

Near-fault Ground Motions
for Seismic Design of Bridge Structures

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Wu Shuangle

Abstract

A number of significant inland earthquakes tragically occurred, such as the 1994 Northridge earthquake (M_w 6.7), the 1995 Kobe earthquake (M_w 6.9), the 1999 Chi-Chi earthquake (M_w 7.6 - 7.7), the 2008 Wenchuan earthquake (M_w 7.9), the 2016 Kumamoto earthquake (M_w 7.0), the 2017 Iran-Iraq earthquake (M_w 7.2), and the 2018 Hualien earthquake (M_w 6.4). These seismic events had devastating effects on urban infrastructures, especially the near-fault zones. The near-fault ground motions are very different from far-field ground motions and cause damage or collapse due to both ground acceleration and fault ruptures (permanent displacement), especially the structures adjacent to and/or spanning a potentially active fault. For example, during the 2008 Wenchuan earthquake (M_w 7.2), the SFB station (at a source distance of 14 km) recorded peak ground displacements (PGDs) of 376 cm for vertical directions, and the largest horizontal and vertical permanent ground displacement was 530cm and 620cm, respectively.

At present, seismic design of bridges takes into account only the shaking caused by ground acceleration. However, seismic design of bridges under earthquake excitation with fault displacement needs to consider the spatial variations and internal forces caused by the ground acceleration and the relative displacement between the bridge supports. Time histories which include fling displacements are required when conducting dynamic time-history analysis for structures located near potential active faults. Thus, in this research, it proposed an analytical procedure for evaluating the seismic behavior of near-fault bridge structures considering this permanent displacement. First of all, based on previous researches, a hybrid synthesis method (a combination of theoretical Green's function method and modified stochastic Green's function method), for simulating near-fault strong ground motions is presented. It could well consider the pulse-like velocity and fling-step displacement of near-fault ground motions. Then, by adoption of this proposed method, seismic response of a fault-crossing bridge was conducted, using multi-input ground motions with permanent displacements, which could offer some useful references for near-fault bridge designing.

This thesis consists of six chapters, which yields most information on work done in this research. These works included the seismic evaluation, theoretical analysis, and numerical simulations. The contents in each chapter can be briefly described as the following.

Chapter 1 presented a general overview and detailed objectives of this research. In this chapter, reviews on previous studies related to simulations and evaluations on near-fault ground motions, and the dynamic response of near-fault bridge engineering was provided. It also tried to look into the current practices as well as research in this field.

Chapter 2 detailed the characteristics of near-fault ground motions, the work done in this chapter contains:

(a) The complete analytical solution of wave propagation and the focal mechanics analysis are presented.

(b) Main 5 specific characteristics, including the concentration effects, the pulse-like velocity effects, the permanent displacement (fling-step) effects, the hanging-wall effects, and the directivity effects, were clarified by exact recording data and occurred earthquakes.

(c) The effects of spatial varied near-fault ground motions on bridge engineering was presented.

(d) A detail damage evaluation on near-fault buildings of 138 houses was conducted as an example for prescribing the damage of near-fault.

Chapter 3 provided a hybrid synthesis method for near-fault ground motions. It conducted the following work:

(a) The theoretical analysis on a new hybrid synthesis method is described. It is a combination by a modified stochastic Green's function and theoretical Green's function. The stochastic Green's function was modified to consider the near-field and intermedia-field terms for complete waveforms of near-fault ground motions. In the modification, two ratios (R_S and R_P) for intermedia-field and near-field components are introduced, the R_S is related to the components along θ, ϕ directions and the R_P is related to the components along r -direction (θ, ϕ and r are spherical polar coordinates). Theoretically, these two ratios are related to the ratio of $(R\omega)/V_s$. Simple examples of strike-slip fault model and dip-slip fault model resulted that, they (R_S and R_P) are independent of focal mechanism, and R_P is larger than R_S when $(R\omega)/V_s$ is smaller than 10, which is satisfied with the theoretical conclusions.

(b) Additionally, this hybrid synthesis method was adapted to two simple fault models for further introduction, the comparison between only dynamic and both static and dynamic are conducted, the strike-slip fault model resulted that, located along the fault normal direction, the static term-induced displacements were dominant in the time history along the near fault area, while the dynamic term-induced displacements were dominant along the further distance from the fault line; the dip-slip fault model indicated that, the fling-step was much obvious and dominant in the hanging-wall, and the same tendency occurred in strike-slip fault along the distance from the fault trace on the surface.

(c) The simulation on the 1999 Chi-Chi earthquake was conducted. The comparisons between the observed and simulated displacement time histories, the vibrations of the simulated results are not so obvious in the waveforms, which meant the dynamic parts are smaller than the static terms. The simulated displacement along NS-direction for both TCU052 and TCU072 (TCU052 and TCU072 are two observation sites) are with large differences compared with the observed recordings. And the displacement along UD-direction of TCU052 is a little smaller than the observed one. And the other

simulated displacement are fitted a good agreement with observed ones, which showed the proposed hybrid method could well simulate the near-fault strong ground motions.

Chapter 4 offered the evaluations on the near-fault ground motions generated by the proposed hybrid synthesis method. and the following works are achieved:

(a) Spatial variation of near-fault displacement-, velocity- and acceleration field was conducted on a dip-slip fault model, with 2500 observation sites located in the near-fault area (the source-distance is smaller than 20km) is calculated. The 3D vectors of acceleration field indicated that the near-fault accelerations are rather complicated. The velocity field showed the pulse-like during 0.5s~3.0s, and the rupture propagation from the central area of the dislocation destruction is easy to seen; for the 3D vector of displacements, the hanging wall effect can be seen from 0.1s; from 0.1s~3.0s, the vectors, including the direction and the values, changed a lot, and after 5s, the permanent displacement remains the same, which means the rupture of surface are with 5.0s, which well shown that the complicated features is the phenomenon resulting from combination factors, while mainly due to the complexity dipole focal mechanism, the propagation characteristic of S-waves, and the directivity effects.

(b) The effects of buried depth indicated that, the permanent displacements are not obvious (less than 3.0 cm) when the buried depth is more than 700m. And the soil conditions also affected the results significantly. The magnitude as well as its focal mechanism affected the permanent displacements.

Chapter 5 described the seismic design method considering the permanent displacement on fault rupture crossing bridge, and the multi-excitation method was presented for seismic analysis of bridge.

(a) First of all, linear analysis on a 5-span bridge was conducted to check the probabilistic of the displacement time histories input, the bridge model located near fault exhibited large residual displacements, deformations, and bending moments under the consideration of permanent displacement excitation, which should be carefully considered when designing bridge. the bearing behavior are over the efficient limitation level, which demonstrated the non-linear analysis is necessary.

(b) The non-linear analysis on the same 5-span fault crossing bridge was achieved under three different excitations (both static and dynamic terms, only dynamic terms synthesized by the hybrid method, and the ground motions from JRA), and a comprehensive analysis on the general seismic behavior of bridge components (superstructures (including three items: the time history of displacement, time histories of bending moment of girder, and the maximum bending moment of girder), pier behaviors (column drift, bending moment curvature, the shear-deformation and the moment-curvature ($M-\phi$))

hysteresis curves), and bearing behaviors (time histories of deformation and the shear-deformation hysteresis curves) were examined, it resulted that, the residual response (deformation, curvature) for girder, pier and bearing are larger when considering the fling-step displacement than by the conventional method, also dynamic analysis considering only accelerations cannot estimate residual moments and displacements, and it is necessary to conduct dynamic analysis using displacement time histories to gain more exact seismic behaviors of bridge structures during earthquake.

(c) Furthermore, we also explored the effects of dip-, strike-, and rake-angles on the bridge crossing fault-rupture zones. The dip angle has a significant affect on the vertical components of the ground waveforms and the seismic response of the bridge; if the dip angle is smaller than 15° , the vertical components are slightly smaller or even equal to the horizontal components, while when the dip angle is larger than 45° , they become larger than the horizontal terms. Therefore, the vertical responses are correlated with the dip angle and also need special attention. Because the limit state of performance level for curvature didn't consider the changing axial force effect in this analysis, it suggested the factor: normalized maximum strain, is a good option to evaluate the seismic performance of piers of near-fault bridge structures.

Chapter 6, the conclusions, discussions and recommendation of this research were presented.

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Character 1 Introduction

1.1 Background

A number of significant inland earthquakes tragically occurred successively, such as the 1994 Northridge earthquake (M_w 6.7), the 1995 Kobe earthquake (M_w 6.9), the 1999 Chi-Chi earthquake (M_w 7.6 - 7.7), the 2008 Wenchuan earthquake (M_w 7.9), the the 2016 Kumamoto earthquake (M_w 7.0), the 2017 Iran-Iraq earthquake (M_w 7.2), and the 2018 Hualien earthquake (M_w 6.4) in Tawan. These seismic events had devastating effects on urban infrastructures, especially the near-fault zones (which is defined as the distance close to the hypocenter less than 20km in this research), (e.g., the situation shown in Figure 1-1). In the near-fault areas, ground motions (GMs) are different from ordinary GMs in that they often contain strong coherent dynamic long period pulses and permanent ground displacements, which might cause the near-fault, spatially extended and flexible structures (such as long-span bridges, embankment, highways, etc.) subjected to the static deformation that occurs close in time to the arrival of dynamic pulse-like ground motions. For example, during the 2008 Wenchuan earthquake (M_w 7.2), the SFB station (at a source distance of 14km) recorded peak ground displacement of 376 cm along vertical direction, and the largest horizontal and vertical permanent displacements are 530 cm and 620 cm, respectively, which caused large collapse and failure on the near-fault bridges as listed in Figure 1-2 [1]. Therefore the characterization, causative parameterization, analytical modeling, and methods for near-fault GMs, as well as their effects on the responses of near-fault engineering structures, are extremely essential and required to be clarified.

Due to near-fault records (in the distance range of 10 to 100 meters from faults) are essentially nonexistent except for a few cases, also even though there are many researchers are focusing on the near-fault ground motions and these approaches are theoretically sound, but rather time consuming and not efficient at very high frequencies, especially when considering realistic layered half-spaces. Additionally, the importance of effects of the near-fault GM on the near-fault structures was underestimated, and a rational and simple seismic design philosophy for such structures has not been established yet.

Therefore, in this research, it proposed an analytical procedure for evaluating the seismic behavior of near-fault bridges considering the fling-step displacement. First of all, a simple hybrid synthesis method for simulating the near-fault strong GMs is presented, it can well consider the velocity pulse and step-like displacement; then by adoption of this proposed method, seismic response of near-fault bridge engineering was conducted, using multi-input displacement waveforms considering the fling-step effects.

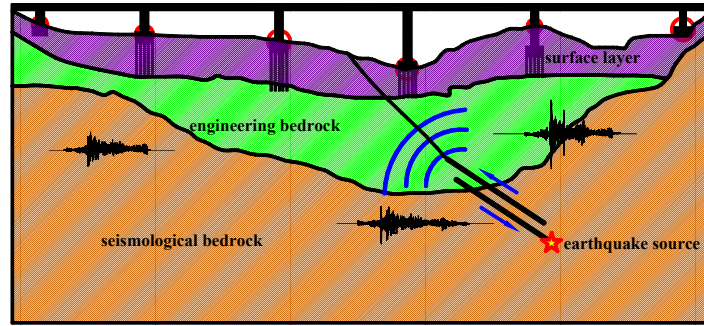


Figure 1-1 Near-fault bridge structure

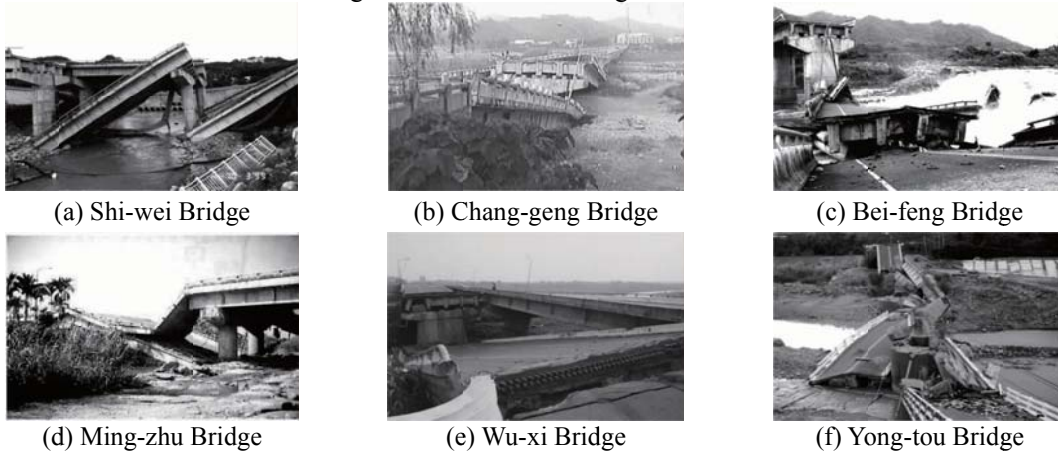


Figure 1-2 Destructive and collapse of bridges near-source [1]

1.2 Significance of the research

There are many engineering applications that require an understanding of the nature of strong GMs adjacent to and/or spanning across potential faults. Our method combines the stochastic Green's function ([2][3][4]) with a theoretical Green's function method ([5][6][7]). The stochastic Green's function method is widely used, especially in Japan. For example, it has been used to estimate the waveforms for the anticipated gigantic Nankai Trough Earthquake ([8], we can fill in the application form to obtain data from the Cabinet Office, Government of Japan). Based on previous work, this study is motivated by the engineering requirements that, near-fault displacement waveforms are needed for developing and simplifying the procedures for seismic design of near-fault bridges. Aimed at these requirements, this research is to develop an analytical procedure to near-fault bridge structures and it contains the following steps:

- 1) Offered a better understanding on near-fault GMs and proposed a synthesis method on the near-fault GMs;
- 2) Evaluated the seismic response of near-fault bridge structures.

This research will give engineers some proper tools necessary to incorporate the findings into the seismic analysis, design and retrofitting of near-fault bridge structures.

1.3 Literature review

1.3.1 Synthesis methods on strong ground motion

Unlike typical far-field motions, near-source earthquake ground motions may be characterized by high-amplitude; long-period pulses in velocity and displacement time-histories, and thus the near-fault region, ground motions at a particular site are significantly influenced by the rupture mechanism and the rupture direction relative to the site, as well as the permanent ground displacement at the site resulting from tectonic movement.

The research of near-fault strong ground motion is a very active field of seismology and earthquake engineering in past decade. Many destructive earthquakes occurring at the end of last century and the beginning of 21 century cause severe damage in near-fault area, and show some specified characteristics of near-fault strong ground motion. The research of near-field strong ground motion is a fundamental research on characteristics and regularity of surface vibration of ground motion, involving a series of basic problems, such as mechanism and modeling of hypocenter, analysis and simulation of the influence of transmission medium, numerical simulation of the influence of site condition, and coherence of near-fault ground motion. Research results can be directly used for the calibration of essential parameter and engineering prediction of ground motion. The engineering prediction, which is the reference for seismic design of various kinds of engineering structures and lifeline systems, is all important basic work in disaster prevention and reduction.

Seismic wave propagation simulation in 3D media is crucial in evaluating failure process of strong ground motion and estimating the damage of earthquake disaster. As for the various simulation and synthesis methods on estimation of strong ground motion, it can mainly be divided into the four types: semi-analytical method, the geometric ray method, integral equation method and wave equation method.

Semi-analytical method for solving seismic wave field through the base function expansion method, the commonly used method are Have localization method [9][10] a global generalized inverse transmission coefficient method [11][12][13]. Aki, and Laner method [14] although the computational efficiency is fast, but it does not deal with problem of seismic wave propagation in complex media; Geometric ray method under the high frequency approximation to simplify the seismic wave theory into ray theory, the main consideration the kinematics characteristics of seismic wave propagation, the method to calculate speed but unable to get full wave field; Integral equation method is using green function to solve the boundary of the area to the boundary integral equation method and boundary element method, etc. In the integral equation method, the calculation of Green's Function is the basis of the near fault strong ground motion simulation content, one of the most commonly used to solve the green's function method is to experience Green's Function method, the

theory of Green's Function method and broad band green function method, one of the most widely used is experience green's function method [15][16] The basic idea of empirical Green's function method is as follows: hypocenter of large earthquake is composed of a series of hypocenters of small earthquakes, records of small earthquakes are selected properly as ground response caused by point sources, namely Empirical Green's Functions which are then overlaid by specified cracking ways to obtain the time-history curve of large earthquake.

In the numerical method for solving the wave equation, the commonly used method with finite element method, finite difference method, the spectral element method and pseudo spectrum method, etc. Based on the control in the form of vibrational equation, the finite element can use unstructured grid, is suitable for dealing with complex boundary. Lysmer and Drake [15][16] as early as the finite element method is applied to the seismic wave method. But finite element method (FEM), a large amount of calculation and computing resources on demand is high, and it is not suitable for processing large scale problems. Pseudo-spectral method is through the Fourier transform derivative term into the space wavenumber domain multiplication operations, and then in the space derivative value is obtained by inverse Fourier transform, first put forward by Kreiss etc. [17], and obtained the development, but the pseudo-spectral method of spatial derivative operator as global operator, conducive to parallel computing, limits the pseudo-spectral method in the application of 3D seismic wave simulation. Spectral element method is a relatively new numerical method for solving the wave equation, it first by the Patera, etc. [18] proposed in fluid mechanics, and then quickly spread to the numerical simulation of seismic wave propagation, the method is combined with regional algorithm using vibrational equation in the form of a spectrum method, the grid must be strict with the internal interface matching, the spectral element method of mass matrix is a diagonal matrix, and it is easy to get effective parallel implementation plan, which is an important tool in seismic wave simulation, has great potential, but also on the method is not very perfect and higher requirements for computer. Finite difference method is one of the methods for solving partial differential equations; the finite difference is first put forward in all kinds of numerical simulation. Alterman, etc. [19] as early as finite difference method for solving double medium P-V wave equation, after many researchers of in-depth study, after decades of development, the solution of the finite difference method has perfect the seismic wave simulation and the strong ground motion model. Proposed including viscoelasticity, all the members of the opposite sex, and in the porous media such as the processing of non-homogeneous medium, and the processing of absorbing boundary, this method is easy to implement parallel algorithms, become a mature method of the seismic wave simulation. But in finite difference in simulation of seismic wave, accurate implementation of the free surface condition is to simulate the real condition in the process of the propagation of seismic wave, Zhang and Chen [9][10] curve grid is

used to simulate heterogeneous medium containing terrain undulating terrain of seismic wave propagation problems, and use the traction mirror method and treatment of free surface condition, Zhang and Chen [20] the method to the variable in the grid, that this method can accurate processing seismic wave propagation problems under the condition of the terrain ups and downs, and greatly improve the operation efficiency, in this article, will use this method to simulate the process of ground motion. Finite difference method has been widely used in conventional strong ground motion simulation due to its accuracy and efficiency.

1.3.2 Theoretical Green's function method

Within a homogeneous, isotropic, unbounded elastic semi-infinite medium or the homogeneous, layered half-space, the seismic motions can be represented by integration over the spatial and temporal region within which the seismic source is acting. Under these simple medium models, the integration is a synthesis from the displacement field of a certain Green Function, and it can be calculated by the generalized R/T (reflection/transmission) method, discrete wavenumber method, the finite element method (FEM) and finite difference method (FEM). The first successful attempt for theoretical calculation of strong motions was made by Aki [14] and Haskell [21] using kinematic source model, given by a propagating dislocation over a fault plane in an infinite homogeneous medium. Kawasaki *et. al.* [22] gave the exact solutions of seismic motions due to a double couple point source in a semi-infinite medium. Sato [23] proposed an approach to derive exact expressions of a series of “ray” for layered media by applying the Cagniard-deHoop method. Sato and Hirata [24] gave a new approach using integral evaluation to compute the seismic motions for layered media including the contributions from dispersive surface waves. Heaton and Helmberger [25] succeeded also in synthesizing strong motions on the basis of a generalized ray theory with the Cagniard-deHoop method for layered models. Bouchon [26] developed a method to compute strong motions for a propagating fault in layered media, based on a discrete wave number method. These synthesis methods calculating the strong motions calculating the strong motions based on a deterministic fault model have been successful in low frequency ranges (< 1 Hz). At high frequencies these methods underestimate the strong motions, when coherent rupture propagation is assumed. Indeterminable factors in the source and the propagating medium may strongly influence the high frequency ground motions. Hartzel and Helmberger [27] attempted to determine a localized area of larger dislocation, based on the analysis of some excellent set of records obtained in the 1979 Imperial Valley, California earthquake and the highly accurate calculation of the discrete wave number/finite element method (Aleksiev and Mikkailenko [28]). They proposed a more complex model with two localized sources better to explain the data.

There are two big problems limiting the development of application of theoretical Green function. The first is, the theoretical Green function is only appropriate in the layered medium, while in reality, the earth is an imperfect elastic medium. Thus, the calculations on theoretical Green function within three-dimensional inhomogeneous medium are highly required, resulting in the complexity and large computation and time consuming. Secondly, since the impenetrability in the earth, it is rather difficult to come out the theoretical Green function mainly due to the lack of geological structures and velocity parameters within the three-dimensional space and localized information.

1.3.2 Stochastic Green's function method

Earthquake is the probabilistic events; the seismologists normally conduct the synthesizing on ground motions by stochastic methods. The basic work were done by Hanks [29][30] and McGuire [31]. They combined the seismic model based on the spectrum of peak amplitude and the concept the high-frequency ground motions is random, and based on the assumption that the far-field acceleration in the elastic half-space is the Gaussian white noise process with Limited band and bandwidth, the spectrum of seismic sources can be represented by a single corner frequency, which is determined by the earthquake size [32]. The most common way to calculate the earthquake spectrum of one site is to divide it into the product of transform functions of seismic source, propagation paths and sites.

Aki [33], Atkinson [34], Boore and Boore [35] focused on the research on seismic source, Aki [36], Boore [35] Atkinson and Boore [37] conducted the research on the propagation path and its relative effects, Boore [34] made a complete conclusion on site effects and the transform function of earthquake motions. By stochastic Green's function, the stochastic point source model is used for small events, and finite fault plane model for near-fault and large ones.

In the theoretical analysis by stochastic method, the seismic radiated energy simulated by static corner frequency is related to the divisions on sub-fault (Beresnev and Atkinson [38], Wang Guoxin *et. al.* and Motazedian *et. al* [39]). And a new variation of this approach is introduced based on a "dynamic corner frequency" in Motazedian *et. al.* [39]. The corner frequency is a function of time, and the rupture history controls the frequency content of the simulated time series of each subfault. The rupture begins with a high corner frequency and progresses to lower corner frequencies as the ruptured area grows. Limiting the number of active subfaults in the calculation of dynamic corner frequency can control the amplitude of lower frequencies. While the dynamic corner frequency meets the conservation of energy physically and qualitatively; aimed at quantitatively describe the conservation of energy, Motazedian *et. al* [39]. proposed the concept of a scaling factor that we will apply to conserve the high-frequency spectral level of subfaults, while it indicated that the unnecessary due to the unambiguous physical meaning in the dynamic corner frequency.

In order to problem that the simulated results calculated by stochastic methods overestimate the low-frequency parts, Motazedian *et. al* [39] proposed the concepts of the ratio of pulsing and sub-fault based on the research Wald *et. al* [40] Due to this concept is established on the real situation that only parts of the fault slip during the whole rupture progress, it is with clear physical meaning, but the exact method of how to determine the ratio of pulsing sub-fault has not presented. If there is lack of small earthquake recordings, the stochastic methods advantages calculation easily and no need the small earthquake recordings.

1.3.3 Other research methods on near-fault ground motion

Somerville *et. al.* [41] developed modifications to empirical strong ground motion attenuation relations to account for the effects of rupture directivity on strong motion amplitudes and durations. The modifications are based on an empirical analysis of near-fault data. The ground motion parameters that are modified include the average horizontal response spectral acceleration, the duration of the acceleration time history, and the ratio of strike-normal to strike-parallel spectral acceleration. Since both of these parameters can be derived from the hypocenter location and the fault geometry, the model of rupture directivity effects on ground motions that we have developed can be directly included in probabilistic seismic hazard calculations. And the developed quantitative models of rupture directivity effects that are based on a set of fault parameters. These models can be used to modify existing attenuation relations to incorporate directivity effects in ground motions used for seismic design.

By using a large number of actual near-fault ground-motion records, Mavroeidis and Papageorgiou [42] proposed a mathematical model for the representation of the coherent component of the near-fault seismic excitations was introduced, along with a simplified methodology for the generation of realistic, broadband, ground motion time histories in the near-fault region incorporating forward directivity. This simple, yet effective, analytical model is proposed for the representation of near-field strong ground motions. The model adequately describes the impulsive character of near-fault ground motions both qualitatively and quantitatively. In addition, it can be used to analytically reproduce empirical observations that are based on available near-source records.

In order to associate particular features of the near-fault ground motions with specific aspects of the faulting process, forward ground-motion simulations are performed and the concept of the S-wave isochrones is exploited. And the computation of the low-to-intermediate frequency synthetic motion is carried out using the discrete wavenumber representation method (Aki [43]). The generalized transmission and reflection coefficient technique (Luco and Apsel [44]) is utilized for the propagation of the wave field through the layered half-space.

A new method of synthesizing artificial accelerogram synthesis, based on the SYNACC method first proposed by Trifunac [45] was presented by the strong motion research group on USC earthquake engineering (V.W. Lee, Todorovska Trifunac and H.L. Wong). It is a combination of an empirical and physical model-based method, and involves unfolding in time a site-specific Fourier amplitude spectrum of ground acceleration obtained for a scenario earthquake by an empirical scaling model. The unfolding consists of representing the ground motion as a superposition of traveling surface Love and Rayleigh waves and of body P and S waves, which propagate with phase and group velocities consistent with the dispersion characteristic of the site geology, approximated by parallel layers. Because the coefficients of expansion are scaled so that the Fourier spectrum of the synthetic motion matches a site specific empirical spectrum, the synthetic motions are consistent statistically with observations within the recording range of typical accelerographs (0.02 ~ 25 Hz). A Uniform Hazard Fourier spectrum or any specified Fourier spectrum can also be used as a target spectrum. The output consists of synthetic accelerations, velocities and displacements, and point strains, rotations and curvatures, all at a point or at an array of points. Such time histories are useful for the design of spatially extended structures, like pipelines, bridges and tunnels.

Graves and Pitaka [46] proposed a broadband ground motion simulation procedure, which is a hybrid technique that computes the low frequency and high frequency ranges separately and then combines the two to produce a single time history (Hartzell [47]). At frequencies below 1 Hz, the methodology is deterministic and contains a theoretically rigorous representation of fault rupture and wave propagation effects, and attempts to reproduce recorded ground motion waveforms and amplitudes. At frequencies above 1 Hz, it uses a stochastic representation of source radiation, which is combined with a simplified theoretical representation of wave propagation and scattering effects. The use of different simulation approaches for the different frequency bands results from the seismological observation that source radiation and wave propagation effects tend to become stochastic at frequencies of about 1 Hz and higher. And then Graves and Pitaka [48] extended this to incorporate the functions developed using equivalent linear response analysis as implemented in the empirical model of Campbell and Bozorgnia [49].

Due to the very near-fault locations of stations and issuers grading the singular nature of frequency-wavenumber-integration Green's functions, Dreger *et. al.* [50] used the finite-difference code, to simulate three- component time histories for finite-source rupture models. With the finite-difference method, the discretization of the source process is matched to the discretization of the wave propagation problem, eliminating the need to interpolate one coarse field when convolving the two. And the finite-difference method does not have explicit singular expressions and satisfies the

complete wave-field elastodynamic solution to a maximum limiting frequency that depends on the discretization of the model and the order of the finite-difference equations.

Based on a comprehensive study of around 50 near-fault ground motion records, Bray and Rodriguez-Marek [51] proposed that near-fault, forward-directivity motions can be adequately represented by simplified time histories consisting of one or a few sine-pulses. The number of pulses is likely related to slip distributions in the causative fault, and consequently is difficult to predict. But, their attenuation relationships have relatively high standard deviations indicating significant uncertainty in the prediction of PGV and pulse period. This uncertainty may be reduced somewhat as additional records from future events become available. Hence, aggressive installation of strong motion stations that are located close to highly active faults is warranted.

