment for time integration was 0.002 seconds.

![Soil profile at vertical array station](image)

**Figure 2.5.2.** Soil profile at vertical array station
The model parameters for all soil layers as shown in Figure 2.5.2 are tabulated in Table 2.5.1. The cyclic elasto-plastic model for sand (EP) in 2.5.2 was applied to reclaimed B layer and alluvial sand As layer. The B layer was divided three layers of B1, B2 and B3. The B1 layer is the upper part on the underground water table. The shear wave velocity of the B2 layer was smaller than that of the B3 layer. The cyclic elasto-viscousplastic model for clay (EVP) in 2.5.3 was applied to alluvial clay Ac layer. The other layers of diluvial sand Ds and diluvial clay Dc were modeled by Ramberg-Osgood model (RO), because the large shear strain might not occur in these hard layers. The model parameters were determined based on the in-situ tests and the laboratory tests with undisturbed soil samples. The following parameters, \( \varepsilon_0, k, \lambda, \kappa, M_m \) and \( M_f \) for B layer were determined by physical property tests and drained monotonic shear tests using undisturbed samples (Suzuki et al, 1997). The remaining parameters for B layer in Table 2.5.1 were determined by adjusting technique in order to describe the liquefaction strength curve (Suzuki et al, 1997). The parameters for Ac layer were determined by physical property tests and undrained monotonic shear tests with several strain rates using undisturbed samples, and were adjusted in order to describe the dynamic strength curve. The parameters for Dc and Ds layers were determined in order to describe the dynamic deformation characteristics, which were the relations between shear modulus, damping and the shear strain amplitude (Kobe City, 1995).

<table>
<thead>
<tr>
<th>Name of soil profile</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
<th>Ac</th>
<th>As</th>
<th>Ds</th>
<th>Dc</th>
<th>Ds</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density ( \rho ) (t/m³)</td>
<td>2.10</td>
<td>2.10</td>
<td>2.10</td>
<td>1.67</td>
<td>2.00</td>
<td>2.00</td>
<td>1.75</td>
<td>2.00</td>
</tr>
<tr>
<td>Initial void ratio ( \varepsilon_0 )</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>1.41</td>
<td>0.5</td>
<td>0.5</td>
<td>1.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Coefficient of permeability ( k ) (m/s)</td>
<td>3.0E-5</td>
<td>3.0E-5</td>
<td>3.0E-5</td>
<td>2.0E-6</td>
<td>1.0E-5</td>
<td>2.0E-5</td>
<td>1.0E-5</td>
<td>2.0E-5</td>
</tr>
<tr>
<td>Compression index ( \lambda )</td>
<td>1.0E-2</td>
<td>1.0E-2</td>
<td>1.0E-2</td>
<td>3.3E-1</td>
<td>1.0E-2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Swelling index ( \kappa )</td>
<td>1.0E-3</td>
<td>1.0E-3</td>
<td>1.0E-3</td>
<td>4.3E-2</td>
<td>1.0E-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial shear velocity ( V_s ) (m/s)</td>
<td>140</td>
<td>140</td>
<td>230</td>
<td>180</td>
<td>230</td>
<td>330</td>
<td>280</td>
<td>450</td>
</tr>
<tr>
<td>Initial shear modulus ratio ( G_0/\sigma_{eq}^0 )</td>
<td>2002</td>
<td>730</td>
<td>1019</td>
<td>326</td>
<td>516</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure stress ratio ( M_f )</td>
<td>1.34</td>
<td>1.34</td>
<td>1.34</td>
<td>1.23</td>
<td>1.26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phase transformation stress ratio ( M_m )</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>1.03</td>
<td>0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardening parameter ( B_0 )</td>
<td>6000</td>
<td>1500</td>
<td>2100</td>
<td>55</td>
<td>5000</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For elasto-plastic model (EP)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>B1</th>
<th>150</th>
<th>140</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference strain parameter ( \gamma'_{c} )</td>
<td>-</td>
<td>0.005</td>
<td>0.004</td>
<td>0.01</td>
</tr>
<tr>
<td>Reference strain parameter ( \gamma''_{c} )</td>
<td>-</td>
<td>0.005</td>
<td>0.004</td>
<td>0.1</td>
</tr>
<tr>
<td>Dilatancy parameter ( D_0 )</td>
<td>-</td>
<td>1.0</td>
<td>1.2</td>
<td>1.0</td>
</tr>
<tr>
<td>Dilatancy parameter ( n )</td>
<td>-</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

For elasto-viscousplastic model (EVP)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscoplastic parameter ( m_0^\theta )</td>
<td>14</td>
</tr>
<tr>
<td>Viscoplastic parameter ( C_m' )</td>
<td>5.5E-6</td>
</tr>
<tr>
<td>Viscoplastic parameter ( C_m'' )</td>
<td>7.8E-7</td>
</tr>
</tbody>
</table>

For Ramberg-Osgood Model (RO)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial shear modulus ratio ( G_0/\sigma_{eq}^{0.5} )</td>
<td>12414</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.35</td>
</tr>
<tr>
<td>Internal frictional angle</td>
<td>degree</td>
</tr>
<tr>
<td>Maximum damping ratio ( h_{max} )</td>
<td>0.23</td>
</tr>
<tr>
<td>Reference strain ( \gamma_0 )</td>
<td>2.0E-03</td>
</tr>
</tbody>
</table>
Numerical results

The time histories of computed acceleration and velocity respectively in NS and EW directions at four levels are shown in Figure 2.5.3 and Figure 2.5.4. The velocity and displacement are relative values to input motion at the bottom of GL-83m. The computed accelerations at GL-16m and GL-32m agreed with the recorded ones, except that the simulation underestimated the peak spiky acceleration. The computed accelerations at GL-0m also agreed with the recorded ones, except that the slight phase difference between recorded and computed waves after about 5 seconds were shown. The computed velocities at all level also agreed with the recorded ones.

Figure 2.5.5 shows the relative effective stress ratio (R.E.S.R.) in B1, B2, Ac and As layers. The R.E.S.R. was calculated by \((1-\sigma'_m/\sigma''_m)\), and becomes 1.0 when the mean effective stress \(\sigma'_m\) becomes almost zero, which means liquefaction. The complete liquefaction in B and As layers occurred at about 10 and 20 seconds respectively. The decrease of mean effective stress in Ac layer was very small. Figure 2.5.6 shows the recorded and computed particle orbits of horizontal acceleration, velocity and displacement at the ground surface. The computed velocity agreed with the recorded one regarding the maximum values and its directions. The numerical method could reproduce the 3-dimensional seismic behavior of sand and clay quantitatively.

Acknowledgements

The soil investigation was conducted in the co-research between EDM, RIKEN and Gifu University. The authors wish to thank Prof. Atsushi Yashima of Gifu University, Associate Prof. Feng Zhang for their worthy advice to the numerical analyses. The verification example analysis for the borehole records in Port Island was carried out with Mr. Takeuchi and Mr. Furuta in graduate school in Gifu University.
Figure 2.5.3. Time histories of recorded and computed acceleration

Figure 2.5.4. Time histories of recorded and computed velocity
Figure 2.5.5. Time histories of computed relative effective stress ratio

Figure 2.5.6. Recorded and computed particle orbits at the ground surface

References


2.6 Models and Methods for Three-Dimensional Building Structural Analysis

Information coming from recent earthquakes revealed extensive damages to building structures even to some earthquake-resistant designed modern structures for intensive ground motion and excessive displacement. It alerts us again the necessity of comprehensive understanding the damage mechanism and structural behavior in nonlinear ultimate stage. For decades researchers and academia have been investigating on numerical methods to simulate the structural earthquake responses. Due to the limitation of computer capacity in early time, simplified structural model, such as single or multiple mass-point model (stick model) that has concentrated mass at each floor level or into a single mass point were used, and are still in use now in investigating structural dynamic responses for the simplicity and low computation cost. By the simplified model, general seismic behavior of structures in terms of lateral displacement and story sidesway resistance can be computed and used to measure the extent of response and safety. However, the simple model may have less reliability and the results obtained are very limited (no results on the responses of individual structural members).

Promoted by the advanced computer technology, simulating the building earthquake responses in sophisticated structural model based on nonlinear force-displacement relations of individual structural members has become realizable and practicable. In addition, various nonlinear member models and a large number of hysteresis models [1]-[5] have been proposed and investigated based on experimental studies of loading destructive test of structural members and their assemblies even real scale structures. Some simple and effective models such as the multi-spring model, fiber model have been developed and calibrated to represent the structural members nonlinear behavior including the interactions among multi-axial loads.

In this section, modeling building structure and members, and numerical methods for three-dimensional nonlinear structural analysis are presented and the reliability of the models and methods are examined and verified, and their application in simulating the building earthquake responses are given in the relevant sections in Chapter 3 and Chapter 5 in this report.

2.6.1 General assumptions and treatments in modeling building structure

The structure is idealized as a finite number of rigid nodes, connected by a finite number of structural members (beam, column, shear wall, and spring). Among the nodes and elements rigid or semi-rigid connection and pin or roller connection are treated. Up to six displacement degrees of freedom (DOFs) are considered at a structural node. That is three translational components and three rotation components in three orthogonal directions. Fig. 2-6-1 illustrates the node DOFs and the positive direction of the displacement and relevant force components. The DOFs at any structural nodes can be treated independent, eliminated (prohibited), prescribed (given displacement) or in coupled relation with others, according to the structural constitution and boundary conditions. Dealing with reinforced concrete (RC) frame structure, the torsional stiffness of structural members is comparatively small and is usually neglected in analysis, and the horizontal beam members are treated as uniaxial bending element in frame plane. Then the DOF of rotation around the vertical Z-axis is to be eliminated, and maximum five DOFs, three translations together with two rotations in the vertical X-Z and Y-Z planes, are considered at structural nodes.

In treating the floor slab of building structure extreme assumption is applied. That is, the floor slab is treated as rigid diaphragm having infinite stiffness in the floor level horizontal plane and having three rigid movements, two horizontal translations and a rotation in the
horizontal plane. The rigid movements govern the lateral displacements of those structural nodes on the floor level and associated with the rigid diaphragm through floor slab. This is usually called rigid floor assumption often used to incorporate the structural torsional oscillation in building structural analysis. Multiple rigid diaphragms in the same floor level are treated for the case of discontinued or multi-segmented floor slabs, as shown in Fig. 2-6-2. The effect of floor slab on the vertical and rotational displacements of structural node is approximately considered by adding the slab stiffness to the horizontal beam element on the floor level.

The mass and inertia load is concentrated at structural nodes corresponding to the node translational displacements. The mass corresponding to node rotational displacements is zero and the inertia moment at structural node is ignored. When rigid diaphragm is used for the floor slab, the lateral inertia load and inertia moment at the gravity center point of a rigid diaphragm are the resultant load of the lateral inertia load at the structural nodes associated with the rigid diaphragm.

Structural members are treated as mass-less straight linear element along with its axial line and connected to structural node at the line element ends. If element axial line is not coincident with node axial line, then offset from the element-end and the structural node is treated as rigid link, as shown in Fig. 2-6-3. In RC frame structure, element-end may also have rigid zone according to the beam-column joint size. The length of element-end rigid zone is adjusted to approximately allow for the deformation of structural joint.

Initial force to structural members subjected to initial constant load (structural self-weight and service load) is taken into account before analysis for dynamic load or static increasing load.

The nonlinear analysis considers the material nonlinearity represented by nonlinear force-displacement relations in individual structural members. The geometrical nonlinearity, however, is taken into account by approximate method that treats the additional lateral force and overturning moment induced by the gravity load over lateral displacement (P-Δ effect). The P-Δ effect is counted at individual structural members allowing for the change of gravity load in members, as shown in Fig. 2-6-4.

The boundary condition at building base (ground floor level or basement level) is treated as fixed, pin-roller supported or spring supported. The support spring may represent the foundation effect (the soil-structure interaction), and the base of building can be treated as a rigid body to incorporate the foundation swaying and rocking (Fig. 2-6-5).

**Fig. 2-6-1 Nodal Displacement Degrees of Freedom**

TZ \((F_z,d_z)\)

TY \((F_y,d_y)\)

RZ \((M_z,\theta_z)\)

RY \((M_y,\theta_y)\)

RX \((M_x,\theta_x)\)

TX \((F_x,d_x)\)

The direction of positive forces and displacement.
(Rigid diaphragm represented by a 2-D supernode at its gravity center point)

Fig. 2-6-2 Rigid Diaphragm Representing Floor Slab and Multiple Rigid Diaphragms for Segmented Floor Slabs

\[
\begin{bmatrix}
F_X \\
F_Y \\
F_Z \\
M_X \\
M_Y \\
M_Z
\end{bmatrix}
= 
\begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & -1 & 0 & 0 & 0 \\
0 & -R_Z & R_Y & 1 & 0 & 0 \\
R_Z & 0 & -R_X & 0 & 1 & 0 \\
-R_Y & R_X & 0 & 0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
F'_X \\
F'_Y \\
F'_Z \\
M'_X \\
M'_Y \\
M'_Z
\end{bmatrix}
\]

Fig. 2-6-3 Treatment of Element Offset

\[
\Delta Q_y' = \frac{N_i \cdot \Delta y}{h}
\]

Fig. 2-6-4 P-Δ Effect and Equivalent Lateral Force in Structural Member
2.6.2 Modeling structural members

In building structural analysis, beam, column and shear wall are the main structural members to be modeled and idealized using various member analysis models.

The beam member is treated as uniaxial flexural member in the horizontal floor level. Its stiffness contribution is considered against the uniaxial flexural ration and shear deformation in vertical frame plane. The beam member does or does not have axial tension/compression and axial torsion. When rigid diaphragm is used to represent floor slab, the beam member is treated with no axial tension and compression. For simplification, in nonlinear analysis the beam member is idealized using one-component rotational spring, shear spring, and axial spring as shown in Fig. 2-6-6. Each spring represents one-component nonlinear force-displacement relation of the beam member.

The column member is considered subjected to biaxial bending and shear with constant or variable axial load. Same with beam one-component spring can be used to represent the column flexural rotation, shear and axial deformation and results in no interaction among the bi-directional bending and shear and axial load. However, for RC columns the axial load has significant effect on its bending capacity as well as the interaction between its bending moments in two directions. To include the axial load effect on bending capacity and the interaction between bi-directional bendings, multi-spring model (MS model) is used to idealize the column member in nonlinear analysis (Fig. 2-6-7). Details of the MS model are described in next section 2.6.3.

The shear wall member is treated as special line element. It is associated with four structural nodes at wall corners. It has stiffness contribution against the flexural and shear in the wall plane only and is treated as uniaxial bending and shear member. Plane section remained plane is assumed for wall member section against bending. It results in the displacement relations between the wall and the corner nodes as shown in Fig. 2-6-8. Shear wall may have side columns and they may have to be modeled together according to the integrity of the wall and side columns. Fig. 2-6-9 shows the wall together with side columns idealized by fiber model to allow for the axial load effect on bending capacity and to ensure the wall and side columns working together. The shear deformation of wall and side columns is treated separately by using one-component shear spring.
The displacement relations among multi-span shear walls and relevant structural nodes are subjected to the integrity of the walls as shown in Fig. 2-6-10. For well-integrated walls, displacement compatible conditions are applied among the walls and the structural nodes. That is, rigid beam is placed through all adjacent walls at the base and topsides. Then the multi-span walls can be idealized as a single element or multiple elements according to the perforation (Fig. 2-6-11).

Using the member models of beam, column and shear wall to model the space frame structure is illustrated in Fig. 2-6-12.

![Diagram of structural elements](image)

**Fig. 2-6-6 One-Component Uniaxial Spring Models**

![Diagram of column idealization](image)

**Fig. 2-6-7 Column Idealized by Multi-Spring Model**
Fig. 2-6-8 Displacement Relations Between Structural Node and Shear Wall

\[ d_{x3} = d_{x4} = d_t, \]
\[ \theta_{y3} = \theta_{y4} = \frac{d_{x3} - d_{x4}}{L_t}, \]
\[ d_{x1} = d_{x2} = d_b, \]
\[ \theta_{y1} = \theta_{y2} = \frac{d_{x1} - d_{x2}}{L_b}. \]

Fig. 2-6-9 Shear Wall Including the Side-Column Idealized by Fiber Model

(a) Discretized Multi-Span Walls

(b) Integrated Multi-Span Walls

Fig. 2-6-10 Displacement Options of Multi-Span Shear Walls

(a) Multi-elements for perforated wall

(b) One element for multi-span wall

Fig. 2-6-11 Modeling of Multi-Span Shear Walls with and without Perforation
2.6.3 Multi-spring model

Multi-spring model (MS model) represents force-displacement relations. It is used to simulate the behavior of flexural rotation coupled with axial deformation of column member, and also used to represent the column biaxial shear. A column element idealized using MS model has a line element and two flexural and axial MS element at the column-end (Fig. 2-6-7). There are then two internal nodes between the line element and the MS elements. The MS element is considered to be zero length in establishing the column force-displacement relation. The line element is elastic in flexural behavior and axial deformation. The line element may include inelastic shear deformation represented by the shear MS element.

Flexural and axial MS element

In the flexural and axial MS element the column section is discretized into several axial springs (Fig. 2-6-7). Each spring carries axial force and resists axial deformation. Invoking the assumption of plane section the spring deformation is determined from the flexural rotations and axial deformation of the MS element. The spring represents the material stiffness of the column section. The number of the spring depends on material properties, section size and reinforcing bar arrangement. For RC column member, each single steel bar is replaced by a steel spring at the bar center point, and the concrete area is properly discretized into several portions and represented by concrete spring at the center point of each portion. The number of springs in the MS element may affect the preciseness in simulating the column flexural and axial behavior. Calibration and reliability examination is given in the reference [3].

The flexural and axial MS element carries moment and axial force but no shear force, so it has to be considered with zero length to have equal magnitude of moments at its both sides. Then including the element at column ends may result in under-estimation of the column initial stiffness, unless using rigid-plastic spring. However, it is not reality to consider the spring in rigid-plastic behavior. Therefore, the flexibility of a small portion, namely called as "plastic zone," of the column is assigned to the spring as its initial flexibility (Fig. 2-6-13).
The spring initial stiffness, yielding strength and displacement are calculated as following by assuming uniform strain over the plastic zone:

\[
K_0^i = \frac{E_i A_i}{p_z} \quad \text{(of i-th spring)}
\]

\[
f_c = \sigma_c A_i, \quad d_c = \varepsilon_c \cdot p_z \quad \text{(for concrete)}
\]

\[
f_{sy} = \sigma_{sy} A_i, \quad d_{sy} = \varepsilon_{sy} \cdot p_z \quad \text{(for steel)}
\]

Where \(K_0^i\) is initial stiffness of i-spring, \(E_i\) is the material young's modulus, \(A_i\) is the spring governed area, and \(p_z\) is the length of assumed plastic zone. \(\sigma_c, \varepsilon_c\) are the concrete material compression strength and corresponding strain, and \(\sigma_{sy}, \varepsilon_{sy}\) the steel material yielding stress and strain. Empirically, \(p_z\) takes the value of \(D/2\) or \(0.1L_0\). \(L_0\) is the column clear length, and \(D\) the dimension of the column cross-section.

\[
\text{Fig. 2-6-13 Assumed Plastic Zone for MS Element Determining Spring Initial Stiffness}
\]

To balance the initial stiffness of the column in the line element and two MS elements, flexibility reduction factors \(\gamma_1, \gamma_2, \gamma_0\) are used to reduce the bending and axial flexibility of the line element:

\[
\begin{align*}
\gamma_1 &= \gamma_2 = 1.0 - \frac{3}{L_0} \cdot \frac{p_z}{\varepsilon} \quad \text{and} \quad \gamma_1 > 0.5 \quad \text{and} \quad \gamma_2 > 0.5 \\
\gamma_0 &= 1.0 - 2 \frac{p_z}{L_0} \quad \text{and} \quad \gamma_0 > 0
\end{align*}
\]

Where, \(\varepsilon = \sum \frac{E_i A_i y_i^2}{EI}\), \(p_z < \frac{\varepsilon L_0}{6}\) for \(\gamma_1, \gamma_2\), and \(p_z < 0.5L_0\) for \(\gamma_0\), and \(EI\) is the flexural stiffness of the column member. The factors make the column initial flexibility in balance as following:

\[
\begin{bmatrix}
\frac{L_0}{3EI} & -\frac{L_0}{6EI} & 0 \\
-\frac{L_0}{6EI} & \frac{L_0}{3EI} & 0 \\
0 & 0 & \frac{L_0}{EA}
\end{bmatrix}
\begin{bmatrix}
\gamma_1 L_0 & -\frac{L_0}{3EI} & 0 \\
\frac{3EI}{6EI} & \gamma_2 L_0 & 0 \\
0 & 0 & \gamma_0 L_0
\end{bmatrix}
\begin{bmatrix}
\frac{p_z}{\sum E_i A_i y_i^2} & 0 & 0 \\
0 & \frac{p_z}{\sum E_i A_i y_i^2} & 0 \\
0 & 0 & \frac{2p_z}{\sum E_i A_i}
\end{bmatrix}
\]

The first item in the right side of the equation (2-6-3) is the reduced flexibility of the line element, and the second item represents the flexibility of the two MS elements.
The force-deformation relation in the MS element is expressed as following:

\[ \{d_j\} = [\delta_j]\{f_j\} \]  \hspace{1cm} (2-6-4)

In which,

\[ \{d_j\} = \{r_{s_j}, r_{y_j}, d_{o_j}\}^T \]
\[ \{f_j\} = \{m_{s_j}, m_{y_j}, f_{o_j}\}^T \]

\[ [\delta_j] = \begin{bmatrix} k_{yy} & k_{xy} & k_y^j \\ k_{xy} & k_{xx} & k_x^j \\ k_y & k_x & k_o \end{bmatrix}^{-1} \begin{bmatrix} \delta_{yy}^j \\ \delta_{xy}^j \\ \delta_x^j \end{bmatrix} = \begin{bmatrix} \delta_{yy} \\ \delta_{xy} \\ \delta_x \end{bmatrix} \]  \hspace{1cm} (2-6-5)

\[
\begin{align*}
    k_{yy} &= \sum k_i \cdot y_i^2, \\
    k_y &= \sum k_i, \\
    k_{xy} &= \sum k_i \cdot x_i \cdot y_i, \\
    k_{xx} &= \sum k_i \cdot x_i^2, \\
    k_x &= \sum k_i, \\
    k_o &= \sum k_i
\end{align*}
\]

In the above expressions the subscript (or superscript) \( j = 1 \) for the MS element at initial end, and \( j = 2 \) for the terminal-end. \( x, y \) = spring location and \( k_i \) = instantaneous stiffness of the \( i \)-th spring.

**Shear MS element**

The shear MS element is used to simulate the biaxial shear behavior of column member subjected to bi-directional lateral loads. It consists of a number of shear springs radially arranged at equal angle in the column mid-span section (Fig. 2-6-7). Each shear spring is subjected to uniaxial lateral force and displacement in the spring direction. The total number of the springs can be in any even number, of them two are parallel to the column section principal axes \( x \) and \( y \). The shear spring is numbered anti-clockwise starting with number 1 for the \( x \)-axis parallel spring. The angle \( \theta_i \) of \( i \)-th spring to the \( x \)-axis is

\[
\theta_i = \frac{(i-1)\pi}{n} \hspace{1cm} (i = 1, 2, \ldots, n)
\]  \hspace{1cm} (2-6-6)

In which \( n \) is an even number representing the total number of the springs. Then the spring initial stiffness, \( k_{si} \), is expressed as

\[
k_{si} = k_{s0} + \Delta k \cdot \sin \theta_i \hspace{1cm} (i = 1, 2, \ldots, n)
\]  \hspace{1cm} (2-6-7)

Where the parameter \( k_{s0} \) and \( \Delta k \) are based on the column initial shear stiffness \( K_{sy}, K_{sx} \) in \( x \)-and \( y \)-axis as following:

\[
\begin{align*}
    \Delta k &= \frac{K_{sy} - K_{sx}}{\sum \sin^3 \frac{(i-1)\pi}{n} - \sum \sin \frac{(i-1)\pi}{n} \cdot \cos^2 \frac{(i-1)\pi}{n}} \\
    k_{s0} &= \frac{K_{sy} - \Delta k \cdot \sum \sin^3 \frac{(i-1)\pi}{n}}{n/2}
\end{align*}
\]  \hspace{1cm} (2-6-8)
Similarly the spring strength is evaluated from the column shear strength $Q_{px}$, $Q_{py}$ in x- and y-axis. Considering $Q_{px} \neq Q_{py}$ in general, the strength $Q_{si}$ of the i-th shear spring is expressed as:

$$Q_{si} = Q_{s0} + \Delta Q_s \cdot \sin \theta_i \quad (i = 1, 2, \ldots, n) \quad (2-6-9)$$

Where $Q_{s0}$ is the strength of the spring in x-axis ($\theta_i = 0$) and $Q_{s0} + \Delta Q_s$ of that in y-axis ($\theta_i = 90^\circ$). $Q_{s0}$ and $\Delta Q_s$ are determined assuming that all the springs reach their strength when the resultant shear force in principal axis is equal to the column shear strength.

$$\begin{align*}
\Delta Q_s &= \frac{Q_{py} - Q_{px}}{n/2 - \sum \sin \theta_i |\cos \theta_i|} \\
Q_{s0} &= \frac{Q_{py} - \Delta Q_s \cdot n/2}{\sum \sin \theta_i} 
\end{align*} \quad (2-6-10)$$

When $Q_{px} = Q_{py}$ then $\Delta Q_s = 0$, and it results in equal strength, $Q_{s0}$, for all shear springs.

Using only two shear springs locating in the principal x- and y-axis direction for the shear MS element, the spring initial stiffness and strength from the above equations become $k_{sx} = K_x$, $k_{sy} = K_y$, $Q_{sx} = Q_{px}$, $Q_{sy} = Q_{py}$. It represents the column shear force-deformation relations in the principal x- and y-axis separately (without the interaction between them).

The relations of incremental shear force $\{\Delta Q\} = \{\Delta Q_x, \Delta Q_y\}^T$ and deformation $\{\Delta \delta\} = \{\Delta \delta_x, \Delta \delta_y\}^T$ in column section principal axis direction can be expressed as:

$$\{\Delta \delta\} = [\delta] \{\Delta Q\} \quad (2-6-11)$$

The shear flexibility matrix $[\delta]$ is formed as following:

$$[\delta] = \begin{bmatrix}
\delta_{sx} & \delta_{sxy} \\
\delta_{sxy} & \delta_{sy}
\end{bmatrix} = \begin{bmatrix}
\sum k_{st} \cdot \sin^2 \theta_i & \sum k_{st} \cdot \sin \theta_i \cdot \cos \theta_i \\
\sum k_{st} \cdot \sin \theta_i \cdot \cos \theta_i & \sum k_{st} \cdot \cos^2 \theta_i
\end{bmatrix}^{-1} \quad (2-6-12)$$

**Column member force-displacement relations**

The force-deformation relations at column member ends is

$$\{D\} = [\delta]\{F\} \quad (2-6-13)$$

$$\{D\} = \{d_1, d_2, \theta_1, \theta_2, \theta_y, \theta_2\}^T$$

$$\{F\} = \{M_{x1}, M_{x2}, M_{y1}, M_{y2}, P_z\}^T$$

The flexibility matrix in the equation (2-6-12) is the sum of $[\delta_1]$ of the line element, $[\delta_1]$ and $[\delta_3]$ of the flexural and axial MS elements, and $[\delta_5]$ of the shear MS element, expressed as following:
\[ [\delta] = [\delta_L] + [T_1]^T [\delta_1][T_1] + [T_2]^T [\delta_2][T_2] + [T_3]^T [\delta_3][T_3] \]  
(2-6-14)

\[
[\delta_L] = \begin{bmatrix}
\gamma_1 L_{0y} & -L_{0y} & 0 & 0 & 0 \\
\frac{3EI_x}{6EI_x} & 0 & 0 & 0 & 0 \\
\gamma_2 L_{0y} & 0 & 0 & 0 & 0 \\
3EI_x & \gamma_1 L_{0x} & L_{0x} & 0 & 0 \\
3EI_y & \frac{3EI_y}{6EI_y} & 0 & 0 & 0 \\
3EI_y & \gamma_2 L_{0x} & 0 & 0 & 0 \\
Sym. & \gamma_0 L_0 & 0 & 0 & 0 \\
& 0 & EA_0 & 0 & 0
\end{bmatrix}
\]

\[ [\delta_1] \] and \[ [\delta_2] \] are from the equation (2-6-5) and \[ [\delta_3] \] from the equation (2-6-12). The transformation matrix \[ [T_1], [T_2], [T_3] \] are as following:

\[
[T_1] = \begin{bmatrix}
-1 & 0 & 0 & 0 & 0 \\
0 & 0 & -1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1
\end{bmatrix}, \quad [T_2] = \begin{bmatrix}
1 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1
\end{bmatrix}
\]

\[
[T_3] = \frac{1}{L_0} \begin{bmatrix}
1 & 1 & 0 & 0 & 0 \\
0 & 0 & 1 & 1 & 0
\end{bmatrix}
\]

2.6.4 Fiber model based on material stress-strain relations

Fiber model is based on uniaxial material stress-strain relation. It represents moment-curvature and axial load-strain relations of a flexural member section. No plastic zone is required. It becomes simple using the fiber model to simulate the flexural and axial deformation of shear wall member (Fig. 2-6-9). The nonlinear moment-curvature and axial load-strain relations are evaluated in the two sections at base and topside of shear wall, and it is assumed linear distribution of the flexibility between the two sections.

As shown in Fig. 2-6-14, the force and deformation at member-end and at z-section are:

\[
\{D\} = [\delta] \{F\} \quad (2-6-15)
\]
\[
\{F\} = \{M_{x1}, M_{x2}, M_{y1}, M_{y2}, N\}^T
\]
\[
\{D\} = \{\theta_{x1}, \theta_{x2}, \theta_{y1}, \theta_{y2}, d_0\}^T
\]

\[
\{d(z)\} = [f(z)] \{p(z)\} \quad (2-6-16)
\]
\[
\{p(z)\} = \{m_x(z), m_y(z), p_0(z)\}^T
\]
\[
\{d(z)\} = \{\phi_x(z), \phi_y(z), \varepsilon_0(z)\}^T
\]

Where \[ f(z) \] is the flexibility at z-section and is calculated from the flexibility \[ f_1, f_2 \] of the fiber slices at member-end by linear distribution as following:
\[ [f(z)] = (1 - z/L_0) \cdot [f_1] + z/L_0 \cdot [f_2] \] (2-6-17)

The flexibility at the member-end fiber slice is the contribution of all fibers, calculated as following:

\[
[f_j] = \begin{bmatrix}
k_{yy}^j & k_{xy}^j & k_y^j \\
k_{xy}^j & k_{xx}^j & k_x^j \\
k_y^j & k_x^j & k_0^j
\end{bmatrix}^{-1} (j = 1, 2) \tag{2-6-18}
\]

\[
k_{yy} = \sum E_i A_i \cdot y_i^2, \quad k_y = \sum E_i A_i \cdot y_i, \quad k_{xy} = \sum E_i A_i \cdot x_i \cdot y_i,
\]

\[
k_{xx} = \sum E_i A_i \cdot x_i^2, \quad k_x = \sum E_i A_i \cdot x_i, \quad k_0 = \sum E_i A_i
\]

Where \( E_i A_i \) is the stiffness and \( x_i, y_i \) = the location of i-th fiber.

The relations between the force and deformation at z-section and member-end can be expressed as:

\[
\{D\} = \int_0^{L_0} [T_z]^T \{d(z)\} \cdot \partial z \tag{2-6-19}
\]

\[
\{p(z)\} = [T_z] \{F\} \tag{2-6-20}
\]

Where the geometrical transformation matrix \([T_z]\) is:

\[
[T_z] = \begin{bmatrix}
-(1 - z/L_0) & z/L_0 & 0 & 0 & 0 \\
0 & 0 & -(1 - z/L_0) & z/L_0 & 0 \\
0 & 0 & 0 & 0 & 1
\end{bmatrix} \tag{2-6-21}
\]

![Fiber slice force-deformation](image)

Fig. 2-6-14 Force-Deformation Relations at z-Section and Member-End

Then the member flexibility matrix \([\delta]\) is calculated from the above equations (2-6-15) to (2-6-21) as following:
\[
[J] = \int_0^{L_0} \left[ \begin{array}{c}
3f_{yy} + f_{xy}^2 - f_{xy}^1 - f_{xy}^2 \\
 f_{xy}^1 + 3f_{yy} \\
3f_{xx} + f_{xx}^2 - f_{xx}^1 - f_{xx}^2 \\
 f_{xx}^1 + 3f_{xx}
\end{array} \right] \cdot \left[ f_1 \right] \cdot dZ
\]
\[
= \frac{L_0}{12} \left[ \begin{array}{c}
3f_{yy} + f_{xy}^2 - f_{xy}^1 - f_{xy}^2 \\
 f_{xy}^1 + 3f_{yy} \\
3f_{xx} + f_{xx}^2 - f_{xx}^1 - f_{xx}^2 \\
 f_{xx}^1 + 3f_{xx}
\end{array} \right]
\]

\[
\text{Sym.}
\]

(2-6-22)

2.6.5 Section discretization for MS and fiber model

Using MS or fiber model for structural member idealization, the member critical section is discretized in to a number of springs or fibers according to the section details and material properties. For RC or SRC section, a single reinforcing bar is replaced by a steel spring/fiber, and the concrete area is discretized and replaced by a fairly large number of concrete springs/fibers with or without distinguishing the confined core concrete and unconfined cover concrete. The steel of SRC section is also properly divided into small portions and replaced by steel springs/fibers.

Discretization of rectangular concrete area

The discretization of a rectangular concrete area in dimension of B×D is illustrated in Fig. 2-6-15. The core concrete is discretized in to \(k_1\)-column and \(k_2\)-row and the cover concrete on B-side is divided into a row of \(k_1+1\) elements and on D-side a column of \(k_2+1\) elements. The total number of the concrete springs/fibers used to replace the concrete area is decided between 50 ~ 400 according to the size of the discretized area (usually considering the concrete spring/fiber governed area about 5×5 cm\(^2\) to 10×10 cm\(^2\)).

Fig. 2-6-15 Discretization of Rectangular Concrete Area
Discretization of circular concrete area

For circular concrete area, the discretization first makes the area into \( m \) rings (Fig. 2-6-16). Then dividing the ring again into a series of small elements, 4 for the first ring (from the circle center), 12 for the second ring, 20 for the third ring, and so on. It reaches \((8m-4)\) elements in the outer ring, and total \( 4m^2 \) concrete elements. Thus it makes equal area for each element:

\[
a_i = \frac{\pi r^2}{4m^2}
\]  
(2-6-23)

Then the equal number of concrete springs/fibers is placed at the center point of each concrete element. Of the \( j \)-th ring the center point is on the circumference with radius of \( \bar{r}_j \) (Fig. 2-6-16).

![Fig. 2-6-16 Discretization of Circle Concrete Area](image)

Discretization of steel-section

Steel section or steel in SRC section is considered an assembly of thin steel plates. Each steel plate is discretized into a row or a column of steel elements and replaced by steel springs/fibers at the element center point. Fig. 2-6-17 illustrates the discretization of I-steel (H-steel) and L-steel.

![Fig. 2-6-17 Discretization of Steel Sections](image)
2.6.6 Hysteresis models

Hysteresis model specifies the relation for force-displacement of structural members or spring components. It is carefully determined according to the material properties of concrete and steel and the mechanical properties of the force-displacement component in structural members.

The hysteresis models shown in Fig. 2-6-18 represent the force-displacement or stress-strain relations of the steel and concrete spring/fiber in MS element and fiber model. The steel model with stiffness reduced before yielding is considered to allow for reinforcing bar pulling out from beam-column joint or the deformation of reinforcing bar over its anchored length. For the steel spring/fiber representing steel section or SRC section, the stiffness reduction before yielding may not be included by making the parameter $\kappa = 1.0$. The concrete model has optional bilinear skeleton curve or high-order function curve in compression ascending branch, and also optional descending branch or horizontal line after reaching the maximum compression strength (point C). The concrete crack-closing effect is taken into account to allow the concrete spring/fiber carrying compression force/stress before reloading into compression deformation.

For force-displacement relation:

$$F_y = \sigma_{sy}A_s, \quad d_{sy} = \varepsilon_{sy}P_z$$

$$K_s = \frac{E_sA_s}{\varepsilon_{sy}P_z}$$

For stress-strain relation:

$$\kappa = \begin{cases} 1 + (\eta - 1)/\eta, & \eta > 1 \\ 1, & \eta \leq 1 \end{cases}$$

$\sigma_{sy}$, $\varepsilon_{sy}$: steel yielding stress & strain
$E_s$, $A_s$: steel Young's modulus and bar area
$\eta$: column shear span ratio
$P_z$: assumed plastic hinge zone

For concrete hysteresis model:

$$F_c = \sigma_c \cdot A_{ci}, \quad d_c = \varepsilon_c \cdot P_z$$

$$F_t = \sigma_t \cdot A_{ti}, \quad d_t = \varepsilon_t \cdot P_z$$

$$K_c = \frac{E_c^2 A_{ci}}{\sigma_c \cdot P_z}$$

For stress-strain relation:

$$\sigma_c, \varepsilon_c: concrete \text{ Young's modulus and strength}$$

$$\sigma_c = f'_c + \alpha \rho_c \sigma_{sy}, \quad \varepsilon_c = \varepsilon_0 + \beta \rho_c \sigma_{sy} / f'_c$$

$\alpha, \beta$: concrete confinement factor
$\rho_c$: hoop bar volume ratio ($\rho_c \leq 0.018$)
$\sigma_{sy}$: hoop bar yielding stress

Fig. 2-6-18 Hysteresis Model for the Force-Displacement or Stress-Strain Relations of the Spring/Fiber of MS and Fiber Model
The hysteresis model shown in Fig. 2-6-19 specifies the force-deformation relations of uniaxial bending and shear. It has the features of optional trilinear or bilinear skeleton curve, unloading stiffness degradation, strength deterioration and pinching effect. The model is used to specify stiffness and restoring force of the rotational spring and shear spring of beam members (Fig. 2-6-6), and the shear spring of column and shear wall (Fig. 2-6-7, 2-6-9).

![Hysteresis model diagram]

Fig. 2-6-19 Force-Displacement Relations Representing Uniaxial Bending and Shear

2.6.7 **Step by step analysis method and equilibrium iteration procedure**

The nonlinear analysis is calculated step by step in adequate small load/time step. Solving the force-displacement responses and other responses (acceleration, velocity) in the load/time step is carried out based on the assumption of (1) linear relations between force and displacement increments within the load/time step — piece-wise linear relations; (2) compatibility relation of displacements at structural nodes; (3) equilibrium relation of forces (inertia, damping, resistance, and external loads) at structural nodes; and (4) differential relation among time response functions (accelerations, velocities and displacements).

At any time/load step, the incremental displacements of structural members are determined from the displacements at corresponding structural nodes by compatible condition, and the apparent incremental forces of structural members are calculated from the incremental displacements under the assumption of linear relation during the time/load interval. The calculated force and displacement increments may not satisfy a specified force-displacement relation for the member (Fig. 2-6-20) because a nonlinear force-displacement relationship is approximated by an instantaneous linear relationship during the time/load step. Then the member apparent force increments are to be modified to fit the specified force-displacement relationship, though the resultant force may disrupt the equilibrium condition in the member and at corresponding structural nodes. The resulted unbalanced forces are corrected by iteration procedures.

Two iteration procedures, overall iteration and member iteration, are adopted to maintain the equilibrium for the analysis based on nonlinear force-displacement relations of individual structural members. The member iteration procedure is carried out for all members to maintain the force components at member-end in balance, while the overall iteration is performed to keep the equilibrium between the external load and structural restoring forces.
The displacement compatible condition, however, shall be kept during both iteration procedures.

![Diagram](image)

**Fig. 2-6-20 Unbalanced Force to Meet Specified Force-Displacement Relation**

**The overall iteration**

In overall iteration procedure by Newton-Raphson method is employed to find the convergent results. It renews the stiffness in every iteration step (Fig. 2-6-21). If the unbalance (due to stiffness change) becomes too large, then the iteration may result in divergence. To prevent the iteration from failure, re-step method (Fig. 2-6-22) is combined. By reducing the load/time step, the unbalance would be reduced as well, so to stabilize the iteration procedure in reaching convergence.

![Diagram](image)

**Fig. 2-6-21 Newton-Raphson Method for Overall Equilibrium Iteration**

**Fig. 2-6-22 Re-step Reducing Unbalance Force for Stabilizing Overall Iteration**

**The member iteration**

For structural member in nonlinear flexural, shear and axial tension/compression deformation, unbalance between moment and shear force and between axial forces at member-end may be induced due to different stiffness change and hysteresis loop in the force-displacement components. Therefore, member iteration is carried out to find out a set of balanced member forces. Note that the balanced member forces may cause again the unbalance between the forces at the member-end and at relevant structural nodes. Such unbalance is to be solved by the overall iteration procedure.
For 3-D flexural-shear-axial tension/compression member, the member-end force components of apparent force \( \{F\} \) at the beginning of the step, and of real force \( \{F'\} \) at the end of the step are as following:

\[
\{F\} = \{M_{x1}, M_{x2}, Q_y, M_{y1}, M_{y2}, Q_x, N_1, N_2\} \\
\{F'\} = \{M'_{x1}, M'_{x2}, Q'_{y}, M'_{y1}, M'_{y2}, Q'_x, N'_1, N'_2\}
\]  
(2-6-24)

If

\[
Q'_{y} \neq -\frac{M'_{x1} + M'_{x2}}{L_0}, \text{ or } Q'_{x} \neq -\frac{M'_{y1} + M'_{y2}}{L_0}, \text{ or } N'_1 \neq N'_2
\]

Then, a set of force adjustments \( \{\Delta F\} = \{\Delta M_{x1}, \Delta M_{x2}, \Delta Q_{y}, \Delta M_{y1}, \Delta M_{y2}, \Delta Q_{x}, \Delta N_{1}, \Delta N_{2}\} \) are applied at the member (Fig. 2-6-23 with fixed member-end to keep the displacement compatible condition between the member-end and relevant structural nodes) to cause the member internal deformation redistribution and to reach new balances as following:

\[
\begin{align*}
Q'_{y} + \Delta \tilde{Q}_{y} &= -\frac{M'_{x1} + \Delta \tilde{M}_{x1} + M'_{x2} + \Delta \tilde{M}_{x2}}{L_0} \\
Q'_{x} + \Delta \tilde{Q}_{x} &= -\frac{M'_{y1} + \Delta \tilde{M}_{y1} + M'_{y2} + \Delta \tilde{M}_{y2}}{L_0} \\
N'_1 + \Delta \tilde{N}_{1} &= N'_2 + \Delta \tilde{N}_{2}
\end{align*}
\]  
(2-6-25)

Finding the proper force adjustments is repeated until the condition of (2-6-25) is satisfied. Then the unbalance between the forces of the member-end and the relevant structural node is calculated as \( \{F\} - \{F'\} \) and is to be corrected in the overall iteration procedure.

**Fig. 2-6-23 Finding Force Adjustments to Balance the Member Force Components**
2.6.8 Static pushover analysis

Static pushover analysis is usually carried out to find the complete later load-displacement relations of building structure under gradually increased lateral loads. Load distribution pattern or displacement pattern is needed for the pushover analysis. Given displacement pattern the pushover analysis may result in finding no weak story. Therefore, load pattern is given for pushover analysis in the study. The load pattern is a set of load factors that represents the load distribution on structure, and it governs the displacement mode of the structure. During loading the load may increase or decrease, however, all the load components are kept in the proportion specified by the load distribution factors.

For building structure with a displacement mode dominated by the side-sway of the floor level, weighted reversed-triangle load distribution (Fig. 2-6-24) is one of the reasonable load patterns [16] and is applied for the pushover analysis in this study. The load pattern is unitized to make the maximum factor value in 1.0. Such load pattern forms the unit load vector \( \{ \bar{F} \} \), and the displacement of structure caused by the unit load vector is the unit displacement \( \{ \bar{D} \} \).

![Anti-Triangle Load Pattern and the Master Displacement for Loading Control](image)

The analysis is carried out by displacement control. The dominant lateral displacement component, usually at the top level of building structure (Fig. 2-6-24), is used for the analysis control, and is called as master displacement. The displacement increment \( \Delta D_i \) of a load step at the master displacement point is determined as:

\[
\Delta D_i = 2^\sigma \frac{K_0}{K_i} \frac{H}{100000} \leq \frac{H}{100} \quad (2-6-26)
\]

In which, \( K_0 \) and \( K_i \) are the initial stiffness and instantaneous stiffness evaluated at the master displacement component, and \( H \) is the total height of the building structure. The equation \((2-6-26)\) gives a load step size equal to \( H/100000 \) at the beginning (suppose the exponent parameter \( \sigma = 0 \)). It increases the load step size as stiffness degrading. The upper limit of the load step size is set to be \( H/100 \) to prevent too large load step size after structural yielding occurred.
There is a factor $2^\sigma$ used in the equation (2-6-26) to adjust the load step size. The parameter can be set as $\sigma = 0, \pm 1, \pm 2, \pm 3$, etc. $\sigma = 0$ may result in appropriate evaluation of the load step size for most building structures. However, for special structure every stiff or soft, it may cause too much unbalance (too large load step) or too slow computation (too many load steps). It is well said that a reasonable nonlinear solution cannot be obtained at only once or twice attempts. After a few attempts, it may become known what is the proper value for the parameter $\sigma$ to control the pushover analysis.

During the load steps of pushover analysis, the master displacement is increased by $\Delta D_i$ step by step until it reaches a destination displacement (usually given as $H/100 \sim H/20$). The analysis procedure is as following.

1. Using the stiffness $[K]$ at the beginning of current load step to determine the unit displacement vector $\{\bar{D}\}$. If there are any residual unbalanced forces $\{UF\}$ during the last load step, then also calculating the displacement $\{UD\}$ caused by the unbalanced forces.

   \[
   \{\bar{D}\} = [K]^{-1}\{\bar{F}\}
   \]

   \[
   \{UD\} = [K]^{-1}\{UF\}
   \]  

(2-6-27)

2. Estimating the incremental force vector, $\{\Delta F\}$ and the displacement vector $\{\Delta D\}$. From the assumption of piece-wise linear relations during a load step, there are

   \[
   \{\Delta F\} = \frac{\Delta D_i}{\bar{D}_i} \{\bar{F}\}
   \]

   \[
   \{\Delta D\} = \frac{\Delta D_i - UD_i}{\bar{D}_i} \{\bar{D}\}
   \]  

(2-6-28)

Where, $\Delta D_i$ is the master displacement increment determined by the equation (2-6-26), $\bar{D}_i$ is the unit displacement component at the master displacement point, $UD_i$ is the displacement due to unbalanced forces at the master displacement point.

3. Calculating the incremental displacements of individual structural members and finding the corresponding incremental forces and new stiffness through specified hysteresis models. If any stiffness change occurred then it is done to re-assemble the stiffness matrix $[K]$, and carry out iteration procedures described in section 2.6.7, and collect the new residual unbalanced forces in to $\{UF\}$.

4. Examining the displacement at the master displacement component, and terminating the analysis if it has reached or become over the destination displacement. Otherwise, determining new master displacement increment $\Delta Di$ by the equation (2-6-26), and repeat the procedure from step 1.
2.6.9 Dynamic response analysis

Dynamic response analysis is carried out to simulate building earthquake responses to ground motion (acceleration) input. In dynamic analysis, mass is given at corresponding displacement degrees of freedom (DOF) to incorporate inertia forces, and the equilibrium condition includes the inertia force, damping force and restoring force as well as any time-dependent external loads (dynamic equilibrium). However, in the building structural analysis of multi-DOFs system, some of the DOFs may have less effect in response and may be assumed zero mass for simplicity. For the DOFs with zero mass the equilibrium condition includes no inertia force (static equilibrium). For such multi-DOFs system, the equilibrium conditions (the equations of motion to a relative coordinate system) including the two groups of the DOFs with and without mass are expressed as following:

\[
\begin{bmatrix}
[M_1] & [C_{11}] & [C_{12}] & [K_{11}] & [K_{12}] \\
0 & [C_{21}] & [C_{22}] & [K_{21}] & [K_{22}]
\end{bmatrix}
\begin{bmatrix}
\{\ddot{X}_1\} \\
\{\ddot{X}_2\}
\end{bmatrix}
+ 
\begin{bmatrix}
[0] \\
[0]
\end{bmatrix}
= 
\begin{bmatrix}
[M_1] & [0] \\
0 & [0]
\end{bmatrix}
\begin{bmatrix}
\{\ddot{a}\} \\
\{\ddot{f}\}
\end{bmatrix}
+ 
\begin{bmatrix}
\{f_t\} \\
\{f_0\}
\end{bmatrix}
\tag{2-6-30}
\]

Where, \(\{X_1\}, \{\dot{X}_1\}, \{\ddot{X}_1\}\) relative displacement and velocity of the group of DOFs with mass,
\(\{X_2\}, \{\dot{X}_2\}, \{\ddot{X}_2\}\) displacement and velocity of the group of DOFs without mass,
\(M_1\) = the mass matrix,
\([C_{11}] [C_{12}] [C_{21}] [C_{22}]\) damping matrix,
\([K_{11}] [K_{12}] [K_{21}] [K_{22}]\) the stiffness matrices corresponding to two DOFs groups,
\(\{\ddot{a}\}\) = the acceleration of the relative coordinate system (ground motion),
\(\{\ddot{f}\}\) = any time-varying external loads at the DOFs with mass, and
\(\{\ddot{f}_0\}\) = any external loads at zero-mass DOFs.

The acceleration responses for the group of zero-mass DOFs are undetermined, and the damping matrices \([C_{12}], [C_{22}]\) exist only when stiffness-proportional damping is included. The velocity \(\{\dot{X}_2\}\) of the zero-mass DOFs is evaluated when including the damping \([C_{12}], [C_{22}]\).

The two groups of massed and mass-less DOFs are not separated during computation, because it convenient to perform auto-renumbering over all the DOFs for optimum numbering system and minimum matrix size. Therefore, the equation (2-6-30) is re-written as:

\[
[M] \{\dddot{X}\} + [C] \{\dddot{X}\} + [K] \{X\} = -[M] \{\dddot{a}\} + \{f\}
\tag{2-6-31}
\]

Where, the mass matrix \([M]\), damping matrix \([C]\) and stiffness matrix \([K]\) and the vectors of displacement \(\{X\}\), acceleration \(\{\dddot{X}\}\), velocity \(\{\dot{X}\}\), input acceleration \(\{\dddot{a}\}\) and external load \(\{f\}\) include all DOFs. Thus matrix automatic renumbering is applicable.