Hisada [5][6], Hisada and Bielak [7] proposed an efficient mathematical method for computing the near-fault strong ground motions in a layered half-space, giving explicit consideration to the static offset due to surface faulting. In addition, the combined effects of “fling step” and “rupture directivity” on the near-fault ground motions are investigated.

Harada and Ohsumi [52] presented the definition of the differential ground motions on free-field ground surface, and then describe numerical examples to show the characteristics of the differential ground motions as well as those of near-field ground motions. They proposed a discrete three fold Fourier transform of the analytical forms of seismic wave field to synthesize the differential ground motions and the ground motions. The ground motions are derived by a stiffness matrices-based formulation of the physical processes of propagation of the seismic waves generated by a kinematic fault rupture model buried in horizontally layered media. In consequence, the differential ground motions and the ground motions in this paper can be synthesized in quite accuracy, because the seismic wave field in our method is represented as a simple sum of plane waves with the complex amplitudes given by the analytical forms in frequency wave number domain.

Because the seismic wave motions near a fault are quite complex in time and space by showing the vortexes of waves, it may be considered to be appropriate that a response analysis of long structures is performed on the basis of three dimensional model of them, thus accordingly they [53] simulated the input seismic wave motions to conduct the three dimensional model nonlinear response analysis of viaduct crossing a strike slip fault compared with those of the two dimensional model.

Miyatake [54][55] conducted the numerical simulations of earthquake source process by a three-dimensional crack model on rupture process and seismic waves and spectrum. In the rupture process [54], the displacement-time history on the crack is obtained from numerical solutions for equations of motion with a finite stress fracture criterion. If the strength is uniform and slightly above the initial stress the rupture propagates elliptically with a nearly P wave velocity in the direction of the

initial stress and with a nearly S wave velocity in the direction perpendicular to it, but if the strength is at a considerably high level, the rupture propagation tends to become nearly circular with a velocity close to that of S wave. Non-uniform strength distributions considered here are the cases with a line barrier which divides the fault into two parts, with a square-shaped block barrier in the center of the fault, random strength distribution of Weibull's type and a random barrier distribution. And the cases study suggests that the triggering mechanism of multiple shocks is the arrival of "waves." Block and line barrier can generate nearly the same multiple shock's seismic waves. In the case of random strength distribution with no strong barriers, no unbroken region remains. In the case of random barriers, the rupture propagates irregularly and in some cases there remain unfractured regions. The displacement time history in these cases does not show very complex shapes. For the seismic waves and spectrum [55], they dealt with seismic waves in a very near-field above the fault, and in near-fields in an ordinary sense, and results that: 1) Seismic waves immediately above the fault are largely affected by the fracture of a fault segment nearest to the observing point; 2) When the rupture passed by this point, a large sharp phase appears. The rupture velocity may be estimated from the arrival time of the phase; 3) Seismic waveforms become complicated due to the existence of a barrier. An estimate of the barrier's shape and its location from these phases is a problem left unsolved. If we can get data at many points very close to the fault, however, it might be possible to find the existence of a barrier and estimate its location from irregularities of large phases described in the above. Also complex seismic waves including multiple shocks are generated mainly by irregular propagation of the rupture, rather than by complex source time functions. This complex rupture propagation radiated high frequency seismic waves.

Onishi and Horike [56] examined behaviors of close-fault strong motions calculated by the extended stochastic simulation method. Horizontal and vertical peak ground velocities (PGV) calculated by this method behave quite differently from PGV calculated by the attenuation relationships. With decreasing the source distance, horizontal PGV grows monotonically and vertical PGV rapidly decreases, while PGV for both components calculated by the attenuation relationships saturates in close-fault zones. Comparison between the near-field term, the intermediate term, the far-field term in the elastic-wave Green's function and the sum of the three shows that these differences are generated because of a neglect of the near-field term and the intermediate term in the stochastic simulation method. This comparison also suggests the frequency range dealt with by the extended stochastic method should be confined to higher frequencies for shallower earthquakes.

The Okada [57] model calculates analytical solution for surface deformation due to shear and tensile faults in an elastic half-space. This model is widely used to simulate ground deformation produced by local perturbation like tectonic faults (earthquakes) or volcanic dykes (magmatic

intrusion). Given rectangular fault geometry (length, width, depth, strike, dip) and 3-component dislocation amplitude (rake, slip and open), it computes the displacements, tilts and strains at the free-surface.

Okada [58] proposed a complete suite of closed analytical expressions is presented for the surface displacements, strains, and tilts due to inclined shear and tensile faults in a half-space for both point and finite rectangular sources. These expressions are particularly compact and free from filed singular points which are inherent in the previously stated expressions of certain cases. The expressions derived here represent powerful tools not only for the analysis of static filed changes associated with earthquake occurrence but also for the modeling of deformation fields arising form fluid-derived crack sources. A complete set of closed analytical expressions [58] is presented in a unified manner for the internal displacements and strains due to shear and tensile faults in a half-space for both point and finite rectangular sources. Theses expressions are particularly compact and systematically composed of terms representing deformations in an infinite medium. A term relater to surface deformation and that is multiplied by the depth of observation point. Several practical suggestions to avoid mathematical singularities and computational instabilities are also presented. The expressions derived here represent powerful tools both for the observational and theoretical analyses of static field changes associated with earthquake and volcanic phenomena.

Kagawa [59] proposed a developed stochastic Green's function method having more accuracy to be used in the hybrid method to simulate near field strong ground motion reasonably in wide period range. Furthermore, frequency dependent radiation pattern and site response of layered media are introduced. Calculated near-filed ground motion by the proposed stochastic method has sufficient accuracy use even for longer period range at a site on basement of flat layered structure.

Baker [60] proposed a quantitative scheme for detecting pulses is being developed. The procedure uses a wavelet-based signal decomposition to identify and extract the largest velocity pulse from a ground motion, and if the extracted signal is large relative to the remaining signal, the ground motion is classified as a pulse-like motion. The identified pulse-like ground motions are then being used in dynamic nonlinear structural analyses to identify relationships between near-fault directivity effects and structural response.

In order to tackle the near-fault ground motions but complicated problem, Kojima Kotaro and Takewaki Izuru [61][62] introduced the double impulse input as a substitute of the fling-step near-fault ground motion and a closed-form solution of the elastic-plastic response of a structure by the "critical double impulse input" is derived, it is intended to simplify these typical near-fault ground motions by a double impulse (Kojima *et al.* [61]) and a triple impulse. This is because the double impulse and triple impulse have a simple characteristic and a straightforward expression of the

response can be expected even for elastic–plastic responses based on an energy approach to free vibrations. Furthermore, the double impulse and triple impulse enable us to describe directly the critical timing of impulses (resonant frequency) which is not easy for the sinusoidal and other inputs without a repetitive procedure.

1.3.4 Hybrid simulation methods

Hybrid methods that combine deterministic and stochastic estimations for lower and higher frequencies are widely used to simulate broadband strong ground motions (e.g., Kamae *et al.* [63], Pitarka *et al.* [64]). The matching frequencies between the two estimations are usually around 0.5 ~ 2 Hz, which are the most important frequency range for engineering structures. However, the two estimations sometimes show large differences in amplitudes, arrival times, and waveforms in that frequency range because of completely different methodologies. Two approaches have been proposed to remedy this problem and connect the two results more smoothly. One approach is to introduce stochastic randomness in deterministic methods at higher frequencies. For example, Herrero and Bernard [65] proposed the k -squared model, which is an omega-squared model based on a kinematic fault model using randomness in slip distribution at higher frequencies. Hisada modified the model by introducing pseudo-dynamic slip and the k -squared random distributions not only in slip but also in rupture time at higher frequencies [66][67]. More recently, various similar models considering pseudo- dynamic slips and randomness for slip and rupture time (or rupture velocity) have been proposed (e.g., Guatteri *et al.* [68], Hartzell *et al.* [69]). These approaches are theoretically sound, but rather time consuming and not efficient at very high frequencies, especially when considering realistic layered half- spaces, because huge numbers of Green's functions are necessary to maintain the continuity of the rupture front (about 8–10 source points per wavelength).

The second approach introduces deterministic coherency in stochastic methods at lower frequencies. Kamae *et al.* (1991) proposed the stochastic Green's function method, in which the far-field S-waves from the stochastic point source (Boore [2]) are super- posed on an extended fault plane using the empirical Green's function technique [70]. To extend this method to lower frequencies, Kagawa [59] proposed a trial-and-error technique, in which several stochastic Green's functions are simulated, and one can choose synthetics that are consistent with deterministic results. Onishi and Horike [56] improved the method by introducing the theoretical radiation coefficients of the P-,SV-, and SH-waves and array tracing technique in layered half-spaces. These stochastic approaches are much faster to compute than the theoretical approaches because it basically requires only one source point per sub-fault using simple Green's functions.

Thus, in this research, the stochastic Green's function method is widely used, especially in Japan. For example, it has been used to estimate the waveforms for the anticipated gigantic Nankai

Trough earthquake [8], we can fill in the application form to obtain data from the Cabinet Office, Government of Japan). Our motivation is to simulate ground motions based on this widely used stochastic Green's function method in combination with a method that can accurately express the permanent displacement. And then we introduce the proposed hybrid method for near-fault ground motions, including permanent displacements to conduct the seismic response of near-fault bridge, and it resulted that the proposed method is a useful tool for evaluating near-fault ground motions for designing bridges and other structures.

1.3.5 Seismic design methods of near-fault bridge structures

The strong earthquake events occurred in recent decades caused serious damage to or even the collapse of a considerable numbers of major bridges that were apparently designed to withstand strong seismic forces, as shown in Figure 1-3. These failures include lateral offsets and rising or sinking of the ground. In some cases the fault movement may occur beneath bridge structures as shown in Figure 4-2. Such failures are of great concern to earthquake engineers because they may reoccur near the potential active fault area any time in the near future.

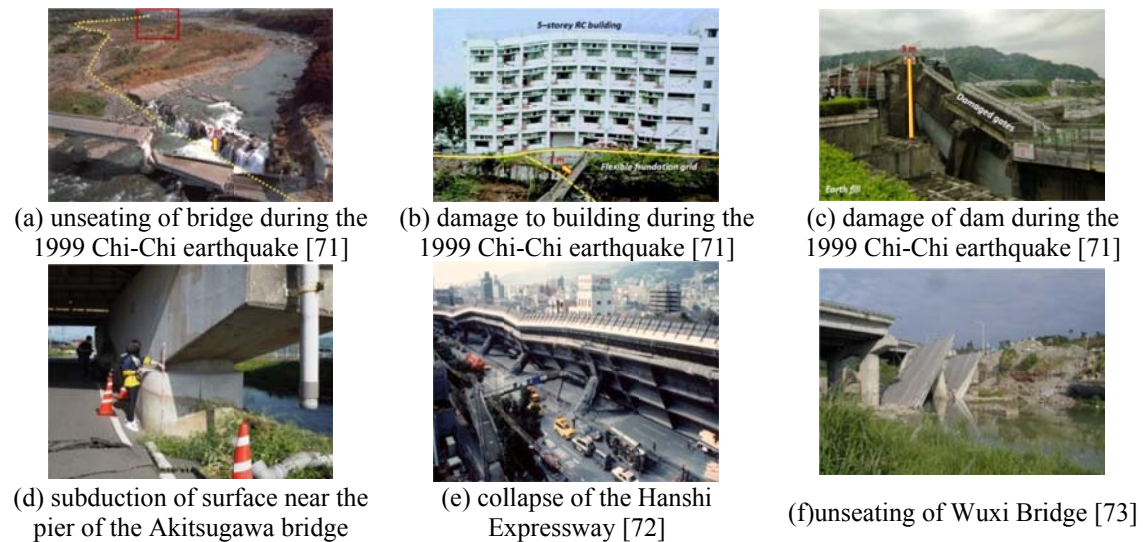


Figure 1-3 Damage to structures caused by strong earthquakes

At present, there are many research conducted the seismic response of near-fault bridges. Goel and Chopra [74] developed a response spectrum-analysis procedure and a linear static-analysis procedure for estimating the dynamic part of the seismic demand responses in linearly elastic bridges crossing fault rupture zones. They demonstrated that both these procedures, when combined with the quasi-static demands due to the ground offset across the fault, provided estimated peak responses that were close to the exact results from observed response analysis. Goel and Chopra[75] extended these

methodologies to estimate the seismic demand for ordinary bridges deforming into their inelastic range. Ucak et al. [76] investigated the effect of the fault rupture zone traversing a seismically isolated bridge utilizing a finite element model of a section of the Bolu Viaduct, Turkey; the input signal was a set of synthetic broadband strong ground motions simulated for the Bolu Viaduct site based on the 1999 Duzce earthquake. They concluded that the large difference in the response of the piers crossing the fault was a direct result of the uniform spatial excitation, the fault rupture traversing the bridge, and the imposed substantial permanent tectonic displacement along the fault trace. The effects of fling are typically removed by standard processing of strong ground motion, namely high-pass (HP) filter and baseline correction, performed to remove low-frequency noise. Yang et al. [77] used ground motions of the Lucerne Valley record from the 1992 M_w 7.2 Landers earthquake with and without fault crossing considerations to study the seismic performance of a 10-span seismically isolated bridge crossing a fault. They found that all the response values related to the permanent offset were significantly underestimated; their results indicate the strong effect of permanent ground displacement on the dynamic response of spatially extended engineering structures crossing fault rupture zones. Other studies [78][79] revealed that an impulse ground motion near an earthquake fault, and in particular the amplitude and period of the velocity pulses, may affect the performance of structures in that area.

Near-fault ground motions are ground motions that occur near an earthquake fault. Near-fault motions recorded near the epicenter are known as near-epicenter records, while those obtained along the fault in the direction of rupture are referred to as being in the region of forward directivity. To evaluate the inelastic demand on a structure due an earthquake, it is necessary to have a method of defining the level of inelastic demand or severity. This method should work well for far-fault shaking and be modified easily to consider the effect of near-fault shaking. At present, some methods are adopted to evaluate on the seismic demand on a structure due to an earthquake. Here lists the main three methods.

a) Substitute structures approach

ATC-40 [80] and ATC-43 [81] use approaches based on the substitute structure method [82]. The substitute structure method idealizes a yielding structure as an equivalent elastic structure with an elongated natural period and additional viscous damping. It assumes that the structure oscillates about the zero displacement position. Both the elongated period and additional damping are generally considered to be functions of the ductility of the structure. For structures with different periods T and the same target ductility demand μ , a plot of peak spring force divided by structural mass F_u/m versus peak displacement du may be made. According to the substitute structure method, points on this plot should lie directly on top of a plot of the elastic spectral peak spring force divided by the structural

mass F_e/m versus the elastic displacement d_e using an increased damping associated with the target ductility demand μ .

Because near-fault excitation may cause large displacements in predominantly one direction, the substitute structure method may significantly underestimate inelastic demands due to near-fault shaking [83]. According to the substitute structure method the inelastic response for a target ductility μ of 2 (F_μ/m , d_μ) should be nested inside the elastic ($\mu=1$) response (F_e/m , d_e) in the same way as increased damping would decrease the response. Instead, the figure shows that, for periods near 0.9 and 1.8 s, the inelastic response is sometimes outside the elastic response range. This implies a negative damping ratio would be needed to predict the actual response for these periods using the substitute structure method. For oscillators with other periods, different damping ratios would be required. Because the effective damping ratio is not constant for certain ductility and because negative damping ratios may be required to predict the inelastic response from the elastic response, it is obvious that the present form of the substitute structure method cannot always adequately estimate inelastic displacement demands due to near-fault shaking.

b) Pulse modeling approach

If a ground motion record consisting of a pulse of a certain shape gives inelastic demands that are similar to those from actual near-fault records, then that pulse may be used to assess near-fault shaking demands. Procedures that use pulses have been proposed [84][85], and sophisticated computer models are under development. However, simple reliable design methods have not yet been developed because reasonable methods for determining pulse magnitude and period at a site are not yet available.

c) Displacement amplification approach

Methods for obtaining peak displacements most commonly used in design involve scaling yield or elastic displacements. They are included in documents such as FEMA273 [86], JRA [87], UBC [88], and FEMA302 [89].

The FEMA273 [86] linear static procedure computes the ultimate displacement for an inelastically responding system du as $C1C2C3d_e$. For normal, long period structures $C1C2C3$ is unity and du is equal to the elastic displacement d_e . The displacement demand du becomes greater than d_e when the period of the structure T_s is less than a characteristic period. For short-period structures that are strong enough to respond elastically or near-elastically, this method unreasonably predicts displacements that may be significantly greater than the elastic level displacement because $C1C2C3$ is greater than unity [85].

The FEMA273 [86] nonlinear static procedure computes the ultimate displacement for an inelastically responding system du as $C0C1C2C3d_e$, where $C1$ is given in Section 1). The elastic

displacement magnification factor CI recognizes the increase in displacement demand of short-period structures; CI becomes unity when the lateral force reduction factor R is unity correctly indicating that the total displacement will be the elastic displacement d_e when the structure is strong enough to remain elastic. However, since the hysteresis shape modification factor $C2$ does not necessarily equal unity when R is unity, the predicted peak displacement may be greater than d_e even when the structure is strong enough to remain elastic.

JRA [87] considers the lateral force reduction factor R and yield displacement d_y to compute the ultimate displacement d_u . The approach is based on the “equal energy method” [90], and it is used for structures of all periods.

According to FEMA302 [89], the total displacements of normal structures of all periods d_u are C_d/R times the elastic displacement d_e , where C_d/R ranges from 0.70 to 0.90. UBC (ICBO 1997) displacements are computed as $d_u = 0.7d_e$. Both of these methods estimate total displacements d_u , which are significantly less than the elastic displacement d_e even though the elastic displacement is generally regarded as a good rule for estimating the design displacement of normal structures on normal sites with periods greater than about 0.7 s [90][91]). FEMA302 and UBC displacements from these methods are also significantly less than those from FEMA273 [86] and JRA [87], where d_u is commonly in the range of 1.0-2.0 times the elastic displacement d_e .

It may be seen that present code methods used to estimate displacements of inelastically responding structures give widely differing displacement predictions. Some methods consider higher demands for short-period structures while others do not. Some methods do not even correctly predict the response of a structure that is strong enough to remain elastic and others have inconsistencies meaning that they cannot easily be modified to describe inelastic demands caused by near-fault as well as far-fault shaking.

1.4 Research objectives

1.4.1 A procedure for near-fault bridge analysis

Owing to the complexities of near-fault ground motion, an analytical procedure to model the seismic response of near-fault bridges is proposed for engineering design purposes, which contains two important topics: (1) generate the near-fault ground motions by a hybrid synthesis method (2) seismic response of bridge engineering. It is schematic in the following Figure 1-4.

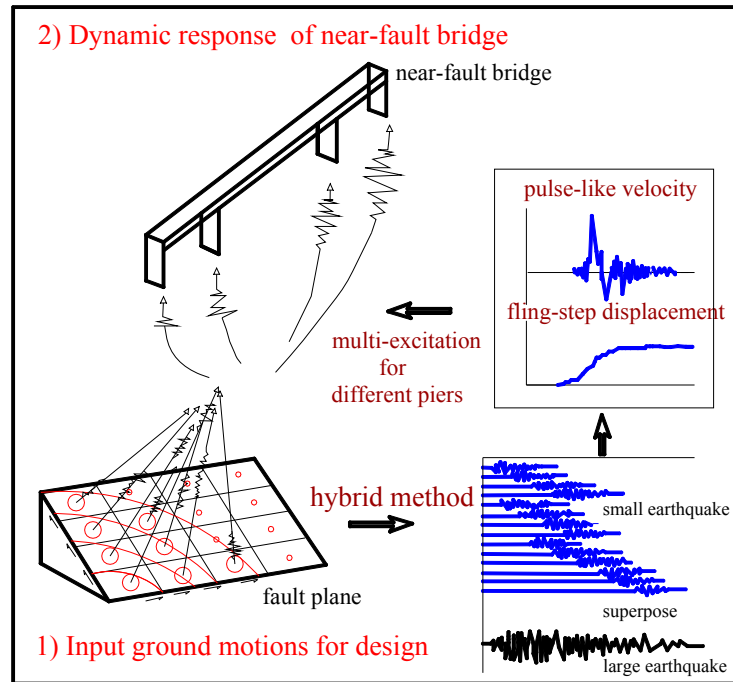


Figure 1-4 Analytical procedure for seismic analysis of near-fault bridge

1.4.2 Hybrid synthesis method

The main objectives of this research are to generate the near-fault GMs efficiently and to develop an appropriate analytical procedure for evaluating the seismic response of bridge engineering considering the near-fault dislocation. The specific objectives are:

- Make it clear the characteristics of near-fault ground motions and its effects on the near-fault bridge structures;
- Propose a new hybrid synthesis method for near-fault GMs: a hybrid synthesis method was proposed to get effective earthquake excitations. Additionally, the 1999 Chi-Chi earthquake was calculated for verify the validation of this hybrid method, and further evaluated by two simple fault models (a strike-slip and dip-slip model);
- Evaluated three affected factors: the buried depth, soil conditions and Mw.

1.4.3 Evaluation on seismic behavior of near-fault bridge structures

The following listed the evaluations on near-fault bridge structures.

- Evaluate the seismic behavior of near-fault bridge engineering: the linear and non-linear seismic response of near-fault ground motions will be conducted by numerical calculation;
- Evaluate the effects of strike angle, dip angle and rake angle: the effects of different crossing angle between the fault and bridge, and the focal mechanism on seismic response of near-fault bridges will be evaluated.

1.5 Summary and outline

Figure 1-5 illustrates the research flowchart of all chapters in this study. The thesis consists of six chapters, which yields most information on work done in this research. These works included an analytical procedure of near-fault GMs and its evaluation, and numerical simulations on near-fault bridge. The content on each chapter can be briefly described as follows.

Chapter 1 presented a general overview of this study and detailed objectives and aims of this research. In this chapter, reviews on previous studies related to simulations and evaluations on near-fault GMs and dynamic response of near-fault bridge engineering, it also tried to summarize the current practices as well as research in this field.

Chapter 2 detailed the characteristics of near-fault ground motions, including the focal mechanics analysis and 5 main features, including concertation effect, permanent displacement effect, directivity effect, hanging wall effect and intensive pulse-like velocity effects, as well as the effects of spatial varied near-fault ground motions on bridge engineering.

Chapter 3 included the hybrid synthesis method for near-fault ground motions. The hybrid synthesis method is a combination between the stochastic Green's function and theoretical Green's function, in further, the stochastic Green's function was modified to consider the complete waveform of near-fault ground motions. Additionally, this hybrid synthesis method was adapted to two simple fault models for further introduction.

Chapter 4 conducted the evaluation on the near-fault ground motions generated by the proposed hybrid synthesis method, mainly focused on the 3D vectors on near-fault area of ground motions: its acceleration-, velocity-, and displacement field. Moreover, the evaluation on the effects of buried depth, soil conditions and M_w were explored by this hybrid synthesis method.

Chapter 5, based on the multi-excitation method, the OpenSees was performed as a useful numerical simulation tool for linear and non-linear analysis of a fault-crossing bridge. It focused on general seismic responses of bridge. Furthermore, the effects of strike-, dip- and rake-angles on seismic behaviors of fault-crossing bridge were explored.

Chapter 6, the conclusions, discussions and recommendation of this research were presented.

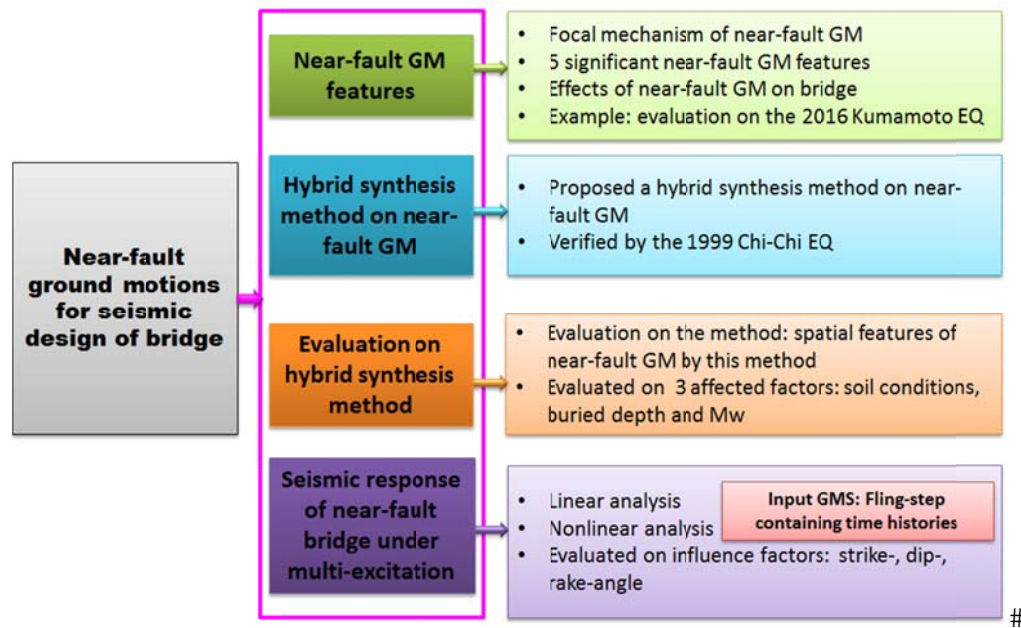


Figure 1-5 Research flowchart

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Chapter 2 Characteristics of near-fault ground motions and effects on bridge structures

The near-fault GMs are significantly different from the far-field GMs, it mainly features the concentration effect, the permanent (fling-step) effect, the directivity effect, the pulse-like velocity effect, and the hanging wall effect. Even though these characteristics may not appear in a single earthquake, they could be testified by strong motion recordings and numerical simulation of near-fault GMs. In order to make clear these basic characteristics, this section clarified and detailed these effects, it first presented the focal mechanics of near-fault ground motions, then based on occurred earthquake database and/or observation data, it clarified the near-fault GMs, and the field survey on the 2016 Kumamoto earthquake was presented to further clarify the characteristics of near-fault GMs and the seismic behavior of near-fault buildings.