As the mass of structure is concentrated at structural nodes, the mass matrix \([M]\) is diagonal matrix and is treated as constants during the computation. The damping matrix \([C]\) is formed using Rayleigh's damping as following:

\[
[C] = a_m[M] + a_k[K]
\tag{2-6-32}
\]
Where, $a_m$ = damping factor proportional to mass matrix $[M]$, and $a_k$ = damping factor proportional to the time-varying stiffness matrix $[K]$. For RC building analysis, damping constants $h_1$ and $h_2$ corresponding to the first mode and second mode are given at $2 \sim 5\%$. Then the damping factors $a_m$ and $a_k$ are determined based on the circular frequency $\omega_1$, $\omega_2$ of the modes.

\[
\begin{align*}
2 \cdot h_1 \cdot \omega_1 &= a_m + a_k \cdot \omega_1^2 \\
2 \cdot h_2 \cdot \omega_2 &= a_m + a_k \cdot \omega_2^2
\end{align*}
\] (2-6-33)

When only $h_1$ is given, then $a_k = 2 \cdot h_1 / \omega_1$ is computed for the stiffness-proportional damping, or $a_m = 2 h_1 \omega_1$ is calculated for the mass-proportional damping.

For nonlinear analysis the equation of motion of (2-6-31) is re-written at time $t+\Delta t$ as following:

\[
[M] \{\dot{X}\}_{t+\Delta t} + (a_m[M] + a_k[K]) \{\ddot{X}\}_{t+\Delta t} + \{R\}_{t} + [K] \{\Delta X\}_{t+\Delta t} = -[M]\{\Delta a\}_{t+\Delta t} + \{f\}_{t+\Delta t} + \{F_u\}_{t}
\] (2-6-34)

Where $\{R\}_t$ is the restoring force vector at time $t$, and $\{\Delta X\}_{t+\Delta t}$ the displacement increment at time $t+\Delta t$. By the piece-wise linear assumption the restoring force increment is calculated using the stiffness $[K]$, at time $t$ times the displacement increment $\{\Delta X\}_{t+\Delta t}$. The item $\{F_u\}_t$ is added for the probable residual unbalance forces at time $t$. At time $t+\Delta t$ the stiffness and damping matrices may change to be $[K]_{t+\Delta t}$ and $a_m[M]+[K]_{t+\Delta t}$, and may again cause unbalance $\{F_u\}_{t+\Delta t}$. The unbalance is calculated as following:

\[
\{F_u\}_{t+\Delta t} = \{f\}_{t+\Delta t} - [M](\{\Delta a\}_{t+\Delta t} + \{\Delta \dot{X}\}_{t+\Delta t}) - (a_m[M] + a_k[K]_{t+\Delta t})\{\Delta \ddot{X}\}_{t+\Delta t} - \{R\}_{t+\Delta t}
\] (2-6-35)

The equation of motion (2-6-34) is solved by Newmark $\beta$-method using the differential relations of the time functions as following:

\[
\{\Delta \ddot{X}\}_{t+\Delta t} = (1-\gamma)\Delta t \{\ddot{X}\}_t + \gamma \cdot \Delta t \{\Delta \dot{X}\}_{t+\Delta t}
\]

\[
\{\Delta X\}_{t+\Delta t} = \Delta t \{\dot{X}\}_t + \left(1/2 - \beta\right)\Delta t^2 \{\ddot{X}\}_t + \beta \cdot \Delta t^2 \{\Delta \ddot{X}\}_{t+\Delta t}
\] (2-6-36)

The parameter $\gamma = 1/2$ and $\beta = 1/4$ is used (average acceleration assumption) in the computation. The displacement increment $\{\Delta X\}$, velocity $\{\dot{X}\}$ and acceleration $\{\ddot{X}\}$ at time $t+\Delta t$ are calculated as following:

\[
\begin{align*}
\{\Delta X\}_{t+\Delta t} &= [\tilde{K}]^{-1}\{\Delta \tilde{F}\} \\
\{\ddot{X}\}_{t+\Delta t} &= C_0\{\Delta X\}_{t+\Delta t} - C_1\{\dot{X}\}_t - C_2\{\ddot{X}\}_t \\
\{\dot{X}\}_{t+\Delta t} &= C_3\{\Delta X\}_{t+\Delta t} - C_4\{\dot{X}\}_t - C_5\{\ddot{X}\}_t
\end{align*}
\] (2-6-37)

Where the effective stiffness matrix $[\tilde{K}]$ and load vector $\{\Delta \tilde{F}\}$ are calculated as following:
\[
\begin{align*}
[\ddot{K}] & = C_{13}[K]_t + C_{10}[M] + C_3(a_0[K_0] + [C_e]_t) + C_3[C_v]_t \quad (2-6-38) \\
\{\Delta \ddot{F}\} & = \{f\}_{t+\Delta t} + \{F_u\}_t + [M]\{C_{14}\{\ddot{x}\}_t + C_{12}\{\ddot{x}\}_t - \{a\}_{t+\Delta t}\} + [K]_t[C_{14}\{\ddot{x}\}_t + C_{15}\{\ddot{x}\}_t] - \{R\}_t \quad (2-6-39) \\
C_0 & = \frac{1}{\beta \Delta t^2}, \quad C_1 = \frac{1}{\beta \Delta t}, \quad C_2 = \frac{1}{2 \beta}, \\
C_3 & = \frac{\gamma}{\beta \Delta t}, \quad C_4 = \frac{\gamma}{\beta} - 1, \quad C_5 = \left(\frac{\rho}{2 \beta} - 1\right) \Delta t, \\
C_{10} & = C_0 + C_3 \cdot a_m, \\
C_{11} & = C_1 + C_4 \cdot a_m, \\
C_{12} & = C_2 + C_5 \cdot a_m, \\
C_{13} & = 1 + C_3 \cdot a_k, \\
C_{14} & = C_4 \cdot a_k, \\
C_{15} & = C_5 \cdot a_k. 
\end{align*}
\]

2.6.10 Multi-directional input for dynamic response analysis

The input of ground motion (acceleration) to a building structure is considered uniform over all points of the building. Up to three translational acceleration components \((A_x, A_y, A_z)\) can be given as input at all structural nodes on the base level of a building structure, as shown in Fig. 2-6-25.

When the base level of a building structure is treated as rigid diaphragm that has infinite stiffness in the level and complete soft out of the level, then the lateral acceleration components of \(Ax\) and \(Ay\) are inputted at the gravity center point of the rigid diaphragm, and a rotational acceleration component \(A_{xy}\) in the diaphragm plane can be added to the input, as shown in Fig. 2-6-26. The vertical acceleration component, if given, is made input at individual structural nodes on the diaphragm.

![Fig. 2-6-25 Uniform Input Up to Three Translational Components at All Structural Nodes on the Base Level of Structural Model](image-url)
2.6.11 Mode shape and vibration period analysis

The mode shape and vibration period of building structure in its elastic status and at any load/time steps in plastic stiffness are calculated for determining the mass and stiffness proportional damping in dynamic response analysis and for considering the damage extent of building structures during or after earthquake response. The mode shape extraction is carried out using subspace method and Jacobi-iteration. Details of the methods can be found in the reference [13].

2.6.12 Reliability of the analysis models and numerical methods

The reliability of the analysis models and numerical methods developed and described in the chapter is examined and verified by reproducing structural lab test and simulating the earthquake responses of an instrumented building. The calculated results are comparable with the observed test results and recorded building responses. Therefore, the reliability of the models and methods are verified.

Reproducing lab test results

The lab test specimen is a reinforced concrete cantilever column, circular section diameter 610 mm with 20 #6 (D19) reinforcing bars arranged at diameter 540 mm. The specimen was tested under cyclic loading applied in one-direction at the column top 244 cm high from the fixed base-end, and subjected to constant axial load of 654 kN. The steel bar Young's modulus \( E_s = 2 \times 10^3 \) MPa, yielding strength \( \sigma_y = 303.3 \) MPa. Concrete Young's modulus \( E_c = 24130 \) MPa, compression strength \( \sigma_c = 38.0 \) MPa for core and \( \sigma_c = 34.5 \) MPa for cover concrete, tension strength \( \sigma_t = 3.5 \) MPa. Details of the specimen and lab test are reported in reference [17].

In analysis modeling, the circular concrete area is divided into 28-concrete spring (16-core and 12-cover concrete springs). The plastic zone 24.4 cm (0.1 \times shear span) is assumed and used to determine the spring yielding displacement. The specimen load-deflection relations by lab test and by analysis were compared in Fig. 2-6-27. The analysis results are very comparable with the test results.
Simulating building earthquake responses

The building model is the Holiday Inn located in Van Nuys, California and was damaged during the 1984 Northridge Earthquake. It was a seven-story RC frame structure, designed and constructed in the middle 1960s, about 20 m high, with 8 frame spans in the longitudinal direction and 3 frame spans in the transverse direction (or about 46.0 m by 18.5 m). The typical floor level is shown in Figure 2-6-28. The longitudinal frame is in East-West direction, and the transverse frame is in the North-South direction. The foundation system is made groups of two to four cast-in-site RC friction piles with deep pile caps. The pile caps are connected though rigid foundation beams. According to inspections conducted at the site after the earthquake, no footing damage and liquefaction has been reported. Typical member sections are given in Fig. 2-6-28 and material properties are listed in Table 2-6-1. Details about the building structure and the damage observed during various earthquakes are given in reference [18].

The building located at 7 km northeast to the epicenter of 1994 Northridge earthquake and had severe damage in structural members (beams and columns, flexural and shear failure), but not collapsed. The building responses on typical floor levels were recorded by the instruments installed and operated by the California Strong Motion Instrument Program (CSMIP). The data channels of recorded responses were shown in Fig. 2-6-28 and 2-6-29.

The building is modeled as a beam-column frame system. A typical frame model is illustrated in Fig. 2-6-29. Fixed support at ground floor level (1F) is assumed because of comparatively high rigid foundation to the column. The cross-section size of the first story column is about 500 mm square, while the typical pile cap is 3048 mm square. The floor slabs is relatively well integrated to connect all columns. Therefore, each floor slab is treated as a rigid diaphragm.

Records of acceleration at the ground floor level (CSMIP record data channel 1, 13,15,16) during the first 30 seconds of recorded motion are used for input in the analysis (Fig. 2-6-30). The input has four components in two lateral translational directions and the vertical direction, as well as a rotational component in the horizontal plane. The rotational component is found from the records from channel 1 and 13 at the two ends of the building ground floor level. Numerical integration to solve the equations of motion is carried out at a time interval 1/200 sec. Internal viscous damping in constant 5% proportional to mass and instantaneous stiffness is assumed.
Fig. 2-6-31 and 2-6-32 show the calculated building responses at selected instrument locations and compared with the recorded responses. All the calculated acceleration, velocity and displacement responses are in excellent agreement with the records over the 30 seconds time duration.

Fig. 2-6-28 Typical Floor Plan and Member Sections (Spandrel Beam Size in inch)

Fig. 2-6-29 Frame Model in Longitudinal Direction and Instrument Locations

Table 2-6-1: Material Properties Used in the Analysis (unit: N, mm)

<table>
<thead>
<tr>
<th>Concrete materials</th>
<th>Column 1F~2F</th>
<th>Column 2F~3F Beam &amp; slab 2F</th>
<th>All others 3F~Roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young modulus $E_c$</td>
<td>28960</td>
<td>25510</td>
<td>22753</td>
</tr>
<tr>
<td>Compression strength $f'_c$ and strain $\varepsilon_0$</td>
<td>34.5, 0.002</td>
<td>27.5, 0.002</td>
<td>20.7, 0.002</td>
</tr>
<tr>
<td>Unconfined ultimate strength and strain</td>
<td>0, 0.01</td>
<td>0, 0.01</td>
<td>0, 0.01</td>
</tr>
<tr>
<td>Tension strength</td>
<td>3.0</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>Steel bar for beam and slab</td>
<td>$E_s = 200000$</td>
<td>$\sigma_y = 276$</td>
<td>$\sigma_{max} = 1.5\sigma_y$</td>
</tr>
<tr>
<td>Steel bar for column</td>
<td>$E_s = 200000$</td>
<td>$\sigma_s = 414$</td>
<td>$\sigma_{max} = 1.5\sigma_y$</td>
</tr>
</tbody>
</table>
Fig. 2-6-30 The Records on Ground Floor Level Used for the Input

Fig. 2-6-31 The Recorded and Simulated Responses on Floor Level 2F in E-W Direction

Fig. 2-6-32 The Recorded and Simulated Responses on Top Level in N-S Direction
References


Chapter 3. Case Study on Ground and Building Responses in the 1995 Hyogo-ken Nanbu Earthquake

3.1 Introduction

The earthquake attacked Kobe-city and the vicinity areas in January 17, 1995 caused extensive damage even to the modern buildings that were designed and constructed to sustain earthquake impact. From the viewpoint of structural engineering, the disaster alerted us again the importance and necessity of comprehensive understanding of structural earthquake response and damage mechanism and reliable assessment of vulnerability of urban structures to probable very strong seismic action. Based on the data and information collected from the earthquake disaster in Kobe area, the Structural Performance Team in EDM started working on a new system that integrates the advanced analytical technologies so far individually developed in earthquake engineering, civil and structural engineering areas. The team planned to implement the system in the team research to investigate the generation, propagation and amplification of earthquake ground motion considering uneven soil structures and edge effects, and to evaluate the responses of subsurface soil, soil and foundation, and superstructures including the interactions among all these phases. As the first step of developing the system, the reliability and performance of the analysis models developed and used in the individual areas are to be examined by comparing the evaluated ground motion and simulated soil-foundation-structure responses with the observed data during the earthquake. For this purpose, a residential building located in the extensively damaged belt area in Kobe-city but suffered only light damage, and a North-South ground section across the building location from the mountainside to the seaside are selected and modeled for the analytical study.

The methods employed in the analytical study are illustrated in the picture show in the front color pages with the capital of "Case Study on Earthquake Impacted Structure". That is, the response analyses of deep basin structure, subsurface soil, soil-foundation, and superstructure are carried out separately at the first attempting of performing an integrated evaluation and simulation of the series of earthquake impact and damage phenomenon. By these ways, the ground motion at the building location, the responses and damage of the soil and foundation and the building structure are obtained. The results are compared with the records and post-earthquake onsite observations to examine the reliability and appropriateness of the analysis models and numerical methods. The analysis conditions, methods and results are presented in the following sections in this chapter.
3.2 Simulation of Ground Response and Ground Motion Distribution

3.2.1 Outlines of Analyses

In the 1995 Hyogo-ken Nanbu Earthquake, damaged area in Hanshin district was extended linearly along the coast, the so-called "Damage Belt". This expression, however, only shows a general trend. We will come to know that the damage distribution of buildings is not distributed uniformly if we carry out a close inspection. The explanations for the phenomenon may be the effect of ground motion characteristics including following factors; long-period pulse, spatial distribution of ground amplifications caused by deep and shallow ground structures, dynamic characteristics of each buildings whose structural type or date are different, and the interactions between them. In this section, results of dynamic response analysis using 2-D model is shown, that is aimed to simulate the actual ground motion in target site (Nada-Ward, Kobe) during the 1995 Hyogo-ken Nanbu Earthquake. The simulated motion will be used directly or indirectly in following studies, e.g. simulation of liquefaction, pile foundation response and building response.

For the purpose of estimating the ground motion at Point A, where the target building for following analysis were located, deep ground model was made for the analysis, with 2.0km depth and 3.5km length along the N332E line (shown in Figure 3.2.1). The line almost agrees with normal direction to Suwayama Fault, which is almost parallel to source fault plain.

![Image](image_url)

Figure 3.2.1. Map showing microtremor observation sites and Location of model section.

Regarding the inverted Vs structure obtained from ground exploration by reflection method (Society for the Active Fault Investigation in Hanshin Area, 1996), we made
two-dimensional linear model of cross section along the line passing through Point A as shown in Figure 3.2.2. The Holocene and upper Pleistocene layer was removed from the model surface, and the layer of Osaka Group and Granite were modeled.

Using the model, dynamic response analyses are performed by the finite element method in frequency domain. Maximum element size of each layer was determined so as to keep the effective frequency range up to 5Hz for shear waves, and transmitting and viscous boundaries were used on both sides and at the bottom of the soil model, respectively. Soil parameters of each layer used in the analysis are shown in Table 3.2.1.

As the input motion, the fault normal component of inverted outcrop motion at the depth of 2.0km, was used; thus, which is the vertically incident S-waves with in-plain particle motions. The inverted acceleration motion was derived from observed velocity motion at KBU (Kobe University) so as to remove the amplification in weathered granite on the surface, by the method of 1-D linear multi reflection method. The observed motion and inverted outcrop motion acceleration are shown in Figure 3.2.3, and those peak accelerations are 265cm/s² and 324cm/s², respectively.

![Figure 3.2.2. Supposed two-dimensional Vs profile along the line passing through KBU strong motion station and Point A.](image)

**Table 3.2.1.** Soil parameters used in the dynamic response analysis.

<table>
<thead>
<tr>
<th>ρ (t/m³)</th>
<th>V_p (m/s)</th>
<th>V_s (m/s)</th>
<th>h</th>
<th>Deposits</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8</td>
<td>1800</td>
<td>500</td>
<td>0.03</td>
<td>Upper Osaka Group</td>
</tr>
<tr>
<td>1.9</td>
<td>1900</td>
<td>700</td>
<td>0.02</td>
<td>Middle Osaka Group</td>
</tr>
<tr>
<td>2.1</td>
<td>2500</td>
<td>1000</td>
<td>0.02</td>
<td>Lower Osaka Group</td>
</tr>
<tr>
<td>2.7</td>
<td>5400</td>
<td>3200</td>
<td>0.01</td>
<td>Granite</td>
</tr>
</tbody>
</table>
3.2.2 Results of Analyses

Figure 3.2.4 shows the Fourier acceleration spectrum of simulated outcrop motion. Simulated motion has remarkable peak in the period of 1.3s. Spatial distribution of amplification factors along the model line are shown in Figure 3.2.5, that was computed as amplitude ratios between Fourier spectra of the simulated ground motions and those of the input motions.

The amplification factors at outcrop area (X= -0.5 to 0.0km) are almost unity for all periods, while those at basin are larger than unity, and their peak periods vary according to thickness of sedimentary layer. Amplifications in the period over 0.5s, predominant peak can be seen in the seaside area (X>0.5km).

Figure 3.2.4. Fourier spectrum of simulated outcrop motion.
Figure 3.2.5. Special distribution of amplification factors along the model line.

Spatial distribution of pseudo velocity response spectra along the model line is shown in Figure 3.2.6. The remarkable peak at X=2.8km can be explained by the dominant period of 1.3s in inverted outcrop motion (see Figure 3.2.4) and the peak of 1.3Hz at X=2.8km in the spatial distribution of amplification factors (see Figure 3.2.5).

As shown in the paste-up of acceleration time histories along the model line (Figure 3.2.7), long period pulse of 1.3s, that was dominant in the input motion (see Figure 3.2.4), transfers from mountain to seaside and is amplified especially in the seaside area. Figure

Figure 3.2.6. Special distribution of pseudo velocity response spectra along the model line (h=5%).
3.2.8 shows the variation of peak ground accelerations (PGA) and velocities (PGV) of the simulated horizontal motions along the observation line. The simulated PGA and PGV vary considerably along the line and their values lie in the range of 200-640 cm/s² and 50-150 cm/s, respectively. The highest peak of PGA and PGV are estimated in X=2.8 km, and another peak is distributed around Point A, between X=1.5 and 2.0 km. Figure 3.2.9 shows the variation of the collapse ratios of buildings along the line (Building Research Institute, 1996). Comparing Figure 3.2.8 with Figure 3.2.9, assumed PGA and PGV values in the damaged zone over 50% exceed 400 cm/s² and 90 cm/s, respectively. As shown in Figure 3.2.9, there were few damaged buildings around Point A, despite the PGA and PGV are assumed to be relatively large there. This could be explained by the fact that few sample existed in the
statistical unit. The simulated ground motion (acceleration) at Point A is shown in Figure 3.2.10, and its PGA and PGV are 458 cm/s² and 95.7 cm/s, respectively.

![Figure 3.2.9. Variation of the collapse ratios of buildings along the observation line (calculated from the of research results by Building Research Institute, 1996).](image)

![Figure 3.2.10. Simulated motion (acceleration) at Point A](image)

**Acknowledgements**

The strong motion record at KBU (Kobe University) station used in this study was provided by CEORKA (The Committee of Earthquake Observation and Research in the Kansai Area). The authors express their sincere thanks to the organization.

**References**


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3.3 Simulation of Subsurface Soil Response and Liquefaction

In this section, subsurface soil responses and ground motion characteristics during the 1995 Hyogo-ken Nanbu earthquake are estimated at a strong motion station in Rokko Island (Sekisui House, 1996), hereinafter called Site RKI, and at A residential building in Nada-ku, Kobe, employing the one-dimensional effective stress analysis reviewed in the chapter 2.4. The locations of the sites are shown in Figure 3.2.1. Figure 3.3.1 shows a caricature of the earthquake response analyses for the subsurface soil at the sites. In the analyses, the ground motions in fault normal direction (in-plain direction in Figure 3.3.1) estimated on upper surface of Osaka Group in the previous section 3.2 are used as input outcrop motions for subsurface soil models at the sites.

Figure 3.3.1. Caricature of one-dimensional response analyses for subsurface soils at Rokko Island and A building.
3.3.1 Rokko Island Site

Figure 3.3.2 shows soil profile down to a depth of 120 m from boring and PS logging data at Site RKI (Sekisui House, 1995). The upper surface of Osaka Group appears at a depth of 117 m, which is a control point of the input motion for earthquake response analysis. At the site, an accelerogram was installed on 7 m deep on the 1st basement floor of a building, (see Figure 3.3.2), and strong ground motion during the 1995 Hyogo-ken Nanbu earthquake was successfully recorded and the digitized record is available (Sekisui House, 1995).

One-dimensional effective stress analysis using Eqs. (2.4.1)-(2.4.26) is performed for the soil profile shown in Figure 3.3.2. Soil parameters required in the analysis are inferred from boring data at the site, and are shown in Table 3.3.1. In the table, \( n_e \) denotes number of elements in each layer, and the values of \( \gamma_i \) and \( m_i \) are those under the condition \( \sigma_0' = 1 \) kgf/cm\(^2\).

Broken lines in Figure 3.3.3 are (a) acceleration and (b) velocity time series of the computed ground motions in the fault normal direction at a depth of 7 m. Solid lines in the figure are those of the recorded motions at that depth. The computed and observed ground motions show good agreement in both acceleration and velocity waveforms.

Figures 3.3.4(a)-(c) show variations with a depth of maximum acceleration, shear strains, and excess pore water pressure ratios from the result of the analysis for the site. In Figure 3.3.4(a), the computed peak acceleration is amplified in the subsurface soils down to a depth of 40 m by a factor of 2, and in Figure 3.3.4(c), maximum excess pore water pressure ratio is less than 0.9 at any depth. This suggests that no/ few soil layer liquefied at the site during the 1995 Hyogo-ken Nanbu earthquake. Besides, the depth at which pore water pressure ratio reaches the maximum value are 12-26 m, and are deeper than those at the estimated liquefied layers for Site EKB during the 1995 event, 2-15 m, as shown in Figure 2.4.9(c). These findings are consistent with those from field observations in which very few remarks of sand boils could be seen at Site RKI after the earthquake (Hamada et al., 1995).

The above results also reveal that the estimated ground motions on Osaka Group in the section 3.2 could be reliably reasonable.

![Soil profile at Site RKI (Sekisui House, 1995)](image_url)
Table 3.3.1. Soil parameters in 1-D effective stress analysis at Site RKI.

<table>
<thead>
<tr>
<th>$H$ (m)</th>
<th>$n_e$</th>
<th>$p$ (kPa)</th>
<th>$V_s$ (m/s)</th>
<th>$\phi_d$ (deg.)</th>
<th>$\gamma_{ri}$</th>
<th>$h_{max}$</th>
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Figure 3.3.3. Observed and computed (a) acceleration and (b) velocity waveforms at Site RKI (Fault normal direction).

Figure 3.3.4. Variation with depth of (a) maximum acceleration, (b) shear strain, and (c) excess pore water pressure ratio at Site RKI (Fault normal direction).
3.3.2 A Building Site

Around a building, bore-hole data at four sites, hereby called Sites B01-B04, are available (Kobe city, 1980), and their locations are shown in Figure 3.3.5. Figures 3.3.6(a)-(d) show shallow soil profiles at Sites B01-B04, respectively, based on the bore-hole data and results of microtremor array measurements near the sites (Tokimatsu et al., 1997). Deep soil profiles down to Osaka Group are inferred from the results by Tokimatsu et al. (1997).

One-dimensional effective stress analyses are then conducted for the soil profiles shown in Figures 3.3.6(a)-(d). Soil parameters required in the analyses are determined from the bore-hole data at the sites, and are shown in Tables 3.3.2-3.3.5. In the tables, $n_e$ denotes number of elements in each layer, and the values of $\gamma_i$ and $m_v$ are those under the condition $\sigma_0' = 1$ kgf/cm$^2$. The computed acceleration and velocity waveforms on ground at the sites are shown in Figures 3.3.7(a) and (b), respectively. Figures 3.3.8(a)-(c) show variations with a depth of maximum acceleration, shear strains, and excess pore water pressure ratios. Figure 3.3.9 shows the computed pore water pressure ratios in the soil layers with maximum values shown in Figure 3.3.8(c).