2.1 Focal mechanics of near-fault ground motions

Theoretically, relayed on Aki and Richards (Eq. 4.32) [1], the complete solution of wave propagation from double-couple in an infinite elastic and homogeneous half-space can be obtained by Eq. (2-1) in a form that natural brings out the radial and transverse components of motion (the Cartesian and spherical polar coordinates are shown in Figure 2-1):

$$\mathbf{u}(\mathbf{x}, t) = \frac{M_o}{4\pi\rho} \left[\begin{aligned} & \mathbf{R}_{\theta\phi}^N \frac{1}{r^4} \int_{r/\alpha}^{r/\beta} \tau f(t-\tau) d\tau \\ & + \frac{1}{\alpha^2} \mathbf{R}_{\theta\phi}^{IP} \frac{1}{r^2} f(t-\frac{r}{\alpha}) + \frac{1}{\beta^2} \mathbf{R}_{\theta\phi}^{IS} \frac{1}{r^2} f(t-\frac{r}{\beta}) \\ & + \frac{1}{\alpha^3} \mathbf{R}_{\theta\phi}^{FP} \frac{1}{r} \dot{f}(t-\frac{r}{\alpha}) + \frac{1}{\beta^3} \mathbf{R}_{\theta\phi}^{FS} \frac{1}{r} \dot{f}(t-\frac{r}{\beta}) \end{aligned} \right] \quad (2-1)$$

where,

$f(t)$: is the normalized slip-time function, and the final slip is assumed equal to 1;

$\dot{f}(t)$: is differential of $f(t)$;

$R_{\theta\phi}^N$ $R_{\theta\phi}^{IP}$ $R_{\theta\phi}^{IS}$ $R_{\theta\phi}^{FP}$ $R_{\theta\phi}^{FS}$: represent the radiation patterns of near-field terms, the intermedia-field, and the far-field terms respectively, and they are expressed in Eq. (2-2).

$$\begin{aligned} R_{\theta\phi}^N &= 9\sin 2\theta \cos \phi \hat{\mathbf{r}} - 6 \left(\cos 2\theta \cos \phi \hat{\boldsymbol{\theta}} - \cos \theta \sin \phi \hat{\boldsymbol{\phi}} \right) \\ R_{\theta\phi}^{IP} &= 4\sin 2\theta \cos \phi \hat{\mathbf{r}} - 2 \left(\cos 2\theta \cos \phi \hat{\boldsymbol{\theta}} - \cos \theta \sin \phi \hat{\boldsymbol{\phi}} \right) \\ R_{\theta\phi}^{IS} &= -3\sin 2\theta \cos \phi \hat{\mathbf{r}} + 3 \left(\cos 2\theta \cos \phi \hat{\boldsymbol{\theta}} - \cos \theta \sin \phi \hat{\boldsymbol{\phi}} \right) \\ R_{\theta\phi}^{FP} &= \sin 2\theta \cos \phi \hat{\mathbf{r}} \\ R_{\theta\phi}^{FS} &= \cos 2\theta \cos \phi \hat{\boldsymbol{\theta}} - \cos \theta \sin \phi \hat{\boldsymbol{\phi}} \end{aligned} \quad (2-2)$$

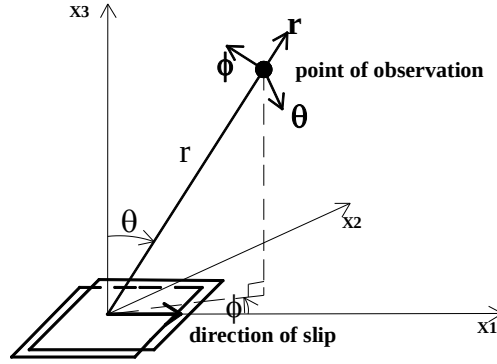


Figure 2-1 Cartesian and spherical polar coordinates for analysis of radial and transverse components of displacement radiated by a shear dislocation and average slip

These radiation patterns explicitly display a radial component, proportional to $\sin 2\theta \cos \phi \hat{r}$, and a transverse component, proportional to $(\cos 2\theta \cos \phi \hat{\theta} - \cos \theta \sin \phi \hat{\phi})$. The important property brought out by Eq. (2-1) is that these are the only two radiation patterns needed to obtain a complete picture of all the different terms in the displacement field radiated from a shear dislocation (double couple). Figure 2-2 (a) shows the way in which the radial component varies in magnitude for different directions (θ, ϕ) , and transverse component varies in both magnitude and direction in Figure 2-2 (b), detailed can be viewed in [1]. Only the radial component is present for the far-field P-wave, and only the transverse component is present for the far-field S-wave. However, the intermediated-field displacements, both P-waves and S-waves, involve both radial and transverse components, as does the near-field displacement. Thus, the near-source ground motions are rather complicated.

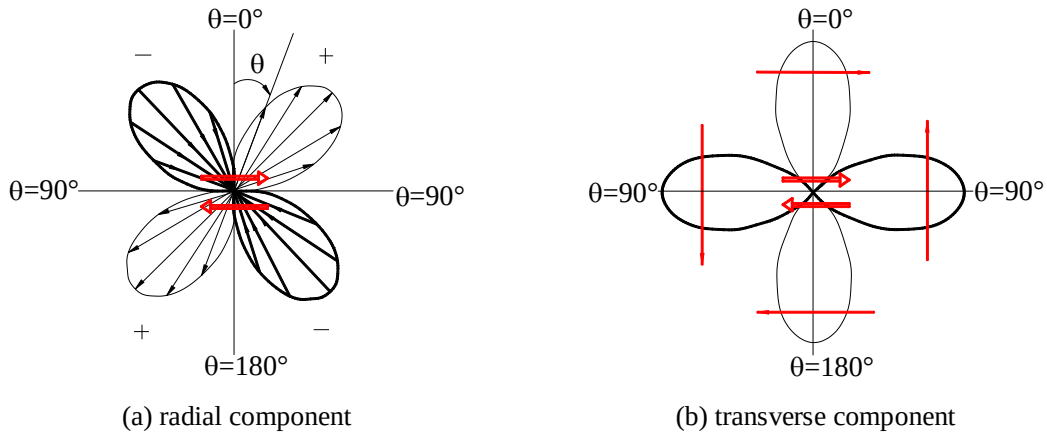


Figure 2-2 Diagram for the radiation pattern of displacement due to a double couple

The complete waveform of equation (2-1) emphasized that, the near-field and intermedia-field terms are proportional to the dislocation time function and source time function, while the far-field

term is proportional to the rupture velocity and the geometric decay of the near-field and intermediate-field terms are much larger than that of the far-field terms.

The geometric decay of the near-field term corresponding to different source time functions exhibits different attenuation behavior. Generally speaking, the near-field terms are attenuated on the order of $1/r^4$ to $1/r^2$, intermediate-field terms decrease as $1/r^2$, and $1/r$ for far-field terms, therefore, in the far-field area, near-field and intermediate-field terms are rather small and can be neglected, thus the permanent displacement in this area can also be neglected. While in the near-fault area, because the near-field and intermediate-field terms are very large and can not be ignored, so the general permanent displacement of the ground are relatively large, except the buried depth of the fault is corresponding large, which illustrated that the near-field and intermediate-field terms must be taken into consideration when calculating the Green's function for GMs by kinetic source model, and large permanent ground deformation exists only in a limited area vicinity to the fault.

2.2 Characteristics of near-fault ground motions

From recording database and numerous analyses, the 1966 Parkfield earthquake and the 1971 San Fernando earthquake can be considered as the milestones in recognition of the damaging effects of near-fault ground motions since both occurred in the vicinity of populated areas. In order to conduct the seismic analysis and evaluations on near-fault structures, it is necessary to make it clear the characteristics of near-fault GMs.

2.2.1 Concentration effect

The results of strong earthquake observation data and numerical simulation show that, the near-fault GM is attenuated rapidly with the increasing of fault distance and the strong GM is concentrated in a narrow range centered on the surface of the fault, and seismic evaluation analysis showed that this area is the most serious damage area, which was well demonstrated in Figure 2-3, it is the contours of PGA during the 1999 Chi-Chi Earthquake (data was available at [2]). The concentration of strong earthquakes in the near-fault area is almost all close to the fault on both sides of the fault, the earthquake damage outside of this area is significantly reduced; the 1999 Kocaeli earthquake in Turkey is also concentrated in the narrow range of both sides of the fault [3].

The main reason for the serious damage caused by the near fault is the large and/or surface rupture of the GM intensity. Numerical simulations also show the concentration of near-fault strong ground motion [4] [5] [6].

The concentration effect of near-fault ground motions show that the distribution of near-fault ground motion is regular, but this is only a general qualitative conclusion. At present, the qualitative and quantitative relationship between the distribution area and magnitude of strong earthquakes, the

fault types, the buried depth, and the rupture process of faults are still a major research topic of near-fault ground motions. Inoue *et al.* [5], Liu Qifang *et al.* [6] studied the relationship between the distribution of near-fault strong earthquakes and fault depth by the kinetic and kinetic source models, respectively. These studies show that the fault depth closing to the fault dynamic distribution has an important effect: with the increase of fault depth, the intensity of near-fault GM is obviously decreased. The quantitative prediction of the strong earthquakes for potential fault is urgently required for seismic-resistance engineering structures.

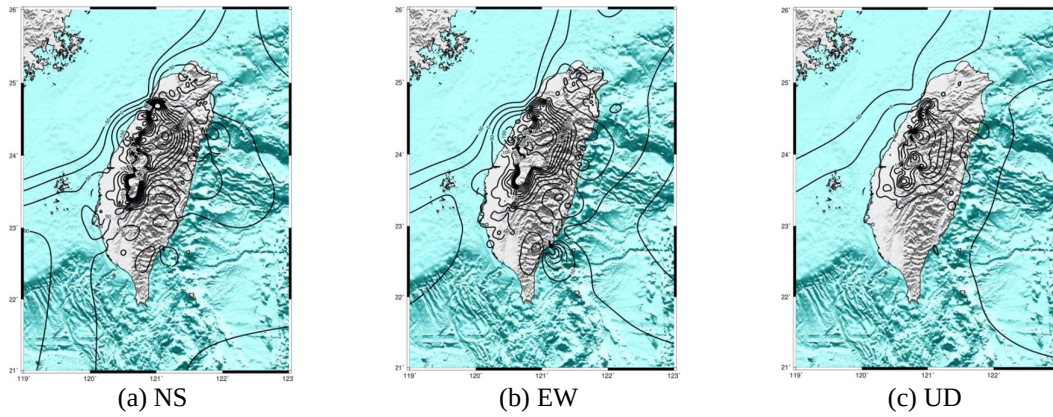


Figure 2-3 PGA contour of the 1999 Chi-Chi earthquake, Taiwan

2.2.2 Permanent (fling-step) effect

Fling is the engineering term for the effects of the permanent tectonic offset of a rupturing fault in the recorded GMs near the fault. It is expressed by a one-sided pulse-like velocity and a nonzero final displacement at the end of shaking.

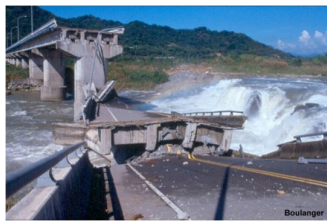
Fling effects are polarized in strike-parallel and strike-normal components for strike-slip and dip-slip events respectively. The 1985 Michoacan Mexico earthquake, the 1999 permanent earthquake and the 1999 Kocaeli earthquake in Turkey obtained significant permanent ground displacement observations. For example, the 1999 Chi-Chi earthquake in Taiwan observed the surface rupture length more than 100 km caused by the Chelungpu fault, the eastern side of the fault (hanging wall) within the range of 15 km moved towards northwest direction and with uplift movement, the relative displacement of hanging wall to the footwall is, 1 m in the southern section and 8 m in the northern section of the fault [7].

The permanent deformation of the ground can be explained by the theory of elastic dislocation. Under the action of tectonic stress, the elastic strain on both sides of the fault is accumulated to the rupture strength of the fault plate, the fault suddenly ruptures, the accumulated strain is released and the elastic bounce occurs, causing the tectonic earthquake. Elastic rebound produced the ground motion and also caused the permanent static ground displacement. The theory of elasticity and static

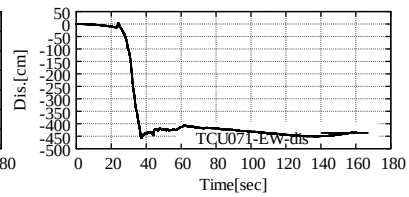
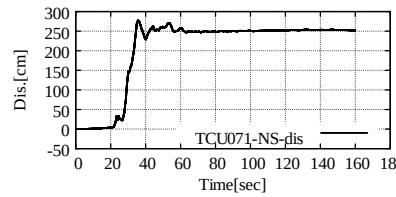
offset is successfully used to analyze the ground displacement field, strain field and stress field caused by fault displacement [8][9][10].

The surface rupture will cause devastating damage to the structure located in the near-fault area, as shown in Figure 2-4 (b) and (c), in (b) (data was available at [2]). The waterfall in the river channel did not exist before the earthquake, but resulted from about 6 m of fault offset in the bedrock. The fault cut across this bridge, separating the supporting columns and causing collapse of the decks, and about 9 m of fault offset developed across the Shinkan dam in (c), respectively.

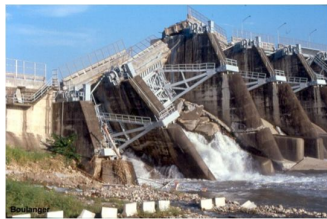
However, the influence of surface permanent deformation on the failure mechanism of the near-fault structure is not clear and under research. In addition, the static offset caused by fling is typically removed from ground motion records via processing (generally a combination of filtering and baseline correction), which is used to remove noise and errors due to baseline offsets. At present, it is consisted that the surface permanent displacement might cause some damages to the long-span structures. For example, if a bridge is crossing or located near a potential strike-slip fault, due to the permanent displacement in the near-fault area attenuates very fast, the spatial various of permanent displacement might cause quite different seismic responses of the bridge in the far-field and near-field sections, causing the bridge to twist, or even collapse of the bridge.



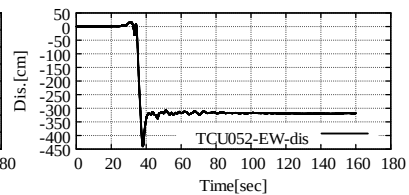
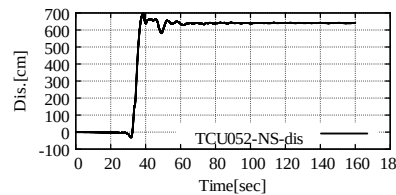
(a) damage of bridge [11]



(c) recording of fling-step displacement at TCU071



(b) Shinkan dam [11]



(c) recording of fling-step displacement at TCU052

Figure 2-4 Permanent displacement (fling-step) and its effects on structures damage in the 1999 Chi-Chi earthquake, Taiwan

2.2.3 Directivity effect

When the fault rupture propagates toward an observation point, and the propagation velocity of the rupture is close to the shear wave velocity, most of the energy radiated from the fault plane might reach the observation point simultaneously, and form a noticeable recording long-period pulse at the beginning of recording [12], this is the so-called rupture directivity effect.

In order to understand the directivity effects, the supershear rupture dynamics should be made clear. The supershear rupture dynamics can be explained as following [13]. There are two modes of shear rupture as shown in Figure 2-5, which are distinguished by the direction of slip relative to the rupture front. In the diagram on the left, the blue arrows indicate the displacement of the two sides of the fault (the fault is the dashed line). Slip in mode II ruptures excites only shear waves, so the S-wave speed, c_s , becomes the limiting rupture speed in this geometry. Mode I slip generates both shear and dilational waves, which can interact in rather interesting ways (e.g., to produce Rayleigh surface waves). Mode I ruptures typically propagate below the Rayleigh speed, c_R , but can also propagate at interionic speeds (speeds between the S-wave speed, c_s , and the P-wave speed, c_p). The difficulty is that ruptures cannot propagate in the narrow speed range between c_R and c_s (mathematical solutions for such ruptures have the unphysical feature that energy flows out of the rupture front, rather than into it to be dissipated).

Rupture speed determines how waves from different parts of the fault interfere with each other. At the bottom of the Figure 2-5, two diagrams can illustrate this rupture mechanism. They show shear wavefronts emitted when the rupture passes the solid dots. For supershear ruptures (down), the source outruns the waves and a Mach front is formed. When the rupture is subshear (up), which is more common in the earthquake due to the rupture velocity is 0.72 times the shear wave velocity; the wavefronts are concentrated in the forward direction and separated in the backward direction. This leads to larger amplitudes and higher frequencies in the forward direction.

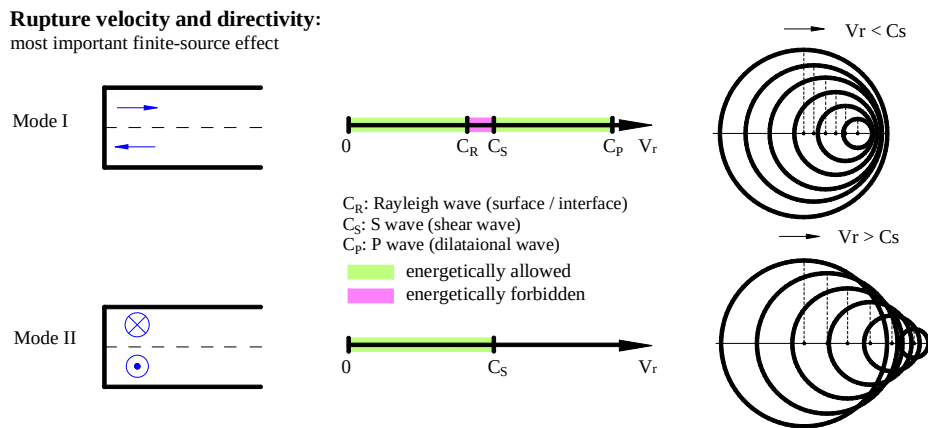


Figure 2-5 Permissible values of the rupture speed, v_r , are determined by the rupture mode [13]

Based on the rupture dynamics above, the areas and regions, where experience near-fault earthquakes are divided into three zones: zones where their ground motions are influenced by ‘*Forward directivity*’, ‘*Backward directivity*’ and ‘*Neutral directivity*’ [12]. To be specific, the three zones are plotted in Figure 2-6 below.

(a) Firstly, the region within the angles of faulting direction and the path through which earthquake waves go ahead is where the forward directivity effects might occur [14]. This angle is small in nature and is shown in Figure 2-6.

(b) Second, in the opposite direction to which the forward directivity happens, earthquakes are not generally impulsive at all.

(c) Additionally, regions that are located away from the aforementioned locations are expected to experience a neutral directivity effect in their ground motions which is nearly the same as the ground motions of far-fault earthquakes.

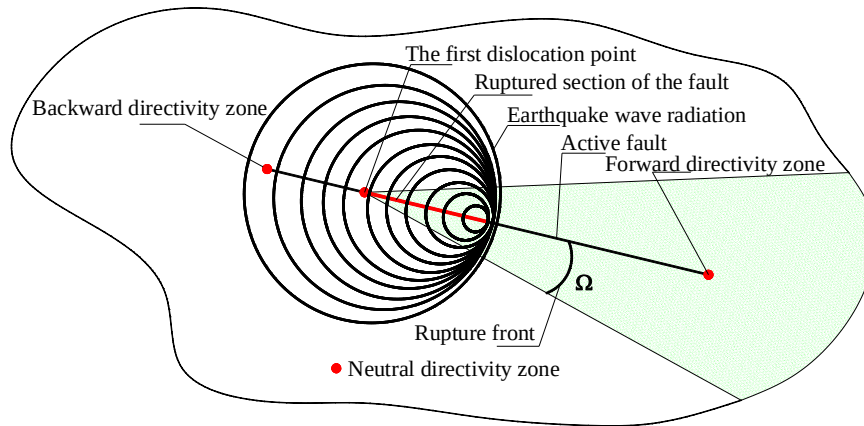


Figure 2-6 Forward, backward and neutral directivity zones in a region near to an active fault [14]

It should be mentioned that, just one of these zones experience impulsive earthquakes that cause structures to get heavily damaged, as mentioned the forward directivity leads to larger amplitudes and higher frequencies. There are two distinctive characteristics in the ground motions of the near-fault earthquakes influenced by forward directivity:

(a) First, these ground motions are always followed by immense Peak Ground Velocities (PGVs) in one of those pulse-like shapes in their velocity time histories;

(b) Second, the earthquakes influenced by forward directivity have a shorter strong motion duration compared to the ordinary quakes because the pulses usually existed at the onset of these quakes; induce a huge amount of shaking energy to the engineering structures.

Therefore, these huge amounts of energy which is cumulative in nature and released within short period are correspondingly followed by an extraordinary PGV, which also create severe harmful effects on the infrastructures.

The forward directivity effects are observed both in the cases of strike-slip and dip-slip events but seldom at normal events (Someriville [12]). Figure 2-7 shows the velocity and displacement time histories perpendicular to the fault strike direction from the fault front Lucerne Valley (LUC) station and the backward Joshua (JSH) station in the 1992 Lancers earthquake [15]. It can be seen that, the displacement and velocity time histories at the fault rupture front (LUC station) exhibited strong long period pulse, and the amplitude is large while the duration is short. In contrast, the amplitude of the ground motion at the JSH station (in the fault rupture backward) is obviously reduced, and there is no pulse phenomenon, the duration is also significantly longer.

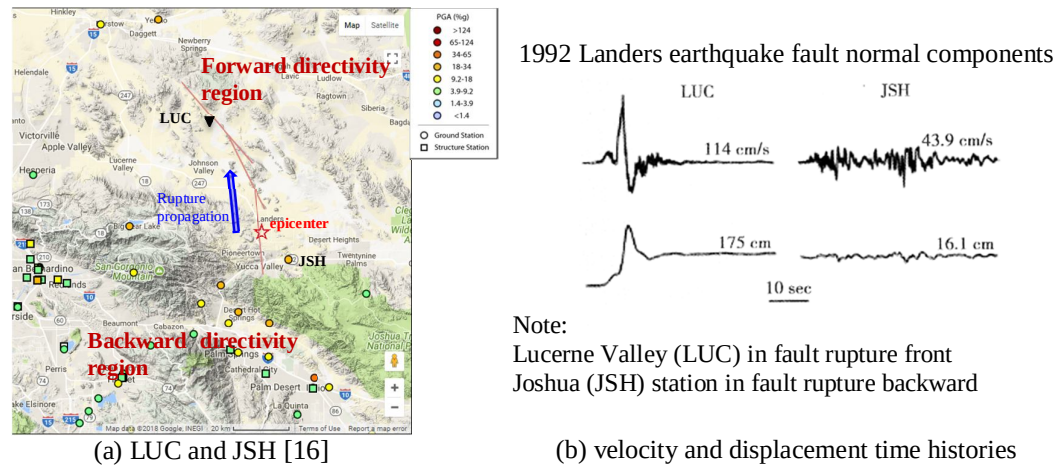


Figure 2-7 Rupture directivity (forward- and backward- effects) in the 1992 Lancers earthquake [16]

Although the directivity effect of rupture was found very early, the study on the influence of its destruction on the structure was late. Berrero *et al.* [17] first recognized that it could have a damaging effect on the structure, and he studied the directivity pulses effect of on flexible structure. Until the 1994 Northridge earthquake, the engineers realized the destructive effects of the directivity pulse on the long-period structure, and started to consider the influence of the specificity of the near-fault ground motion on the structure in engineering design. Because of the amplitude, spectral components caused by the directivity effect along fault parallel and fault normal directions are very different, so in the near-fault structure design, the two different components of excitation of ground motions (response spectrum) should be treated differently. The spectral amplitude is consistently improved (as opposed to the previous ground motion prediction), i.e., the near-fault ground motion effect is treated as a broadband effect. Recent studies of several earthquakes have shown that the directivity effect

does not uniformly amplified the ground motion of all the bands in the near-fault zone, but just amplifying the earthquakes in several bands and moving towards the low frequency with earthquake magnitude increasing. This phenomenon is called the narrowband directivity pulse [18]. Therefore, for a ground motion with period of about 1 s, moderate earthquakes might occur beyond the stronger earthquake. This phenomenon might be related to factors such as the rise time of the source time function. In this way, the near-fault directivity effect is more complicated.

There are many geometrical parameters determining the directivity effects, mainly the rupture propagation direction, the angle between seismic wave propagation and the observed points, and the size of the rupture fault between the source and the observation point. Somerville *et al.* introduced two parameters to describe the degree of influence of the directional effect for strike-slip and dip-slip fault in separate [12]. There are many seismic factors influencing the directivity effect, including the type of fault, rupture velocity, fault depth, dislocation distribution, sliding direction and so on. The method of using a magnitude or several simple parameters to describe the directivity effect is too simple. It is necessary to study the qualitative and quantitative relationship between the different source parameters and the directivity effect, and to estimate the affected area caused by directional effect reasonably, as well as incorporated into the seismic code.

2.2.4 Pulse-like velocity effects

The pulse-like velocity contains the following two types:

Type I: The velocity pulse is expressed as a double pulse-like velocity due to the directional velocity pulse caused by the Doppler effect of the rupture propagation. This velocity pulse is mainly expressed in the direction perpendicular to the fault plane, and it is mainly in the direction perpendicular to the fault direction when the inclination of the fault is large.

Figure 2-8 shows the location of observation array of the EI Centro array in the Imperial Valley earthquake on October 15, 1979, the pulse is perpendicular to the strike direction; these stations are located in the front of the rupture to meet the conditions of directional effects. It should be noted that, due to the limited space, only recordings of the observation site ec04, ec07, and ec12 located on the fault normal direction are listed. From the time history of velocity, it indicated that the double pulse-like is obvious. Simultaneously, it also showed that the pulse-like velocity becomes more and more obvious along with the fault distance decreasing, in the stations ec12, the pulse-like velocity with small pulse-like velocity, which indicated the pulse-like velocity appear only in a certain distance.

Type II: The second type of pulse-like velocity was caused by the permanent displacement of ground, which is related to the size and time of the permanent displacement. It is mainly in the fault parallel component and it's only single pulse-like velocity.

Figure 2-9 shows the velocity and displacement records parallel to the fault (east-west) recorded at the Sakarya and YPT stations in the 1999 Kocaeli earthquake, Turkey. A one-side velocity pulse corresponding to the permanent displacement of ground can be seen in the velocity time history.

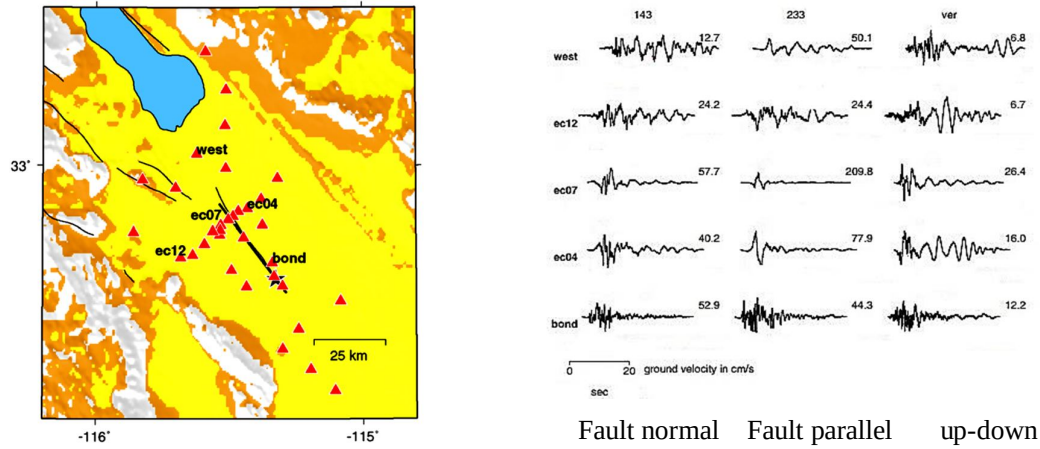


Figure 2-8 Recorded velocity time histories during the 1979 Imperial Valley [19]

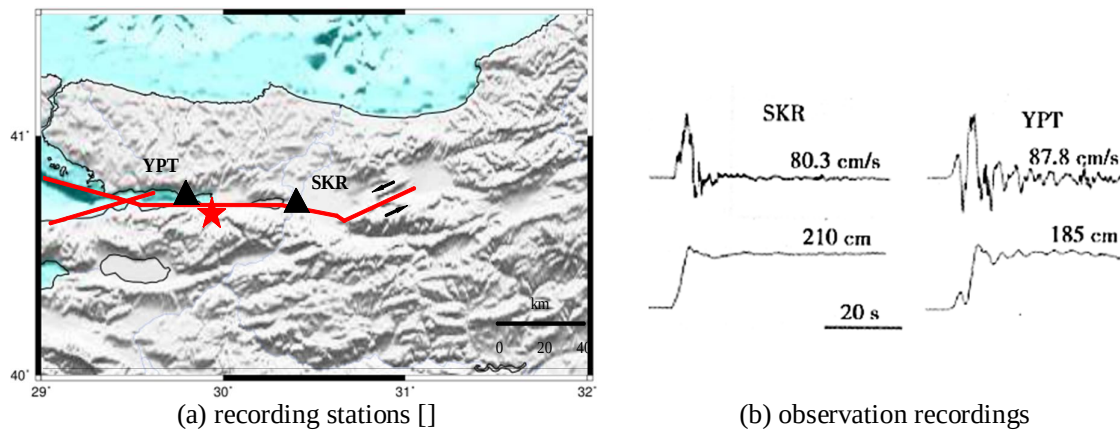


Figure 2-9 One-side impulse-like velocity recorded in 1999 Kocaeli Turkey Earthquake

Velocity pulses are a major cause of near-fault damage. The analysis on elasticity and inelastic response of the multi-degree-of-freedom system under the action of pulse shows that, the pulse amplitude and period are the main parameters to control the deformation of structure [20][21][22][23][24]. Recent researches mainly focus on the pulse peak value and period correlated with fault parameters and their effects on failure mechanism of different types of structures. Krawinlder and Adavi [25] proposed that, the application of simple velocity pulses as input ground motions can well describe the predominant response of the structures under near-fault ground motion. At present, the study on velocity pulses is limited to statistical regression analysis based on strong earthquakes, and most scholars do not distinguish between double-side pulse caused by velocity directivity and one-

side pulse caused by permanent displacements [21][26]. These studies show that, the pulse period increases along with magnitude increase, and under the same conditions, the velocity pulse period in the soil site is slightly larger than that of the bedrock site. However, there are obvious shortcomings in these statistical relations. First, the number of records with impulsive features is small, so the data performed large discrete. Secondly, under the assumptions that velocity pulse is independent of the fault distance, and all the statistical relations do not consider the fault-types-factor, and neglect the effects of fault depth, rupture velocity, they just conduct the macroscopic statistics analysis.

The numerical simulation of near-fault ground motion shows that [27][28] near-fault ground motions have a great relationship with the depth, fault type, rupture velocity, rise time and dislocation distribution of faults. Therefore, it is necessary to study the qualitative and quantitative relationship between the velocity pulse and the source parameters of near fault, and provide the theoretical basis for empirical statistics.

2.2.5 Hanging-wall effect

It has been observed that sites located on the hanging wall of a dip-slip fault experience larger ground motions than those located on the foot wall at the same closet distance, which called the hanging-wall effect [29][30].

A significant hanging wall effects were observed in the 1994 Northridge earthquake and the 1999 Chi-Chi earthquake. In Figure 2-10 [31], it shows the PGA distribution along with the rupture distance in the 1999 Chi-Chi earthquake, Taiwan. It is clear to seen, during the range of 11 ~ 29 km, the PGA in the hanging wall (the solid marks) is larger than the PGA in the foot wall (the hollow marks).

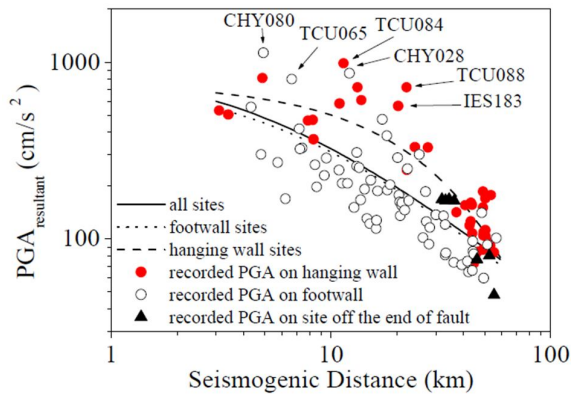


Figure 2-10 Hanging wall effect: distribution of PGA in the 1999 Chi-Chi earthquake [31]

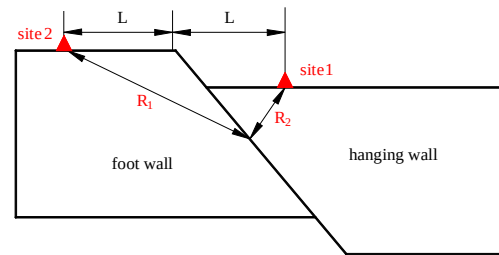


Figure 2-11 Hanging wall effect of near-fault area

The hanging wall effect is mainly due to the fact that, the two observation points which are located on the hanging wall and footwall with the same trace distance (the surface distance) as shown in Figure 2-11, the source distance of site 1 is smaller than that of the site 2 to the footwall, $R_1 < R_2$, for the sake of the proximity the forward directivity effects are more severe since the attenuation of ground motion amplitude is proportional to the distance from site to the surface projection of fault-plane (also referred to as Joyner-Boore distance), the difference in the proximity of the source to the site is a result of using the shortest distance to the rupture plane as the definition of the closest distance, so the dynamic ground motions on the hanging wall are larger than that on the footwall.

In addition, for the hanging wall, the multi-reflected and refracted seismic waves radiated from the fault plane will be reflected back to the fault surface and then reflected from the fault plane to the surface repeatedly, could amplify the ground motion on the hanging wall. The hanging wall effects prompted us to consider the differences for designing the near-fault structures and treated the two sections separately.

As described by Somerville *et al.* [32], rupture directivity effects increase the long-period ground motion on the fault normal component to the fault parallel components because the near-fault parallel component is nearly nodal due to radiation pattern effects, whereas the fault-normal component is near a maximum radiation pattern near the fault. Rupture directivity effects are largest in the region centered on the up-dip projection of the rupture surface, and this region occupies more of the footwall than the hanging wall. For the fault-normal peak velocity, the effect of rupture directivity on the footwall increases the ground motions and causes them to be as large as on the hanging wall, counteracting the tendency for the proximity effect to make the hanging-wall motion larger.

2.3 Spatial variations of near-fault ground motions on bridges

2.3.1 Spatial variations of near-fault ground motions

Theoretically, the spatial variation of near-fault ground motions result from a combination of wave propagations involving seismic sources, the propagation path, site conditions. In general, earthquake excitation is the result of a combination of more than one wave pattern, each one characterized by its particle motion shown in Figure 2-12 and its own apparent velocity of propagation. For extended-in-plane structures such as long-span bridges, differences in ground motion among different supports due to travelling seismic waves may result in quantitative and qualitative differences in seismic response as compared with those that produced by in phase ‘uniform’ motion at all support [33][34][35][36]. Earthquake ground motions in the three orthogonal directions of a long-span bridge may be transmitted to the superstructure through the two piers and the two abutments or anchor piers in Figure 2-12.

If the bridge is large-span with respect to the wavelengths of the input motion, in the frequency range of importance to its earthquake response, then different portions of the bridge may be subjected to significantly different excitations. Any two supporting points, a given distance apart, on the surface of the ground may receive long-period earthquake motions that are nearly equal, but at the same time, these two points may also experience short-period motions that are dissimilar and uncorrelated. In general, the correlation of the motion at these support points is extremely complicated, particularly in the case of a long-span bridge, with different foundation conditions, subjected to seismic waves with different angles of incidence and different travel paths (multi-reflections and refractions, and transitions etc.). In that case, marked differences in amplitude as well as phase could occur over distances on the order of a few wavelengths of the motion Figure 2-12.

2.3.2 Effects of spatial variations on near-source bridges

It is known that at a distance less than 20 km around the seismic source (the near-fault zone), the ground motions can exhibit significantly different characteristics than those recorded further from the seismic sources. An important feature that sometimes occurs in near-fault ground motions is a large pulse in velocity [37]. In addition, due to the permanent ground displacement at the site resulting from tectonic movement, ground motions close to the surface rupture may contain a significant permanent static displacement [38]. The near-fault ground motions also characterize hanging-wall effect, concentration effect, surface rupture, and directivity effect, thus, in the near-source area, the ground motions are rather complicated.

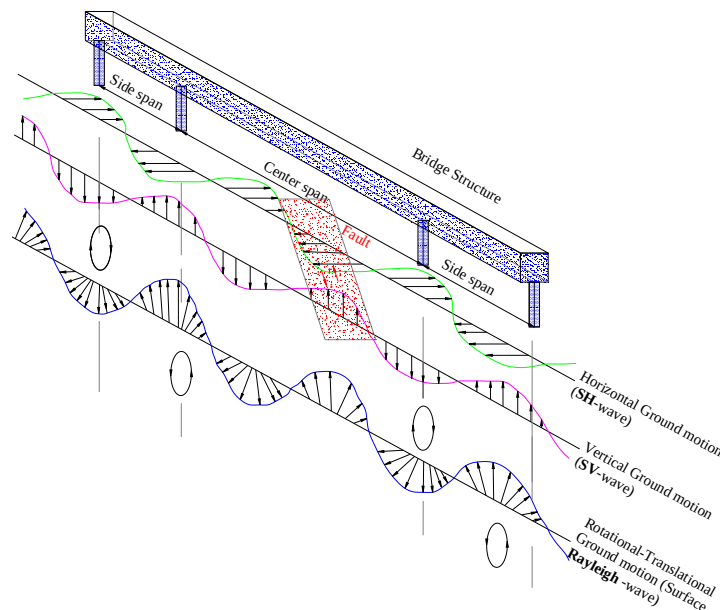


Figure 2-12 Different types of seismic waves impinging on piers of a fault-crossing bridge

2.4 Evaluation on seismic risk during the 2016 Kumamoto Earthquake

2.4.1 Seismic activity of the 2016 Kumamoto Earthquake

A large and destructive earthquake (M_{JMA} 7.3) occurred on April 16, 2016 (Coordinated Universal Time (UTC), local time), in Kumamoto region, Kyusyu Island, Japan, following the largest foreshock (M_{JMA} 6.5) occurred on 14 April, 2016. Numerous buildings collapsed due to the double shocks. In particular, near-fault region (around 20km away from the fault plane), the effects of the ground displacements were remarkable, and buildings and infrastructure that were right above the fault rupture were severely damaged. We conducted the site survey for two times, 22nd~24th, April and 5th May, especially focused on the seismic behavior of near-fault structures, and a detail damage evaluation on 138 buildings on Kurukawa area, about 26km far away from the hypocenter, and 4km perpendicular to the fault trace, the site 1 and site 2 are shown in Figure 2-13. Thus, in this section, based on the open data [39] and the damage survey in the field, a seismic evaluation on the near-fault ground motions are conducted.

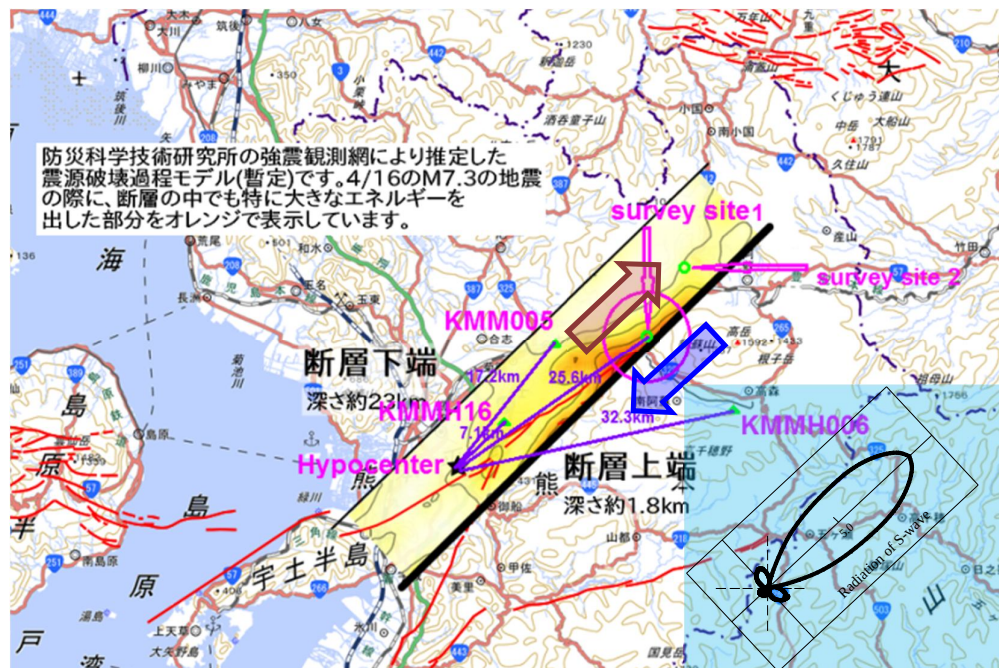


Figure 2-13 Fault plane, observation sites and survey sites [40]

2.4.2 Strong ground motion characteristics in the near-fault region

Ground motion data recorded at KMMH16 (Mashiki; 32.7967°N, 130.8199°E) are analyzed in detail, noting that the earthquake damage surveys were conducted near this station during the field survey. The KMMH16 station belongs to the KiK-net, and thus two sets of three component

recordings at the ground surface and in borehole are available, enabling site amplification effects to be investigated. Another important aspect of the selected records is that KMMH16 is in the hanging wall region of the mainshock (i.e., within a projected fault plane on the ground surface), and thus intense ground shaking was observed during the mainshock. Moreover, at KMMH16, strong shaking due to the foreshock preceded the mainshock, resulting in double-shock ground motions [41].

Figure 2-14 shows observed acceleration as well as velocity time-histories (three components) at KMMH16 for the foreshock and mainshock [42]. The blue curves are for the ground surface recordings, whereas the red curves are for the borehole recordings. The significant amplification as well as different dominant frequency content of the ground motions can be observed by comparing the blue and red curves. Another notable observation is that for the velocity time-histories of the mainshock (i.e., Figure 2-14 D), relatively large velocity waves are present at both ground surface and borehole (particularly for vertical motions). This indicates that site amplification for short-period components is significantly influenced by near-surface soil borehole. The latter may also be attributed to the ground surface rupture near the Mashiki areas. Longer period components are more coherent at ground surface and borehole. The latter may also be attributed to the ground surface rupture near the Mashiki areas.

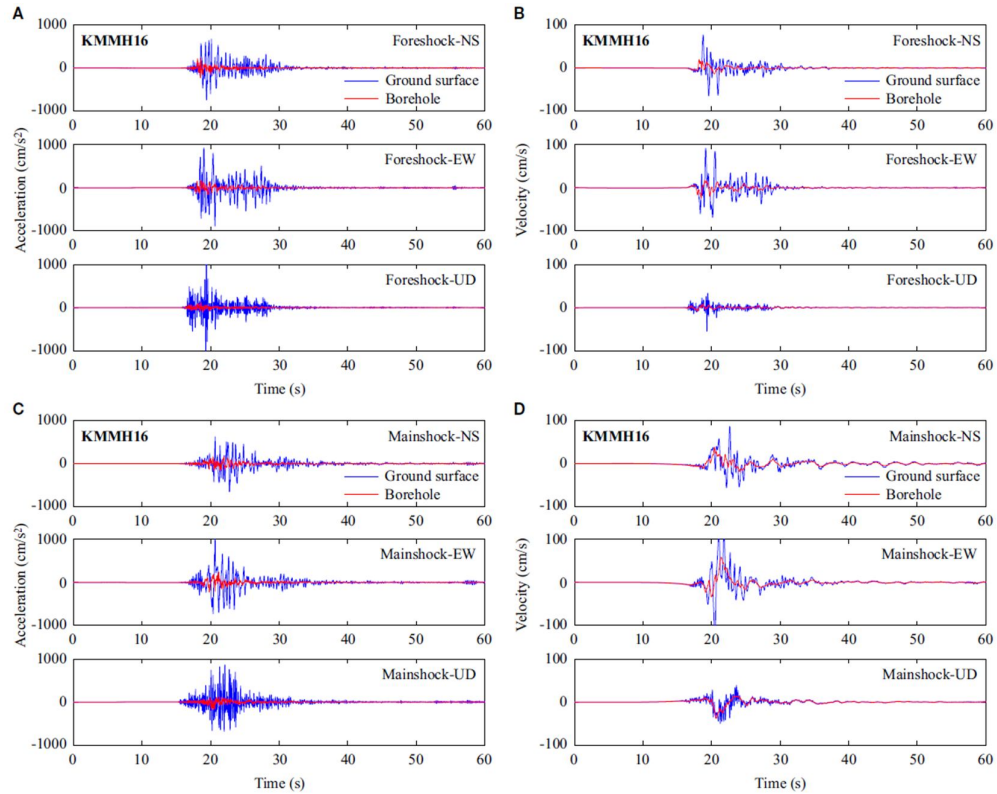


Figure 2-14 Recordings of acceleration and velocity at KMMH16 [42]

2.4.3 Earthquake damage survey

A detail damage evaluation on near-fault buildings of 138 houses, including different materials (e.c., RC (12), wood (124), steel (1), and unknown (1)) in Kurokawa area on 5th May was conducted on site 2 (shown in Figure 2-13). The evaluations resulted that, under the high frequency, high amplitude, impulse-like and permanent displacement ground motions, 36 houses without damage, 13 houses with slight damage, 6 with moderate damage, 7 with heavy damage, and 76 houses are collapsed. The evaluation results are shown in Figure 2-15.

The building exhibited collapse, rotation with large cline angle, shear failure of columns, large relative rupture when cross the fault (f) as shown in Figure 2-16, which showed the complicated characteristics of near-fault ground motions. For the seismic behavior of near-fault buildings, mainly due to the impulsive velocity and permanent displacement, and especially the radiation pattern of S wave (shown in Figure 2-13), and very high frequency and large amplitude, the collapse and inclined direction are approximated to the fault normal direction, as well as the trapped of vehicles. And they exhibited total collapse due to foundation subduction, or rotation with large incline angle, while there are also some buildings without damage even located along the fault trace, thus further quantitative numerical analysis combined with the evaluation results should be conducted in the future research.



Figure 2-15 Field survey on damage of buildings, collapse of vehicles (the collapse of vehicles was surveyed by Google Earth)

2.5 conclusions

In this chapter, the following items could be obtained:

(1) It started with the focal mechanics analysis, especially the radiation patterns, which might significantly reflect the seismic behavior of structures in the near-fault area;

(2) It detailed 5 significant characteristics of near-fault ground motions, including concentration effect, permanent displacement (fling-step) effect, directivity effect, pulse-like velocity effect, and the hanging-wall effect, and each character is clarified by the database of recordings and occurred earthquake. What's more, the effects of spatial varied near-fault ground motions on bridge engineering are provided, which makes it clear to understand the reason and necessity of multi-support excitation method for dynamic analysis on near-fault bridge;

(3) As an example for describing the damage of near-fault ground motions on structures and make clear its characteristics, a detail damage evaluation on near-fault buildings in Kurokawa area on May 5th in Kurokawa District of Minami Aso Village was conducted, and the evaluations resulted that, under the high frequency, high amplitude, pulse-like velocity, permanent displacement ground motions, and especially the radiation pattern of S wave, the building exhibited collapse, rotation with large cline angle, shear failure of columns, large relative rupture when cross the fault, and the collapse of inclined direction almost along fault normal direction, which showed the complicated characteristics of near-fault ground motions.

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Chapter 3 Hybrid synthesis method on near-fault ground motion

3.1 Introduction

Recent large-scale inland earthquakes, especially the 1999 Chi-Chi earthquake, Taiwan and the 1999 Kocaeli earthquake, Turkey, dramatically demonstrated the strong effects of surface faulting on near-fault ground motions (e.g., Shin and Teng [1]). The effects of surface rupture due to tectonic deformations have been recently called “fling step” (e.g., Richard and Aki [2], Archuleta and Hartzell [3], Somerville *et al.* [4]), the fling effects exhibit long-period pulses and permanent static offsets along fault parallel direction. In particular, when the fault rupture reaches the free surface, the faulting slip and the subsequent GMs are strongly amplified (e.g., Archuleta and Frazier [5]). Thus, the combination of fling and directivity could have destructive effects on various lifeline facilities and long-period structures, such as high-rise or base-isolated buildings, in the vicinity of the surface fault.

Even though various numerical methods have been available to reproduce the near-fault ground motions considering the surface faulting (e.g., Archuleta and Frazier [5], Oglesby and Day [6], Ramachandra and Meguro [7]), they are either too simple, or much complicated, or time consuming, which could not be well performed as earthquake excitations for near-fault engineering designing. Thus, in this chapter, aimed at providing more effective estimates of the ground motions, it proposed a hybrid synthesis method, which combined the stochastic Green's function method and theoretical Green's function method for simulating the near-fault ground motions using an extended kinematic source, it advantaged. Then several applications of the proposed method are described.

3.2 Hybrid synthesis method

The displacement due to a kinematic fault model can be expressed as follows in the frequency domain [2]:

$$U_k(Y; \omega) = \int_{\Sigma} T_{ik}(X, Y; \omega) D_i(X; \omega) d\Sigma \quad (3-1)$$

where,

U_k : the k th component of displacement in the Cartesian coordinate system at an observation point;

Y and X : a source point on the fault plane;

ω : the circular frequency;

Σ : the fault plane;

T_{ik} : the traction Green's Function;

D_i : the i th component of the fault slip.

We use the summation convention for subscript i . The Green's function of the layered half-spaces is obtained using the following wavenumber integration:

$$T_{ik}(X, Y; \omega) = \int_0^\infty t_{ik}(X, Y; \omega) dk \quad (3-2)$$

where,

t_{ik} : the integrand of the traction Green's Function.

When the observation point is located on the fault plane, the Green's Functions become singular as the order of $(1/r^2)$.

Based on Hisada and Bielak [8], they pointed out that, when the observation point is close to the fault plane, the dynamic traction Green's functions exhibit sharp peaks within the area close to the observation point. However, subtracting and adding the static Green's functions can eliminate the singularities of the original Green's function. And they proposed the following representation theorem is preferred, instead of Eq. (3-1):

$$U_k(Y; \omega) = \int_\Sigma \{T_{ik}(X, Y; \omega) - T_{ik}^s(X, Y)\} D_i(X; \omega) d\Sigma + \int_\Sigma T_{ik}^s(X, Y) D_i(X; \omega) d\Sigma \quad (3-3)$$

where,

T_{ik} : the traction Green's function of the layered half-space at circular frequency;

T_{ik}^s : the static traction Green's function of the layered half-space ($\omega=0$).

Based on the new idea of the calculations on static terms and dynamic terms proposed by Hisada and Bielark [8], there are many options to calculate these two integrations effectively. The theoretical green's function method can generate both dynamic and static ground motions and has the advantage that the displacement near the fault almost directly reflects the source characteristics. However, its application is limited to idealized layered media and time-consuming for dynamic terms. Since the stochastic Green's function method is very popular, flexible, applicable to structural designs, and can also generate dynamic ground displacements in the vicinity of the fault. However, it is difficult to simulate fault movements such as a "static" fling-up (permanent displacement) of the fault.

Thus, to incorporate the displacements induced by active faults, a hybrid synthesis method of modified stochastic (for dynamic calculations) and theoretical (for static calculations) Green's functions is proposed for near-fault ground displacement based on above Eq. (3-3) is preferable.

3.3 Dynamic terms: stochastic Green's function

Strong ground motion is simulated using stochastic Green's function method. The keys of stochastic Green's function method are appropriate underground structural models of the crust, deep ground and shallow ground and also the asperity models calibrated by strong motion data obtained near the earthquake sources, and some detailed key points are listed as following.

3.3.1 Basic principle of stochastic Green's function method

1) *Empirical Green's function*

In seismology, records of small earthquakes that share the source area and observation station with the main shock are often called empirical Green's function. Empirical Green's function shares the path and the local amplification effects with the main shock but has significant difference in the source effect (Hartzell [9]). This difference has been called the relation of small and large earthquakes. ω^{-2} model of the source spectra (Aki, [10]) and the scaling law of the fault parameters (Kanamori and Anderson [11]) gave the way to estimate this relation for earthquakes up to intermediate size up to M_w 6 (e.g., Yokoi and Irikura [12]). For large earthquakes, heterogeneous rupture has to be considered (e.g., Kikuchi and Fukao [13]), the parts of earthquakes source fault that emit the seismic waves stronger than the rest is called Asperities (e.g., Boatwright [14]). Applying ω^{-2} model of the source spectra to asperities, strong ground motion due to big earthquakes can be synthesized. This way is called empirical Green's function method, which was shown in Figure 3-1.

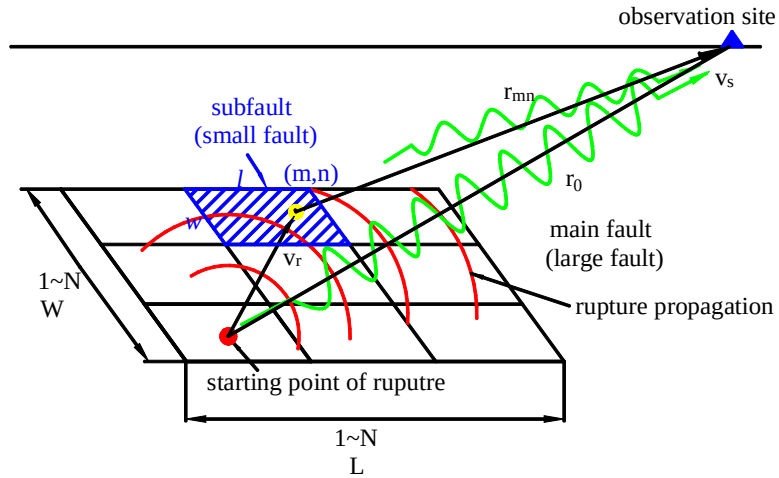
Empirical Green's function method can be applied without any information about local amplification effect that affects much the waveforms, because this is already included in empirical Green's function (e.g., Irikura [15], Irikura and Kamae [16]). But it has the limitation that the effective small observation earthquake is required.

2) *Stochastic Green's function*

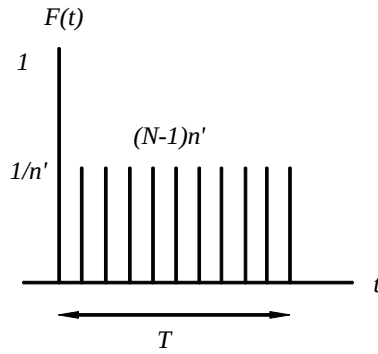
In order to extend empirical Green's function method to the cases if recording of small earthquakes is not available, a method to calculate them based on the stochastic theory has been developed. This calculated small earthquake's record is called stochastic Green's function. The way to synthesize strong motion using stochastic Green's function is called stochastic Green's function method, for which the observed records are not required but at least the underground velocity structure beneath the observation stations need to be clarified (e.g., Kamae and Irikura, [17]).

3) *The basic principle*

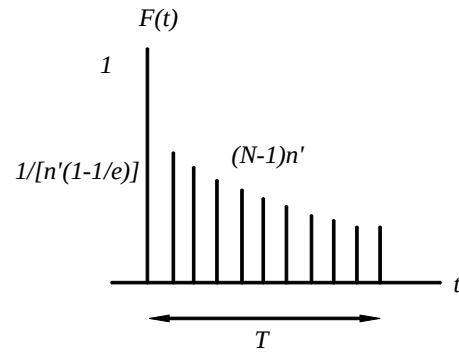
Because the above stochastic Green's function is used instead of empirical Green's function, the basic principle of these two methods are almost the same that the technique by which waveforms for large events are synthesized follows the empirical Green's function method proposed by Hartzell [9]. And some revisions have been made by Kanamori [11], Irikura [15]. In this research, based on the scaling law of fault parameters for large and small events (Kanamori and Anderson [11] and the ω^{-2} source spectra (Aki [10]), the waveform for a large event is synthesized by summing the records of small events with corrections for the difference in the slip velocity time function between the large and small events following the above scaling laws. This method does not require knowledge of the explicit shape of the slip velocity time function for the small event.



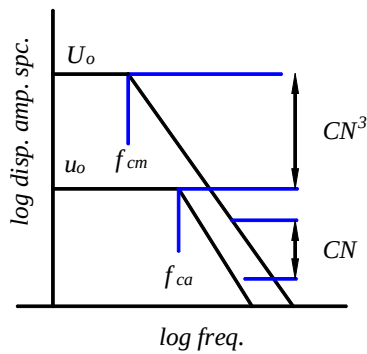
(a) fault area of large and small events are defined to be $L \times W$ and $l \times w$, respectively, where $L/l = W/w = N$.



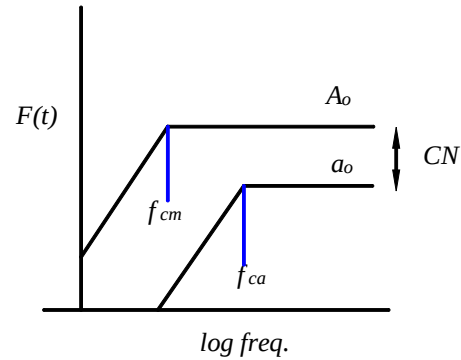
(b) filtering function $F(t)$ (after Irikura [15]) to adjust to the difference in slip velocity function between the large and small events. This function is expressed as the sum of a delta and a boxcar decaying function.



(c) modified filtering function (after Irikura *et al.*, [18]) with an exponentially decaying function instead of a boxcar function. T is the rise-time for the large event.