In Figures 3.3.7(a) and (b), before 11 seconds, the computed ground motions at the four sites are same in both acceleration and velocity waveforms, however, after that time, acceleration waveforms are significantly different from each other. In Figure 3.3.9, at the four sites, excess pore water pressure ratios reach 0.9 at 10-11 seconds, and after that time, the pore water pressures change with a trend of cyclic mobility. In Figures 3.3.8(a) and (b), besides, maximum accelerations and shear strains change in the soil layers down to a depth of 10 m at the four sites, which have different S-wave velocities, soil type structures, and SPT N-values as shown in Figures 3.3.6(a)-(d). Thus, these suggest that the differences of accelerations after 11 seconds in Figure 3.3.7(a) could be associated mainly with those of $V_S$ structures and cyclic mobility characteristics in the shallow soil layers.

Figure 3.3.5. Map showing bore-hole sites B01-B04 near A building, Nada-ku, Kobe.
Figure 3.3.6. Shallow soil profiles at Sites B01-B04 (Kobe city, 1980; Tokimatsu et al., 1997).

Table 3.3.2. Soil parameters in 1-D effective stress analysis at Site B01.

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88
Table 3.3.3. Soil parameters in 1-D effective stress analysis at Site B02.

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Table 3.3.4. Soil parameters in 1-D effective stress analysis at Site B03.

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Table 3.3.5. Soil parameters in 1-D effective stress analysis at Site B04.

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<td>37.0</td>
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<td>1.00</td>
<td>0.60</td>
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<td>1.e-3</td>
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<tr>
<td>1.30</td>
<td>1.83</td>
<td>201.5</td>
<td>40.0</td>
<td>6.e-4</td>
<td>0.22</td>
<td>0.33</td>
<td>0.74</td>
<td>1.00</td>
<td>0.60</td>
<td>5.e-3</td>
<td>1.e-3</td>
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<tr>
<td>2.80</td>
<td>1.87</td>
<td>314.7</td>
<td>40.0</td>
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<td>40.0</td>
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<td>-</td>
<td>5.e-3</td>
<td>1.e-3</td>
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<td></td>
</tr>
</tbody>
</table>

- 2.00 500.0

- 2.00 500.0

- 2.00 500.0
Figure 3.3.7. Computed (a) acceleration and (b) velocity waveforms on ground at Sites B01-B04 (Fault normal direction).

Figure 3.3.8. Variation with depth of (a) maximum accelerations, (b) shear strains, and (c) excess pore water pressure ratios from effective stress analyses for Sites B01-B04 (Fault normal direction).

Figure 3.3.9. Computed excess pore water pressure ratios in the layers with maximum values at Sites B01-B04.
From the filed observation after the earthquake, remarks of sand boils are found on
ground near Site B01 (Hamada et al., 1995) and Site B04 are located near an old river (see
Figure 3.3.5). In Figures 3.3.8(b), (c), and 3.3.9, it is suggested that the following soil layers
could liquefied during the earthquake; at a depth of 1.6-8.1 m, 5.4-6.4 m, and 2.2-9.4 m in
Site B01, B02, and B04. These estimated results are consistent with the observed ones
stated above. Furthermore, the values of peak ground acceleration and velocity from the
analyses (520 cm/s² and 110 cm/s, respectively) are in good agreement with those from the
other ground motion evaluations in this area during the 1995 Hyogo-ken Nanbu earthquake
(600-800 cm/s² and 120-150 cm/s, e.g., Motosaka and Nagano, 1996; Tokimatsu et al., 1997;
Hayashi et al., 1997). These results indicate that the estimated subsurface soil responses
and ground motions during the 1995 Hyogo-ken Nanbu earthquake are reasonable, and
confirm again the effectiveness of the one-dimensional effective stress analysis employed in
this report.

References
ground displacement and soil condition in Hanshin area,” Assoc. Development of Earthq.

Hayashi, Y., et al. (1997). “Study on the distribution of peak ground velocity based on


motions in Kobe city taking account of deep irregular underground structure,
-Interpretation of heavily damaged belt zone during the 1995 Hyogo-ken Nanbu

Sekisui House (1996). “Rokko Island City strong motion record during the 1995 Hyogo-ken

characteristics in Sumiyoshi area, Kobe city, based on microtremor measurements,” J.
3.4 Simulation of Pile Foundation Response and Damage

Many buildings with pile foundation on reclaimed land were seriously damaged during the 1995 Hyogo-ken Nanbu earthquake. On the other hand, some pile foundations were not damaged in the area where the strong motion was observed. In this section, the seismic response of the pile foundation, which was not severely damaged in the damaged zone, was discussed through the 2-dimensional effective stress analysis. The numerical method described in section 2.5 was applied to the soil-pile-building system.

3.4.1 Pile Foundation

The damaged building was five stories building made of RC structural members completed in 1961. Figure 3.4.1 shows the plan of the foundation. The each footing has three or five piles with the diameter of 300 mm. The pile is a RC pile with the length of 6 m. The investigation showed that the cracks were observed at the ditches around the building and the bottom concrete of the entrance, however no inclination and settlement of the building were measured. Therefore, the pile foundation might not suffer from major damages.

3.4.2 Numerical Conditions

2-dimensional plain strain analysis with the effective stress method described in section 2.5 was conducted using soil-pile-building finite element model. The finite element model for north-south section including footing F3 is shown in Figure 3.4.2. The soil, pile and building were modeled for a span of 5 m. The soil profile was obtained from the existing boring data near the building, and was modeled to the depth of GL-14.5 m. The cyclic elasto-plastic model for sand was applied to all soil deposits, and their parameters were determined based on the soil properties, which were empirically estimated from the N values.

![Figure 3.4.1. Plan of pile foundation](image)
of standard penetration tests. The liquefaction parameters for As1 layer were determined to reproduce the liquefaction strength, which was the cyclic shear stress ratio of 0.245 for the double amplitude shear strain of 5%. Although the model parameters should be determined on the mechanical laboratory tests such as undrained cyclic shear tests with undisturbed samples, the liquefaction strength was empirically estimated from the N values because no laboratory tests were carried out. The soil material parameters are summarized in Table 3.4.1.

The building was modeled with elastic beam elements and shell elements. The

![Diagram showing FE model for north-south section]

Figure 3.4.2. FE model for north-south section

<table>
<thead>
<tr>
<th>Table 3.4.1. Soil parameters</th>
<th>As1</th>
<th>As2</th>
<th>Ag</th>
<th>Dg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density ( \rho ) (t/m(^3))</td>
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<td>1.87</td>
<td>1.95</td>
<td>1.87</td>
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<tr>
<td>Initial void ratio ( e_0 )</td>
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<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
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<td>1.0E-04</td>
<td>1.0E-04</td>
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<td>2.50E-2</td>
<td>2.50E-2</td>
<td>2.50E-2</td>
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<tr>
<td>Swelling index ( \kappa )</td>
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<td>2.50E-3</td>
<td>2.50E-3</td>
<td>2.50E-3</td>
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<tr>
<td>Initial Shear modulus ratio ( G_0/\sigma_{\text{m}} )</td>
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<td>2240</td>
<td>3679</td>
<td>4135</td>
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<tr>
<td>Failure stress ratio ( M_f )</td>
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<td>1.34</td>
<td>1.34</td>
<td>1.34</td>
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<tr>
<td>Phase transformation stress ratio ( M_m )</td>
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<td>0.91</td>
<td>0.91</td>
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<td>9300</td>
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<tr>
<td>Reference strain parameter ( \gamma_f )</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Dilatancy parameter ( D_0 )</td>
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<td>-</td>
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<tr>
<td>Dilatancy parameter ( n )</td>
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<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
columns and girders were modeled with beam elements, and the girders were assumed rigid. The walls in the north-south direction were modeled with the shell elements. The piles were modeled by non-linear beam elements with tri-linear hysteresis relation between moment and curvature. The moment-curvature relation was calculated using the properties of RC pile as shown in Figure 3.4.3. The numerical conditions for the axial load, the compression strength of concrete and the yield strength of steel bar are summarized in Figure 3.4.3. A few piles for a footing were modeled by a single equivalent pile, which had a simply summed moment of inertia and sectional area for a footing.

The bottom of the FE model was fixed, and the wide elements, which simulated the free field behavior, were attached on the both sides of the FE model. The only underground water surface was drained boundary, and the other all boundaries were impermeable.

The calculated acceleration as shown in Figure 3.4.4, which was obtained at GL-14.5 m in the 1-dimensional effective stress analysis, was input from the bottom of the FE model.

Rayleigh damping proportional to initial stiffness, which was determined by assuming that the damping factor is 2%, was used in order to obtain numerical stability in the high frequency domain. The increment for time integration was 0.001 seconds.
3.4.3 Numerical Results

Figure 3.4.5 and 3.4.6 show the time histories of the response acceleration and displacement respectively at the black squares in Figure 3.4.2. These output nodes are on the free field, top of the building and top of the both piles. The waveforms at each output node showed a similar tendency, but the maximum values were slightly different. The maximum acceleration and displacement occurred on the top of the building. The response acceleration at the pile top was used as the input motion for the building response analysis in section 3.5. Figure 3.4.7 shows the time histories of the relative effective stress ratio (R.E.S.R.: \(1 - \sigma_{w} / \sigma_{w0}\)) in the As1 layer at black circles in Figure 3.4.2. The peak acceleration and displacement occurred at about 11 seconds, however the complete liquefaction, which the R.E.S.R. became 1.0, occurred at about 14 seconds. Thus, the large

![Image](image1.png)

Figure 3.4.5. Time histories of response acceleration

![Image](image2.png)

Figure 3.4.6. Time histories of response displacement

![Image](image3.png)

Figure 3.4.7. Time histories of relative effective stress ratio

95
acceleration amplitude did not input after the complete liquefaction. This is one of the reasons why the maximum ground surface displacement was not very large.

The deformed configuration at 11.4 seconds and the distribution of excess pore water pressure ratio after the earthquake for the whole model are shown in Figure 3.4.8 and 3.4.9 respectively. Although the most of the ground deformation was due to the shear strain in liquefied As1 layer, the maximum ground surface displacement of about 3 cm was not as large as that at other liquefied sites near the coastal line because of the thin thickness of liquefied layer.

Figure 3.4.10 shows the time histories of moment at the pile top and the bottom of As1 layer at the black circles in Figure 3.4.2. These moment values were converted to the value for a single pile, and the crack moment also is shown in Figure 3.4.10. The maximum moment of each element passed the crack moment, however it did not reach the yield moment. Therefore, it was suggested that the residual deformation due to the bending failure of the pile be not generated after the earthquake. These numerical results coincide with the actual behavior of the building that did not tilt or settle after the earthquake.

3.4.4 Summary

The seismic response of the pile foundation, which was not severely damaged in the damaged zone, was discussed through the 2-dimensional effective stress analysis. The numerical method described in section 2.5 was applied to the soil-pile-building system. The cyclic elasto-plastic model for sand was applied to all soil deposits, and their parameters were determined based on the soil properties, which were empirically estimated from the N values of standard penetration tests. Therefore, more soil investigations might be needed for more accurate analysis. As a result, the maximum moment of each element passed the crack moment, however it did not reach the yield moment. Therefore, it was suggested that the residual deformation due to the bending failure of the pile be not generated after the earthquake. These numerical results coincide with the actual behavior of the building that did not tilt or settle after the earthquake.

Acknowledgements

The authors wish to thank Hyogo Prefecture for providing the building data.
Figure 3.4.8. Deformed configuration

Figure 3.4.9. Distribution of excess pore water pressure ratio

Figure 3.4.10. Time histories of moment of the piles
3.5 Simulation of Building Response and Damage

The earthquake in 1995 attacked Kobe-city and the vicinity areas caused great damage and collapse of modern buildings that were designed and constructed as earthquake-resistant structures. Many analytical studies have been carried out to investigate the earthquake responses and the damage mechanism of the heavily damaged and destroyed buildings [1]–[2]. However, some buildings remained undamaged or minor damaged though that located in the extensively damaged belt area in Kobe-city. Investigating the ground motion input and simulating the responses of such undamaged or minor damaged buildings become our interests as that can be a good example to examine the reliability of our analytical methods and models in evaluating the ground motion input to the building and in predicting the structural earthquake responses.

The analytical study is carried out on a 5-story reinforced concrete apartment building, which located in the area severe-affected by the 1995 Hyogo-ken Nanbu Earthquake. The building was observed only minor cracks on shear walls and non-structural members after the earthquake. The analysis using three-dimensional frame model is carried out to investigate the building responses and to examine the reliability of the analysis models and the appropriateness of the evaluated input ground motion to the building. The input ground motion is obtained from the soil and foundation response analysis in two-dimensional plain strain model (Section 3.4) and in one-dimensional effective stress model (Section 3.3). The building response analysis results in only crack in beams and columns of main frames and some damage in shear walls. The results generally agree with the observed building behavior.

The building outline

The building structure to investigate its seismic behavior is a 5-story (14.29 meters high) RC frame shear wall apartment building designed and constructed before the major revision of the design code in Japan in 1971 and located in Kobe-city in the severe damage belt area affected by the 1995 Hyogo-ken Nanbu Earthquake. The building supported on pile foundation has long rectangle floor plan 49.6 x 6.3 m², as shown in Fig. 3-5-1, single span in transverse direction (about North-South direction), and 8-units 12-span in longitudinal direction (about East-West direction). The size of the beam and column structural members is listed in Table 3-5-1 and the section details of typical beam and column members in Fig. 3-5-2. The details of the shear wall were given in Table 3-5-2. The shear walls in transverse direction were used as the boundary of the residential units, and the shear walls in the longitudinal direction had large perforation for doors and windows. The steel reinforcement was round bar SR30. The concrete material could not be identified. In the analysis the concrete was assumed to be 300 N/mm² of compression strength.

According to observation, the building suffered only minor damage to the structure during the 1995 earthquake. It was found concrete cracks on some shear walls, but no obvious damage to the beam-column frames and no damage to the foundation in contrast of heavy damage of buildings surrounding.

The building analysis model and analysis methods

The structural members of beam, column and shear wall in the mainframe planes are considered in the analysis. The analysis frame model and the frame name system are shown in Fig. 3-5-3 and 3-5-4. The shear walls around the staircases are non-structural members and are not included in the analysis. The contribution of the pile foundation to the vertical and lateral reaction at the footing beam-column joint is taken into account. The contribution is
represented by support springs, inelastic vertical spring and elastic lateral spring. The vertical spring reduces stiffness and yields only in tension. Table 3-5-3 lists the spring stiffness. The foundation contribution to the footing rotation is neglected.

The analysis uses three-dimensional frame model. The floor slab is treated as rigid diaphragm in the horizontal floor plane. Treatment of the displacement degrees of freedom of structural nodes and the rigid diaphragms and idealization of the structural members follow the method given in the section 2.6. The structural model has the fundamental period of 0.339 second in the building transverse Y-direction, and 0.282 second in the longitudinal X-direction. It is quite disparity in the stiffness in the building X and Y directions. Therefore, response to the input in the building weak direction (transverse Y-direction) is calculated and discussed.

The input for calculating the building responses are one-component horizontal acceleration in the building transverse Y-direction. Two acceleration waves A1 and A2 are available (Fig. 3-5-5) that are the results from the subsurface soil analysis at building site (Section 3.3) and from the soil-foundation-building analysis (Section 3.4). The input then is one-component acceleration made in the building transverse Y-direction. This is considered base on the building location where the strong component of the ground motion was in about the North-South section conforming to the building transverse direction. The input peak ground acceleration is 521 GAL of A1 and 387 GAL of A2. The input duration is 20 seconds cover the major peaks of the acceleration waves. Time interval of 1/200 second is used in the numerical integration to solve the equations of motion. The time interval is about 1/6 of the fundamental period of the building in transverse direction. Damping of the structural system is assumed proportional to the structural instantaneous stiffness in damping constant of 5%.

Fig. 3-5-1 Typical Floor Plan of the Objective Building

Fig. 3-5-2 Sections of Typical Beams and Columns
Table 3-5-1 Section of Beam and Column (cm)

<table>
<thead>
<tr>
<th>Beam</th>
<th>G2</th>
<th>G4</th>
<th>Other</th>
<th>Column</th>
</tr>
</thead>
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<tr>
<td>RF</td>
<td>30×75</td>
<td>25×45</td>
<td>25×75</td>
<td>Story BxD</td>
</tr>
<tr>
<td>5F</td>
<td>30×60</td>
<td>25×45</td>
<td>25×60</td>
<td>5-RF 35×35</td>
</tr>
<tr>
<td>4F</td>
<td>30×60</td>
<td>27×45</td>
<td>27×60</td>
<td>4-5F 40×35</td>
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<tr>
<td>3F</td>
<td>30×60</td>
<td>27×45</td>
<td>27×60</td>
<td>3-4F 45×40</td>
</tr>
<tr>
<td>2F</td>
<td>30×60</td>
<td>30×50</td>
<td>30×60</td>
<td>2-3F 55×40</td>
</tr>
<tr>
<td>1F</td>
<td>30×140</td>
<td>30×90</td>
<td>30×150/90</td>
<td>1-2F 60×40</td>
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Table 3-5-2 Shear Wall Section and Reinforcement

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<th>Notation</th>
<th>W1</th>
<th>W2</th>
<th>W3</th>
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<td>Thickness (mm)</td>
<td>120</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Horizontal bar</td>
<td>φ9@250</td>
<td>φ9@200</td>
<td>φ9@300</td>
</tr>
<tr>
<td>Vertical bar</td>
<td>φ9@250</td>
<td>φ9@200</td>
<td>φ9@200</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>Single</td>
<td>Single</td>
<td>Double</td>
</tr>
</tbody>
</table>

Fig. 3-5-3 Frame Model (X1, X3,……)

Fig. 3-5-4 The Frame Name System and the Main Frame Members Included in the Analysis

Table 3-5-3 Mechanical Properties of the Foundation Support Springs (unit: kN, m)

<table>
<thead>
<tr>
<th>Spring location</th>
<th>Vertical stiffness</th>
<th>Lateral stiffness</th>
</tr>
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<tbody>
<tr>
<td>Y1 and Y2: X1, X13</td>
<td>4.71×10^5</td>
<td>5.44×10^4</td>
</tr>
<tr>
<td>Y2: X2, X3, X5,X6, X8,X9, X11, X12</td>
<td>6.29×10^5</td>
<td>7.26×10^4</td>
</tr>
<tr>
<td>All others</td>
<td>7.86×10^5</td>
<td>9.07×10^4</td>
</tr>
</tbody>
</table>

Fig. 3-5-5 Input Acceleration A1 and A2 Evaluated at the Building Location
The calculated building responses

The building responses indicate only flexural cracks but no yielding damage and no shear damage in all beams and columns to both the input acceleration waves A1 and A2. Some shear cracks are predicted in the first-story shear walls to both input A1 and A2. Some of the first-story shear-walls reach the shear strength and may have shear failure to the input of A1. Generally, the calculated responses to the evaluated input accelerations identified the building in minor damage and agreed with the observed behavior during the earthquake.

The calculated displacement acceleration and velocity responses at the roof level and the base shear force responses in the building transverse Y-direction are shown in Fig. 3-5-6 and 3-5-7 according to the input acceleration waves. The two input acceleration waves A1 and A2 result in similar response time history but different peak responses. The extreme values of responses are summarized and listed in Table 3-5-4.
Table 3-5-4 Summary of the Maximum Responses (unit: cm, sec)

<table>
<thead>
<tr>
<th>Input acceleration</th>
<th>Base shear (W)</th>
<th>Top D</th>
<th>Top A</th>
<th>Top V</th>
<th>Interstory drift</th>
<th>Torsion(rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>0.457</td>
<td>6.57</td>
<td>775.8</td>
<td>60.21</td>
<td>1/187</td>
<td>0.000376</td>
</tr>
<tr>
<td>A2</td>
<td>0.384</td>
<td>4.28</td>
<td>664.3</td>
<td>45.31</td>
<td>1/299</td>
<td>0.000297</td>
</tr>
</tbody>
</table>

The response of maximum inter-story displacement angle is about 1/300 for the input A2 and about 1/190 for the input A1. The response results are compared in Fig. 3-5-8 with the results by pushover in anti-triangle load pattern. In the figure, across the curves of story shear and inter-story displacement angle by pushover analysis, there are the broken lines that show the beginning crack and reaching shear strength of the shear walls by pushover analysis, and the solid lines with solid circles illustrate the maximum inter-story displacement by dynamic response analysis. The comparison indicates clearly that the building responses mainly in cracks and has no story yielding or severe damage. Some shear walls in the first story reached shear strength in the case of A2 input. Damage to the shear walls is hard to avoid as the high stiffness and relatively poor reinforcement (single reinforced) in most shear wall indicated W1 and W2 in Fig. 3-5-4.

Concluding remarks
The study is carried out to investigate the seismic behavior of a minor-damaged building during the 1995 Hyogo-ken Nanbu Earthquake. Using 3-D frame model based on the nonlinear force-displacement relations of individual structural members, the numerical simulation of the building responses to the evaluated ground motion results in minor damage of cracks and agreed with the observed seismic behavior. The simulated building responses also proved the appropriateness of the evaluated ground motion at the building location.

References

102
3.6 Summary

The chapter presented the case study on evaluating ground motion and simulating the building response and damage by a series of response analyses of deep basin structure, subsurface soil, soil-foundation and superstructure. The results are summarized as following.

The analysis of deep ground using two-dimensional finite element model resulted a reasonable distribution of the evaluated acceleration and velocity on the top of Osaka Group. It becomes well correlated with the distribution of the rate of damaged buildings identified block-by-block after the earthquake. At the point GL-113 meters under the selected objective building, the evaluated PGA and PGV are 460 cm/s$^2$ and 96 cm/s, and the acceleration wave at the point is obtained and used for the input for subsurface soil analysis.

The second step of the analytical study employs the one-dimensional effective stress method to evaluate the responses of subsurface soil and the ground motion characteristics. The soil parameters are inferred from the boring data at the site where the ground motion is to be evaluated. The input makes use of the results from the deep ground analysis. The analysis is first carried out to evaluate the ground motion at an instrumented point on Rokko Island. The estimated ground motions of both acceleration and velocity agreed well with the records at the point. It proves the reliability of the model and method, and the reasonability of the input from the deep ground analysis. Then the ground motion at selected points within the objective building location is estimated in the same way. The results are used for the input of responses analysis of the soil and foundation and the superstructure.

The third step of the study was carryout to simulate the responses of the soil-foundation-building system using two-dimensional effective stress method and applying the cyclic elasto-plastic model for sand to all soil deposits. The model parameters were determined based on the soil properties, which were empirically estimated based on the N-value of standard penetration tests. The results indicate the maximum responses of element moment beyond the crack moment but before the yielding moment. It means that pile foundation has no bending failure. The simulated results coincide with the observed behavior of the building during the earthquake.

The last step of the analytical study is the simulation of the building responses to the evaluated ground motion. Three-dimensional frame model is used in the analysis that is based on the nonlinear force-displacement relations of individual structural members. The building responses indicated that it would suffer cracks but no yielding in all the beams and columns, and almost shear walls except some poor-reinforced first story shear walls. The maximum responses of inter-story displacement are 1/190 to 1/300 according to the input accelerations. The displacement is small than the story yielding displacement. The building responses to the evaluated ground motion could be determined as light damage. The results proved the appropriateness of the evaluated ground motion at the building location.

Through the case study, the reliability of evaluating ground motion and simulating building responses by the analytical technologies developed in individual areas is verified. It supports us to apply the technologies in high seismic-risked area to estimate the ground motion and to predict the damage for a scenario earthquake and provide the information to the society in the area for management and preparedness for future earthquake.
Chapter 4. Case Study on Ground and Building Responses in the 1999 Kocaeli Turkey Earthquake

4.1 Introduction

The Kocaeli, Turkey earthquake hit the western region of Turkey including cities of Izmit in Kocaeli Province, Adapazari in Sakarya Province, and Istanbul, at 00:01, August 17, 1999 (GMT); 03:01 AM in local time. The earthquake occurred on the active North Anatolian fault with the epicenter 17 km below the surface, located west of Izmit at 40.702N and 29.987E. The magnitude was reported 7.4 as moment magnitude and 7.8 as surface wave magnitude from the United States Geological Survey (USGS), USA.

The death toll could reach about 20,000, and the injury tally could climb to around 24,000. The economic losses, although not definitely estimated, have been reported to be about 16 billion US dollars (USD) including socio-economic losses, with physical losses of 5 billion USD for buildings, 2 billion USD for industrial facilities, and another dollars for railway, harbor facilities and highway systems. The heavily damaged zones, where the number of reported death was greater than 2,000 and/or damaged buildings exceeded 20,000, are the provinces of Kocaeli, Golcuk, Sakarya and Yalova, locating along the North Anatolian fault.

In this chapter, we try to evaluate ground motions and building damage distributions during the earthquake using the simulation methodologies presented in the previous chapter 2, based on the information of earthquake fault, ground structures, and Turkish buildings.

In section 4.2, firstly, broadband frequency ground motion simulation of the earthquake is performed in the region around the fault plane. The analytical method described in section 2.1 is used for the simulation. The procedure is also applied to predict bedrock motion in Golcuk at a depth of 300 m.

In section 4.3, based on H/V inversion method reviewed in section 2.2, microtremor measurements are performed in Golcuk, and a two-dimensional $V_S$ structure across the heavily damaged area is estimated.

In section 4.4, with the bedrock motion estimated in section 4.2 and the $V_S$ structure from section 4.3, ground surface motions in Golcuk during the main shock are simulated employing both one- and two-dimensional dynamic response analyses presented in section 2.3 considering site effects.

In section 4.5, finally, response ductility factors of simplified non-linear single-degree-of-freedom (SDOF) building systems are computed for the simulated ground motions in section 4.4.