(d) Schematic displacement amplitude spectra following the omega-squared source scaling model, assuming a stress drop ratio C between the large and small events.



(e) Acceleration amplitude spectra following the omega-squared source scaling model.

Figure 3-1 Schematic illustrations of the empirical Green's Function method

The numerical equations to summing records of small events are:

$$U(t) = \sum_{i=1}^N \sum_{j=1}^N \frac{r}{r_{ij}} F(t) * (C \cdot u(t)) \quad (3-4)$$

$$F(t) = \delta(t - t_{ij}) + \frac{1}{n'} \sum_{k=1}^{(N-1)n'} \left[\delta \left(t - t_{ij} - \frac{(k-1)T}{(N-1)n'} \right) \right] \quad (3-5)$$

$$t_{ij} = \frac{r_{ij} - r_0}{V_r} + \frac{\xi_{ij}}{V_r} \quad (3-6)$$

where,

$U(t)$: the simulated waveform for the large event;

$u(t)$: the observed waveform for the small event;

N and C : the ratios of the fault dimensions and stress drops between the large and small events in respect;

$*$: indicates convolution;

$F(t)$: the filtering function (correction function) to adjust the difference in the slip velocity time functions between the large and small events;

V_s and V_r : the S-wave velocity near the source area and the rupture velocity on the fault plane;

t_{ij} : the rise-time for the large event, and defined as duration of the filtering function $F(t)$ (in Figure 3-1 (b) and (c)). It corresponds the duration of slip velocity time function on sub-fault from the beginning to the time before the tail starts;

n' : an appropriate integer to weaken artificial periodicity of n , and to adjust the interval of the tick to be the sampling rate.

Other parameters are given in Figure 3-1(a).

Regarding the filtering function $F(t)$, Irikura *et al.* [18] proposed a modification to Eq. (3-5) in order to prevent sag at multiples of $1/T$ (Hz) from appearing in the amplitude spectra. The discretized equation for the modified $F(t)$ is,

$$F(t) = \delta(t - t_{ij}) + \frac{1}{n'(1-1/e)} \sum_{k=1}^{(N-1)n'} \left[\frac{1}{e^{\frac{(k-1)}{(N-1)n'}}} \delta \left(t - t_{ij} - \frac{(k-1)T}{(N-1)n'} \right) \right] \quad (3-7)$$

The shape in Eq. (3-7) is shown in Figure 3-1 (c). In Irikura [15], the scaling parameters needed for this technique, N (integer value) and C , can be derived from the constant levels of the displacement and acceleration amplitude spectra of the large and small events with the formulas:

$$\frac{U_0}{u_0} = \frac{M_0}{m_0} = CN^3 \quad (3-8)$$

$$\frac{A_0}{a_0} = CN \quad (3-9)$$

Here, U_0 and u_0 indicate the constant levels of amplitude of the displacement spectra for the large and small events, respectively. M_0 and m_0 correspond to the seismic moments for the large and small events. A_0 and a_0 indicate the constant levels of the amplitude of the acceleration spectra for the large and small events as shown in Figure 3-1 (d), (e)).

N and C are derived from Eqs. (3-8) ~ (3-9),

$$N = \left(\frac{U_0}{u_0} \right)^{\frac{1}{2}} \left(\frac{a_0}{A_0} \right)^{\frac{1}{2}} \quad (3-10)$$

$$C = \left(\frac{u_0}{U_0} \right)^{\frac{1}{2}} \left(\frac{A_0}{a_0} \right)^{\frac{3}{2}} \quad (3-11)$$

3.3.2 Procedure of stochastic green function method

The stochastic Green's function method should well consider the source, wave propagation and effects of site conditions.

The source time function is converted from the conventional stochastic Green's function method (Boore [19]) that fulfills ω^{-2} model of the source spectra. Then, P-, SV- and SH waves radiated from point source is calculated in the following way.

The path effect is given by the geometrical spreading effect, scattering effect and the radiation pattern of body waves. The former is estimated by the ray tracing method (Cerveny *et al.* [20]) Therefore, only body waves are taken into account. For the latter, the effect of scattering during propagation is taken into account. For the propagation distance much shorter than the wave length, the theoretical radiation coefficients are used, whereas for that much longer than the wave length, their convergent values estimated by the scattering theory are adopted. For intermediate case, the linear interpolation between these two cases is used. The used thresholds for the ratio of the propagation distance to the wavelength are 0.5 and 5.0 as well as Onishi and Horike [21].

The local amplification effect is calculated using the given horizontally layered media beneath observation stations and the incident angles calculated by the ray theory for P-, SV- and SH-waves as mentioned above (Silva [22]).

The velocity models used vary case by case. For few of them, PS-logging data are available but for others only crustal velocity structure model.

The above mentioned stochastic Green's function is used in place of empirical Green's function. The slip velocity function proposed by Irikura *et al.* [18] is used. And stochastic Green's function is calculated for each asperity by locating its point source at the center of asperity. The effect of background region is not calculated because it usually does not affect significantly to acceleration waveforms.

The detailed procedure for stochastic Green's function method is shown in Figure 3-2 below.

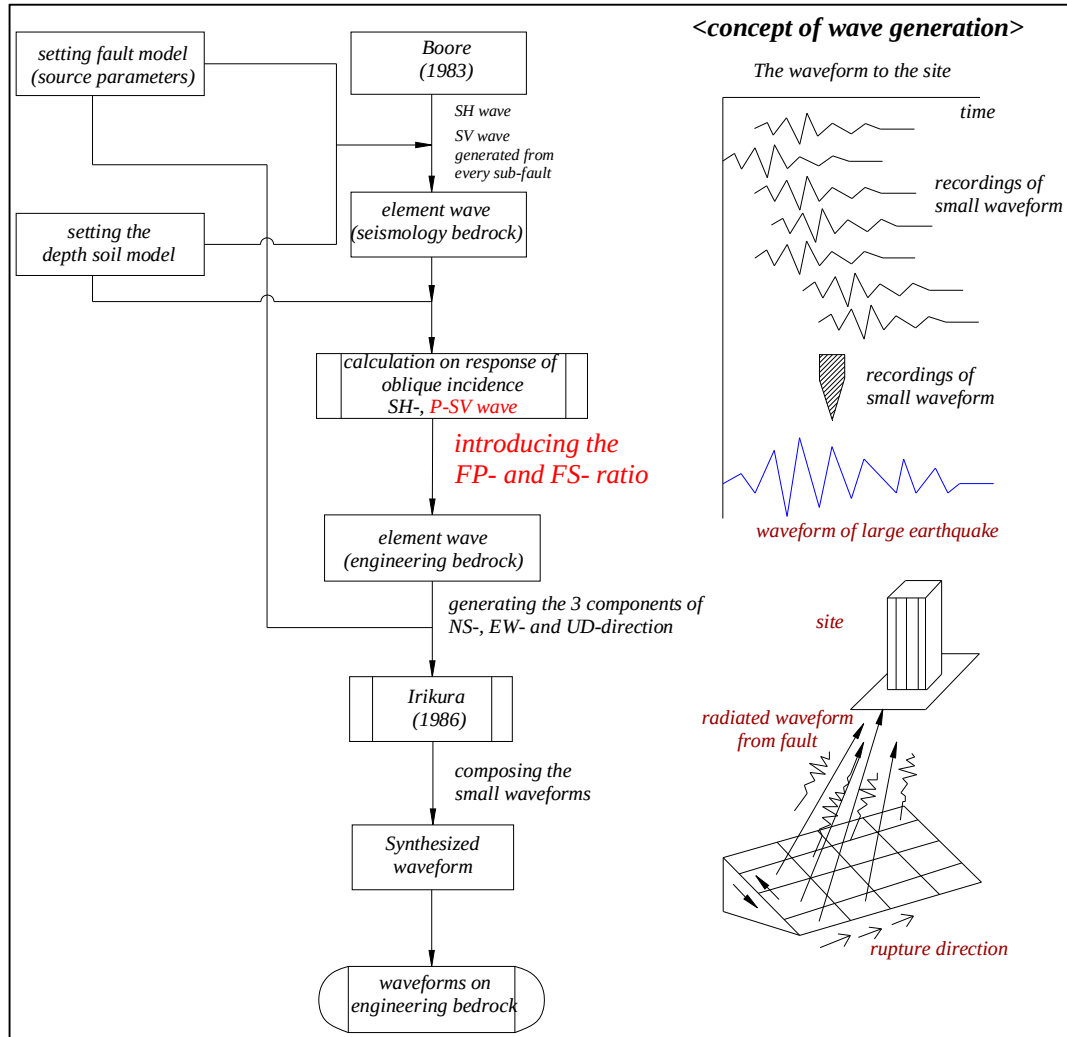


Figure 3-2 Flowchart of schematic of stochastic Green's function

3.4 Modification on stochastic Green's function

3.4.1 The principle of modification

On the basis of Boore [19], the most commonly used and conventional stochastic Green's function method is accurate for far-field ground motions, while in the near-fault zone, the other terms of the complete waveform solutions as shown in Eq. (2-1) (Aki [2]), including the far-field P-waves, the near-field, and the intermedia-field terms characterized complicated and can't be neglected. Due to this research focus on the near-fault ground motions, the modified stochastic Green's function method is adopted.

Relayed on Aki and Richards (Eq 4.32) [23], by using the time-dependent seismic moment $M_o(t) = \mu \bar{u}(t) A$, the complete waveform of displacement field in the infinite elastic half-space $u_n = M_{pq} * G_{np,q}$ can be written in a concise vector form as the complete solution of wave propagation shown in section 2.1, the focal mechanics of near-fault ground motions, thus here it will not repeat the Eqs. (2-1) ~ (2-2).

3.4.2 Introduce the far-field terms of P-waves

Rallied on Kagawa [24], the far-field P-waves was introduced, following which the calculation way of small events are the same as the far-field of S-waves, as shown in Eqs. (3-12) and (3-13).

$$A(\omega) = C_s M_o S(\omega, \omega_c) P(\omega, \omega_m) \frac{e^{-\omega R/2Q\beta}}{R} \quad (3-12)$$

$$C_s = \frac{R_{\theta\phi} \cdot FS \cdot PRTITN}{4\pi\rho\beta^3} \quad (3-13)$$

The velocity of P-wave (α) is adopted instead of S-waves velocity (β) in the Eqs. (3-12) ~ (3-13):

$$A(\omega) = C_s M_o S(\omega, \omega_c) P(\omega, \omega_m) \frac{e^{-\omega R/2Q\alpha}}{R} \quad (3-14)$$

$$C_p = \frac{R_{\theta\phi} \cdot FS \cdot PRTITN}{4\pi\rho\alpha^3} \quad (3-15)$$

And the other calculation procedures are the same as the S-waves.

3.4.3 Introduce the near-field and intermedia-field terms

Based on Nozu [25], and Nozu and Nagasaka [26], the near-field and intermediate-field terms in the frequency domain, shown in Eqs. (3-16) ~ (3-17). In the spherical polar coordinates as shown in Figure 2-1 fore mentioned, the Eq. (3-16) could generate the near-field and intermedia-field terms of

S-waves, related to the components along the θ, ϕ directions, and Eq. (3-17) related to the components along r -direction.

$$(N + IP + IS + FS) / FS = 6i\left(\frac{\beta}{r\omega}\right)^3 \left[1 - e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)}\right] - 6\left(\frac{\beta}{r\omega}\right)^2 \left[1 - \frac{\beta}{\alpha} e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)}\right] + 2i\left(\frac{\beta}{r\omega}\right)\left(\frac{\beta}{\alpha}\right)^2 e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)} - 3i\left(\frac{\beta}{r\omega}\right) + 1 \quad (3-16)$$

$$(N + IP + IS + FP) / FP = 9i\left(\frac{\alpha}{r\omega}\right)^3 \left[1 - e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)}\right] - 9\left(\frac{\alpha}{r\omega}\right)^2 \left[1 - \frac{\alpha}{\beta} e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)}\right] - 4i\left(\frac{\alpha}{r\omega}\right) + 3i\left(\frac{\alpha}{r\omega}\right)\left(\frac{\alpha}{\beta}\right)^2 e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)} + 1 \quad (3-17)$$

Thus, the near-fault complete waveforms can be obtained. The simple example on these two ratios will be shown in section 3.6.1.

3.5 Static terms: theoretical Green's function (Hisada [27][28][8])

The static terms mentioned before is calculated by the theoretical Green's function method [28][29]. The following listed some additional key points of this method.

3.5.1 The Green's function of layered half-spaces

1) Introduction of the solution

First of all, it should be noted that when the source depth is equal or close to the receiver depth, there are difficulties in computing the displacement and stress Green's functions, and it explained the reason here. We illustrate this problem with a spherical wave, because any 3D wave field can be expressed as a superposition of spherical wave and can be decomposed into cylindrical waves using the Sommerfeld integral (see Chapter 6: P194 Eq. (6.7), Aki and Richards [23]).

$$\begin{cases} \frac{1}{R} \exp\left(i\frac{\omega}{c}R\right) = \int_0^\infty \left[\frac{k}{v} \exp\{-v|z-h|\} J_0(kr) \right] dk \\ R = \sqrt{r^2 + (z-h)^2}, v^2 = k^2 - \left(\frac{\omega}{c}\right)^2, \text{Re}(v) \geq 0 \end{cases} \quad (3-19)$$

Where h and z are the depths of source and receiver, respectively, r is the horizontal distance between the source and the receiver, c is the velocity of the medium, ω is the circular frequency, J_0 is the Bessel function of the order 0, and the variable of integration (k) represents the horizontal wavenumber. In order to obtain stresses, we need to differentiate Eq. (3-19) with respect to z or r . For example, the derivative of Eq. (3-19) with respect to r gives us:

$$\left(\frac{1}{R} - i\frac{\omega}{c}\right) \frac{r}{R^2} \exp\left(i\frac{\omega}{c}R\right) = \int_0^\infty \left[\frac{k^2}{v} \exp\{-v|z-h|\} J_1(kr) \right] dk \quad (3-20)$$

When the source depth is equal to the receiver depth ($h=z$), Eqs (3-19) and (3-20) are expressed as:

$$\frac{1}{r} \exp\left(i \frac{\omega}{c} r\right) = \int_0^\infty \left\{ \frac{k}{v} J_0(kr) \right\} dk \quad (3-21)$$

$$\left(\frac{1}{r} - i \frac{\omega}{c} \right) \frac{1}{r} \exp\left(i \frac{\omega}{c} r\right) = \int_0^\infty \left\{ \frac{k^2}{v} J_1(kr) \right\} dk \quad (3-22)$$

The integrations here are more difficult to compute than those of Eqs (3-19) ~ (3-20), because the decaying exponentials $\exp\{-v|z-h|\}$ of Eqs. (3-38) ~ (3-20) disappear and their integrands oscillate with slowly decreasing and increasing amplitudes, respectively, with wavenumber.

The same problem occurs when computing the Green's function of the layered half-space, as described in Apsel and Luco [30], Herrmann and Wang [31], Chang [32] and Herrmann [33]. In order to remedy this problem, two techniques have been proposed. Chang [32] proposed: the repeated averaging method", taking advantage of the well-behaved damped-oscillating property of the latter part of the integrand of the displacement. This method is stable but is rather time-consuming because a set of accurate peak and trough values in the damped-oscillating part are required to determine the average value. For computing the stress, this method is probably unstable because of the increasing-oscillating property. An alternative, more analytical, method is proposed by Apsel and Luco [30] who take advantage of the fact that the integrand of the dynamic Green's function converges to that of the static Green's function with increasing k .

We shall explain Apsel and Luco's [30] idea using our example. The static solutions corresponding to Eqs. (3-19) ~ (3-20) are expressed as follows, by imposing $\omega=0$.

$$\frac{1}{R} = \int_0^\infty \left[\exp\{-k|z-h|\} J_0(kr) \right] dk \quad (3-23)$$

$$\frac{r}{R^3} = \int_0^\infty \left[k \exp\{-v|z-h|\} J_1(kr) \right] dk \quad (3-24)$$

It is clear that the integrands in Eqs (3-19) and (3-20) approach those in Eqs (3-23) and (3-24) as k increases. Therefore, subtracting Eqs (3-23) and (3-24) from Eqs (3-19) and (3-20) and rearranging, we have:

$$\frac{1}{R} \exp\left(i \frac{\omega}{c} R\right) = \int_0^\infty \left[\left\{ \frac{k}{v} \exp\{-v|z-h|\} - \exp\{-k|z-h|\} \right\} J_0(kr) \right] dk + \frac{1}{R} \quad (3-25)$$

$$\left(\frac{1}{R} - i \frac{\omega}{c} \right) \frac{r}{R^2} \exp\left(i \frac{\omega}{c} R\right) = \int_0^\infty \left[\left\{ \frac{k}{v} \exp\{-v|z-h|\} - \exp\{-k|z-h|\} \right\} k J_1(kr) \right] dk + \frac{r}{R^3} \quad (3-26)$$

Respectively.

The above integrands quickly converge to zero with wavenumber, even for $h=z$. In fact, when h is equal to z , Eqs. (3-25) and (3-26) become

$$\frac{1}{r} \exp\left(i \frac{\omega}{c} r\right) = \int_0^\infty \left\{ \left\{ \frac{k}{v} - 1 \right\} J_0(kr) \right\} dk + \frac{1}{r} \quad (3-27)$$

$$\left(\frac{1}{r} - i \frac{\omega}{c} \right) \frac{1}{r} \exp\left(i \frac{\omega}{c} r\right) = \int_0^\infty \left\{ \left\{ \frac{k}{v} - 1 \right\} k J_1(kr) \right\} dk + \frac{1}{r^2} \quad (3-28)$$

We find the $\{\}$ parts in the above integrands converge to zero with increasing k . Thus, we can significantly reduce the integration ranges as compared with those of Eqs (3-19) and (3-20), particularly when the source depth is the same or close to the receiver depth.

In order to find an analytical form for the asymptotic solutions of the integrands of Green's functions, we need static and dynamic Green's functions that do not have growing exponential terms. For this purpose, we adopt the generalized R/T (reflection and transmission) coefficient method (Luco and Apsel [30]) and the stress discontinuity representations (Harkrider [34]; Haskell [35]) to express the boundary and source conditions, respectively.

The R/T coefficient method is well known to be much numerically stable than other conventional propagator matrix methods (e.g., Haskell [36]), because it avoids growing exponential terms by introducing the R/T coefficients of the up/down-going waves in each layer. Kennett [37] and Kennett and Kerry [38] first proposed this method for 2D and 3D cases, respectively. In this study, however, we adopt the other version of the R/T coefficient method by Luco and Apsel [30], because of the following two reasons. First, its form is simplified by introducing "the generalized R/T coefficient", and its physical meaning is clearly understood. Second, they derived the explicit static solution of the R/T coefficient. The static solution requires consideration of the singularity of the coefficient matrix, which is caused by the disappearance of differences between terms associated with P and S waves. Because of this simplicity, together with the numerical stability up to very high frequencies, the generalized R/T coefficient method by Luco and Apsel [30] has been widely used to compute not only Green's functions (e.g., Chang [32] Chin [39]), but also the normal modes solution (Chen [40]). However, Luco and Apsel's formulation includes growing exponentials in the source terms and has numerical instabilities at very high frequencies and/or large wavenumbers.

To avoid this problem, we adopt "stress discontinuities" to express the source conditions, as shown by Kennett and Kerry [38] and derive Green's functions without exponentials growing with wavenumber and/or frequency.

2) The static and dynamic Green's function due to point sources

(i) Theoretical solutions

Figure 3-4 shows a layered half-space consisting of horizontally flat N layers overlying the homogeneous half-space. Point sources and a receiver are located at $(0,0,h)$ and (r,θ,z) , respectively, in the cylindrical coordinate system. We assign the layers with the source and the receiver to the sth and

j th layers, respectively. Assuming the time-dependent term as $\exp(i\omega t)$, static and dynamic Green's functions at the receiver will be the following forms:

$$\left\{ \begin{aligned} U_{r(x)}^j(r, \theta, z; h) &= \int_0^\infty \left\{ V_{1(x)}^j(z; h) \frac{dJ_1(kr)}{dkr} + H_{1(x)}^j(z; h) \frac{J_1(kr)}{kr} \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} = u_{r(x)}^j(r, z; h) \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\ U_{rz}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{1z}^j(z; h) J_1(kr) \right\} dk \\ U_{\theta(x)}^j(r, \theta, z; h) &= \begin{pmatrix} - \\ + \end{pmatrix} \int_0^\infty \left\{ V_{1(x)}^j(z; h) \frac{J_1(kr)}{kr} + H_{1(x)}^j(z; h) \frac{dJ_1(kr)}{dkr} \right\} dk \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} = \begin{pmatrix} - \\ + \end{pmatrix} u_{\theta(x)}^j(r, z; h) \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\ U_{\theta z}^j(r, \theta, z; h) &= 0 \\ U_{z(x)}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{2(x)}^j(z; h) J_1(kr) \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} = -u_{z(x)}^j(r, z; h) \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\ U_{zz}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{2z}^j(z; h) J_0(kr) \right\} dk \end{aligned} \right. \quad (3-29)$$

For displacement, and

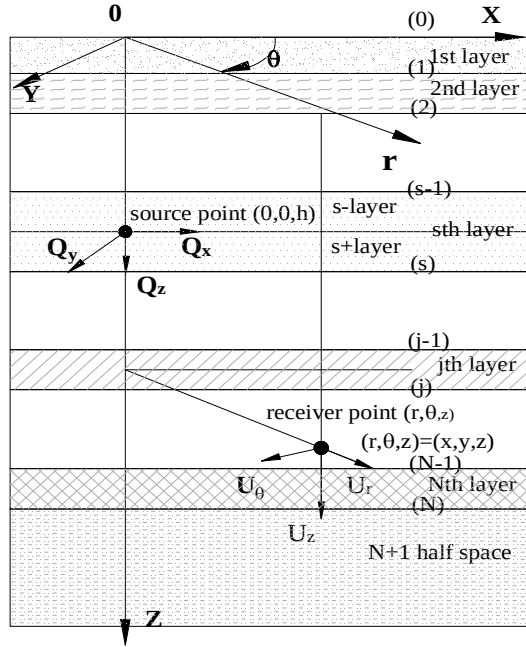
$$\left\{ \begin{aligned} \sigma_{rz(x)}^j(r, \theta, z; h) &= \int_0^\infty \left\{ V_{3(x)}^j(z; h) \frac{dJ_1(kr)}{dkr} + H_{2(x)}^j(z; h) \frac{J_1(kr)}{kr} \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\ \sigma_{rzz}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{3z}^j(z; h) J_1(kr) \right\} dk \\ \sigma_{\theta z(x)}^j(r, \theta, z; h) &= \begin{pmatrix} - \\ + \end{pmatrix} \int_0^\infty \left\{ V_{3(x)}^j(z; h) \frac{J_1(kr)}{kr} + H_{2(x)}^j(z; h) \frac{dJ_1(kr)}{dkr} \right\} dk \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\ \sigma_{\theta zz}^j(r, \theta, z; h) &= 0 \\ \sigma_{zz(x)}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{4(x)}^j(z; h) J_1(kr) \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\ \sigma_{zzz}^j(r, \theta, z; h) &= - \int_0^\infty \left\{ V_{4z}^j(z; h) J_0(kr) \right\} dk \end{aligned} \right. \quad (3-30)$$

For stress.

In the above Eqs, U_r, U_θ, U_z are the components of displacements in the cylindrical coordinate system, and $\sigma_{rz}, \sigma_{\theta z}, \sigma_{zz}$ are those of stresses on the plane normal to the z axis. The superscript j represents the j th layer, and the last subscripts x, y , and z correspond to the solutions due to the point sources with vector components $(Q_x, 0, 0)$, $(0, Q_y, 0)$, $(0, 0, Q_z)$ in the Cartesian coordinate system, respectively. Within the parentheses, the upper and lower values are all ousted to the solutions due to Q_x and Q_y . J_0 and J_1 are Bessel functions of orders 0 and 1, and the variable of integration (k) represents horizontal wavenumber. H_1^j and H_2^j and $V_1^j \sim V_4^j$ are “the displacement-stress vectors” in Kennett and Kerry [41], and “the motion-stress vectors” in Aki and Richards [2]; “vertically polarized waves” in Luco and Aspsel [42] for SH and P - SV waves, respectively, which depend on z, h, k , and the source and boundary conditions. The explicit forms of the displacement-stress vectors, in which there aren't growing exponentials, are given in Appendix A.

(ii) Asymptotic solution of Green's function

We find the asymptotic solutions of Green's functions at large wavenumbers. Since our main concern is the case when the source depth is equal or close to the receiver depth, we assume the source and the receiver are located in the same j th layer for the derivation of our equations. Later, we shall numerically check the validity of these equations for the cases when the source and/or the receiver are located on a boundary between layers and/or in different layers, as shown in Appendix B, the displacement-stress vectors for very large k converge to the following forms:



Point sources are located at $(0,0,h)$ in the s th layer, with the vector components $(Q_x, 0, 0)$, $(0, Q_y, 0)$, $(0, 0, Q_z)$, in the Cartesian coordinate system. The receiver is located at (r, θ, z) in the j th layer, with the displacement components (U_r, U_θ, U_z) in the cylindrical coordinate system.

Figure 3-3 Multi-layered half-space model considered in this study

$$\begin{cases} H_{1q}^j(z;h) \approx \tilde{H}_{1q}^j(z;h) = \bar{H}_{1q}^j \exp\{-k|z-h|\} \\ \begin{Bmatrix} V_{1q}^j(z;h) \\ V_{2q}^j(z;h) \end{Bmatrix} \approx \begin{Bmatrix} \tilde{V}_{1q}^j(z;h) \\ \tilde{V}_{2q}^j(z;h) \end{Bmatrix} = \begin{Bmatrix} \bar{V}_{11q}^j + k\bar{V}_{12q}^j \\ \bar{V}_{21q}^j + k\bar{V}_{22q}^j \end{Bmatrix} \exp\{-k|z-h|\}, (q = x, y, \text{ or } z) \\ V_{12q}^j = V_{22q}^j = 0, \text{ for } z = h \end{cases} \quad (3-31)$$

For the displacement vectors, and

$$\begin{cases}
H_{2q}^j(z;h) \approx \tilde{H}_{2q}^j(z;h) = \tilde{H}_{2q}^j k \exp\{-k|z-h|\} \\
\begin{cases} V_{3q}^j(z;h) \\ V_{4q}^j(z;h) \end{cases} \approx \begin{cases} \tilde{V}_{3q}^j(z;h) \\ \tilde{V}_{4q}^j(z;h) \end{cases} = \begin{cases} \bar{V}_{31q}^j + k\bar{V}_{32q}^j \\ \bar{V}_{41q}^j + k\bar{V}_{42q}^j \end{cases} k \exp\{-k|z-h|\}, (q = x, y, \text{ or } z) \\
V_{32q}^j = V_{42q}^j = 0, \text{ for } z = h
\end{cases} \quad (3-32)$$

For the stress vectors, where the values with upper bars in the right-hand sides of the equations are constants, will be shown later how to determine.