Throughout the sections 4.2-4.5, the simulated results are compared with the observed ground motions and/or building damage distributions during the earthquake, and the possible cause of the large ground motions and/or serious disaster could be discussed.
4.2 Fault Rupture Process and Strong Motion Modeling

4.2.1 Fault Rupture Process

The 1999 Kocaeli earthquake that struck the western Turkey was a complex rupture characterized by a right lateral strike-slip fault with a moment magnitude of 7.4. The earthquake was produced by the western part of the North Anatolian fault east of the Marmara Sea. Approximately 130 km of surface rupture was observed. To understand the rupture process of the earthquake, a kinematic inversion of the source was performed using the strong motion stations and the fault model shown in Figure 4.2.1 (Sekiguchi and Iwata, 2000).

The rupture started 10 km east from Golcuk city and propagated bilaterally. The total rupture time was about 18 sec (Sekiguchi and Iwata 2000). From their kinematic inversion result a very large rupture velocity of 5.8 km/sec was found for the eastward propagation.

The final slip solution shows three patches of large slip, the first one localized in the bottom of the fault, 10 km west of the hypocenter. The second one 15 km to the east of the hypocenter at the bottom of the fault and the third one localized in the upper part of the fault 40 km to the east of the hypocenter (Figure 4.2.2). The final purpose of this study is to predict the ground motion at Golcuk city in order to be able to compare with the building damage distribution.

![Stations and Fault Model](image)

Figure 4.2.1. Surface break and fault model of the 1999 Kocaeli earthquake. The strong motion stations used for the kinematic source inversion are shown (Sekiguchi and Iwata, 2000).

4.2.2 Strong Ground Motion Modeling

Near field stations modeling

We applied the ground motion methodology described in chapter 2, assuming the asperity contribution as a point source for the low frequency part and a finite area for the high frequency part (Pulido et al 2001), to obtain the ground motion at all the near and intermediate field stations recorded during the Kocaeli earthquake.

From the kinematic source model of the Kocaeli earthquake (Sekiguchi and Iwata, 2000) an initial asperity model was determined for the regions of large slip using the methodology.
described in chapter 2 of this volume. The parameters of this initial asperity model like the rupture velocity, rise time, location and stress drop were modified in order to get an optimum fitting of the broadband frequency waveforms, velocity and acceleration of the near field stations. The final asperity model consisted of 4 patches of large slip (asperities) distributed across the fault plane as shown in Figure 4.2.2.

The rise time of the shallow asperities was found to be larger than the deeper ones. The stress drop was smaller for the shallow asperities like No. 3, where a slight increase in this parameter produces a large increase in the acceleration amplitude.

![Figure 4.2.2. Kinematic model of the 1999 Kocaeli earthquake. Final slip distribution (Sekiguchi and Iwata, 2000). The hypocenter is shown by a star. Horizontal distance is measured from the hypocenter. Arrows denote final slip vector. The preferred asperity model is shown.](image)

<table>
<thead>
<tr>
<th>Asperity</th>
<th>Area (km²)</th>
<th>Seismic Moment M₀ x 10¹⁸ (Nm)</th>
<th>Rise Time (sec)</th>
<th>Rupture Velocity (km/s)</th>
<th>Stress Drop (bar)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.5 x 24</td>
<td>44.10</td>
<td>0.4</td>
<td>2.5</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>8.7 x 12</td>
<td>4.62</td>
<td>1.5</td>
<td>2.9</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>8.7 x 24</td>
<td>7.68</td>
<td>1.5</td>
<td>3.6</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>11.6 x 9</td>
<td>5.89</td>
<td>0.3</td>
<td>3.5</td>
<td>200</td>
</tr>
</tbody>
</table>

Table 4.2.1. List of parameters of asperities.

The final parameters of asperities are summarized in Table 4.2.1. In Figures 4.2.3 and 4.2.4, we can see a good agreement between the simulations and observations at GBZ and YPT stations. The response velocity spectra and acceleration response spectra also show a good agreement in the low and high frequency content.

We obtained that the best fitting of the waveforms was obtained for a model that includes a large rupture velocity inside the asperities in the eastern part (asperities No. 3 and No. 4) compared with a smaller rupture velocity for the asperities in the western part (asperities No. 1 and No. 2). These results are constrained by the stations SKR and GBZ, which are located in a forward directivity direction and whose ground motion is determined mainly by asperities No. 3 and No. 1 respectively. We obtained in general that the amplitude of the acceleration waveforms are mainly controlled by the stress drop and rise time inside asperities while the size and rupture duration of asperities controls the velocity waveforms (rupture velocity inside asperity).
Figure 4.2.3. Comparison between observed and simulated ground motions (thicker line) at GBZ station velocity and acceleration waveforms (upper figures), NS component. Velocity and acceleration response spectra (lower figures).

Figure 4.2.4. Comparison between observed and simulated ground motions (thicker line) at YPT station velocity and acceleration waveforms (upper figures), NS component. Velocity and acceleration response spectra (lower figure).
Figure 4.2.5. Prediction of ground motion at Golcuk G03 at a 300m depth, NS component. Velocity and Acceleration waveforms (upper figures). Velocity and Acceleration Response Spectra (lower figures).

Ground Motion Characteristics at Golcuk

By applying the broadband strong ground motion methodology described in chapter 2 (this volume), we estimated the bedrock ground motion in Golcuk at a depth of 300 m (Figure 4.2.5). We obtained that the ground motion at Golcuk (without site effect) was mainly controlled by the asperity No. 1 (Figure 4.2.2), which produced the 4s pulse that can be seen on the predicted velocity waveform (Figure 4.2.5). The maximum predicted acceleration and velocity were 202 cm/s² and 72 cm/s, respectively.

4.2.3 Summary

We estimated the ground motions at several stations during the 1999 Kocaeli earthquake. The good agreement of the waveforms supports the effectiveness of the hybrid ground motion simulation procedure, in estimating the ground motions in a broadband frequency.

Regarding the source contribution to the ground motion we could say that the amplitude and frequency content of the acceleration waveforms are mainly controlled by the stress drop and rise time inside asperities, whereas the velocity waveforms are controlled by the rupture duration and size of asperities.

The estimated rupture velocity of asperities in the eastern part of the fault was larger than that in the western part.

We successfully estimated a bedrock broadband frequency ground motion at heavily damaged city of Golcuk. In the next chapters we will use this bedrock ground motion as an input motion to evaluate the response of a shallow 2-D velocity structure in order to explain the damage distribution at Golcuk.
References


4.3 Estimation of $V_S$ Structure

In Golcuk, Kocaeli Province, a large number of medium-rise buildings sustained either partial or complete collapse typically of the first soft story (for example, Photo 4.3.1). Figure 4.3.1 shows spatial distribution of collapsed building ratios in Golcuk, which is based on the results of reconnaissance survey performed by Architectural Institute of Japan (AIJ) team (AIJ Reconnaissance Team, 1999). Damage to buildings was concentrated in several areas on the north of Ataturk street, which is the main street running from east to west. The concentration of the building damage could be due to the effects of surface geology on ground motions so-called "site effects." In fact, most of the northern area of the main street is located on a plain while the south is on a hill, where the building damage was slight.

In order to evaluate the site effects quantitatively, two- or three-dimensional shear wave velocity ($V_S$) profiles down to bedrock should be properly determined. It is, however, difficult to estimate multi-dimensional deep $V_S$ profiles using conventional geophysical methods with boreholes.

To that end, the microtremor horizontal-to-vertical (H/V) spectrum method reviewed in the chapter 2.2 can be applicable. Microtremor measurements can be conveniently performed on ground surface without drilling any borehole. In this section, based on the H/V inversion method, microtremor measurements are performed in Golcuk and a two-dimensional $V_S$ structure across the damaged area is estimated.

![Figure 4.3.1. Map showing microtremor observation sites and distribution of damage to buildings in Golcuk (AIJ Reconnaissance Team, 1999).](image-url)
4.3.1 Microtremor Measurement

Microtremor measurements with a three-component sensor are conducted at five sites in Golcuk, hereinafter called Sites G01-G05. These sites are on the line crossing the damaged area and extend from the southern hill to the north of Golcuk, the shore of Izmit bay, as shown in Figure 4.3.1. The distance between two adjacent observation stations are about 240-550 m. Sites G03-G05 are located in the heavily damaged area while Sites G01 and G02 are on the southern hill, where the building damage was slight. There was little traffic density at each observation site, except Site G03 which is close to the main street.

The measurement system used consists of amplifiers, 16-bit A/D converters and a note-type computer, which are built in a portable case, with a three-component velocity sensor unit whose natural period is 1 s (Photos 4.3.2 and 4.3.3). At each site, microtremor ground motions are measured for 5 minutes, and digitized with equi-interval of 0.01 s. Figures 4.3.2 and 4.3.3 show the velocity time series of the vertical and two orthogonal horizontal (N-S and E-W components) microtremor motions recorded at Sites G01 and G05. Ten sets of data with 2048 points each are selected from the digitized motions excluding traffic-induced vibrations, and are used for the following analyses.

Figure 4.3.2. Velocity time series of microtremor ground motions at Site G01.

Figure 4.3.3. Velocity time series of microtremor ground motions at Site G05.
Photo 4.3.1. Damage to buildings in the northern area of Ataturk street.

Photo 4.3.2. Test equipments used in microtremor measurements.

Photo 4.3.3. Microtremor measurement at Site G01.
4.3.2 H/V Spectra of Microtremors

In this chapter, microtremor H/V spectral ratio, \((H/V)_{m}\), is defined as

\[
(H/V)_m = (S_{NS} S_{EW})^{1/2} / S_{UD}
\]

where \(S_{UD}\) is Fourier amplitude of vertical motion, and \(S_{NS}\) and \(S_{EW}\) are that of two orthogonal horizontal motions. In this definition, \((H/V)_{m}\) could correspond to the theoretical H/V spectral ratio of Rayleigh waves, \((H/V)_R\), determined by Eq. (2.2.8) at a site (see chapter 2.2).

The microtremor H/V spectra at Sites G01-G05 are shown in Figures 4.3.4(a)-(c) by solid lines. At Sites G01 and G02 located on the hill, the observed H/V spectra have no significant peaks (Figures 4.3.4(a) and (b)). This suggests that the bedrock could outcrop at these sites. At Sites G03-G05 located on the plain, which are in the heavily damaged area, on the other hand, the observed H/V spectra have significant peaks (Figures 4.3.4(c)-(e)). The H/V peak period increases northward, i.e., from 0.4 s at Site G03 to 1.2 s at Site G05. The variation of H/V peak period suggests that \(V_S\) profile varies drastically along the line between Sites G01 and G05.

![Figure 4.3.4. H/V spectra of microtremors compared with those of Rayleigh waves theoretically computed for the inverted soil layer models at Sites G01-G05.](image)

4.3.3 \(V_S\) Profiling Based on Microtremor Data

Using the H/V data shown in Figure 4.3.4, the inverse analyses reviewed in the chapter 2.2 are conducted, and \(V_S\) structures in the area are estimated. In the inverse analyses, the following presumptions are made: (1) soil model at Sites G01 and G02 is characterized by a half-space while that at other sites by a four-layered half-space; (2) soil models at these sites have a common bedrock; and (3) \(V_S\) values of the soil layers are those from the results of microtremor array measurement near Navy Base (Kudo et al., AJI Reconnaissance Team, 1999). The above assumption leaves only the thickness of the top three layers at Sites G03-G05 unknown. The thickness can be sought so by minimizing the misfits between the H/V ratios of observed microtremor and theoretical Rayleigh waves.

Figures 4.3.5(a)-(c) show the inverted \(V_S\) profiles with standard errors (e.g., Matu’ura and Hirata, 1982) of layer thickness at Sites G03-G05. The dashed-lines in Figures 4.3.4(c)-(e) indicate the theoretical H/V spectra of Rayleigh waves for the inverted soil profiles at the sites. The theoretical H/V spectra are in good agreement with the observed ones. Besides, the evaluated errors of layer thickness at the sites are sufficiently insignificant (Figures 4.3.5(a)-(c)). Figure 4.3.5(d) shows the \(V_S\) profile near Navy Base estimated by the AJI Team (Kudo et al., AJI Reconnaissance Team, 1999). The estimated depth to the bedrock at Sites G04 and G05, where are close to the Navy Base, are consistent...
with those at the Navy Base by the AIJ Team. These results indicate that the estimated profiles could be reasonably reliable, and also confirm that the microtremor H/V method used in this report is accurate for estimating S-wave velocity structure at a site.

The determination of the $V_s$ profiles can result in a geophysical cross section along the line between Sites G01 and G05 as shown in Figure 4.3.6. On the south of the main street, the bedrock with $V_s$ over 1300 m/s outcrops. The bedrock, however, dips on the immediate north of the main street creating a vertical gap of about 100 m and then gently slopes north. The estimated depth to the bedrock in the heavily damaged area ranges from about 100 m to 200 m. With this $V_s$ structure, in the next section, ground motions during the main shock are simulated by one- and two-dimensional dynamic response analyses using the bedrock motions estimated in the previous section 4.2.

![Figure 4.3.5](image)

Figure 4.3.5. (a)-(c) $V_s$ profiles estimated from microtremor H/V method at Sites G03-G05 compared with (d) that from array method near Navy Base (Kudo et al., AIJ Reconnaissance Team, 1999).

![Figure 4.3.6](image)

Figure 4.3.6. Estimated two-dimensional $V_s$ profile along the line passing through Sites G01-G05.
Acknowledgements

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References


4.4 Simulation of Ground Response and Ground Motion Distribution

4.4.1 1- and 2-D Earthquake Response Analysis

With the inverted $V_S$ structure shown in Figure 4.3.6, one- and two-dimensional equivalent-linear dynamic response analyses using finite element method reviewed in the chapter 2.3 are performed. In both 1- and 2-D analyses, maximum element size of each layer is determined so as to keep effective frequency range up to 10 Hz for S-waves, and Hardin-Drnevich (H-D) model is employed for the constitutive model of soil. Soil parameters of each layer used in both 1- and 2-D analyses are shown in Table 4.4.1, in which $\gamma$ is reference strain and $h_{max}$ is maximum damping ratio. In two-dimensional analysis, transmitting and viscous boundaries are used on both sides and at the bottom of the soil model, respectively.

The N-S horizontal ground motion at a depth of 300 m in Golcuk, which is estimated in the previous section 4.2, is used as the input motion for the $V_S$ structure, which is the vertically incident S-waves with in-plain particle motions. The input motion acceleration and velocity used are shown in Figure 4.2.5. Peak acceleration and velocity of the input motions are 202 cm/s$^2$ and 72 cm/s, respectively.

4.4.2 Ground Motion Distribution

Ground surface motions at Sites G01-G05 simulated by using one- and two-dimensional response analyses are shown in Figures 4.4.1(a)-(e), respectively. Figure 4.4.2 shows velocity response spectra of simulated ground surface motions at Sites G01-G05. In the figures, both peak ground accelerations and velocity response amplitudes at Sites G03-G05 are larger than those at the other sites.

Solid and broken lines in Figure 4.4.3 show the variations of peak ground accelerations (PGA) and velocities (PGV) of the horizontal motions simulated by two-dimensional response analysis, respectively. The simulated PGV (broken line) is almost constant along the observation line, and their values are 90-100 cm/s. On the other hand, the PGA (solid line) varies dramatically along the line, and their values lie in a range of 200 to 400 cm/s$^2$. Figure 4.4.4 shows the variation of the collapse ratios of medium-rise reinforced concrete (R/C) buildings along the line (AIJ Reconnaissance Team, 1999). Comparing Figure 4.4.3 with Figure 4.4.4, the PGA values in the heavily damaged zone near Sites G03 and G04 exceed 300 cm/s$^2$, and are larger than those in the other zone. This suggests that the peak ground accelerations could have a significant effect on damage to buildings in the area.

Open circles in Figure 4.4.3 show the PGA of the horizontal motions computed by one-dimensional response analysis at Sites G01-G05. On that figure, the amplification of the PGA values near Sites G03 and G04 from two-dimensional analysis is larger than that from one-dimensional analysis by a factor of 1.3, while the both values at the other sites are almost same. This indicates that the large amplification of the PGA values in the heavily damaged zone near Sites G03 and G04 could be mainly due to that for one-dimensional propagating S-waves so-called “local site effects,” at the same time, however, it could be partly resulted from two-dimensional wave propagating effects, e.g., so-called “basin-edge effects” (Kawase, 1996).
Table 4.4.1. Soil parameters used in 1- and 2-D earthquake response analysis.

<table>
<thead>
<tr>
<th>$\rho$ ($t/m^3$)</th>
<th>$V_P$ (m/s)</th>
<th>$V_S$ (m/s)</th>
<th>$\gamma_f$</th>
<th>$h_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.7</td>
<td>1500</td>
<td>280</td>
<td>$5 \times 10^{-3}$</td>
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</tr>
<tr>
<td>1.8</td>
<td>2000</td>
<td>500</td>
<td>$1 \times 10^{-2}$</td>
<td>0.25</td>
</tr>
<tr>
<td>1.9</td>
<td>2500</td>
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</tr>
<tr>
<td>2.0</td>
<td>3000</td>
<td>1300</td>
<td>$5 \times 10^{-2}$</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Figure 4.4.1. Ground surface motions at Sites G01-G05 simulated by 1- and 2-D earthquake response analyses.
Figure 4.4.2. Velocity response spectra of simulated ground motions at Sites G01-G05.

Figure 4.4.3. Variation of PGA and PGV of the simulated ground motions along the observation line.

Figure 4.4.4. Variation of the collapse ratios of medium-rise R/C buildings along the observation line (AIJ Reconnaissance Team, 1999).
References


4.5 Simulation of Building Damage Distribution

In order to examine the effects of peak ground acceleration on the building damage, maximum response ductility factors of the simulated ground motions are computed for simplified single-degree-of-freedom (SDOF) building systems with elasto-plastic force-displacement relations, and are compared with the building damage distribution during the earthquake.

4.5.1 Characterization of Building System

In the non-linear SDOF analysis, base shear coefficient, \( k \), and damping factor of the building system are required. Base shear coefficient, \( k \), for medium-rise R/C building could be inferred to be 0.3-0.6, based on the studies for Turkish medium-rise buildings during the 1992 Erzincan earthquake (e.g., AIJ, 1993), while the one presented by Turkish building code established in 1975 (Ministry of Public Works and Settlement, Turkey, 1975) could be 0.1 in this area. In order to determine damping factor of the building system, microtremor measurements are performed at two R/C buildings for residential use; one is located near Site G01 and the other about 1 km east of Adapazari downtown. These buildings are both five-storied, almost squared, and have a similar structural system, but Golcuk one is no damaged and the other heavily damaged (Photos 4.5.1-4.5.3). The damaged one in Adapazari subsided by about 0.5-1 m, and this subsidence might be due to soil liquefaction. At the top and ground floors of the both buildings, two orthogonal horizontal (longitudinal and transverse) microtremor motions are observed. The test equipments, sampling conditions, and data processing used are all the same as those in the section 4.3.

Photo 4.5.1. Non-damaged building at which microtremor measurements are performed.

Photo 4.5.2. Heavily damaged building at which microtremor measurements are performed.

Photo 4.5.3. Subsidence and damaged infill wall of the building shown in Photo 4.5.2.
Figure 4.5.1 shows the amplification factors between the top and ground floors of these buildings estimated from spectral analyses of the microtremor data, which are in the most damaged direction of the observed two horizontal motions. In the figure, damping factor of the non-damaged building can be estimated from the observed amplification factors, and the estimated value is 0.06.

Fundamental period of the non-damaged building could be 0.3 s in the figure. In Turkish building code in 1975 (Ministry of Public Works and Settlement, Turkey, 1975), approximate fundamental period of building, $T$ (s) is presented as:

$$T = \min (0.09H / D^{0.5}, \lambda N)$$  \hspace{1cm} (4.5.1)

where $H$ is height of building, $D$ is width of building in motion direction, $N$ is number of stories, and $\lambda$ is constant (0.1 or 0.07 for lower or higher stiffness). As for the non-damaged building, $H$, $D$, and $N$ are 15, 8, and 5, respectively, and $\lambda$ could be 0.1 considering lower stiffness due to infill wall. Using Eq. (4.5.1), fundamental period $T$ for the non-damaged building is then approximated, and the resulted value is 0.5 s. This approximated value is larger than the fundamental period from a microtremor measurement, 0.3 s. One of the causes of this disagreement might be due to the stiffness of infill wall, which might have an effect on micro-vibration characteristics of the building.

In Figure 4.5.1, fundamental period of the damaged building could be 0.7 s, which is larger than that of the non-damaged one by the factor of 2. Assuming that the effects of soil-structure interaction on the observed amplification factors are insignificant and that the dynamic response characteristics of both buildings before the earthquake are almost same, it is suggested that fundamental period of the medium-rise R/C building could double that of the non-damaged building because of structural damage. Thus, it is also indicated that maximum response ductility of the medium-rise R/C buildings might be about/over 2-4 during the 1999 Kocaeli earthquake.

![Amplification Factors](image)

Figure 4.5.1. Amplification factors between the top and ground floors of the damaged and non-damaged buildings, which are in the most damaged direction of the observed two horizontal motions.
4.5.2 Evaluation of Building Damage Distribution

Based on the results presented in the previous section, base shear coefficients, $k$, used in the non-linear response analyses are assumed to be equal to 0.3, 0.4, and 0.5, and damping factor of the SDOF system is 0.06. In the analyses, hysteresis rule of the force-displacement relations of the system is defined by degrading bi-linear model.

Figure 4.5.2(a) shows spatial variations of the computed maximum ductility factor spectra for the SDOF systems along the line using the gradations shown in legend, when $k$ is taken equal to 0.4. Maximum ductility factor spectra in the heavily damaged zone have a significant peak in a period range of 0.2-0.3 s. This peak period is equal to the observed fundamental period of non-damaged medium-rise R/C buildings shown in Figure 4.5.1. Then, spatial variations of the maximum ductility factors computed for the SDOF system with a fundamental period of 0.25 s are shown in Figure 4.5.2(b), when $k$ is 0.3, 0.4, and 0.5. In each base shear coefficient, the spatial variation of the computed ductility factors is in fairly good agreement with that of the collapse ratios of medium-rise R/C buildings shown in Figure 4.4.2 and that of the PGA shown in Figure 4.4.1. Besides, the values of the maximum ductility factors estimated in the heavily damaged zone are over 3, which are consistent with those for the heavily damaged building in the previous section, indicating that the medium-rise Turkish buildings in the zone might be either heavily damaged or collapsed during the main shock. This suggestion from the simulation results corresponds well to the observed building damage distribution during the earthquake as shown in Figures 4.3.1 and 4.4.2.

These above facts reveal that the predicted ground motions and building damage distributions are reliably reasonable and that the damage distributions of the medium-rise R/C building in Golcuk could be mainly controlled by peak ground acceleration during the earthquake as discussed previously.
Figure 4.5.2. Variation of maximum ductility factor spectra computed for the simplified SDOF building systems along the observation line.

Reference

4.6 Summary

We estimated the ground motions during the 1999 Kocaeli earthquake at several stations. The good agreement between the observed and simulated waveforms supports the effectiveness of the hybrid ground motion simulation procedure presented in section 2.1, for estimating the ground motions in a broadband frequency. The amplitude and frequency content of the acceleration waveforms are mainly controlled by the stress drop and rise time inside asperities, whereas the velocity waveforms are controlled by the rupture duration and size of asperities. The estimated rupture velocity of asperities in the eastern part of the fault was larger than that in the western part.

Microtremor measurements using a three-component sensor were performed at five sites in Golcuk along the line crossing the heavily damaged area, and H/V spectra of microtremors were determined for the sites. Inverse analyses of the observed H/V spectra successfully resulted in a two-dimensional $V_S$ profile down to a depth of 300 m in the area. This confirms that the geophysical method using microtremors is an economical and reliable means of estimating multi-dimensional shear wave velocity structure.

With the estimated bedrock motion and $V_S$ profile in Golcuk, ground surface motions in the area during the main shock were simulated employing both one- and two-dimensional response analyses, and then response ductility factor spectra of simplified SDOF building systems in the area were computed for the simulated ground surface motions, and were compared with the observed building damage distributions. The evaluated ground and building responses were consistent with the observed damage distributions.

These above results reveal that the simulated ground motions and building responses are reasonable and reliable, and that the earthquake response simulation methodologies employed in this study could be promising means for evaluating $V_S$ profiles, ground motions, and building responses during earthquakes.
Chapter 5. Prediction of Ground Motion and Building Damage in a Scenario Earthquake for Tottori Region

5.1 Introduction

The ground motion and Structural Performance estimation for earthquake-prone regions is a very important issue for the earthquake disaster mitigation and a very active subject for researchers in natural disaster sciences. The analytical models and methodologies being developed by the Structural Performance Team at EDM could be applied to perform both the strong ground motion prediction for past or future (scenario) earthquakes. In this chapter we selected the Sakaiminato and Yonago cities in the Western part of the Tottori Prefecture and consider a scenario earthquake that could take place in this region to perform the prediction of strong ground motion. We made the prediction of ground motion that could be generated from a fault plane corresponding to the actual fault plane that ruptured during the 2001 Tottori-ken Seibu earthquake, and study the plausible range of ground motions that could be generated by only modifying the earthquake source parameters like asperity location. The actual Tottori-ken Seibu earthquake ($M_{\text{JMA}} = 7.3$, $M_w = 6.6$), hit the Tottori and Shimane prefectures of Western Japan on October 2000 and produced more than 100 injured people and around 900 buildings heavily damaged. The peak ground acceleration close to 1G was recorded at a point in mountainside near the earthquake fault. No serious damage in RC or traditional wooden structures, however, was observed. In the seaside, extensive liquefaction occurred in the reclaimed land around the Yumigahama Peninsula.

We considered two scenario earthquakes for the generation of the broadband frequency bedrock ground motion. The first scenario consists of the same fault plane that ruptured during the actual Tottori-ken Seibu earthquake but considering an asperity located at the northernmost edge of the fault getting very close to Yonago. The second scenario consists of the same fault plane as above described but the asperity is located at southernmost edge of the fault. Since the asperity from the actual Tottori-ken Seibu earthquake is located approximately in between our two-scenario asperity models this will produce a range of predicted ground motions that will be compared with the actual recordings.

For determining the underground structure to be used for the ground motion estimation, microtremor measurements are conducted at 26 sites in Yonago city and H/V spectra of microtremors are determined for the sites. Based on the observed H/V data, a two-dimensional S-wave velocity underground structure in the area is successfully estimated in sections 5.3 and 5.4. Then the ground motion distribution in Yonago region is calculated and presented in section 5.4, using the two-dimensional velocity structure and making the input of the bedrock motion from the first analysis.

Finally in Chapter 5.5 and 5.6, the analysis of ground surface response is carried out to estimate the ground motion for the input to buildings, and the building response analysis is conducted to estimate the damage extent. In ground surface analysis soil liquefaction is included and the analysis is performed at seven sites in Sakaiminato city and Yonago city. The ground surface accelerations obtained then is used for the input of building analysis. The building responses and damage estimation are carried out using 3-dimensional structural model based on the nonlinear force-displacement relations of individual structural members.
5.2 Earthquake Source Characterization and Ground Motion Modeling

5.2.1 Earthquake Source Model

The Tottori-ken Seibu earthquake (Mw=6.1) was characterized by a strike-slip left-lateral fault with almost no surface rupture. The rupture started at the center of the fault and propagated bilaterally. The rupture area was approximately 30 km by 15 km. The kinematic solution of the earthquake (Iwata et al. 2000) shows an asperity slightly south west of the epicenter with a maximum slip of 3 meters. The rupture duration was approximately 9 sec. The final slip distribution from the earthquake is shown in Figure 5.2.2.

5.2.2 Source Model Characterization for Ground Motion Prediction

The purpose of the present work is to make a prediction of broadband frequency ground motion for a scenario earthquake in the Tottori-ken Seibu region. A range of plausible scenario earthquakes that would produce from large to small ground motions at Yonago and Sakaiminato are evaluated (Figure 5.2.1).

Two extreme cases of rupture models are considered for the simulation. The first model consists of the same fault plane that ruptured during the Tottori-ken Seibu earthquake, but the main asperity is located at the northern edge of the fault (asperity model No.1), getting very close to the Yonago area (Figure 5.2.2).

The second source model assumes the same fault plane as in the previous case but the main asperity, which has the same characteristics as before, is located at the southern edge of the fault (asperity model No.2, Figure 5.2.3).

Figure 5.2.1. Asperity models for the prediction of the Scenario earthquake of the Tottori-ken Seibu region.
Figure 5.2.2. Asperity model No.1, the asperity is located in the northern part of the fault. The star shows the starting point of the rupture. The final slip distribution of the Tottori-ken Seibu earthquake is shown.

Figure 5.2.3. Asperity model No.2, the asperity is located in the southern part of the fault. The star shows the starting point of the rupture.

The starting point of the rupture is from the center of the fault as can be seen from Figures 5.2.2 and 5.2.3. In Figure 5.2.1, 5.2.2 and 5.2.3 we can observe that the main asperity of the actual Tottori-ken Seibu earthquake is located approximately in between our two scenario asperity models. For estimating the source parameters like rupture area, asperity area, rise time, stress drop and rupture velocity we used the methodology described in section 2.1. The source parameters used for our simulations are summarized in Table 5.2.1.

5.2.3 Ground Motion Prediction at Yonago and Sakaiminato

Using the asperity models No.1 and No.2 we performed the bedrock ground motion prediction at Yonago and Sakaiminato regions for the simulation points shown in Figure 5.2.4. The ground motions were calculated using the velocity structure shown in Figure 5.2.5, which is the one, used by the Research Center for Earthquake Prediction (RCEP) of DPRI Kyoto University for the location of epicenters. A “shallow” 1000 m thick layer with an S-wave velocity of 1000 km/sec was included, and assumed as the bedrock. The ground motions were calculated at a depth of 100m. In the chapter 5.5 the nonlinearity effect of the shallow layers (<100 m) into the ground motion is studied.
Table 5.2.1. Source Parameters for the Tottori-ken Scenario Earthquake.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rupture area (km²)</td>
<td>18 x 27</td>
</tr>
<tr>
<td>Fault Mechanism</td>
<td>150,900,0</td>
</tr>
<tr>
<td>Seismic moment (Nm)</td>
<td>1.02 x 10¹⁸</td>
</tr>
<tr>
<td>Number of asperities</td>
<td>1</td>
</tr>
<tr>
<td>Asperity area (km²)</td>
<td>12 x 9</td>
</tr>
<tr>
<td>Seismic moment asperity (Nm)</td>
<td>0.48 x 10¹⁸</td>
</tr>
<tr>
<td>Stress Drop (Mpa)</td>
<td>11</td>
</tr>
<tr>
<td>Asperity Rise Time (sec)</td>
<td>0.4</td>
</tr>
<tr>
<td>Rupture velocity outside asperity (km/sec)</td>
<td>2.6</td>
</tr>
<tr>
<td>Rupture velocity inside asperity (km/sec)</td>
<td>2.2</td>
</tr>
<tr>
<td>Starting Point of the rupture</td>
<td>Center of the fault</td>
</tr>
<tr>
<td>Number of subfaults inside asperity</td>
<td>12</td>
</tr>
<tr>
<td>Average S-wave velocity (km/sec)</td>
<td>3.4</td>
</tr>
<tr>
<td>Average density (kg/m³)</td>
<td>2690</td>
</tr>
<tr>
<td>I/Q (Ω) (for Tottori region)</td>
<td>0.0312 f⁻¹.⁴⁷⁵²</td>
</tr>
<tr>
<td>Frequency range predicted ground motions</td>
<td>0.1 to 10.0 Hz</td>
</tr>
</tbody>
</table>

Ground Motion Characteristics at Yonago

In Figure 5.2.6 we can observe a comparison between the bedrock ground motions generated from the asperity model No.1 and No.2 at the K-net Yonago site (TTR008). We can appreciate that the peak ground acceleration and velocity from the asperity model No.1 are much larger than the ones from the asperity model No.2. We can also observe that the acceleration and velocity response spectra are larger for the asperity model No.1 compared with the asperity model No.2 (Figure 5.2.7). The waveforms from the asperity model No.2 are characterized by a larger duration and lower frequency content than the ones from asperity model No.1. The actual recording at TTR008 is also shown for comparison (Figure 5.2.6). A comparison with the ground motion including the nonlinearity of the shallow layers (<100 m) at the K-net Yonago site will be made later.

Figure 5.2.4. Location of the stations where the strong ground motion simulations were performed at Sakaiminato and Yonago.

130
Figure 5.2.5. Velocity structure model for the bedrock strong ground motion simulations. The bedrock ground motion are calculated at -100 m level. A shallow velocity structure (<100 m) for including the site effect, nonlinearity and liquefaction in the simulated ground motions will be consider in the section 5.5.

The peak ground acceleration at all the 4 simulation points at Yonago (NS and EW components) from the asperity model No.1 are much larger than the ones from the asperity model No.2 (Figure 5.2.8). A similar observation can be made from the comparison of the acceleration response spectra of the predicted ground motions at Yonago (Figure 5.2.9).

We can also observe that the waveforms from asperity model No.1 have much more high frequency content compared with the waveforms from asperity model No2. This can be explained by a forward directivity effect towards Yonago from asperity model No.1 compared with the backward directivity effect from asperity model No.2.

In the case of the predicted velocity waveforms we can appreciate as before that the amplitude of the NS component ground motions from asperity model No.1 are much larger than the waveforms from the asperity model No.2 (Figure 5.2.10).

The velocity ground motion of the EW components from the asperity model No.1 are similar in amplitude compared with the EW waveforms from asperity model No.2. The velocity spectra of the waveforms from asperity model No.1 are in general larger or equal than the spectra from asperity model No.2.

All the previous observations could be explained by the fact that in model No.1 the asperity is located much closer to the Yonago region compared with the asperity in model No.2. This produces a forward directivity from the asperity model No.1 compared with a backward directivity from asperity model No.2, which produces larger amplitudes and higher frequencies in the waveforms from asperity model No1 compared with the waveforms from asperity model No.2.
Ground Motion Characteristics at Sakaiminato

The Sakaiminato predicted acceleration waveforms from asperity model No.1 are as for the Yonago case larger than the acceleration amplitudes from the asperity model No.2 (Figure 5.2.12). A similar observation can be made from the comparison of the respective acceleration response spectra.

The acceleration waveforms also show a reduction in amplitude compared with the Yonago waveforms due to the attenuation through a larger distance from the asperities.

The Sakaiminato waveforms at the different simulation points look very similar, in part because they are very close each other and relatively far away from the asperity. These waveforms will be used later on as an input bedrock ground motion to study the liquefaction at Sakaiminato (section 5.5).

5.2.4 Bedrock vs. Nonlinear Ground Motions

The nonlinearity effect of the shallow low velocity layers (<100m) (section 5.5) is included here for comparison with the input bedrock ground motion at the K-net Yonago site TTR008 (Figure 5.2.14).

As we can observe in Figure 5.2.14 the amplitude of the acceleration waveforms is very similar compared with the bedrock ground motion acceleration amplitudes (Figure 5.2.6). However if we compare the velocity amplitudes, the velocity waveforms from the nonlinear calculation experience a large increase in amplitude compared with the bedrock velocity waveforms for both the asperity model No.1 and No.2 as we can see from Figures 5.2.6 and 5.2.14.

As for the frequency content of the waveforms, we can observe that the inclusion of the nonlinearity brings a very large reduction in the high frequency content whereas the low frequency content is increased (Figure 5.2.15), compared with the bedrock ground motion spectra (Figure 5.2.7), for both asperity models No.1 and No.2.

5.2.5 Summary

We studied the influence of the location of asperities in the fault plane of a scenario Tottori earthquake into the simulated broadband frequency strong ground motion and compare it with the actual recordings from the Tottori-Seibu earthquake.

We obtained that the location of the asperities in the fault plane has a large influence into the amplitude and frequency content of the predicted waveforms.

The difference in amplitude of the waveforms at the Yonago region can be as large as 3 times depending on whether we consider an asperity with identical characteristics, located in the northernmost or southernmost part of the fault plane ruptured during the Tottori-ken Seibu earthquake.

The inclusion of the nonlinearity effect into the ground motion produces a large change in the frequency content of the waveforms. A large decrease in the high frequency and a large increase in the low frequency content of the waveforms are observed.
Figure 5.2.6. Comparison of the bedrock (-100 m) strong ground motion prediction from asperity models 1 and 2 at the TTR008 station (K-net Yonago) with the actual recording (Acceleration and Velocity). No site effect is included in the simulations.

Figure 5.2.7. Comparison of the bedrock (-100 m) strong ground motion prediction spectra from asperity models 1 and 2 at the TTR008 station (K-net Yonago) with the actual recording (Acceleration and Velocity response Spectra). No site effect is included in the simulations.
Figure 5.2.8. Comparison between the predicted bedrock strong ground motions from asperity models 1 and 2 at the Yonago simulation points (Acceleration waveforms).

Figure 5.2.9. Comparison between the predicted bedrock strong ground motions spectra from asperity models 1 and 2 at the Yonago simulation points (Acceleration response spectra).
Figure 5.2.10. Comparison between the predicted bedrock strong ground motions from asperity models 1 and 2 at the Yonago simulation points (Velocity waveforms).

Figure 5.2.11. Comparison between the predicted bedrock strong ground motion spectra from asperity models 1 and 2 at the Yonago simulation points (Velocity spectra).
Figure 5.2.12. Comparison between the predicted bedrock strong ground motions from asperity models 1 and 2 at the Sakaiminato simulation points (Acceleration waveforms).

Figure 5.2.13. Comparison between the predicted bedrock strong ground motion spectra from asperity models 1 and 2 at the Sakaiminato simulation points (Velocity spectra).
Figure 5.2.14. Comparison of the strong ground motion prediction from asperity models 1 and 2 at the TTR008 station (K-net Yonago) with the actual recording (Acceleration and Velocity). The nonlinearity effect of the shallow layers (<100 m) is included in the simulations.

Figure 5.2.15. Comparison of the strong ground motion spectra prediction from asperity models 1 and 2 at the TTR008 station (K-net Yonago) with the actual recording (Acceleration and Velocity). The nonlinearity effect of the shallow layers (<100 m) is included in the simulations.
References


Research Center for Earthquake Prediction, Disaster Prevention Research Institute, Kyoto University, Report on the Western Tottori earthquake, http://www2.rcep.dpri.kyoto-u.ac.jp/TOTTORI/index_j.html
5.3 Estimation of $V_S$ Structure

Ground motion characteristics during earthquakes are mainly controlled by S-wave velocity ($V_S$) structures at a site, and the effects of subsurface geology on ground motions are called "site effects" or "local site effects." In order to evaluate the site effects quantitatively, two- or three-dimensional $V_S$ profiles down to bedrock should be properly determined. It is, however, difficult to estimate multi-dimensional deep $V_S$ profiles using conventional geophysical methods with boreholes.

For this purpose, the microtremor horizontal-to-vertical (H/V) spectrum method reviewed in the chapter 2.2 can be applicable. Microtremor measurements can be conveniently performed on ground surface without drilling any borehole. In this section, based on the H/V inversion method, microtremor measurements are conducted in Yonago city and $V_S$ structures in the area are then estimated.

5.3.1 Microtremor Measurement

Microtremor measurements with a three-component sensor are conducted at 26 sites in Yonago city, hereinafter called Sites A01-A26, as shown in Figure 5.3.1. These sites are on the line between two strong ground motion observatories, one was installed by K-net system of National Research Institute for Earth Science and Disaster Prevention (K-net, NIED) and the other was of Japan Meteorological Agency (JMA). The locations of strong motion stations correspond to Site A08 and A16, respectively. The observation line extends from Yonago port north-eastward to a shore of Miho bay as shown in Figure 5.3.1. The distance between two adjacent observation sites are about 40-400 m, depending on variations of microtremor H/V spectra with distance and on other geological information. At each observation site, traffic was not very heavy.

The measurement system used consists of amplifiers, filters, 24-bit A/D converters, and a note-type computer, which are built in a portable case, with a three-component velocity sensor unit whose natural period is 2 s, and the system is shown in Photo 5.3.1. At each site, microtremor ground motions are measured for 5-10 minutes, and are digitized with equi-interval of 0.01 s. Figure 5.3.2 shows velocity time series of vertical and two orthogonal horizontal (N-S and E-W components) microtremor motions recorded at Site A12. Eight-16 sets of data with 2048 or 4096 points each are selected from the digitized motions excluding traffic-induced vibrations, and are used for the following analyses.

Photo 5.3.1. Test equipments used in microtremor measurement.
Figure 5.3.1. Map showing microtremor and strong motion observation sites (Refer to color figure 2).

Figure 5.3.2. Velocity time series of microtremor ground motions at Site A12.
5.3.2 H/V Spectra of Microtremors

In this chapter, microtremor H/V spectral ratio, \((H/V)_m\), is defined as

\[
(H/V)_m = \left( S_{NS}^2 + S_{EW}^2 \right)^{1/2} / S_{UD}
\]  

(5.3.1)

where \(S_{UD}\) is Fourier amplitude of vertical motion, and \(S_{NS}\) and \(S_{EW}\) are that of two orthogonal horizontal motions. In this definition, \((H/V)_m\) could correspond to the theoretical H/V spectral ratio of surface waves, \((H/V)_S\), determined by Eq. (2.2.9) at a site (see chapter 2.2).

The observed microtremor H/V spectra at several sites are shown in Figure 5.3.3 in solid lines. At Site A03 near Yonago port, no apparent peak exists in the observed H/V ratio, suggesting that bedrock could outcrop at the site. At the other sites, however, the observed H/V spectra have significant peaks, and the peak periods of H/V spectra vary with a range of 0.4-1.2 s. The peak periods changes largely, in particular, between Sites A03 and A08, and Sites A10 and A16. At Sites A20 and A26, besides, two distinct peaks might be found in the observed H/V spectra. The variation of H/V peak period suggests that \(V_S\) profile varies drastically along the observation line.

To investigate whether such a trend exists in the area, the variation of H/V spectra with distance is shown in Figure 5.3.4. In the figure, the value of H/V ratio is indicated by gradation as shown in the legend. The figure confirms that the above trend exists throughout the area in which the peak period increases abruptly from 0.2 to 1-1.2 s between Sites A05 and A08-A10 and then gently decreases north-eastward to 0.4-0.6 s, and two peaks could be distinguished at Sites A20-A26.

![Figure 5.3.3. H/V spectra of microtremors compared with those of theoretical surface waves.](image-url)
5.3.3 $V_S$ Profiling Based on Microtremor Data

Using the H/V data observed at Sites A08 and A16, K-net and JMA sites, respectively, the inverse analyses reviewed in the chapter 2.2 are conducted on the presumption that soil profile at each site consists of 5 layered half-space and that $V_S$ values of shallow soil layers are from bore-hole data near the sites (e.g., K-net Homepage, NIED).

Figures 5.3.5(a) and (b) show the inverted $V_S$ profiles down to a depth of 60 m at Sites A08 and A16, respectively. Soil type of each layer is evaluated from boring data near the sites, and is also shown in the figure. Solid lines in Figures 5.3.3(b) and (d) are the theoretical H/V spectra of surface waves, $(H/V)_S$, computed for the inverted profiles at the sites. At each site, the computed H/V spectrum is in fairly good agreement with the observed one, indicating that the estimated $V_S$ profiles are reasonably reliable.

Based on Figures 5.3.3 and 5.3.5, it is indicated that the longer the H/V peak period is, the deeper the depth to the bedrock layer with $V_S$ about 1000 m/s becomes. Figure 5.3.6 shows the relation between the observed H/V peak periods, $T_p$, and the estimated bedrock depth, $D_B$, at Sites A08 and A16. The figure implies that there could be a good correlation between $D_B$ and $T_p$ defined as

$$D_B \approx C \cdot T_p$$  \hspace{1cm} (5.3.2)

in which $C$ is about 50 for the sites (see Figure 5.3.6). Similar relationships between microtremor H/V peak period and bedrock depth at a site can be found in recent studies (e.g., Arai et al., 1996; Tokimatsu et al., 1998). Thus, the bedrock depth in the area may be approximately estimated using Eq. (5.3.2), when microtremor H/V spectrum at a site has a significant peak. From Figure 5.3.4 and Eq. (5.3.2), it could be indicated that the bedrock dips sharply between Sites A05 and A08 creating a vertical gap of about 40-50 m, while the bedrock outcrops on the southwest of Site A05. The bedrock depth then gently decreases north-eastward to about 20-30 m at Sites A16-A18.
Figure 5.3.5. S-wave velocity profiles inverted from microtremor H/V spectra at Sites A08 and A16.

Figure 5.3.6. Correlation of H/V peak period with bedrock depth.
5.3.4 Summary

Microtremor measurements using a three-component sensor are performed at 26 sites along the line between two strong motion stations in Yonago city, and H/V spectra of microtremors are determined for the sites. The variation of H/V spectra indicates that $V_S$ profile varies drastically along the observation line. Besides, inversion of the observed H/V spectra successfully results in $V_S$ profiles down to a depth of 60 m at the strong motion stations. These confirm that the geophysical method using microtremors is an economical and reliable means of estimating shear wave velocity structure at a site.

References


5.4 Simulation of Ground Response and Ground Motion Distribution

5.4.1 Outlines of Analyses

As shown in the 1995 Hyogo-ken Nanbu Earthquake, ground motion distribution under earthquake could be greatly affected by deep and shallow ground structures. Herein, with the inverted $V_s$ structure in Yonago city center, two-dimensional equivalent-linear dynamic response analyses are to be performed using the finite element method in frequency domain (Lysmer et al., 1975), in order to estimate the ground motion distribution under scenario earthquake.

East side area of Yonago city spreads on the mouth of Hino River, and the main town is located in the western side of the city, faced to Naka-Umi. As for the soil layer under old town area, sediment composed of gravel or sand is not often observed, but volcanic sediment is observed. These situations had been caused by gentle flow of Hino River. In this area, weathered rock is observed in the depth of about 50m.

For the purpose of estimating the ground motion distribution around Yonago old town area, range with 3km length along the microtremor measurement line (as shown in Figure 5.4.1) was selected for the analysis. The line almost agrees with fault normal direction, comparing with assumed source model (e.g., Sekiguchi and Iwata, 2000). Regarding the inverted $V_s$ structure in K-net and JMA observation site (as shown in Chapter 5.3), we made two-dimensional equivalent-linear model of cross section along the line passing through both sites as shown in Figure 5.4.2. In order to determinate the spatial distribution of bedrock depth in analysis model, distribution of $H/V$ (shown in Figure 5.3.4) was used complementarily. That is, the thickness of subsurface layer was determined approximately by Eq. (5.3.2).

![Figure 5.4.1. Map showing microtremor observation sites and location of model section.](image-url)
Using the model, dynamic response analyses are performed by the finite element method in frequency domain. Maximum element size of each layer was determined so as to keep the effective frequency range up to 7Hz for shear waves, and transmitting and viscous boundaries were used on both sides and at the bottom of the soil model, respectively. In order to take the nonlinear behavior of subsurface soils into account, Hardin-Drnevich (H-D) model was inferred for strain-stiffness and strain-damping relationships of soils. Soil parameters of each layer used in the analysis are shown in Table 5.4.1, in which $\gamma_t$ is reference strain and $h_{\text{max}}$ is maximum damping factor.

As the input motion, the fault normal component of simulated broadband outcrop motion (Chapter 5.2) at the depth of 100m, was used; thus, which is the vertically incident S-waves with in-plain particle motions. The input motion acceleration used is shown in Figure 5.2.6. Peak acceleration of the input motion is 493cm/s$^2$.

Table 5.4.1. Soil parameters used in the dynamic response analysis.

<table>
<thead>
<tr>
<th>$\rho$ (t/m$^3$)</th>
<th>$V_p$ (m/s)</th>
<th>$V_s$ (m/s)</th>
<th>$\gamma_t$</th>
<th>$h_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>1460</td>
<td>90</td>
<td>$4 \times 10^{-4}$</td>
<td>0.25</td>
</tr>
<tr>
<td>1.7</td>
<td>1460</td>
<td>200</td>
<td>$5 \times 10^{-4}$</td>
<td>0.25 / 0.15</td>
</tr>
<tr>
<td>2.0</td>
<td>2100</td>
<td>500</td>
<td>$5 \times 10^{-4}$</td>
<td>0.25</td>
</tr>
<tr>
<td>2.1</td>
<td>2500</td>
<td>1000</td>
<td>$4 \times 10^{-4}$</td>
<td>0.25</td>
</tr>
</tbody>
</table>

5.4.2 Results of Analyses

Figure 5.4.3 shows the pseudo velocity response spectrum of simulated outcrop motion. In the figure, spectrum of observed motion at JMA site is also shown, that were recorded during 2000 Tottori-ken Seibu Earthquake [10/6/2000, Mw=6.6]. Simulated outcrop motion is larger than in the 2000 Earthquake despite its assumed Mw is same with that, which may be explained by closer asperity than actual condition. Simulated motion has remarkable acceleration and velocity response in the period of 0.2 s and 1.3s, respectively.

Special distribution of amplification factors along the model line are shown in Figure 5.4.4, that is computed from amplitude ratios between Fourier spectra of the simulated ground motions and those of the input motions. The amplification factors at outcrop area (X=0 to 200) are almost unity for all periods, while those at basin are larger than unity, and their peak periods vary according to thickness of sedimentary layer. Amplifications in the period of 1s and from 1s to 2s are remarkable in the basin edge (X=200 to 500m) and the old
town area (X=600 to 1600m), respectively.

Figure 5.4.5 shows the variation of peak ground accelerations (PGA) and velocities (PGV) of the simulated horizontal motions along the model line.

The simulated PGA and PGV are remarkably large in basin edge (X=300m) and old town area (X=600 to 1600m), respectively. These are in consequence of input motion and amplification characteristics mentioned above. In the end of sediment area that accumulated
like a shape of convex lens (X=600, 1400m), PGA is larger than nearby area and the value are about 100 cm/s. As shown in the paste-up of velocity time histories along the model line (Figure 5.4.6), long period pulse of 1.3s, that was dominant in the input motion (see Figure5.4.3), is amplified especially in the basin of old town area.

![Graph showing peak ground accelerations (PGA) and velocities (PGV) vs. distance](image)

**Figure 5.4.5.** Variation of peak ground accelerations (PGA) and velocities (PGV) of the simulated horizontal motions along the model line.

![Graph showing velocity time histories](image)

**Figure 5.4.6.** Paste-up of velocity time histories along the model line.

**Acknowledgements**

The strong motion record at JMA Yonago station used in this study was provided by Japan Meteorological Agency. The authors express their sincere thanks to the organization.
References


5.5 Soil Liquefaction and Ground Motion

The soil liquefaction and ground surface responses at seven sites in Sakaiminato and Yonago City were predicted in this section. We estimated not only the occurrence of liquefaction, but also the ground deformation by the effective stress analyses described in section 2.4. The 3-dimensional dynamic finite element analyses were carried out using bi-directional input motions in the case of the northern asperity obtained in section 5.2, because the amplitude of the input motions in northern asperity case were much larger than that in southern asperity case.