Substituting the asymptotic solutions (3-31) and (3-32) into the corresponding displacement-stress vectors of Eqs (3-29) and (3-30), and by analytic integration using the relations given in Appendix B, it can obtain,

$$\begin{cases}
\tilde{U}_{r(x)}^j(r, \theta, z; h) = \left\{ \bar{V}_{11(x)}^j \left(S_1 - \frac{S_4}{r} \right) + \bar{V}_{12(x)}^j \left(S_2 - \frac{S_5}{r} \right) + \bar{H}_{1(x)}^j \frac{S_4}{r} \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \equiv \tilde{u}_{r(x)}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\tilde{U}_{rz}^j(r, \theta, z; h) = -\left\{ \bar{V}_{11z}^j S_5 + \bar{V}_{12z}^j S_6 \right\} \\
\tilde{U}_{\theta(x)}^j(r, \theta, z; h) = \begin{pmatrix} - \\ + \end{pmatrix} \left\{ \bar{V}_{11(x)}^j \frac{S_4}{r} + \bar{V}_{12(x)}^j \frac{S_5}{r} + \bar{H}_{1(x)}^j \left(S_1 - \frac{S_4}{r} \right) \right\} \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \equiv \tilde{u}_{r\theta(x)}^j \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\
\tilde{U}_{z(x)}^j(r, \theta, z; h) = -\left\{ \bar{V}_{21(x)}^j S_5 + \bar{V}_{22(x)}^j S_6 \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \equiv -\tilde{u}_{z(x)}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\tilde{U}_{zz}^j(r, \theta, z; h) = -\left\{ \bar{V}_{21z}^j S_1 + \bar{V}_{22z}^j S_2 \right\}
\end{cases} \quad (3-33)$$

For displacement, and

$$\begin{cases}
\tilde{\sigma}_{rz(x)}^j(r, \theta, z; h) = \left\{ \bar{V}_{31(x)}^j \left(S_2 - \frac{S_5}{r} \right) + \bar{V}_{32(x)}^j \left(S_3 - \frac{S_6}{r} \right) + \bar{H}_{2(x)}^j \frac{S_5}{r} \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\tilde{\sigma}_{rzz}^j(r, \theta, z; h) = -\left\{ \bar{V}_{31z}^j S_6 + \bar{V}_{32z}^j S_7 \right\} \\
\tilde{\sigma}_{\theta z(x)}^j(r, \theta, z; h) = \begin{pmatrix} - \\ + \end{pmatrix} \left\{ \bar{V}_{31(x)}^j \frac{S_5}{r} + \bar{V}_{32(x)}^j \frac{S_6}{r} + \bar{H}_{2(x)}^j \left(S_2 - \frac{S_5}{r} \right) \right\} \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\
\tilde{\sigma}_{zz(x)}^j(r, \theta, z; h) = -\left\{ \bar{V}_{41(x)}^j S_6 + \bar{V}_{42(x)}^j S_7 \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\sigma_{zz}^j(r, \theta, z; h) = -\left\{ \bar{V}_{41z}^j S_2 + \bar{V}_{42z}^j S_3 \right\}
\end{cases} \quad (3-34)$$

For stress; see Appendix C for S_1 through S_7 .

(iii) Procedure to Compute the Green's function using the asymptotic solution

We describe how to compute static and dynamic Green's functions due to point sources, using the asymptotic solutions obtained above. This procedure is basically the same as Herrmann's [33].

First, we determine the constant values in Eqs. (3-31) and (3-32). We obtain those values by substituting z , h , and large wavenumbers into the corresponding displacement-stress vectors given in Appendix A:

$$\begin{cases}
H_{1q}^j(z; h) = \bar{H}_{1q}^j(\bar{k}) \exp\{-\bar{k}|z-h|\} \\
\bar{V}_{(\frac{1}{2})2q}^j = \begin{cases} \frac{1}{\bar{k}_2 - \bar{k}_1} \left\{ V_{(\frac{1}{2})q}^j(\bar{k}_2) \exp\{-\bar{k}_2|z-h|\} - V_{(\frac{1}{2})q}^j(\bar{k}_1) \exp\{-\bar{k}_1|z-h|\} \right\} & (z \neq h) \\ 0 & (z = h) \end{cases} \\
\bar{V}_{(\frac{1}{2})q}^j = \begin{cases} V_{(\frac{1}{2})q}^j(\bar{k}_1) \exp\{\bar{k}_1|z-h|\} - \bar{k}_1 \bar{V}_{(\frac{1}{2})2q}^j & (z \neq h) \\ V_{(\frac{1}{2})q}^j(\bar{k}) & (z = h) \end{cases}
\end{cases} \quad (3-35)$$

For the displacement vectors, and

$$\begin{cases}
H_{2q}^j(z; h) = \frac{1}{k} H_{2q}^j k \exp\{\bar{k}|z-h|\} \\
\bar{V}_{(\frac{3}{4})2q}^j = \begin{cases} \frac{1}{\bar{k}_2 - \bar{k}_1} \left\{ \frac{1}{\bar{k}_2} V_{(\frac{3}{4})q}^j(\bar{k}_2) \exp\{\bar{k}_2|z-h|\} - \frac{1}{\bar{k}_1} V_{(\frac{3}{4})q}^j(\bar{k}_1) \exp\{\bar{k}_1|z-h|\} \right\} & (z \neq h) \\ 0 & (z = h) \end{cases} \\
\bar{V}_{(\frac{3}{4})q}^j = \begin{cases} \frac{1}{\bar{k}_1} V_{(\frac{3}{4})q}^j(\bar{k}_1) \exp\{\bar{k}_1|z-h|\} - \bar{k}_1 \bar{V}_{(\frac{3}{4})2q}^j & (z \neq h) \\ \frac{1}{\bar{k}} V_{(\frac{3}{4})q}^j(\bar{k}) & (z = h) \end{cases}
\end{cases} \quad (3-36)$$

For the stress vectors, where $\bar{k}, \bar{k}_1, \bar{k}_2$ are large wavenumbers much than the Rayleigh and Love poles and the branch points, satisfying the following conditions:

$$\bar{k}, \bar{k}_1, \text{ and } \bar{k}_2 \gg \max(k_\beta^j \text{ or } k_\alpha^j), \quad (j = 1, 2, \dots, N+1) \quad (3-37)$$

And $\bar{k}_1 \neq \bar{k}_2$.

Second, subtracting asymptotic solutions of Eqs.(3-31) and (3-32) from the corresponding displacement-stress vectors in Eqs. (3-29) and (3-30) and adding the analytic integrations (3-33) and (3-34) to the outsides of the integrals, we obtain the finial solutions:

$$\begin{cases}
U_{r(x)}^j(r, \theta, z; h) = \int_0^\infty \left[\left\{ V_{1(x)}^j - \tilde{V}_{1(x)}^j \right\} \frac{dJ_1}{dkr} + \left\{ H_{1(x)}^j - \tilde{H}_{1(x)}^j \frac{J_1}{kr} \right\} \right] dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{U}_{r(x)}^j \\
U_{rz}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ V_{1z}^j - \tilde{V}_{1z}^j \right\} J_1(kr) \right] dk + \tilde{U}_{rz}^j \\
U_{\theta(x)}^j(r, \theta, z; h) = \begin{pmatrix} - \\ + \end{pmatrix} \int_0^\infty \left[\left\{ V_{1(x)}^j - \tilde{V}_{1(x)}^j \right\} \frac{J_1}{kr} + \left\{ H_{1(x)}^j - \tilde{H}_{1(x)}^j \frac{dJ_1}{dkr} \right\} \right] dk \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} + \tilde{U}_{\theta(x)}^j \\
U_{z(x)}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ V_{2(x)}^j - \tilde{V}_{2(x)}^j \right\} J_1(kr) \right] dk \cdot \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{U}_{z(x)}^j \\
U_{zz}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ V_{2z}^j - \tilde{V}_{2z}^j \right\} J_0(kr) \right] dk + \tilde{U}_{zz}^j
\end{cases} \quad (3-38)$$

For displacement, and

$$\begin{cases}
\sigma_{rz(x)}^j(r, \theta, z; h) = \int_0^\infty \left[\left\{ V_{3(x)}^j - \bar{V}_{3(x)}^j \right\} \frac{dJ_1}{dkr} + \left\{ H_{2(x)}^j - \tilde{H}_{2(x)}^j \frac{J_1}{kr} \right\} \right] dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{\sigma}_{rz(x)}^j \\
\tilde{\sigma}_{rzz}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ \bar{V}_{31z}^j - \bar{V}_{31z}^j \right\} J_1(kr) \right] dk + \tilde{\sigma}_{rzz}^j \\
\sigma_{\theta z(x)}^j(r, \theta, z; h) = \begin{pmatrix} - \\ + \end{pmatrix} \int_0^\infty \left[\left\{ V_{3(x)}^j - \bar{V}_{3(x)}^j \right\} \frac{J_1}{kr} + \left\{ H_{2(x)}^j - \tilde{H}_{2(x)}^j \frac{dJ_1}{dkr} \right\} \right] dk \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} + \tilde{\sigma}_{\theta z(x)}^j \\
\sigma_{zz(x)}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ \bar{V}_{41(x)}^j - \bar{V}_{41(x)}^j \right\} J_1(kr) \right] dk \cdot \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{\sigma}_{zz(x)}^j \\
\sigma_{zzz}^j(r, \theta, z; h) = - \int_0^\infty \left[\left\{ V_{4z}^j - \tilde{V}_{41z}^j \right\} J_0(kr) \right] dk + \tilde{\sigma}_{zzz}^j
\end{cases} \quad (3-39)$$

For stress; since the $\{\}$ parts in the above integrands converge to zero with increasing wavenumber, we can efficiently carry out the numerical integrations.

3) The static and dynamic Green's function due to dipole sources

We show the procedure to obtain Green's functions due to dipole sources using the asymptotic technique presented above. Those Green's functions in the frequency domain are expressed in the Cartesian coordinate system shown in Figure 3-4, as follows:

$$U_k^s(0, 0, h) = M_{ji} U_{ik,j}^j(r, \theta, z; h) = M_{ji} U_{ik,j}^j(x, y, z; h) \quad (3-40)$$

Where the subscripts i, j and k are vector components in the Cartesian coordinate and the summation conventions are used for them, a comma between subscripts is used in spatial derivatives at the point (x, y, z) , M_{ji} is the moment tensor (see, e.g., Aki and Richards [23]). To make direct use of the asymptotic solutions obtained above, we do not use the reciprocity theorem in Eq. (3-40). Therefore, U_k^s is the displacement at the source location $(0, 0, h)$ in the sth layer, due to the moment tensor applied at the receiver location (x, y, z) in the jth layer.

Green's functions in the cylindrical coordinate system, given in Eq. (3-29), are transferred to those in the Cartesian coordinate system as follows:

$$\begin{cases} U_{xk}^j = U_{rk}^j \cos \theta - U_{\theta k}^j \sin \theta \\ U_{yk}^j = U_{rk}^j \sin \theta + U_{\theta k}^j \cos \theta \end{cases} \quad (3-41)$$

The spatial derivatives of Eq. (3-41), which are required in Eq. (3-40), are

$$\begin{cases} U_{xk,j}^j = U_{rk,j}^j \cos \theta + U_{rk}^j (\cos \theta)_{,j} - U_{\theta k,j}^j \sin \theta - U_{\theta k}^j (\sin \theta)_{,j} \\ U_{yk,j}^j = U_{rk,j}^j \sin \theta + U_{rk}^j (\sin \theta)_{,j} + U_{\theta k,j}^j \cos \theta + U_{\theta k}^j (\cos \theta)_{,j} \end{cases} \quad (3-42)$$

For $j=x$, or y , and

$$\begin{cases} U_{xk,z}^j = U_{rk,z}^j \cos \theta - U_{\theta k,z}^j \sin \theta \\ U_{yk,z}^j = U_{rk,z}^j \sin \theta + U_{\theta k,z}^j \cos \theta \end{cases} \quad (3-43)$$

Where

$$\begin{cases} \cos \theta = \frac{x}{r}, \quad \sin \theta = \frac{y}{r}, \quad r = \sqrt{x^2 + y^2} \\ (\cos \theta)_{,x} = \frac{\sin^2 \theta}{r}, \quad (\sin \theta)_{,y} = \frac{\cos^2 \theta}{r} \\ (\cos \theta)_{,y} \equiv (\sin \theta)_{,x} = \frac{\sin \theta \cos \theta}{r} \end{cases} \quad (3-44)$$

The spatial derivatives of Green's functions in the right-hand sides of Eqs. (3-42) and (3-43) are obtained from Eq. (3-29)

$$\begin{cases} U_{r(y),j}^j = \int_0^\infty \left\{ V_{1(y)}^j \left(\frac{dJ_1}{dkr} \right)_{,j} + H_{1(y)}^j \frac{J_1}{kr} \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + u_{r(y)}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix}_{,j} \\ U_{rz,j}^j = - \int_0^\infty \left\{ V_{1z}^j (J_1)_{,j} \right\} dk \\ U_{\theta(y),j}^j = \begin{pmatrix} - \\ + \end{pmatrix} \left[\int_0^\infty \left\{ V_{1(y)}^j \left(\frac{J_1}{kr} \right)_{,j} + H_{1(y)}^j \left(\frac{dJ_1}{dkr} \right)_{,j} \right\} dk \cdot \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} + u_{\theta(y)}^j \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix}_{,j} \right] \\ U_{z(y),j}^j = - \left[\int_0^\infty \left\{ V_{2(y)}^j (J_1)_{,j} \right\} dk \cdot \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + u_{z(y)}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix}_{,j} \right] \\ U_{zz,j}^j = - \int_0^\infty \left\{ V_{2z}^j (J_0)_{,j} \right\} dk \end{cases} \quad (3-45)$$

For $j=x$ or y , where u_{pq}^j is defined in Eq. (3-29), and

$$\begin{cases}
\frac{dJ_1}{dkr} = J_0 - \frac{J_1}{kr}, (J_0)_{,j} = -J_1 k r_{,j}, (J_1)_{,j} = \frac{dJ_1}{dkr} k r_{,j} \\
\left(\frac{J_1}{kr}\right)_{,j} = \frac{r_{,j}}{r} \left(J_0 - 2\frac{J_1}{kr}\right) \\
\left(\frac{dJ_1}{dkr}\right)_{,j} = -\frac{r_{,j}}{r} \left(J_0 - 2\frac{J_1}{kr} + k r J_1\right)
\end{cases} \quad (3-46)$$

$r_{,x} = \cos \theta, \quad r_{,y} = \sin \theta.$

We also obtain

$$\begin{cases}
U_{r(x),z}^j = \int_0^\infty \left\{ V_{1(y),z}^j \frac{dJ_1}{dkr} + H_{1(y),z}^j \frac{J_1}{kr} \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
U_{rz,z}^j = -\int_0^\infty \left\{ V_{1z,z}^j J_1 \right\} dk \\
U_{\theta(x),z}^j = \begin{pmatrix} - \\ + \end{pmatrix} \int_0^\infty \left\{ V_{1(y),z}^j \frac{J_1}{kr} + H_{1(y),z}^j \frac{dJ_1}{dkr} \right\} dk \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\
U_{z(x),z}^j = -\int_0^\infty \left\{ V_{2(y),z}^j J_1 \right\} dk \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
U_{zz,z}^j = -\int_0^\infty \left\{ V_{2z,z}^j J_0 \right\} dk
\end{cases} \quad (3-47)$$

We obtain $H_{1k,z}^j, V_{1k,z}^j$ and $V_{2k,z}^j$ in Eq. (3-66) from $H_{2k,z}^j$ and $V_{1k,z}^j$ through V_{4k}^j in Eqs. (3-48) and (3-49) and Appendix A using the following relations:

$$H_{1k,z}^j = \frac{1}{\mu^j} H_{2k}^j, \quad (\text{for } k = x \text{ or } y), \quad (3-48)$$

And

$$\begin{cases}
V_{1k,z}^j = \frac{1}{\mu^j} V_{3k}^j + k V_{2k}^j \\
V_{2k,z}^j = \frac{1}{\lambda^j + 2\mu^j} \{ V_{4k}^j - k \lambda^j V_{1k}^j \}, (\text{for } k = x, y, \text{ or } z)
\end{cases} \quad (3-49)$$

where μ^j and λ^j are Lamé's constant of the j th layer. For the derivation of the above equations for the 2D case, see Eqs (7.20) through (7.27) of chapter 7 in Aki and Richards [23]. It is easy to confirm that the same relations are true for the 3D case.

4) The procedure to compute the Green's function by asymptotic solutions

Using a similar procedure used from Eq. (3-31) to (3-34) plus Eqs. (3-48) and (3-49), the wavenumber integrations of the asymptotic solutions in Eqs. (3-45) and (3-47) are analytically derived as follows:

$$\begin{cases}
\tilde{U}_{r^{(x)},j}^j = r_{,j} \left\{ -\left(\bar{V}_{11^{(x)}}^j S_6 + \bar{V}_{12^{(x)}}^j S_7 \right) - B_{(x)} + A_{(x)} \right\} \cdot \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{u}_{r^{(x)}}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix}_{,j} \\
\tilde{U}_{rz,j}^j = -r_{,j} \left\{ \bar{V}_{11z}^j S_2 + \bar{V}_{11z}^j S_3 - \frac{1}{r} \tilde{u}_{zz}^j \right\} \\
\tilde{U}_{\theta^{(x)},j}^j = \begin{pmatrix} - \\ + \end{pmatrix} \left[r_{,j} \left\{ B_{(x)} - \bar{H}_{1^{(x)}}^j S_6 - A_{(x)} \right\} \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} + \tilde{U}_{\theta^{(x)}}^j \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix}_{,j} \right] \\
\tilde{U}_{z^{(x)}}^j(r, \theta, z; h) = - \left[r_{,j} \left\{ \bar{V}_{21^{(x)}}^j S_2 + \bar{V}_{22^{(x)}}^j S_3 - \frac{1}{r} \tilde{u}_{z^{(x)}}^j \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} + \tilde{u}_{z^{(x)}}^j \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix}_{,j} \right] \\
\tilde{U}_{zz}^j(r, \theta, z; h) = r_{,j} \left\{ \bar{V}_{21z}^j S_6 + \bar{V}_{22z}^j S_7 \right\}
\end{cases} \quad (3-50)$$

For $j=x$ and y , where $\tilde{u}_{pq}^j$ is defined in Eq (3-52), and

$$\begin{cases}
A_{(x)} = \frac{1}{r} \bar{H}_{1^{(x)}}^j \left(S_1 - \frac{2}{r} S_4 \right) \\
A_{(y)} = \frac{1}{r} \left\{ \bar{V}_{11^{(x)}}^j \left(S_1 - \frac{2}{r} S_4 \right) + \bar{V}_{12^{(x)}}^j \left(S_2 - \frac{2}{r} S_5 \right) \right\}
\end{cases} \quad (3-51)$$

We also obtain

$$\begin{cases}
\tilde{U}_{r^{(x)},z}^j = \left\{ \left(\frac{1}{\mu^j} \bar{V}_{31^{(x)}}^j + \bar{V}_{21^{(x)}}^j \right) \left(S_2 - \frac{S_5}{r} \right) + \left(\frac{1}{\mu^j} \bar{V}_{32^{(x)}}^j + \bar{V}_{22^{(x)}}^j \right) \left(S_3 - \frac{S_6}{r} \right) + \frac{1}{\mu^j} \bar{H}_{2^{(x)}}^j \frac{S_5}{r} \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\tilde{U}_{rz,z}^j = - \left\{ \left(\frac{1}{\mu^j} \bar{V}_{31z}^j + \bar{V}_{21z}^j \right) S_6 + \left(\frac{1}{\mu^j} \bar{V}_{32z}^j + \bar{V}_{22z}^j \right) S_7 \right\} \\
\tilde{U}_{\theta^{(x)},j}^j = \begin{pmatrix} - \\ + \end{pmatrix} \left\{ \left(\frac{1}{\mu^j} \bar{V}_{31^{(x)}}^j + \bar{V}_{21^{(x)}}^j \right) \frac{S_5}{r} + \left(\frac{1}{\mu^j} \bar{V}_{32^{(x)}}^j + \bar{V}_{22^{(x)}}^j \right) \frac{S_6}{r} + \frac{1}{\mu^j} \bar{H}_{2^{(x)}}^j \left[S_2 - \frac{S_5}{r} \right] \right\} \begin{pmatrix} \sin \theta \\ \cos \theta \end{pmatrix} \\
\tilde{U}_{z^{(x)},z}^j = - \frac{1}{\lambda^j + 2\mu^j} \left\{ \left(\bar{V}_{41^{(x)}}^j - \lambda^j \bar{V}_{11^{(x)}}^j \right) S_6 + \left(\bar{V}_{42^{(x)}}^j - \lambda^j \bar{V}_{12^{(x)}}^j \right) S_7 \right\} \begin{pmatrix} \cos \theta \\ \sin \theta \end{pmatrix} \\
\tilde{U}_{zz,z}^j = - \frac{1}{\lambda^j + 2\mu^j} \left\{ \left(\bar{V}_{41z}^j - \lambda^j \bar{V}_{11z}^j \right) S_2 + \left(\bar{V}_{42z}^j - \lambda^j \bar{V}_{12z}^j \right) S_3 \right\}
\end{cases} \quad (3-52)$$

Finally, we obtain Green's functions due to dipole sources using the following procedure. First, we determine the constants $\bar{H}_{pq}^j$ and $\bar{V}_{pq}^j$ using Eqs. (3-35) and (3-36) and construct the asymptotic solutions (3-31) and (3-51) plus (3-48) and (3-49).

Second, we subtract the asymptotic solutions from the corresponding displacement-stress vectors in Eqs. (3-45) and (3-47), and numerically carry out the wavenumber integrations.

Finally, we obtain the Green's function (3-40) by adding the analytical integrations.

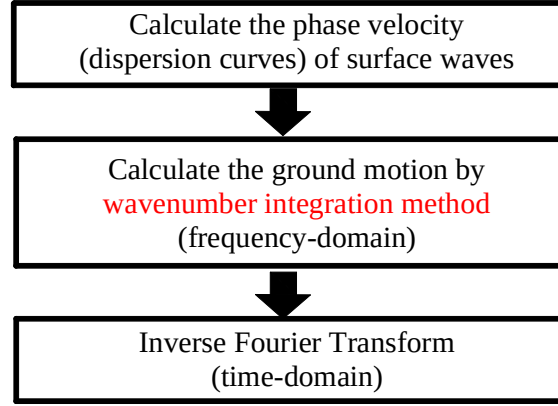


Figure 3-4 Flowchart for the wavenumber integration method

3.5.2 Improvement of the wavenumber integration method (Hisada [29])

As described in Hisada [29], it is difficult to compute Green's functions for layered half-spaces with source and receivers at nearly equidistant depths, because their integrands oscillate with slowly decreasing and increasing amplitudes for displacements and stresses, respectively. To remedy this problem, Apsel and Luco [30] proposed an asymptotic technique, in which we subtract asymptotic solutions from the integrands and integrate them analytically, and numerically integrate the remaining integrands. These procedures can be summarized in the following Eq.:

$$G = \int_0^\infty \left\{ (V - \tilde{V})b_1 + (H - \tilde{H})b_2 \right\} dk S + \Delta\tilde{V} + \Delta\tilde{H} \quad (3-53)$$

Where G is a displacement or stress Green's function, b_j is a Bessel function, S is a sinusoidal function, k is the horizontal wavenumber, and V and H are displacement-stress vectors for P-SV and SH waves, respectively. $\tilde{V}$ is the solution asymptotic to V as k goes to infinity, $\Delta\tilde{V}$ is the analytical integration corresponding to $\tilde{V}$. $\tilde{H}$ and $\Delta\tilde{H}$ are those for SH waves, and see Eqs (3-38) and (3-39) in Hisada [27] for details.

It is clear that the more accurate asymptotic solutions are, the shorter the range of integration is. However, more accurate solutions are also more complicated mathematically. Hisada [27] derived generalized forms for the direct waves and showed that the technique was very efficient even for the case that sources and/or receivers are located in layers different from the source layer. However, Hisada [27] found that the convergences of the asymptotic solutions are rather slow, when sources and/or receivers are very close to layer boundaries, because the reflected waves from the boundaries are not included in the solutions (see case 3 in the Results section of Hisada [27]). In this study, the modified analytic asymptotic solutions of the direct, reflected, and transmitted waves from layer boundaries (Hisada [27]) was applied.

3.5.3 Detailed key points on the numerical calculations

Among various available methods for computing the dynamic and static Green's functions of layered half-spaces, we use the method by Hisada [27], which is modified from Luco and Apsel [42]. This is an appropriate method for this study, as it is completely free from the numerical instabilities at lower and higher frequencies through the introduction of the R/T (reflection/transmission) matrix. In addition, it gives the static solution explicitly (Luco and Apsel [42]).

The procedure for static offset of displacement calculation are as follows. Since the Fourier amplitude of a step-like displacement is infinite at 0 Hz, it first computes velocities in the frequency domain, transforming them into the time domain. Then, we calculate the corresponding displacements by integrating the velocities numerically in the time domain. To check the accuracy of the calculated static offset, we recall that the Fourier amplitude of velocity at 0 Hz is the same as the value of the static offset in the time domain, since the static offset is the displacement as the time goes to infinity.

3.6 Application of hybrid synthesis method

In this section, in order to make a further description of this combined method mention above, the combined method was applied to the synthesis of ground motions for two simple and idealized surface fault models: the strike-slip and dip-slip fault in homogeneous half-space.

3.6.1 Ratios for modifying the stochastic Green's function method

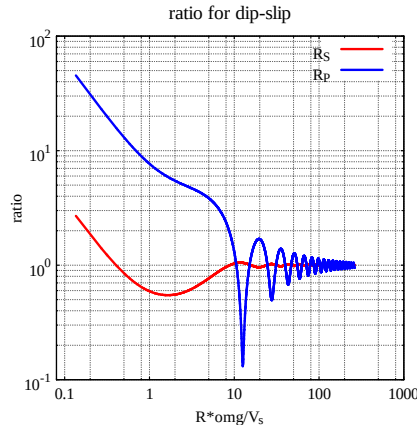
The modification on stochastic Green's function method introduced two ratios, and we defined it as R_s and R_p as below from Eqs. (3-17) and (3-18).

$$R_s = \frac{(N + IP + IS + FS)}{FS} = 6i\left(\frac{\beta}{r\omega}\right)^3 \left[1 - e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)}\right] - 6\left(\frac{\beta}{r\omega}\right)^2 \left[1 - \frac{\beta}{\alpha} e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)}\right] + 2i\left(\frac{\beta}{r\omega}\right)\left(\frac{\beta}{\alpha}\right)^2 e^{-i\omega r\left(\frac{1}{\alpha} - \frac{1}{\beta}\right)} - 3i\left(\frac{\beta}{r\omega}\right) + 1 \quad (3-17)$$

$$R_p = \frac{(N + IP + IS + FP)}{FP} = 9i\left(\frac{\alpha}{r\omega}\right)^3 \left[1 - e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)}\right] - 9\left(\frac{\alpha}{r\omega}\right)^2 \left[1 - \frac{\alpha}{\beta} e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)}\right] - 4i\left(\frac{\alpha}{r\omega}\right) + 3i\left(\frac{\alpha}{r\omega}\right)\left(\frac{\alpha}{\beta}\right)^2 e^{-i\omega r\left(\frac{1}{\beta} - \frac{1}{\alpha}\right)} + 1 \quad (3-18)$$

And simple examples on these ratios are shown here. The simple strike-fault model and dip-slip fault model used in the previous analysis under homogeneous half-space were adopted for the ratio calculations. The velocity ratio of $\alpha/\beta = 3.0/2.5$ is assumed in the calculations. And the results of ratios are shown in Figures 3-5 ~ 3-6. There are two significant points we could from the results clearly. Firstly, the ratios have the same tendency as the results as in Nozu's research [25][26], when $R\omega/V_s$ is smaller than 1, the absolute value of R_s is larger than 1, while $R\omega/V_s$ is in the media parts,

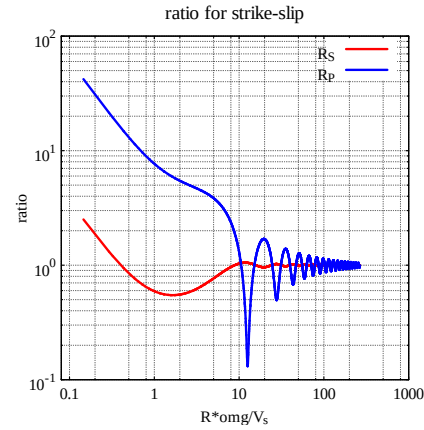
the R_S in Eq. (3-17) is smaller than 1, and it is approximated to 1 along with increasing of $R\omega/V_s$. The absolute value R_P in Eq. (3-18) is also the same as approached to 1 at the high-frequency components, but it is asymptotic speed is slower than the R_S . In any case, when comparing at the same distance, it can be said that the influence of the near-field and intermediate-field terms are larger at the lower frequency components. In addition, the absolute value of R_P shows a significantly larger value than the absolute value of R_S due to the denominator is rather small rather than the molecule is rather large. Secondly, the ratio is free of focal mechanism dependence as the peak value listed in the table below the Figures 3-5 ~ 3-6.