5.5.1 Analyses Sites

The locations of seven sites for the analyses are shown in Figure 5.5.1. The sites of S1-S4 are located in Sakaiminato City, and the sites of Y1-Y3 are located on the line from Yonago City to Miho Bay. The base map shows the surface geology (Institute of Geological Survey, 1982). The white colored area in the land shows a sedimentary layer and all of the seven sites are located on the sedimentary layer. The Yumigahama Peninsula is a large sandbar, which was composed, of sands from the Hino River carried by the long shore current. The deposited sand layer has the depth of about 10 m, and is made of uniform medium and coarse sand.

The soil profiles for seven sites are shown in Figure 5.5.2. These soil profiles were summarized based on the past boring data (Ministry of Construction et al, 1967; Research Committee of soils and foundations in Chugoku District, 1995). The soil properties of the mean SPT-N values, fine particle contents, plastic indexes and liquefaction strengths for each layer are tabulated in Table 5.5.1. The liquefaction strengths were estimated from SPT-N values and fine particle contents by the empirical method (Japan Road Association, 1996). The soil conditions can be summarized as follows before the analysis.

**Upper sand layer (Um, Us):** The alluvial upper sand layers are shown at all sites. The Um and Us layers consist of uniform sand with less fine particle. The soft Um and Us layers with low N values have high liquefaction potential, because the underground water level is very high at all sites.

**Upper clay layer (Uc):** The alluvial upper clay layer is shown at only Y1 in downtown of Yonago City. The sedimentary structure at Y1 is very complicated and the Us layer has much fine particle because the sedimentary condition is different from other sites.

**Lower sand layer (Ls):** The diluvial lower sand layers are shown at the western sites in Sakaiminato City and Yonago City. The Ls layers consist of uniform sand with less fine particle as same as the upper sand layers. The liquefaction potential is low, because the Ls layers have large N values.

**Lower clay layer (Lc):** The thick Lc layers deposit in Sakaiminato City. Although the Lc layers are overconsolidated, N values are relatively small.

**Lower gravel and clay layer (Lm):** The alternation gravel and clay layers are shown at all sites. The N values of Lm layers are over 30, therefore the common structures are supported by Lm layers.

**Base rock (B):** The B layer was treated as the base layer for acceleration input in the analyses, assuming that the shear velocity of the B layer was 1000 m/s. The depth of the B layer at S4 is about 70 m. The depth of the B layer at other sites in Sakaiminato City was assumed to be 70 m as well as S4, because the deep boring data was not reported. The depth of the B layer in Yonago City become deeper toward to Miho Bay.
Figure 5.5.1. Location of analyses sites.

Figure 5.5.2 (a). Soil profiles in Sakaiminato City.
Figure 5.5.2 (b). Soil profiles in Yonago City.

Table 5.5.1. Soil properties.

<table>
<thead>
<tr>
<th>Site</th>
<th>Layer</th>
<th>Layer of N-value</th>
<th>Averaged N-value</th>
<th>$F_c$ (%)</th>
<th>$T_d$</th>
<th>$V_s$ (m/s)</th>
<th>$R_c$</th>
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<tr>
<td>S1</td>
<td>Us</td>
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<td>4</td>
<td>90</td>
<td>50</td>
<td>171</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lm</td>
<td>30</td>
<td>50</td>
<td>0</td>
<td>300</td>
<td></td>
<td></td>
</tr>
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<tr>
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<td>0</td>
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<td>Us</td>
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<td>50</td>
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<td>300</td>
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<td>0</td>
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<td>0</td>
<td>300</td>
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</tr>
</tbody>
</table>
5.5.2 Numerical Conditions

The 3-dimensional soil column at the seven sites was modeled by the rectangular solid finite elements. The bottom of the model, which was the top of the base rock, was the viscous boundary with the shear wave velocity of 1000 m/s, and the four nodes at the same depth were constrained to yield the same displacement for all components. The underground water surface was drained boundary, and the other boundaries were impermeable. The predicted accelerations for two components (NS and EW directions) at the base rock were input from the bottom of the FE model. Figure 5.5.3 shows the input accelerations in EW and NS directions. We used the predicted accelerations for the northern asperity in section 5.2, because the amplitude of the acceleration for the northern asperity was much larger than that for the southern asperity. The EW components in Sakaiminato sites were twice larger than the NS components. The maximum acceleration in EW components was about 130 Gals. The input acceleration at Y1 was largest because the Y1 was closest to the earthquake fault. The maximum acceleration at Y1 was about 450 Gals in NS component. Rayleigh damping proportional to initial stiffness, which was determined by assuming that the damping factor is 2%, was used in order to obtain numerical stability in the high frequency domain. The increment for time integration was 0.002 seconds.

Figure 5.5.3 (a). Input motions in Sakaiminato City.
The cyclic elasto-plastic model for sand described in section 2.5.2 was applied to Um and Us layers, which had liquefaction potential. The other layers were modeled by Ramberg-Osgood model, because the large shear strain might not occur in these hard layers. The parameters were determined based on the soil properties as shown in Table 5.5.1, in which the liquefaction strengths and shear wave velocities (Imai, 1977) were empirically estimated from the N values of standard penetration tests. The liquefaction parameters for Um and Us layer were determined to reproduce the liquefaction strength, which was the cyclic shear stress ratio for the double amplitude shear strain of 5%. Although the model parameters should be determined on the mechanical laboratory tests such as undrained cyclic shear tests with undisturbed samples, the liquefaction strength was empirically estimated from the N values because no laboratory tests were carried out. The parameters for other hard layers were determined in order to describe the dynamic deformation characteristics, which were the relations between shear modulus, damping and the shear strain amplitude, for typical Japanese sand and clay.

Figure 5.5.3 (b). Input motions in Yonago City.
5.5.3 Numerical Results

The time histories of response acceleration on the ground surface at seven sites are shown in Figure 5.5.4. Figure 5.5.5 shows the distributions in depth of 1) the relative effective stress ratio (R.E.S.R.) after the earthquake, 2) the maximum shear strain, 3) the maximum acceleration and 4) the maximum displacement for seven sites. The R.E.S.R. was calculated by \((1 - \sigma'_u / \sigma'_w)\), and becomes 1.0 when the mean effective stress \(\sigma'_u\) becomes almost zero, which means liquefaction. The horizontal two components of the maximum shear strain, acceleration and displacement were combined to one component by root mean square. As the numerical results, the following soil behavior was predicted as shown in Figure 5.5.6.

1) Liquefaction occurred in Um or Us layers at S3 and S4 sites in Sakaiminato City. However, the maximum shear strain in the liquefied layers was smaller than that in Lc layers, and was not very large. Therefore, the ground surface displacement was less than 20 cm. Liquefaction might not cause the serious damage such as the building settlement and pile rupture.

2) Extreme liquefaction occurred in Us layer at Y1 site in Yonago City. Liquefaction caused the large shear strain and ground surface displacement. The complete liquefaction occurred at about 13 seconds, and the amplitude of the response acceleration after that decreased as shown in Figure 5.5.4 (b). However, the liquefied Us layer was not thick, and the upper non-liquefied layer with the width of 5 m covered the Us layer. Therefore, the structural damage might be generated only for the underground structure such as a pile foundation.

3) Shallow liquefaction occurred in Us layer at Y2 site in Yonago City. The maximum shear strain and displacement was not large, because the width of the liquefied layer was not thick and the amplitude of the input motion was not as large as that at Y1. Therefore, the structural damage might be generated only for the light structure on the ground such as houses and pipelines.

The building response on the ground surface at Y1 in Yonago City was discussed in the following section 5.6. The recalculated response acceleration without liquefaction was used as the input motion for the building response analyses, because the liquefied Us layer was not thick and might not exist widely in Yonago City.
Figure 5.5.4 (a). Time histories of response ground surface accelerations in Sakaiminato City.

Figure 5.5.4 (b). Time histories of response ground surface accelerations in Yonago City.
Figure 5.5.5 (a). Distribution of response values in Sakaiminato City.
Figure 5.5.5 (b). Distribution of response values in Yonago City.
Figure 5.5.6. Liquefaction and damage.

5.5.4 Summary

The soil liquefaction and ground surface responses at seven sites in Sakaiminato and Yonago City were predicted in this section. We estimated not only the occurrence of liquefaction, but also the ground deformation by the effective stress analyses described in section 2.4. The 3-dimensional dynamic finite element analyses were carried out using bi-directional input motions in the case of the northern asperity obtained in section 5.2. Liquefaction and damage summarized in Figure 5.5.6 were predicted as the result of the numerical analysis.

1) Liquefaction occurred in Um or Us layers at S3 and S4 sites in Sakaiminato City. However, liquefaction might not cause the serious damages such as the building settlement and pile rupture.
2) Liquefaction occurred in Us layer at Y1 and Y2 sites in Yonago City. Liquefaction might cause the structural damages for underground structures such as pile foundations at Y1 site and for light structures on the ground such as houses and pipelines at Y2 site.

References


5.6 Estimation of Building Response and Damage

Introduction

To estimate the building response and damage to the predicted ground motion in the Western Tottori Area, three building models, 5-story and 10-story reinforced concrete (RC) shear-wall apartment buildings and 20-story RC/SRC frame office building, are used in the analytical study. Such building models may not be seen in that area right now. However, as the research aims to evaluate the ground motion and estimate the damage to future earthquake in any seismic-risked areas, the buildings generally designed and constructed based on the current design code in Japan are assumed to encounter the scenario earthquake.

To estimate the possible damage of the building models the predicted ground motion from the results of surface soil responses (see section 5.5) are used here as the input to calculate the responses of the building models. The response analysis is carried out using three-dimensional frame model based on the nonlinear force-displacement relations of individual structural members. The results show that the responses depend on the dynamic characteristics of the building models. According to the aspect of dominant frequency of ground motion, different responses are evaluated among low and middle-rise buildings and high-rise buildings.

Building models in the objective area

Three building models shown in Fig. 5-6-1 are used in the analysis. They are existent buildings designed and constructed during the last decade in Japan. They are supposed taking place in the concerned area in Yonago-city, where the predicted ground acceleration waves are shown in the figure as well.

The building model A5 is a 5-story RC frame shear-wall apartment building. The detail of the building model can be found in the section 3.5 Simulation of Building Response and Damage.

The building model A10 shown in Fig. 5-6-2 and 5-6-3 is a RC frame shear wall apartment building as well, which is 10-story or 28.15 m high, long rectangle floor plan 78.5x15.5 m². Shear wall is used as the boundary of residential units in the transverse direction, while beam-column frame is in the longitudinal direction. The beam, column and shear wall are in equal thickness. It is a new type of building structure developed and promoted in Japan since later 1980s. Therefore, the research and development for the design and construction of this kind of building structure in detail have been carried out to make the beam-column frame in moment-resistant mechanism and to ensure the capacity of the shear walls. The typical member sections are shown in Fig. 5-6-4 and the list of the section detail of structural members is given in Table 5-6-1 to 5-6-3, and the material properties in Table 5-6-4. The structural weight concentrated at each floor level is shown in Table 5-6-5. The fundamental vibration period of the building model A10 is 0.49 second in longitudinal frame direction, and 0.38 second in transverse shear wall direction.

The building model S20 is an office building. As shown in Fig. 5-6-5 and 5-6-6, it is regular beam-column frame structure, rectangular floor plane in 42x23 m², 7 spans with equal span length 6 meters in longitudinal direction, and 3 spans with span length from 7 to 8 meters in transverse direction. The building is 20-story or 66.15 m high. The lower stories (from floor level 1F up to 6F) are steel reinforced concrete (SRC) while upper stories are reinforced concrete (RC). Detail of the beam and column member sections is shown in Fig. 5-6-7 and 5-6-8 and listed in Table 5-6-6. The material properties are given in Table 5-6-7. The base shear capacity is 0.25 W (W= the building total weight) in design of this building model.
The fundamental vibration period is 1.33 sec in transverse direction and 1.26 sec in longitudinal direction. More details of the building model can be found in reference [2].

The building models selected for the study is based on the consideration of the response spectra of the predicted ground accelerations that have dominant responses for multi-story to high-rise building structures.

Fig. 5-6-1 Predicted Ground Motions and Building Models in the Objective Area

Fig. 5-6-2 Building Model A10: Typical Floor Plan
Fig. 5-6-3 Model A10: Frame Elevation in Transverse Direction

Fig. 5-6-4 Model A10: Typical Member Section

### Table 5-6-1 Building Model A10: Detail of Beam Sections

<table>
<thead>
<tr>
<th>Section BxD(mm)</th>
<th>Longitudinal bar D25 top/bottom</th>
<th>Hoop stirrups ([ ] 2D13, [ ] 3D13, [ ] 3D16, [ ] 4D13)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor 5F~11F</td>
<td>4/3</td>
<td>2D13@200</td>
</tr>
<tr>
<td>G1</td>
<td>450x700</td>
<td>9-11F G1 G6, 11F G5 G8, 10-11F G7, 7-11F G9</td>
</tr>
<tr>
<td>G2-G5</td>
<td>430x700</td>
<td>7-8F G1, 11F G2 G3, 10F G5 G8, 8F G6, 9F G7, 6F G9</td>
</tr>
<tr>
<td>G6-G9</td>
<td>450x670</td>
<td>2D13@150</td>
</tr>
<tr>
<td>Floor 2F-4F</td>
<td>6/5</td>
<td>2D13@100</td>
</tr>
<tr>
<td>G1</td>
<td>450x750</td>
<td>7F G2, 7-9F G3, 8-9F G4, 6-7F G5, 4-6F G8, 2-5F G7, 2-4F G1 G6</td>
</tr>
<tr>
<td>G2-G5</td>
<td>430x750</td>
<td>6F G2 G3, 2-5F G5, 2-3F G8</td>
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<tr>
<td>G6-G9</td>
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### Table 5-6-2 Building Model A10: Column Sections

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<th>Section Size BxD(mm)</th>
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<th>C2</th>
<th>C4</th>
<th>C5(12D16)</th>
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<td>450x800</td>
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<td>10-11F 10D25</td>
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<td>450x1000</td>
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<td>6-7F 16D25</td>
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<td>C3</td>
<td>C4</td>
<td>C5(12D16)</td>
<td>C7</td>
<td>C9</td>
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</table>

Hoop and stirrups ([ ] 2D13, [ ] 3D13, [ ] 3D16, [ ] 4D13)

- 3D13@100: 2-3F C2 C5 C6 C11, 1-2F C10 4D13@100 1-2F C2 C5
- 3D16@100: 2-3F 9C1, 1-2F 9C9 C11 2D13@100 All others

162
Table 5-6-3 Building Model A10: Shear Wall Sections (double bars, unit: mm)

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Table 5-6-4 Building Model A10: Structural Weight (unit: kN)

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<th>2F</th>
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Table 5-6-5 Building Model A10: Material Properties (unit: N/mm²)

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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24.0</td>
<td>24500</td>
<td>9800</td>
<td>Young’s modulus Es=2000000</td>
</tr>
<tr>
<td></td>
<td>22.5</td>
<td>23700</td>
<td>9500</td>
<td>σ_y</td>
</tr>
<tr>
<td></td>
<td>21.0</td>
<td>22900</td>
<td>9200</td>
<td>SD35 (D19 and above)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SD30 (D16 and below)</td>
</tr>
</tbody>
</table>

Fig. 5-6-5 Building Model S20: Typical Floor Plan

Fig. 5-6-6 Model S20: Elevation

Fig. 5-6-7 Model S20: Column Member Sections
Table 5-6-6 Building Model S20: Beam Member Section and Reinforcing Bars (unit: mm)

<table>
<thead>
<tr>
<th>Floor</th>
<th>b×D</th>
<th>Top bar</th>
<th>Bottom bar</th>
<th>Floor</th>
<th>b×D</th>
<th>Top bar</th>
<th>Bottom bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>600×800</td>
<td>3-D35</td>
<td>3-D35</td>
<td>RF</td>
<td>13F</td>
<td>700×800</td>
<td>7-D41</td>
</tr>
<tr>
<td>20F</td>
<td>600×800</td>
<td>3-D35</td>
<td>4-D35</td>
<td>20F</td>
<td>10F~12F</td>
<td>750×800</td>
<td>7-D41</td>
</tr>
<tr>
<td>19F</td>
<td>600×800</td>
<td>5-D38</td>
<td>5-D38</td>
<td>19F</td>
<td>7F~9F</td>
<td>800×800</td>
<td>7-D41</td>
</tr>
<tr>
<td>18F</td>
<td>650×800</td>
<td>6-D38</td>
<td>6-D38</td>
<td>18F</td>
<td>3F~6F</td>
<td>500×1000</td>
<td>6-D41</td>
</tr>
<tr>
<td>16F~17F</td>
<td>650×800</td>
<td>6-D41</td>
<td>6-D41</td>
<td>16F~17F</td>
<td>2F</td>
<td>500×1100</td>
<td>6-D41</td>
</tr>
<tr>
<td>14F~15F</td>
<td>700×800</td>
<td>6-D41</td>
<td>6-D41</td>
<td>14F~15F</td>
<td>1F</td>
<td>600×1100</td>
<td>6-D41</td>
</tr>
</tbody>
</table>

Fig. 5-6-8 Model S20: Typical Beam Sections (Hoop 1F~18F: U11@100, 19F~RF: D13@100)

Table 5-6-7 Building Model S20: Material Properties (MPa)

<table>
<thead>
<tr>
<th>Floor level</th>
<th>$\sigma_c$</th>
<th>$E_c$ (×10^6)</th>
<th>$G_c$ (×10^4)</th>
<th>Steel type</th>
<th>Diameter</th>
<th>$\sigma_y$</th>
<th>$\sigma_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>18~RF</td>
<td>23.5</td>
<td>2.26</td>
<td>0.97</td>
<td>SD295A</td>
<td>D10 ~ D16</td>
<td>294.1</td>
<td>294.1</td>
</tr>
<tr>
<td>1F,14~17F</td>
<td>26.5</td>
<td>2.39</td>
<td>1.03</td>
<td>SD345</td>
<td>D19 ~ D25</td>
<td>343.1</td>
<td>294.1</td>
</tr>
<tr>
<td>10~13F</td>
<td>29.4</td>
<td>2.52</td>
<td>1.08</td>
<td>SD390</td>
<td>D29 ~ D35</td>
<td>392.2</td>
<td>294.1</td>
</tr>
<tr>
<td>2~9F</td>
<td>32.6</td>
<td>2.65</td>
<td>1.14</td>
<td>SSBPD</td>
<td>U6.4 ~ U13</td>
<td>—</td>
<td>588.3</td>
</tr>
</tbody>
</table>

Ec: Young modulus, Gc: Shear modulus
$\sigma_c$: compression strength
$\sigma_y$: Longitudinal bar yielding strength
$\sigma_w$: Hoop bar yielding strength

Idealization of the building models

The responses of the building models to the predicated ground acceleration are calculated using three-dimensional frame model based on the nonlinear force-displacement relations of individual structural members. The frame model consists of rigid structural nodes, line elements (beam, column and shear wall) and floor slabs. Floor slabs are treated as rigid diaphragm having rigid movement in the floor plane and zero resistance out of the floor plane. Each structural node has five degrees of freedom, one vertical and two lateral translations, and two rotations in the vertical X-Z and Y-Z plane. The lateral translations at structural nodes are governed by the movement of the rigid floor diaphragm when the structural node is associated to the rigid diaphragm.

The mass of the building is lumped at each floor gravity center point for lateral inertia loads and at concentrated at individual structural nodes for vertical inertia loads. Zero mass is assumed for the rotational displacement component at structural nodes. That means no inertia moment is considered at structural nodes.

The building model A5 and A10 is supported on pile foundation. Support springs are placed under the joints of footing beam and first story column to represent the contribution of
the piles, as shown in Fig. 5-6-3. The spring properties are determined according to the recommendations given in the references [4] and [5]. The mechanical properties of the springs representing the pile stiffness and strength are listed in Table 5-6-8. For vertical supporting springs the stiffness reduction and yielding in tension is considered (Fig. 5-6-9). Other springs as well as the vertical springs in compression are treated as elastic.

The foundation of the building model S20 is not considered and is assumed pin-roller supports at the ground floor level (Fig.5-6-6) in the analysis.

The analysis methods and models for the structural members and their nonlinear force-displacement relations can be found in Chapter 2 section 2-6.

Table 5-6-8 Building Model A10: Properties of Spring Representing Pile Contribution (unit: kN, m)

<table>
<thead>
<tr>
<th>Pile location</th>
<th>P2</th>
<th>P4</th>
<th>P6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>1.2</td>
<td>1.4</td>
<td>1.6</td>
</tr>
<tr>
<td>Pile length</td>
<td>7.6</td>
<td>7.6</td>
<td>7.6</td>
</tr>
<tr>
<td>Vertical stiffness</td>
<td>31.0×10^5</td>
<td>42.2×10^5</td>
<td>55.1×10^5</td>
</tr>
<tr>
<td>Lateral stiffness</td>
<td>2.37×10^5</td>
<td>2.76×10^5</td>
<td>3.20×10^5</td>
</tr>
<tr>
<td>Rotation stiffness</td>
<td>13.9×10^5</td>
<td>22.6×10^5</td>
<td>33.6×10^5</td>
</tr>
</tbody>
</table>

Pile material young's modulus $E_p = 2.45$ kN/m$^2$

Fig. 5-6-9 Axial Stiffness Properties of Vertical Support Springs

![Fig. 5-6-9 Building Model A10: Pile Locations](image)

**Lateral load-displacement relations of the building models by pushover analysis**

Pushover analysis is carried out to the building models to find out the loading carrying capacity and the lateral resistance against the inter-story drift of the building models. The results are used for comparing with the dynamic responses and for determining the damage extent of the building models subjected to the predicted ground accelerations. The pushover analysis assumes weighted anti-triangular distribution of lateral load at each floor level. The results are shown in Fig. 5-6-10 to 5-6-12.

The building model A5 has relatively high base shear capacity over 0.45W and 0.55W (W= the total weight) in longitudinal and transverse directions, respectively. In longitudinal direction, obvious story yielding starts in a relative story drift of 1/200. In transverse direction the results indicate shear wall yielding starts in relative story displacement of 1/200 but story
resistance increases until the displacement reaching 1/100. The building model A10 has the base shear capacity at 0.26W in the longitudinal frame direction and near 0.5W in the transverse shear-wall direction, and yielding starts at the relative inter-story displacement among 1/400 to 1/200 in both directions. The building model S20 has similar base shear capacity about 0.26W in both directions and yielding starts at a relative inter-story displacement of about 1/200.

![Graphs showing story resistance vs. inter-story displacement for different models and directions](image)

Fig. 5-6-10 Building Model A5: Story Resistance vs. Inter-Story Displacement

Fig. 5-6-11 Building Model A10: Story Resistance vs. Inter-Story Displacement

Fig. 5-6-12 Story Resistance vs. Inter-Story Displacement (Building Model S20)
Input acceleration and analysis method

The input acceleration waves shown in Fig. 5-6-1 are the two lateral acceleration components of the response results at ground level in the objective area by sub-soil and ground analysis (see section 5.5). The peak ground acceleration is about 0.3G in EW-component and 0.37G in NS-component. It has the aspect of low frequency dominance for probable soil liquefaction. According to the elastic response spectra (Fig. 5-6-13) the acceleration waves result in major responses in the period among 0.3 sec to 1.5 sec. It implies the dominant responses to tall building structure about 15 to 50 meters or 5- to 20-story high. The building models used in the analytical study are just among the structures in dominant responses.

The acceleration waves are used as bi-directional lateral input to the building models at the ground floor level. The input makes the two components of the acceleration waves in the building longitudinal and transverse directions alternatively to carry out each two calculations on one building model. The input duration lasts 15 seconds that cover the major peaks of the acceleration waves.

The response is calculated in step-by-step numerical integration using Newark $\beta$-method ($\beta = 0.25$). Stiffness and mass proportional damping (Rayleigh's damping) is assumed at damping constant of 5%. The integration is made at a time interval of 1/200 second, or 1/66 of the fundamental period of the building model A5. At the beginning of a time step, linear relation among the force and displacement increments is assumed. Then at the end of the time step, the stiffness change is evaluated and the unbalanced forces owing to the stiffness change are brought in to next time step to be corrected.

The responses and damage of the building models

The responses of the building models to the predicted ground motion are shown in Fig. 5-6-14 to 5-6-17. The distribution of the extreme relative inter-story displacements and the story shear forces in plus and minus responses in longitudinal X-direction and transverse Y-direction are shown in Fig. 5-6-14 to 5-6-16 for the three building models.

The building model A5 results in maximum inter-story displacement of 1/364 and base shear force 0.393W in longitudinal X-direction, and of 1/206 and 0.423W in transverse Y-direction. Compared with the pushover analysis results shown in Fig. 5-6-10, the building has not yet reached the story yielding. It could be determined as minor or no damage to sustain the predicted ground motion.

The results of the building model A10 indicates obvious story yielding in the longitudinal X-direction as its maximum inter-story displacement reached 1/105 and base shear force 0.324W in longitudinal X-direction. It could be determined as severe damage. The base shear force is higher than the base shear capacity by pushover analysis (Fig. 5-6-11). It may attributed to the effect of dynamic load distribution (higher order of vibration modes) resulted in different load pattern than that used in pushover analysis. In the building transverse shear wall direction the extreme inter-story displacement 1/406 and base shear force 0.408W
indicate no story yielding or minor damage in the direction compared with the pushover analysis results.

The building model S20 results in excessive responses. The maximum inter-story displacement in some middle stories reaches 1/72 in longitudinal direction, and 1/80 in transverse direction. Comparing the maximum inter-story displacement responses with the story shear force and inter-story displacement curves (Fig. 5-6-12) of pushover analysis, it indicates obviously the severe damage or near collapse of the high-rise building model. The responses of base shear force of 0.295W in longitudinal X-direction and 0.288W in transverse Y-direction, however, are comparable with the pushover results. It shows that the dynamic load distribution on the frame-building model is almost similar to the anti-triangle load pattern used in the pushover analysis.

Fig. 5-6-14 Building A5: Extreme Inter-story Displacement and Story Shear (EW→X, NS→Y)

Fig. 5-6-15 Building A10: Extreme Inter-story Displacement & Story Shear (EW→Y, NS→X)
The extreme responses of the base shear factor (Q/W), top floor level displacement, structural torsional oscillation and relative inter-story drift of the responses of the building models are summarized in Table 5-6-9. It shows the responses of structural oscillation (in radian) in the Table as well. The dimension of the floor plan of the three buildings is 49.6, 78.55 and 42 meters. Then the possible additional translational displacement by structural torsion at a side frame is about 3.4%, 7.5% and 5.6% to the lateral displacement responses of the building model A5, A10, and S20, respectively. The structural torsional response is not significant for all the building models.
<table>
<thead>
<tr>
<th>Building model</th>
<th>Input direction</th>
<th>Base shear force</th>
<th>Top D (mm)</th>
<th>Top D/H</th>
<th>Torsion (rad.)</th>
<th>Inter-story drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>A5 W=21290 kN H=14.29 m</td>
<td>NS→X</td>
<td>0.422W</td>
<td>34.7</td>
<td>1/412</td>
<td>0.000576</td>
<td>1/394</td>
</tr>
<tr>
<td></td>
<td>EW→Y</td>
<td>0.400W</td>
<td>41.9</td>
<td>1/341</td>
<td>0.001251</td>
<td>1/314</td>
</tr>
<tr>
<td></td>
<td>EW→X</td>
<td>0.393W</td>
<td>38.4</td>
<td>1/372</td>
<td>0.000292</td>
<td>1/364</td>
</tr>
<tr>
<td></td>
<td>NS→Y</td>
<td>0.423W</td>
<td>57.6</td>
<td>1/248</td>
<td>0.000754</td>
<td>1/206</td>
</tr>
<tr>
<td>A10 W=189660 kN H=28.15 m</td>
<td>NS→X</td>
<td>0.324W</td>
<td>193.6</td>
<td>1/145</td>
<td>0.000754</td>
<td>1/105</td>
</tr>
<tr>
<td></td>
<td>EW→Y</td>
<td>0.393W</td>
<td>62.2</td>
<td>1/453</td>
<td>0.000754</td>
<td>1/447</td>
</tr>
<tr>
<td></td>
<td>EW→X</td>
<td>0.307W</td>
<td>151.7</td>
<td>1/185</td>
<td>0.000686</td>
<td>1/133</td>
</tr>
<tr>
<td></td>
<td>NS→Y</td>
<td>0.408W</td>
<td>68.4</td>
<td>1/411</td>
<td>0.000686</td>
<td>1/406</td>
</tr>
<tr>
<td>S20 W=26225 kN H=66.15 m</td>
<td>NS→X</td>
<td>0.280W</td>
<td>394.4</td>
<td>1/168</td>
<td>0.001273</td>
<td>1/81</td>
</tr>
<tr>
<td></td>
<td>EW→X</td>
<td>0.288W</td>
<td>475.2</td>
<td>1/139</td>
<td>0.001273</td>
<td>1/80</td>
</tr>
<tr>
<td></td>
<td>EW→Y</td>
<td>0.295W</td>
<td>448.3</td>
<td>1/148</td>
<td>0.001076</td>
<td>1/72</td>
</tr>
<tr>
<td></td>
<td>NS→Y</td>
<td>0.266W</td>
<td>463.7</td>
<td>1/143</td>
<td>0.001076</td>
<td>1/84</td>
</tr>
</tbody>
</table>

**Evaluation of the seismic damage based on the vibration period**

The damage extent of the building models subjected to the predicted ground acceleration are observed and determined by comparing the maximum responses of displacement and base shear capacities with the pushover analysis results. In other way, the seismic damage can be evaluated based on measures of the change in fundamental period of the structure due to stiffness changes during earthquake impact [6][7]. It defines damage index as followings:

\[
D_m = 1.0 - \frac{T_0}{T_{max}} \quad (5-6-1)
\]

\[
D_f = 1.0 - \left( \frac{T_0}{T_{final}} \right)^2 \quad (5-6-2)
\]

Where, \(T_0\) = elastic fundamental period of structure in initial stiffness; \(T_{max}\) = the maximum period during earthquake response; and \(T_{final}\) = the period at the end of earthquake response. Such the damage index ranges from 0 to 1.0.

The change of the fundamental period of the building models during the response is shown in Fig. 5-6-18. The characteristic value of the fundamental periods and the damage indices based on the equation (5-6-1) and (5-6-2) are given in Table 5-6-10. The two indices based on the maximum period and the final period result in difference. Therefore, the average value from the two indices is used to evaluate the damage. From the correlation of the damage index and the damage extent of building models determined by comparing the inter-story displacement of dynamic response with the pushover results, the structural seismic damage based on the period index would be evaluated.

![Fig. 5-6-18 Change of the Fundamental Period](image_url)
as listed Table 5-6-11.

<table>
<thead>
<tr>
<th>Building Model</th>
<th>( T_0 )</th>
<th>( T_{max} )</th>
<th>( T_{final} )</th>
<th>( D_m )</th>
<th>( D_f )</th>
<th>( 0.5 \times (D_m + D_f) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A5</td>
<td>0.339</td>
<td>0.714</td>
<td>0.455</td>
<td>0.525</td>
<td>0.446</td>
<td>0.486</td>
</tr>
<tr>
<td>A10</td>
<td>0.487</td>
<td>3.212</td>
<td>0.806</td>
<td>0.848</td>
<td>0.635</td>
<td>0.741</td>
</tr>
<tr>
<td>S20</td>
<td>1.335</td>
<td>5.947</td>
<td>1.736</td>
<td>0.776</td>
<td>0.409</td>
<td>0.593</td>
</tr>
</tbody>
</table>

Table 5-6-11 Seismic Damage Evaluations by Index of Fundamental Period

<table>
<thead>
<tr>
<th>Period Index</th>
<th>Damage extent</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ~ 0.3</td>
<td>No damage</td>
</tr>
<tr>
<td>0.3 ~ 0.5</td>
<td>Minor to moderate damage</td>
</tr>
<tr>
<td>0.5 ~ 0.7</td>
<td>Moderate to severe damage</td>
</tr>
<tr>
<td>0.7 ~ 1.0</td>
<td>Severe damage or collapse</td>
</tr>
</tbody>
</table>

**Concluding Remarks**

The predicted ground motion in the liquefaction-prone area has the aspect of low frequency dominance. To the scenario earthquake in the objective area, low-rise and middle-rise buildings with relatively short vibration period may respond with or without minor or moderate damage. However, high-rise buildings may result in significant responses suffering excessive displacement and severe damage.

The damage extent of structures due to earthquake action can be evaluated by comparing the extreme inter-story displacement of dynamic responses and static pushover. It can also be evaluated using the damage index based on the change of the fundamental period of the structure.

**References**


5.7 Summary

In section 5.2, we made a broadband strong ground motion prediction for a scenario earthquake in the Tottori region. We studied the plausible range of ground motions that could be generated in the Tottori region from two extreme asperity models with identical characteristics but different location, assuming in either case the same fault plane ruptured during the Tottori-ken Seibu earthquake. The first model assumed the asperity located in the southernmost part of the fault and in the second model the asperity was located in the northernmost part of the fault. We obtained that the location of the asperities in the fault plane has a large influence into the amplitude and frequency content of the predicted waveforms. The difference in amplitude of the waveforms at the Yonago region can be as large as 3 times depending on whether we consider an asperity with identical characteristics, located in the northernmost or southernmost part of the fault plane ruptured during the Tottori-ken Seibu earthquake. We can finally conclude that the broadband high frequency strong ground motion methodology is a powerful tool to simulate the ground motion for future scenario earthquakes by fully including the generation of the seismic waves from an active fault, the propagation of the waves through the geological structures and the amplification of the waves by the local site effects.

In section 5.3, microtremor measurements using a three-component sensor were performed at 26 sites along the line between two strong motion stations in Yonago city, and H/V spectra of microtremors were determined for the sites. The variation of H/V spectra indicates that $V_S$ profile varies drastically along the observation line. Besides, inversion of the observed H/V spectra successfully resulted in $V_S$ profiles down to a depth of 60 m at the strong motion stations. The estimated $V_S$ profiles were used or referred in predicting ground motions in the cities of Sakai-minato and Yonago during a scenario earthquake.

The soil liquefaction and ground surface responses in Sakaiminato and Yonago City during the scenario earthquake were predicted in section 5.5. We estimated not only the occurrence of liquefaction, but also the ground deformation by the effective stress analyses described in section 2.4. The 3-dimensional dynamic finite element analyses were carried out using bi-directional input motions in the case of the northern asperity obtained in section 5.2, because the amplitude of the input motions in northern asperity case were much larger than that in southern asperity case. As the result of the numerical analysis, the following damage due to liquefaction was predicted. 1) Although liquefaction occurred in eastern area in Sakaiminato City, it might not cause the serious damage such as the building settlement and pile rupture. 2) Liquefaction occurred in western area in Yonago City and it might cause the structural damages for underground structures and light structures on the ground. Although the numerical method for soil-structure system provided the rational results through the case studies, the following researches will be needed for more accurate and useful prediction.

Attempting in earthquake damage estimation is carried out to the building models supposed locating in Yonago-city of Western Tottori Area (section 5.6). Three building models used in the study were of middle-rise and high-rise RC frame-shear wall apartments and high-rise SRC/RC frame office building with the vibration period from 0.33 to 1.35 seconds. The responses of the building models were calculated to the evaluated ground motions of scenario earthquake in the area. The results showed that the middle-rise building model with comparatively short vibration period responded with or without minor or moderate damage, while the high-rise building models had significant responses suffering excessive displacement and severe damage. The results indicated that the aspects of low frequency-dominant ground motions cause minor or no damage in the less-populated area with almost low-rise buildings.
Chapter 6. Concluding Remarks

In the following, we summarize the conclusive remarks obtained during the study presented in the previous chapters 2 through 5. Herein, we itemize the concluding remarks for each phenomenon within a series from generation of seismic wave to response of structural systems during the earthquake disaster mitigation issue.

Concluding Remarks on Fault Characterization and Ground Motion Prediction

We presented a methodology for prediction of a broadband strong ground motion prediction from a characterized asperity model of the fault source. The methodology has two major purposes described in the following.

One is to make a ground motion prediction from a past earthquake or case study, which is especially relevant in cases when a limited number of strong ground motion records of earthquake are available. For that particular case the broadband strong ground motion methodology can give an idea about the spatial ground motion distribution characteristics of the earthquake namely the determination of a shake map from the earthquake which can help explaining the damage distribution patterns. In our case we applied the broadband technique in chapter 4 to produce a ground motion simulation during the 1999 Kocaeli (Turkey) earthquake at the heavily damaged Golcuk city, where there was no available strong ground motion recordings in order to understand the damage distribution.

The other is to make a strong ground motion prediction for a scenario earthquake (future earthquake). This is a very relevant issue for the disaster prevention, to determine the seismic hazard for a particular region from a physical model of the active faults surrounding the urban areas. In chapter 5 we applied the methodology to study the plausible range of ground motions that could be generated from a particular fault in the Tottori-ken Seibu region. We studied the influence of the location of asperities in the fault plane of a scenario Tottori earthquake into the simulated broadband frequency strong ground motion and compare it with the actual recordings from the Tottori-Seibu earthquake. We obtained that the location of the asperities in the fault plane has a large influence into the amplitude and frequency content of the predicted waveforms.

The final goal of the broadband methodology is in either cases to produce Shake-Maps describing the spatial variation of the ground motion and to produce maps describing how different type of buildings would behave from these input ground motions giving an idea about the spatial damage distribution for past or future earthquakes.

The broadband high frequency strong ground motion methodology is a powerful tool to simulate the ground motion for future or past earthquakes by fully including the generation of the seismic waves from an active fault, the propagation of the waves through the geological structures and the amplification of the waves by the local site effects.

Regarding the source contribution to the ground motion we could say in general that the amplitude and frequency content of the acceleration waveforms are mainly controlled by the stress drop and rise time inside asperities, whereas the velocity waveforms are controlled by the rupture duration and size of asperities.

More research is needed in order to clarify the relationship between the characterized source and the source parameters, which has a large influence on the ground motion characteristics. This point is in particular relevant for the ground motion prediction of a scenario earthquake.

The inclusion of the nonlinearity effect into the ground motion produces a large change in the frequency content of the waveforms. A large decrease in the high frequency and a large increase in the low frequency content of the waveforms are observed. The nonlinearity effect of
the shallow layers should be included for the simulation of ground motion. This is relevant in particular for the high frequency ground motion, i.e., frequencies greater than 1.0 Hz.

Concluding Remarks on $V_S$ Structure Evaluation

Possible use of horizontal-to-vertical (H/V) spectral ratios of microtremors was explored for estimating shear wave velocity ($V_S$) structure and the effects of subsurface soil conditions on ground motion characteristics during an earthquake. Theoretical formulas to simulate microtremor H/V spectra were presented in the section 2.2 assuming that microtremors consist of surface waves which are generated from point sources randomly distributed on a layered elastic half-space. Using these formulas, an inverse analysis of microtremor H/V data was then expressed for estimating the layer thickness of subsurface soil, when the prior information of $V_S$ value at a site was given. In order to examine the applicability, effectiveness, and limitation of the microtremor H/V method, microtremor measurements with only one station were conducted at several sites where the results of other geophysical investigations were available, and microtremor H/V spectra were determined. The inversion of the observed microtremor H/V spectra successfully resulted in S-wave velocity profiles down to the bedrock with $V_S$ over 700 m/s. The $V_S$ profiles from microtremor analyses showed fairly good agreement with those from bore-hole method, revealing that the microtremor H/V method reviewed in Chapter 2 could be promising for evaluating S-wave velocity profile at a site.

The Kocaeli earthquake of August 17, 1999 destroyed over 60,000 masonry building used for either residential or office buildings in the northwest area of Turkey. In Golcuk, Kocaeli Province, in particular, the damage to buildings was concentrated in several limited areas. The concentration of the building damage could be due to the effects of surface geology on earthquake ground motions, i.e., the so-called “site effects.”

In order to evaluate the site effects quantitatively, two- or three-dimensional $V_S$ profiles down to the bedrock should be properly determined. It is, however, difficult to estimate multi-dimensional deep $V_S$ profiles using conventional geophysical methods with boreholes. In Chapter 4, based on the H/V method reviewed in the section 2.2, microtremor measurements were performed at five sites in Golcuk and a two-dimensional $V_S$ structure across the damaged area was estimated. With this $V_S$ structure, ground surface motions during the main shock were simulated employing the one- and two-dimensional dynamic response analysis models, and then response ductility factor spectra of simplified building systems in the area were computed for the simulated ground surface motions. The evaluated ground and building responses were consistent with the damage distributions observed in the area, confirming that the geophysical method using microtremor H/V spectra is an economical and reliable mean of estimating multi-dimensional S-wave velocity structure and site effects during earthquakes.

In Chapter 5, microtremor measurements using a three-component sensor were performed at the 26 sites along the line between two strong motion stations in Yonago city, and H/V spectra of microtremors were determined for the sites. The variation of H/V spectra indicates that $V_S$ profile varies drastically along the observation line. Inversion of the observed H/V spectra successfully resulted in $V_S$ profiles down to a depth of 60 m at the strong motion stations. The estimated $V_S$ profiles were used or referred in prediction of ground motions in the cities of Sakaiminato and Yonago for a scenario earthquake.

Concluding Remarks on Evaluating Ground Motion

Earthquake ground motions were simulated both by a two-dimensional analytical model generated from the finite element method. Application has been performed for the motions
obtained during real earthquakes, i.e., one for the Kobe site during the 1995 Hyogo-ken Nanbu earthquake, and the other for the Golcuk site during the 1999 Kocaeli, Turkey earthquake. Application is extended for the motion generated by the so-called scenario earthquake for the Yonago site during the simulated fault model determined for the 2000 Tottori-ken Seibu earthquake. The evaluated ground motions have been utilized for the further analysis within the research works. The following conclusive remarks could be summarized:

1) The estimated spatial distribution of earthquake motions is the result of the distribution of ground amplification factors and input motion characteristics. Generally, the amplification factor varies drastically near the basin edge, arising from the variations of Vs profile and the so-called “edge effects” of macroscopic local soil condition.

2) From the case of past earthquakes, the evaluated ground and building responses are consistent with the observed damage distribution within the area. Either the peak ground acceleration (PGA) or velocity (PGV) can be a good index to measure the ground intensity closely correlated to the damage distribution. The answer for the question of “Which is better index?” is not definitely specified. In some cases, PGA indicates better correlation, and in other cases, PGV better. The combination of the frequency domain characteristics of amplification, input motion and building response will specify which would be better.

3) To investigate the amplification factor of ground and simulate the distribution of ground motion, it is very important to know the Vs profile distribution as accurate as possible. The microtremor H/V inversion method is efficient when we do not have enough information about the profile sufficient for detailed analysis.

**Concluding Remarks on Liquefaction and Soil and Foundation Responses**

The effective stress analysis technique is one of the promising tools that can predict the behavior of soil liquefaction. We have examined the two- and three-dimensional dynamic response finite element analysis that incorporates constitutive models for sand/clay and the Biot’s two-phase mixture theory. The numerical method has been applied to the horizontal ground, buildings with spread footing, those with pile foundation, infrastructures such as embankments, port structures and underground structures. The numerical method quantitatively reproduced the measured records and damage extent in case studies. The brief description of the governing equations and constitutive models for sand/clay were given in the section 2.5. The proposed method was verified based on the vertical array records observed in Port Island during the 1995 Hyogo-ken Nanbu earthquake in the section 2.5. The computed acceleration, velocity and displacement records have fallen in a good agreement with the observed records. The proposed numerical method could reproduce the three-dimensional seismic behavior of sand and clay in a quantitative manner.

Many buildings with pile foundation on reclaimed land were seriously damaged during the 1995 Hyogo-ken Nanbu earthquake. Some pile foundations, however, were not damaged in the area where the strong motion was observed. In the section 3.4, the seismic response of the pile foundation, which was not severely damaged in the severely damaged zone, was discussed through the two-dimensional effective stress analysis. The cyclic elasto-plastic model for sand was exclusively applied to soil deposits. Their analytical parameters were determined based on the soil properties, which were empirically estimated from the N values from the standard penetration test. It leads to the result that the maximum moment calculated for the piles became greater than the cracked moment, while it remained less than the yield moment, i.e., the piles might generate cracks, while not yielded. It can be indicated that the residual deformation due to the bending failure of the piles are not produced during the earthquake. The numerical results coincide with the actual behavior of the building that has not be tilted nor settled down.
after the earthquake.

The soil liquefaction and ground surface responses in Sakaiminato and Yonago City during a scenario earthquake were predicted in the section 5.5. We estimated both the occurrence of liquefaction, and the ground deformation by the effective stress analyses described in the section 2.4. The three-dimensional dynamic finite element analyses was carried out using bi-directional input motions in the case of the northern asperity obtained in section the 5.2, because the amplitude of the input motions in northern asperity case were much larger than that in southern asperity case. As the result of the numerical analysis, the following damage due to liquefaction could be predicted: (1) Although liquefaction occurred in the eastern area in Sakaiminato City, it might not cause the serious damage such as the building settlement and pile rupture; and (2) Liquefaction occurred in western area in Yonago City, and it might cause the structural damages for underground structures and light-weight structures on the ground.

While the numerical analysis for soil-structure system presented herein provided the rational results through the case studies, the following research items are needed for more accurate and reliable prediction.

1) The parameters of the constitutive model for sand/clay need to be determined more rationally and easily. We need more in-situ tests and laboratory tests than usual soil investigations to determine the parameters of the constitutive models. However, it is difficult to conduct sufficient soil investigations for many soil layers because of its high cost. To begin with, it is impossible to get the complete model for complicated and inhomogeneous soil ground. On the other hand, the estimated parameters from SPT-N values given by usual soil investigations can be useful according the conditions such as the case study in Chapter 3. Rational parameter determination by easy soil investigation, which realizes the in-situ soil conditions, will be needed for more accurate analysis.

2) The coupling analysis with the non-linear response of a superstructure is needed for considering the seismic interaction between foundation and superstructure. As the structural members of the building did not reach the failure state in Chapter 3, the structural members of building could be treated as elastic body. According to the structural and input conditions, the failure of both or either superstructures and foundations would be generated. In that cases, we need to consider the non-linear interaction among soil, foundation and superstructure by coupling the numerical method described in the section 2.5 and 2.6.

3) The estimation method of the input motion at the base needs to be established. The input motions and the depth of the base layers were determined according to the analytical conditions in each case study. We need to estimate the input motions based on the understanding the characteristics and analytical conditions of each analysis described in Chapter 2.

4) The simple modeling for pile foundations will be needed for large-scale prediction. In assessment of seismic performance for a whole urban area, it is difficult for the current computing technique to make the complete model for each structure in the wide area. We need to develop the simple model to reproduce the essential behavior of structures through the analyses by the complete model described in the section 2.5 and 2.6.

**Concluding Remarks on Building Response Simulation and Damage Estimation**

Simulating building responses and estimating earthquake damage is the part of the earthquake disaster mitigation research project carried out. The research and development are summarized in this report as in the following.

Reliability of simulating building responses to the earthquake impact is one of the key issues in damage estimation. Therefore, efforts are made to develop sophisticated analysis
models to represent the material properties and the structural mechanical properties in accurate manner. The section 2.6 in Chapter 2 presented the analysis models and numerical methods that are developed for carrying out three-dimensional nonlinear structural analysis. The analysis is based on the nonlinear force-displacement relations of individual structural members. Therefore, it becomes available the responses and damage information in the structural member level. For reliable evaluation of member response, multi-spring model and fiber model are used to represent the nonlinear behavior of structural members to allow for the interactions among the bi-directional lateral loads and varying axial load. The performance and reliability of the models and methods were examined and verified by comparing the calculated results with the observed results from structural lab tests and from instrumented buildings.

The application of the structural model and analysis methods in the case study of damage investigation and earthquake response simulation was presented in the section 3.5 in Chapter 3. A reinforced concrete apartment building located in the extensive damaged belt area in Kobe-city during the 1995 Hyogo-ken Nanbu Earthquake was selected for the study. The calculated building responses agreed with the observed minor damage of the building during the earthquake. It verified again simulating earthquake responses in agreeable reliability by sophisticated analytical models proposed herein, and suggested the possibility of damage estimation in high seismic risk areas for future earthquake events.

Attempting in earthquake damage estimation is carried out to the building models supposed locating in Yonago-city within the western Tottori area. Three building models used in the study are of middle-rise and high-rise RC frame-shear wall apartment buildings and high-rise SRC/RC frame office building with the vibration period from 0.33 to 1.35 seconds. The responses of the building models were calculated to the evaluated ground motions of a scenario earthquake in the area. The results showed that the middle-rise building model with comparatively short vibration period responded with or without minor or moderate damage, while the high-rise building models had significant responses suffering excessive displacement and severe damage. The results indicated that the aspects of low frequency-dominant ground motions cause minor or no damage in the less-populated area with almost low-rise buildings.

Summarizing the study on building response simulation and damage investigation, the results obtained are: (1) the reliability of simulating earthquake responses using detailed structural model based on nonlinear force-displacement relations of individual structural members; and (2) the possibility of estimating future earthquake damage by numerical methods. The study reported here used few building models in the simulation and estimation. It was because of time-consuming and high cost for carrying out the analytical study. Quick damage estimation and assessment of large number of buildings may be compelled during a destructive earthquake event. Then simplified and efficient model and method are required. Also the response analyses of superstructure and soil-foundation currently were carried out separately without considering the interaction between them.

**Concluding Remarks on The Research Activities of the Structural Performance Team**

The research and development in the Structural Performance Team at EDM is continued towards systematized evaluation and simulation from the earthquake source mechanism to response evaluation of urban structural systems through wave propagation, wave amplification, ground motion evaluation, soil response, soil-structure interaction response, foundation response and responses of superstructure considering either interactive phenomena and correlations among all the relevant subjects.

The developed sophisticated analytical technology can be extended to utilize for the damage and vulnerability assessment of urban structures in the high seismic risk areas.
ABOUT THE EDM (Earthquake Disaster Mitigation Research Center, RIKEN)

The Earthquake Disaster Mitigation Research Center (EDM) was established in January 1998 under the framework of the Institute of Physical and Chemical Research (RIKEN), and located in the Mikiyama Green Park of Hyogo Prefecture. This site was selected with the support of the Hyogo Prefectural Government.

The main purpose of the EDM is to produce “Frontier Research on Earthquake Disaster Mitigation for Urban Regions.” The major research activities are performed by three research teams: the Disaster Process Simulation Team, Disaster Information System Team and Structural Performance Team. Based on the lessons learned from the disaster of the 1995 Hyogo-ken Nanbu earthquake, the EDM aims to carry out multi-disciplinary research on earthquake disaster, encompassing physical, engineering and social sciences.

Through the work of the EDM, methods will be developed for comprehensive understanding of disaster processes that involve “physical”, “social” and “information” agendas, and also methods will be devised for the visual presentation of research results using advanced technologies to enable dissemination and promotion of the work of the EDM.

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