(a) Peak value of ratio ($\alpha/\beta = 3.0/2.5$)

Ratio	Frequency (Hz)	value
$(FS)_{min}$	1.6267	0.5476
$(FS)_{max}$	0.1356	2.6937
$(FP)_{min}$	12.6072	0.1311
$(FP)_{max}$	0.1356	45.3248

Figure 3-5 Ratio for modification based on the dip-slip fault model



(b) Peak value of ratio ($\alpha/\beta = 3.0/2.5$)

Ratio	Frequency (Hz)	value
$(FS)_{min}$	1.6202	0.5477
$(FS)_{max}$	0.1461	2.5054
$(FP)_{min}$	12.6151	0.1311
$(FP)_{max}$	0.1461	42.1020

Figure 3-6 Ratio for modification based on the strike-slip fault model

3.6.2 Strike-slip fault model

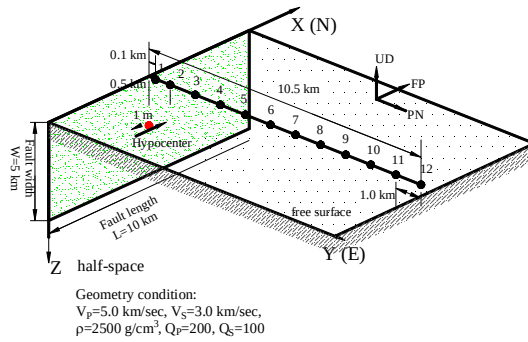
1) Fault model

First of all, a simple and pure strike-slip model of surface faulting (see Figure 3-7) is calculated taking into account both the static- and dynamic- terms of the near-field ground motions. In Figure 3-6 (a), the fault size is assumed 10.0km in length, and 5.0km in width. Surface observation points numbered 1~12 along a line perpendicular to the fault plane are calculated. It should be noted that point 1 is just 100m away from the fault trace, and 500m for point 2. Similarly, points 3-12 are 1.5-10.5 km away from the fault traction arranged with intervals of 1.0km. The characteristic fault model is shown in Figure 3-6 (b), the asperity is with slip of 1.0m, stress drop 141.0 bar, and slip of 0.6285m, stress drop of 28.8 bar for background. Another important parameters is the slip velocity

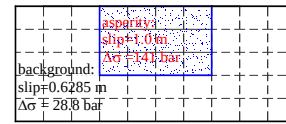
function; here we assume it is an isosceles triangle with 1.0-sec duration. The homogeneous half-space with the physical properties are listed in Figure 3-6 (a), and the fault parameters are given in Table 1. The static terms are calculated by the proposed representation theorem, Eq. (3-3), and the dynamic terms are simulated by adoption of modified statistical Green's function.

Table 3-1 Fault parameters for strike slip

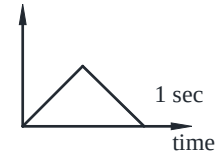
M_0 (dyne.cm)	Strike (°)	Dip (°)	Rake (°)	Length (km)	Width (km)	Hypocenter (km)			Rise time (s)
						x	y	z	
3.825×10^{25}	0	90	0	10.0	5.0	5.0	0.0	-2.5	1.0



(a) strike-slip model with 12 observation point



(b) Slip and stress drop



(c) slip-velocity

Figure 3-7 Strike-slip fault model

2) Comparisons between with and without modification

The comparisons among no modification, with the FP- and FS- ratios and Fourier transform (FFT) and response spectra of acceleration for P1 (100m away from the fault) and P12 (10.5km away from the fault) are checked.

Figure 3-8 (a) shows the results of FFT, it can be seen clearly, the FFT of very near-fault and near-fault are different, P1 is with larger value along NS-, EW- and UD-directions than P12, and with larger modifications for P1 during 0.1~5Hz than P12 during 0.1~0.3Hz in this case. From Figure 3-8 (b), the same tendency related to FFT can be obtained.

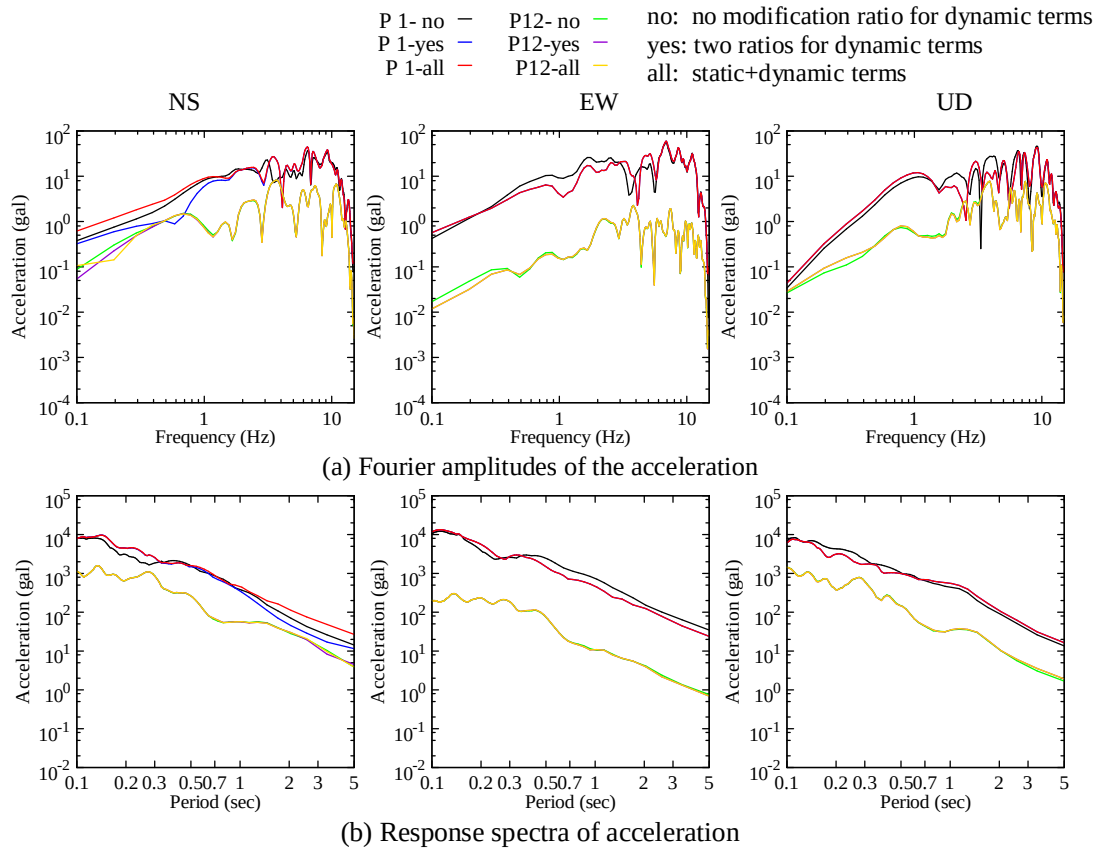


Figure 3-8 Fourier amplitudes (a) and response and spectra of acceleration (b)
(P1 and P12: with and without modification for dynamic terms)

Figure 3-9 listed the results of time history of points 1 and 12, the comparisons among no modification, with the FP-, FS- ratios and complete waveforms, the most significant differences are the NS-components due to this is a strike-slip fault. Significant differences along NS-direction were seen for the displacement time history for both P1 and P12. For P1, the obvious changing can be seen from only the dynamic, considering the FP-, FS-ratio to considering complete waveforms for displacements and velocity along NS-directions, while slight differences for acceleration along NS-, EW-, and UD-directions, velocity along EW- and UD-directions, mainly because this is strike-slip fault. And for P12, only slight differences among the without and with considering FP-, FS-ratio, and the complete waveforms for acceleration, velocity and displacement in this case due to P12 is a little further than P1.

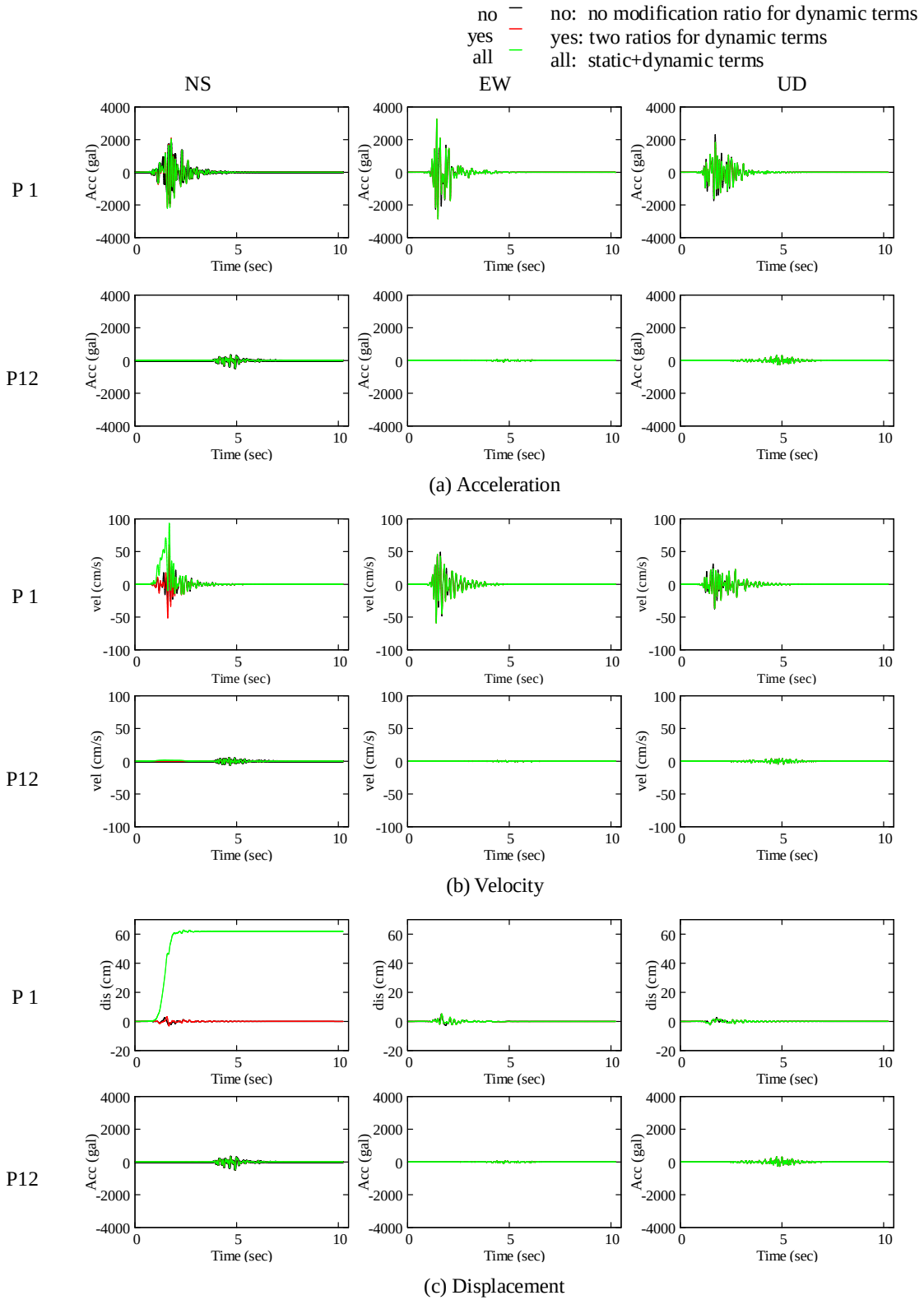


Figure 3-9 Time history of ground motions
(P1 and P12: with and without modification)

3) Simulation results

In Figure 3-10 (a) velocity time history, for the observation points 1~4 (which means the distance of 0.1 ~ 3.5 km away from the fault traction), the peak dynamic terms exhibited random values, and from sites 4 ~ 12, the peak dynamic terms exhibit attenuation tendency. As Somerville [43] obtained an empirical relationship for the variation of the PGV as a function of the moment magnitude and the distance of the station to causative fault, assuming that PGV varies as for distances larger than 3 km. The simulated results can well demonstrate this tendency. In Figure 3-11 (a) displacement time histories of dynamic terms, it also illustrated that the peak displacements of dynamic terms do not only exhibit pure time delay, but the random distribution which located very near the causative fault.

As for the static terms both in Figure 3-10 (b) and Figure 3-11 (b), the peak values of velocity and displacement showed the attenuation relationship that: with the distance become further away from the surface fault traction, and compared with the dynamic terms, it decreases faster. Even under this situation, almost all the static terms are larger than the dynamic displacements, which well showed that the permanent displacements should not be neglected when designing the near-fault bridge or pipeline engineering.

Compared the results with the theoretical Green's function method, the attenuation is different: very near the fault traction, the static terms performed the dominant parts, and the site further away from the fault traction, the dynamic terms taking the leading parts. As relied on different technology analysis methods, these differences can be accepted because the hybrid method proposed in this paper considered the complete waveforms of the dynamic terms; based on Nozu [26], the near-fault ground motions are heavily depend on the frequency and velocity.

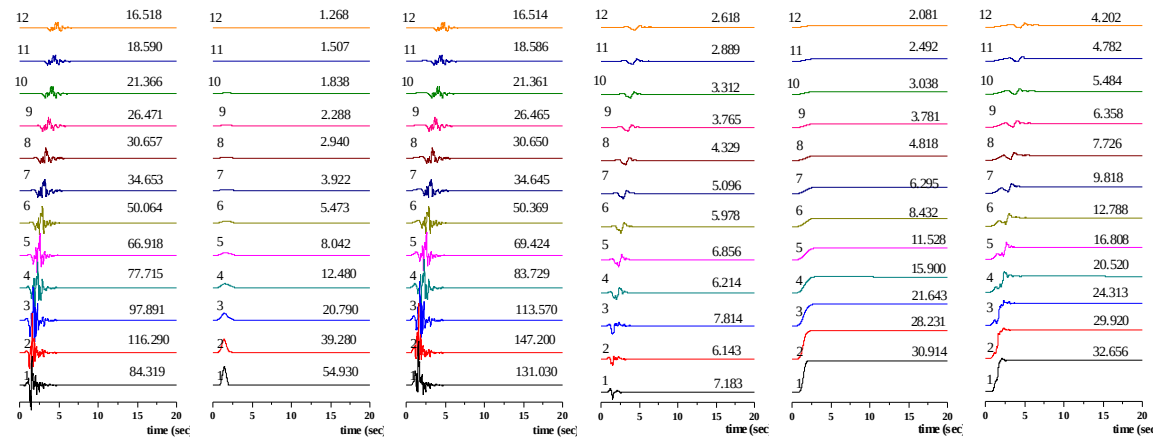


Figure 3-10 Velocity time histories along fault parallel direction

Figure 3-11 Displacement time histories along fault-parallel direction

3.6.3 Dip-slip fault model

1) Fault model

Next, we calculated the near-fault strong motions using a dip-slip model with surface faulting. The fault model is shown in Figure 3-12, here 24 observation points are located on the free surface along the line perpendicular to the fault traction; points 1-12 are located on the foot-wall, and points 13-24 are on the hanging-wall side. The points closest to the fault trace are points 12 and 13, 100m away from the surface fault trace, points 11 and 14 are 500m away from the fault line, and points 1~10, 15 ~ 24 are arranged by interval of 1.0 km. The characteristic fault model with slip distribution, and stress drop, and slip function are schematic the same as the former example 1 in Figure 3-7 (b) and (c), respectively. The material properties of layered half-space are assumed the same as listed in Figure 3-12, and parameters of dip-slip fault are given in Table 3-2.

Table 3-2 Fault parameters for dip-slip

M_0 (dyne.cm)	Strike (°)	Dip (°)	Rake (°)	Length (km)	Width (km)	Hypocenter (km)			Rise time (s)
						x	y	z	
3.825×10^{25}	0	45	90	10.0	5.0	5.0	1.77	1.77	1.0

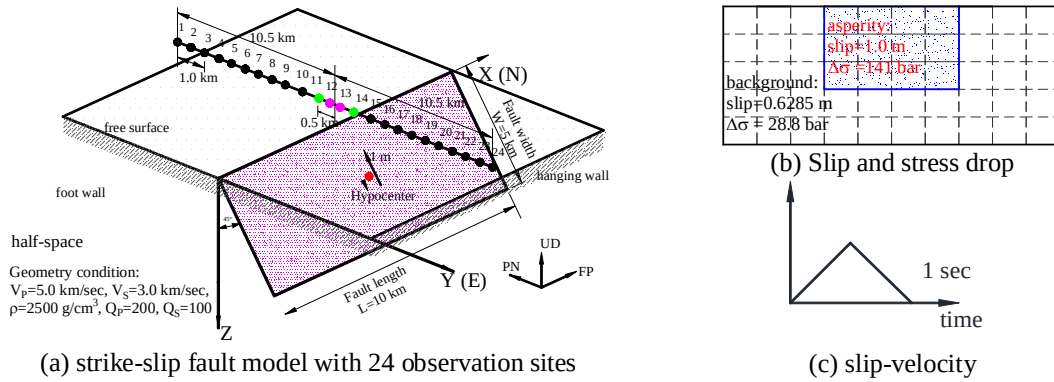


Figure 3-12 Dip-slip model with surface faulting and 24 observation points

2) Compilations between with and without modification

The simulation results of P1, P12 (located on foot wall) and P13 and P24 (located on hanging wall) for dip-slip fault model are shown in Figures 3-13 for Fourier transform and response spectra of acceleration (RSA), and Figure 3-14 for time histories, respectively. The comparisons among without, with two ratios (R_p and R_s) and the complete waveforms are explored. There are not differences for accelerations for all these four observation sites. While for velocity, the impulsive-like velocity can be seen in P12 and P13 in Figure 3-14 (b) and (c) along EW- and UD-directions due to this is a

calculation case of dip-slip fault. For displacement time history, there are no significant differences between without, with two ratios (R_p and R_s), mainly due to the assumption of homogeneous half space, and the rock is relatively hard.

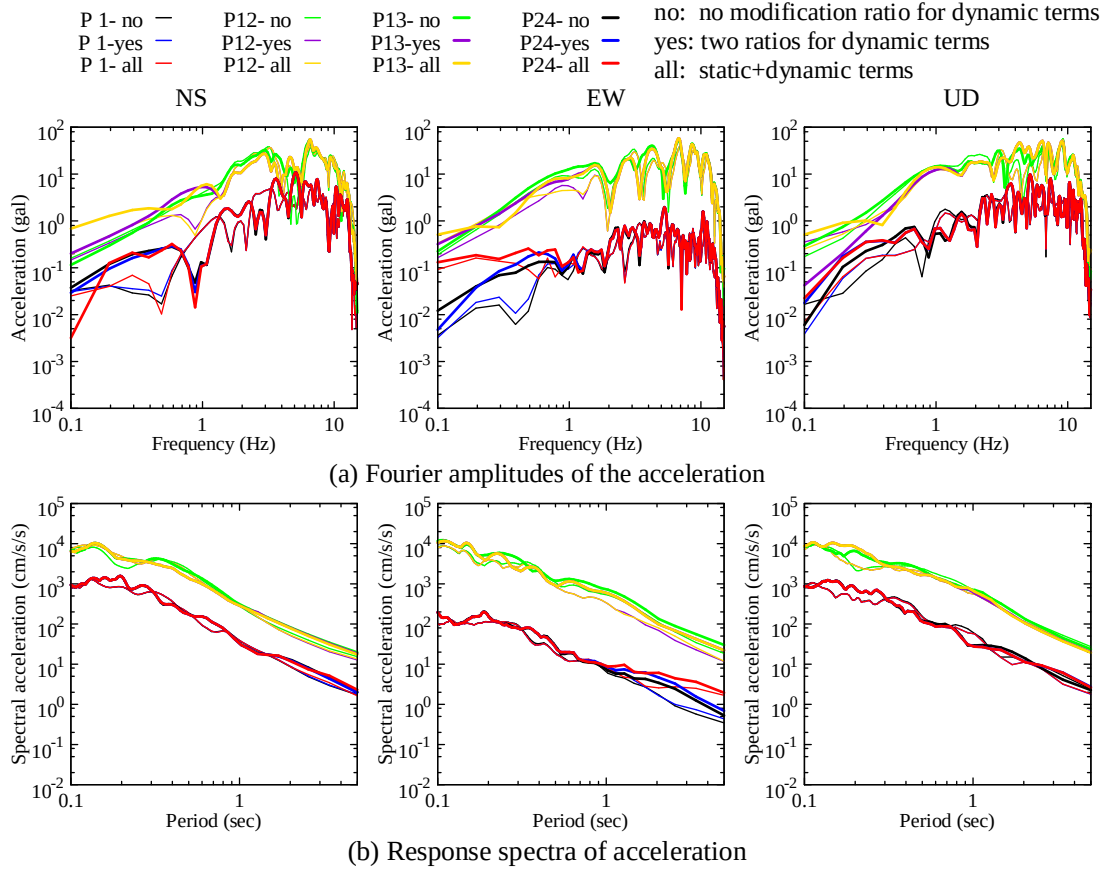
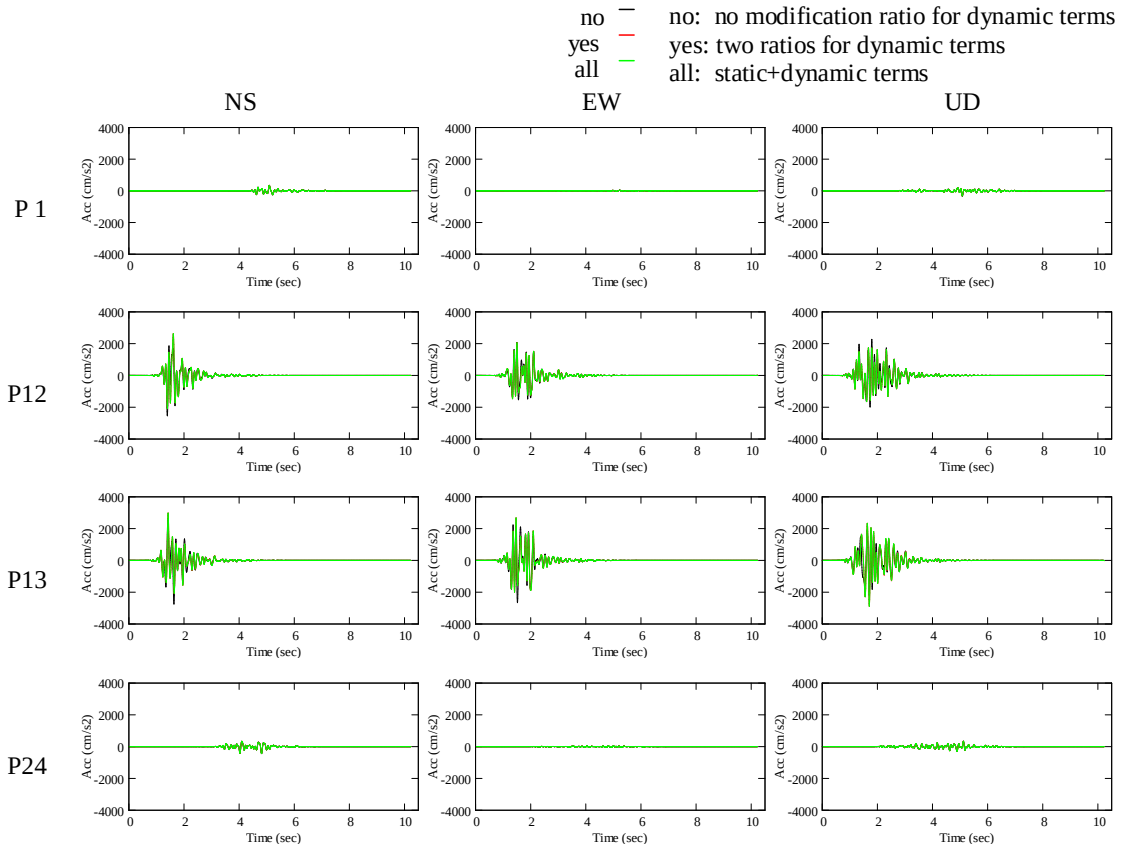
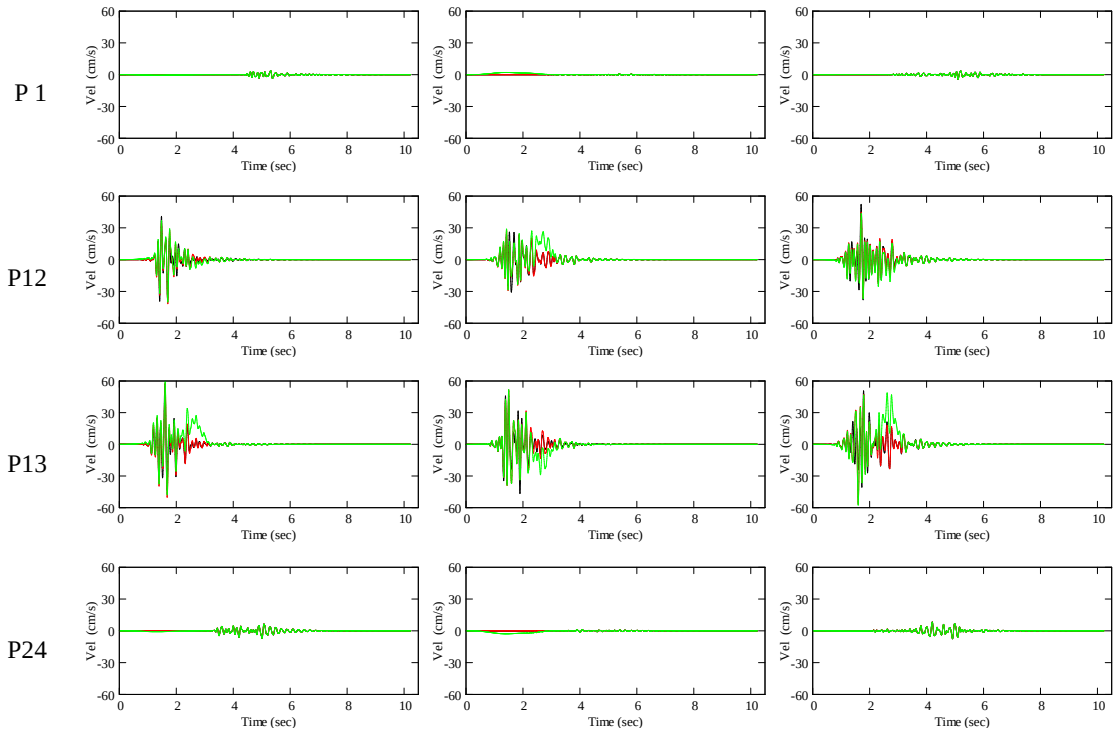


Figure 3-13 Fourier amplitudes (a) and response and spectra of acceleration (b)
P1, P12 (foot wall) and P13, P24 (hanging wall)



(a) Acceleration



(b) Velocity

Continues

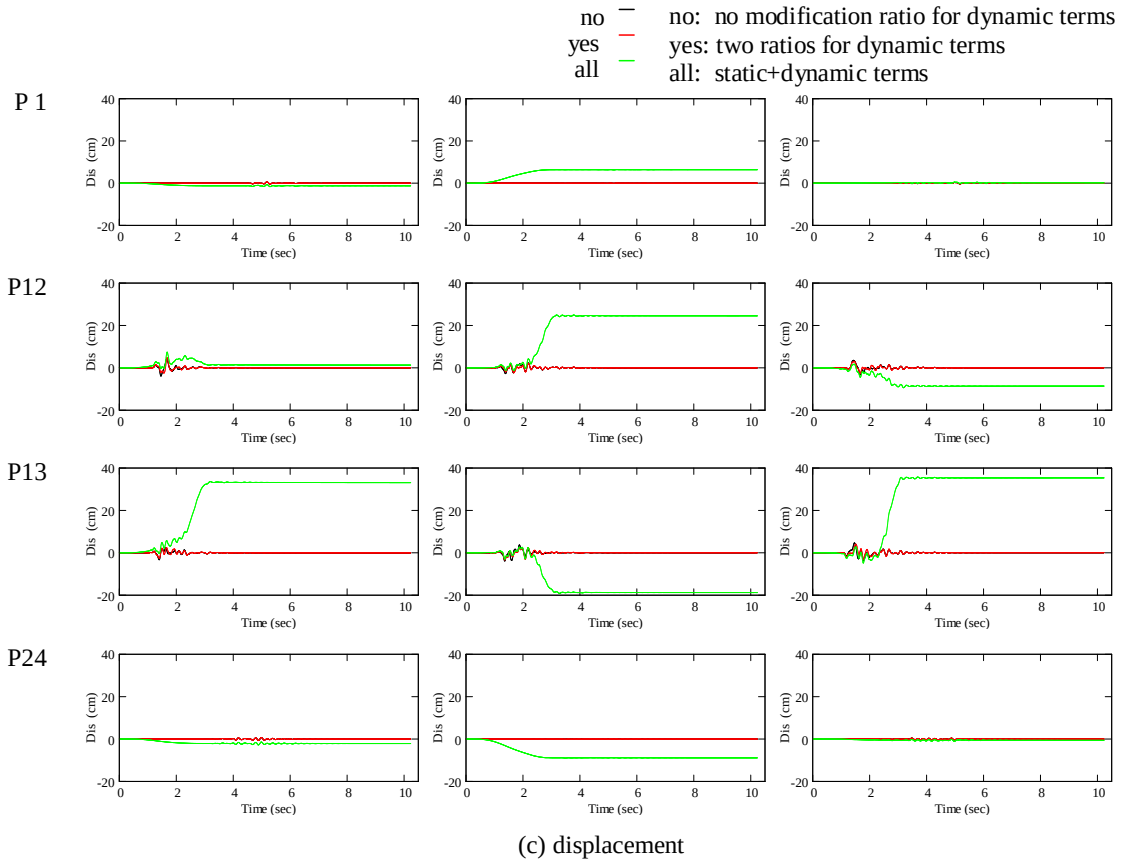


Figure 3-14 Time history of ground motions: P1, P12 (foot wall) and P13, P24 (hanging wall)

3) Results analysis

As this reverse fault model is just a pure dip-slip fault, mainly the simulated results along fault-normal and up-down directions have been analyzed here. Figures 3-15 ~ 3-16 listed the results along fault-normal (EW direction) and Figures 3-17 ~ 3-18 showed the simulation results along up-down direction, including the dynamic terms, static terms, and the total components of velocity and displacement time histories at the 24 observation points, respectively.

Along the EW-direction, the velocity time histories in Figure 3-15 (a) , there are not big differences between the sites located on the hanging wall or the footwall side; even the dynamic terms are all larger than the static velocity, whereas, the dynamic displacements are all larger than the dynamic displacements schematically illustrated in Figure 3-16 (a). Thus, the permanent displacements should be well considered when conducting the seismic analysis on near-fault spatial expanded structures.

In terms of the up-down direction in Figure 3-18, firstly, the results well showed the hang-wall effects, which means the observation points on the hang-wall are larger than the observation sites on the footwall, especially the static terms of both velocities and displacements. The distance becomes

further from the fault traction, the static displacements attenuated very fast, and then the dynamic component takes the dominant parts the same as the strike-slip fault model above.

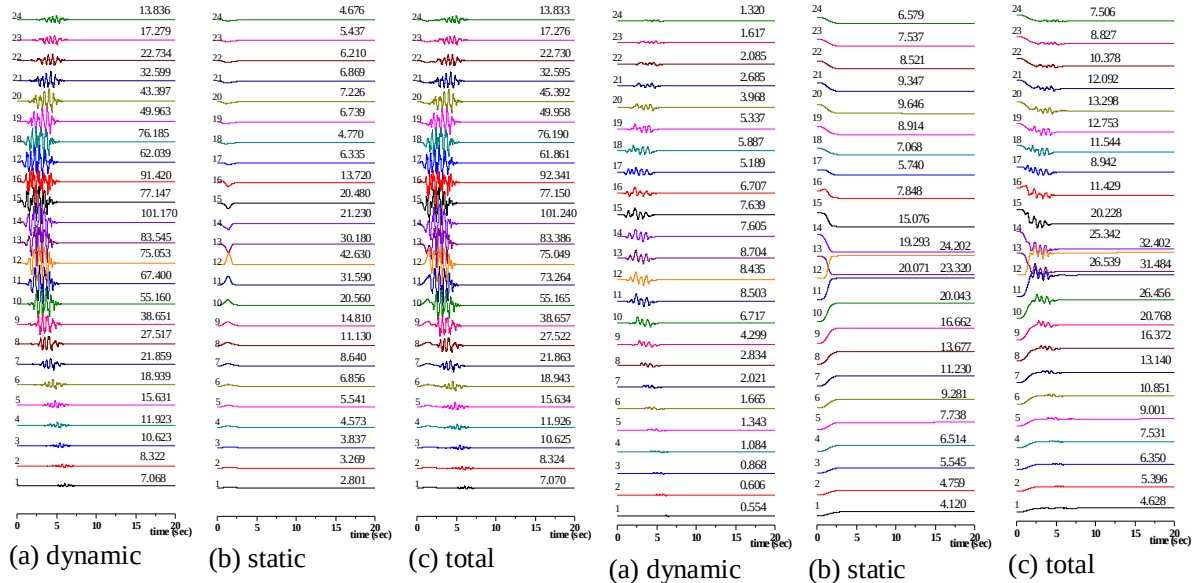


Figure 3-15 Velocity along EW direction (unit: cm/s)

Figure 3-16 Displacement along EW direction (unit: cm)

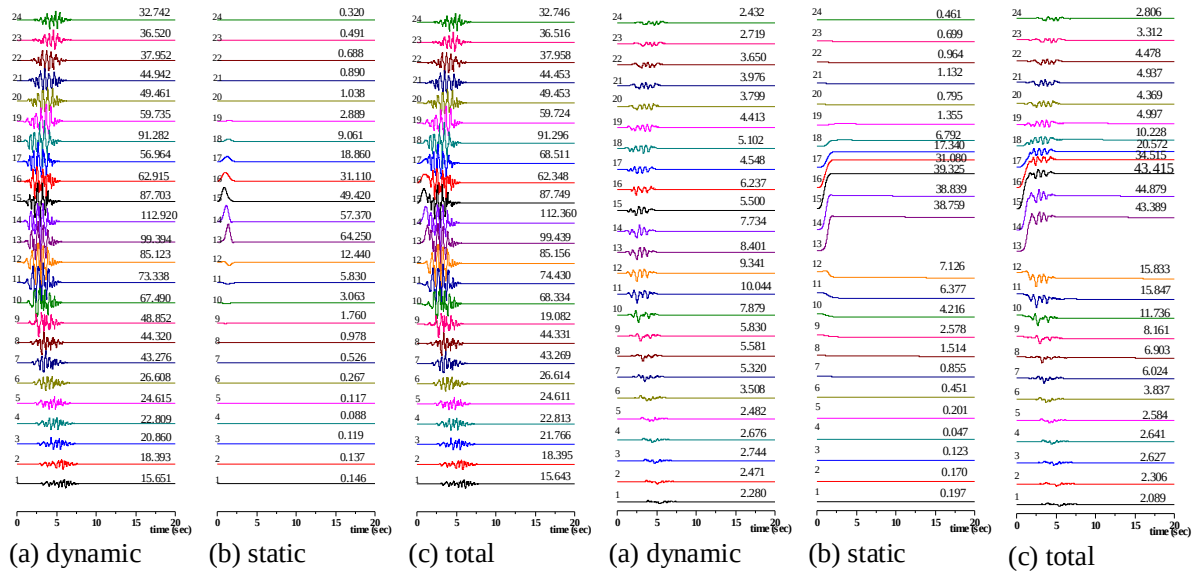


Figure 3-17 Velocity along Up-Down direction (unit: cm/s)

Figure 3-18 Displacement along Up-Down direction (unit: cm)

3.6.4 Simulation on the 1999 Chi-Chi earthquake

1) Fault model

This proposed hybrid method was also adopted to simulate the time histories of near-fault ground motions: the 1999 Taiwan Chi-Chi earthquake, which considered the permanent displacement.

The 1999 Chi-Chi Earthquake (Mw7.6) (20 September 1999, 14:47:15.9 UTC), with hypocenter located at 23.853°N, 120.816°E, and the depth of 7.5 km) inflicted severe regional scale damage to Taiwan. The surface fracture trace runs along the Chelunpu fault, with strike angle of N5°E, dip angle of 30°. According to Wu's fault model A [44] and the same as the simulated model as Hisada [45], its length and width are 82 km and 42 km respectively as shown in Figure 3-19. The characteristic fault model with two asperities is plotted in Figure 3-20. The fault plane was divided into 21×11 sub-fault, for asperity 1 with seismic moment $M_{o_{a1}}=8.2315 \times 10^{25}$, $M_{o_{a2}}=6.3874 \times 10^{25}$ for asperity 2, and $M_{ob}=5.2829 \times 10^{22}$ for background of each sub-fault. In addition, the 16-layered soil conditions are listed in Table 4 as below.

We used a rupture velocity of 3.0 km/sec, rake angle is 60°, and two observation sites, TCU052 (1.7 km away from the surface fault traction) and TCU072 (14.2 km away from the surface fault traction), TCU089 (16.4 km away from the surface fault traction), TCU078 (16.2 km away from the surface fault traction), located on the hanging wall is calculated. Its location and the station configuration are showing in Fig. 15. For the slip velocity, 5 time windows is assumed based on Nakamura and Miyatake [46]. It should be noted that the band-pass filter (0.0 ~ 0.01Hz, and 10.0 ~ 15.0 Hz) and base-line correction are conducted in the results processing.

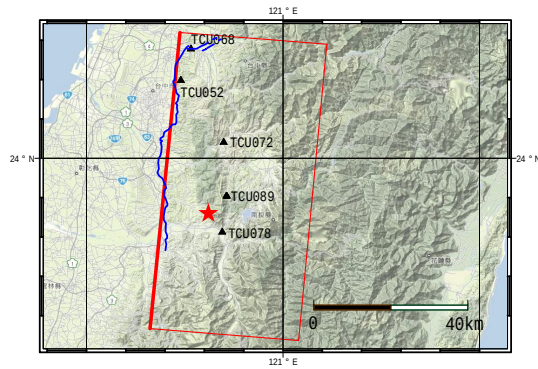


Figure 3-19 Chi-Chi earthquake with observation sites

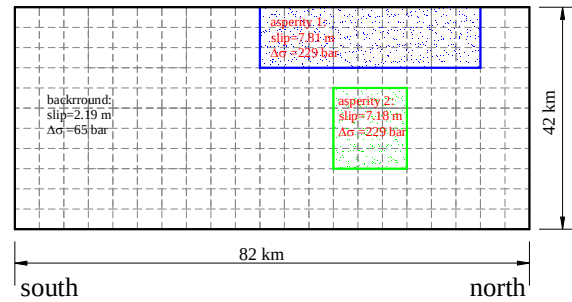


Figure 3-20 Characteristic fault model of Chi-Chi earthquake

Table 3-3 Soil layered condition of hanging wall in the 1999 Chi –Chi earthquake

No. of layer	Density (t/m ³)	V _p (m/sec)	V _s (m/sec)	Q _{po}	Q _{pf}	Q _{so}	Q _{sf}	thickness (km)	depth (km)
1	2.3	3610	2040	500	0	250	0	2	0
2	2.4	4660	2730	500	0	250	0	2	2
3	2.5	5450	3160	500	0	250	0	5	4
4	2.6	5760	3390	600	0	300	0	4	9
5	2.7	6150	3580	600	0	300	0	4	13
6	2.8	6260	3590	800	0	400	0	8	17
7	2.9	6710	3890	1000	0	500	0	5	25
8	3.15	7110	4110	1000	0	500	0	5	30
9	3.15	7500	4320	1000	0	500	0	15	35
10	3.2	8010	4670	1000	0	500	0	20	50
11	3.25	8270	4770	1000	0	500	0	40	70
12	3.25	8470	4970	1000	0	500	0	30	110
13	3.25	8310	4840	1000	0	500	0	30	140
14	3.3	8390	4850	1000	0	500	0	30	170
15	3.3	8510	4920	1000	0	500	0	40	200
16	3.3	8700	5090	1000	0	500	0	0	240

2) Simulation results

Due to limited space, we just show results at two stations, TCU052 and TCU072, as showing in Figures 3-21 ~ 3-22, including the velocity and displacement time histories and comparisons with the observed recordings.

For the velocity time histories in Figure 3-21, the simulated velocities along 3 directions are smaller than the observed waves as in (a) TCU052, and generally fit well with the observed velocity for TCU072. While as the near-fault ground motions are highly depend on the frequency by this proposed method above, it is clearly seen the high-frequency parts affected the simulated results.

As for the comparisons between the observed and simulated displacement time histories as plotted in Figure 3-22, the vibrations of the simulated results are not so obvious in the waveforms, which meant the dynamic parts are smaller than the static terms. The simulated displacement along NS-direction for both TCU052 and TCU072 are with large differences compared with the observed recordings. And the displacement along UD-direction of TCU052 is a little smaller than the observed one. And the other simulated displacement are fitted a good agreement with observed ones, which showed the proposed hybrid method could well simulate the near-fault strong ground motions.

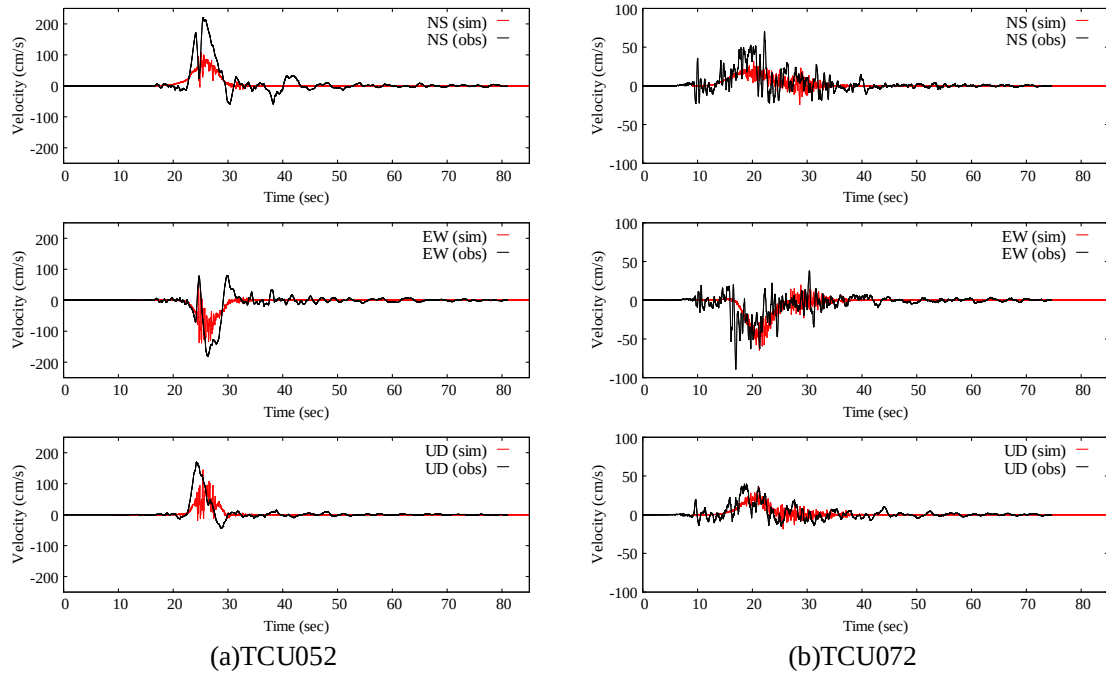


Figure 3-21 Velocity time histories for the 1999 Chi - Chi earthquake (TCU052 and TCU072)

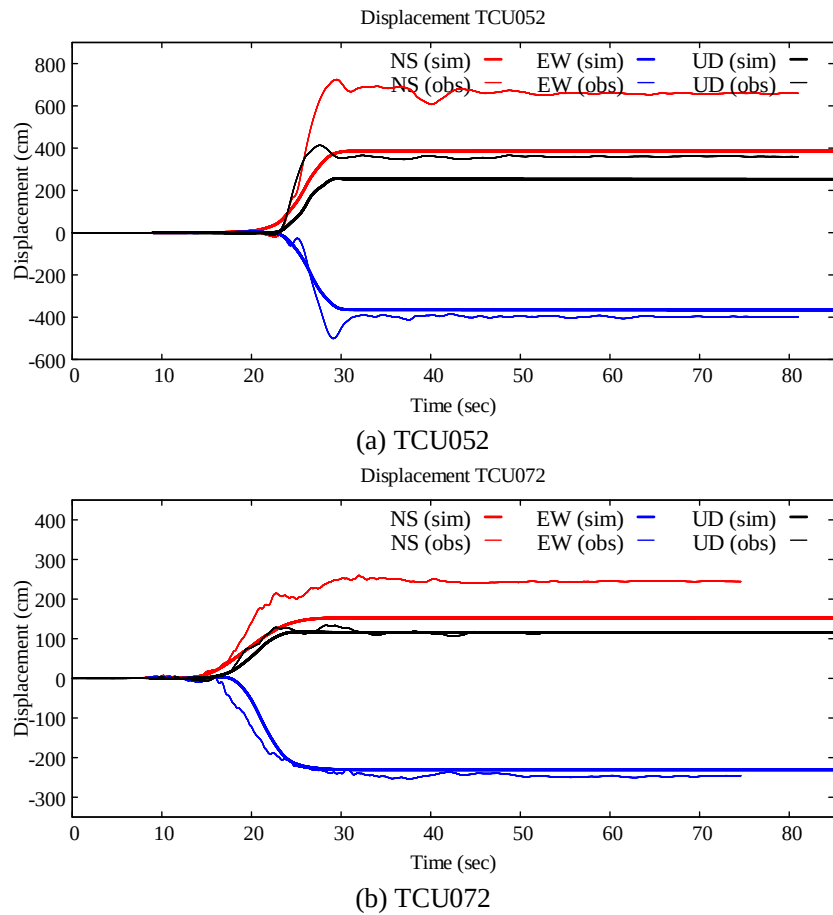


Figure 3-22 Displacement time histories of the 1999 Chi - Chi earthquake

3.7 Conclusions

The Chapter 3 are included the hybrid synthesis method for near-fault ground motions. The hybrid synthesis method is a combination between a modified stochastic Green's function and theoretical Green's function, which could obtain the complete waveform of near-fault ground motions. The detailed calculation procedures of this hybrid synthesis method are presented in this chapter. The following conclusion could be gained in this chapter:

(1) The stochastic Green's function was modified to consider the complete waveform of near-fault ground motions. First of all, it introduced the far-field P-wave by the similar calculation on S-waves. Then, two ratios (R_s and R_p) for intermediate-field and near-field components are introduced, the R_s is related to the components along θ, ϕ directions and the R_p is related to the components along r -direction (θ, ϕ and r are spherical polar coordinates). Theoretically, these two ratios are related to the ratio of $(R\omega)/V_s$. Simple strike-slip fault model and dip-slip fault model resulted that, they (R_s and R_p) are independent of focal mechanism, and R_p is larger than R_s when $(R\omega)/V_s$ is smaller than 10, which is satisfied with the theoretical conclusions.

(2) Additionally, this hybrid synthesis method was adapted to two simple fault models for further, the strike-slip fault model resulted that, located along the fault normal direction, the static term-induced displacements were dominant in the time history along the near fault area, while the dynamic term-induced displacements were dominant along the further distance from the fault line; the dip-slip fault model indicated that, the fling-step was much obvious and dominant in the hang-wall, and the same tendency occurred in strike-slip fault along the distance from the fault trace on the surface.

(3) The simulation on the 1999 Chi-Chi earthquake was conducted. The comparisons between the observed and simulated displacement time histories illustrated that, the vibrations of the simulated results are not so obvious in the waveforms, which meant the dynamic parts are smaller than the static terms. The simulated displacement along NS-direction for both TCU052 and TCU072 are differences with the observed recordings, the displacement along UD-direction of TCU052 is a little smaller than the observed one, and simulated displacement components are fitted a good agreement with observed ones, which showed the proposed hybrid method could well simulate the near-fault strong ground motions.

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Chapter 4 Evaluation on the hybrid synthesis method

4.1 Introduction

The evaluation on ground motions near-fault area is very important, it is very different from the far-fault ground motions, and exhibit rather complicated characteristics. The evaluation on the near-fault ground motions generated by new hybrid synthesis method is very important. This chapter aims at making clear the characteristics and the effect of soil conditions by this hybrid synthesis method.

4.2 Spatial variation characters by hybrid synthesis method

4.2.1 Reverse fault model

Here, we set the fault model and parameters as in section 3.6.2, but we change the coordinate from one end of the fault to the middle part of the fault as shown in Figure 4-1 (a). As the definition of near fault area is the distance less than 20 km, thus here, we set the receiver points along the fault plane with 20 km and the station configuration are shown in Figure 4-1 (b).

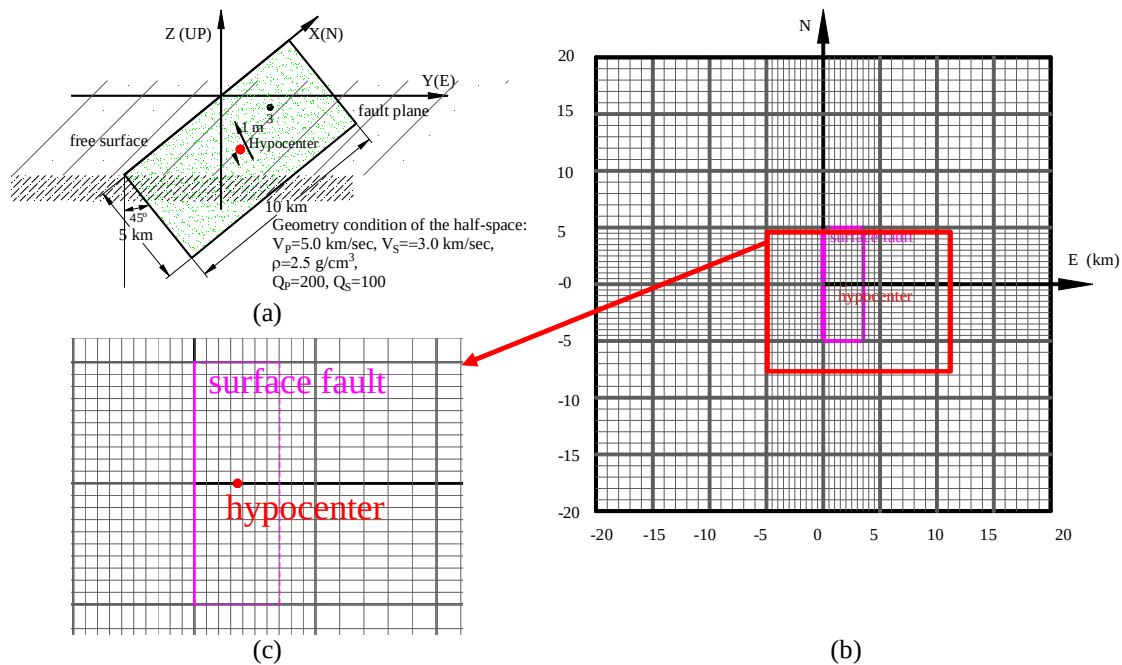


Figure 4-1 3D: fault and receiver locations: every crossing point, in total (2500 receiver points)

4.2.2 Spatial variation of near-fault ground motions by hybrid synthesis method

In order to check the spatial variation characteristics of near-fault ground motions, we plot the time series of acceleration, velocity and displacement of all these 2500 observation points in the following Figures 4-2 ~ 4-4.

In Figure 4-2, the acceleration time histories near-fault ground motion, from time 0.1 s ~ 0.5 s, the acceleration of locations located very near the fault are with very large value, From 3.0 s ~ 10.0 s, the radiation pattern and wave propagation are obvious. And the acceleration exhibited the random features.

From Figures 4-3 ~ 4-3, it can be easily seen that, the velocity pulse, and the fling-step displacement is occurred during 1.0 s ~ 5.0 s. after 5.0 s, the fling-step keep unchange, which means in this potential fault, the fault rupture are completed in the first 5.0 s, and at $t=0.1$ s, 0.5 s, 1.0s, and 2.0, 3.0 s, there are several locations with pulse-like velocities.

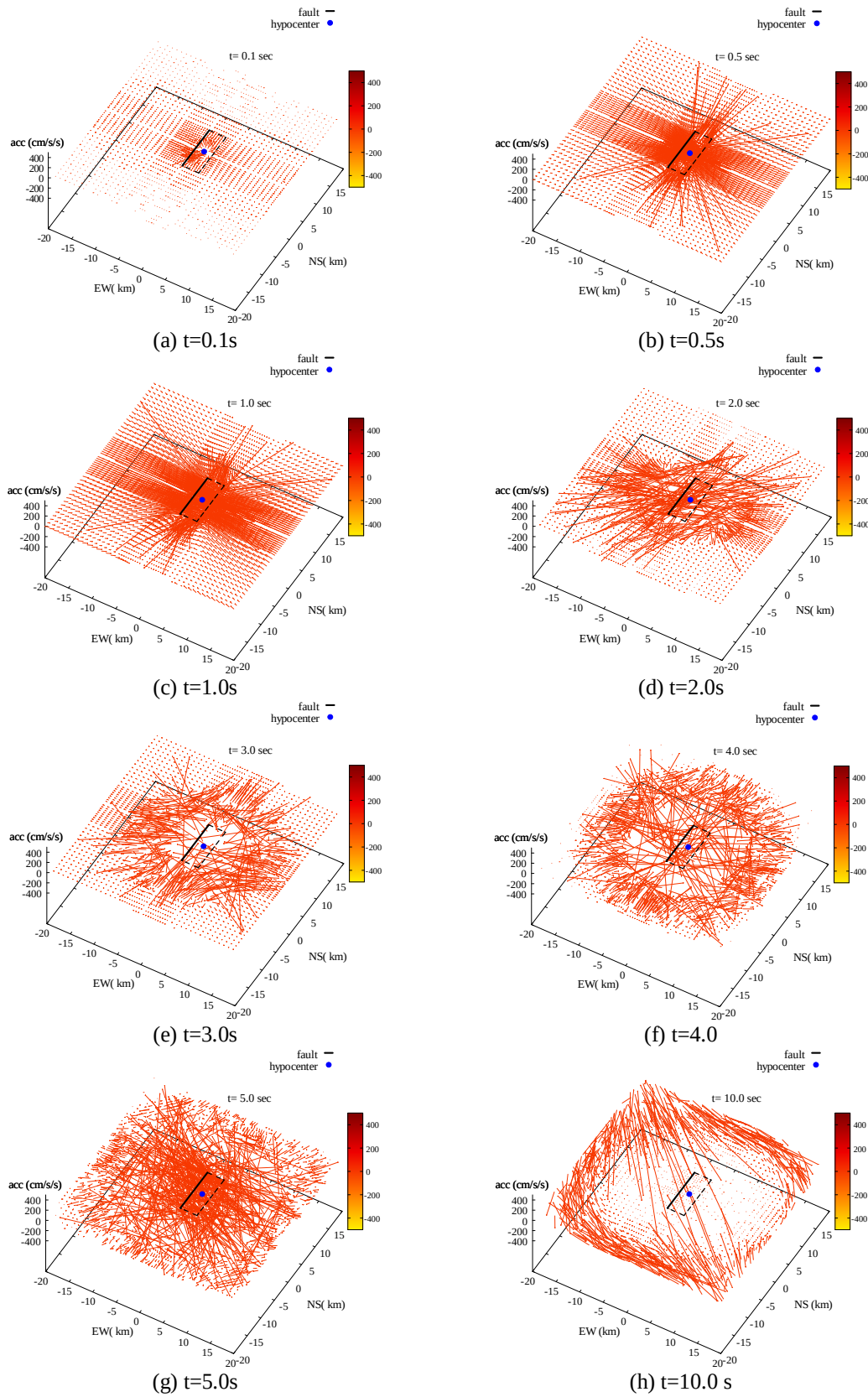


Figure 4-2 Time series of acceleration at near-fault area

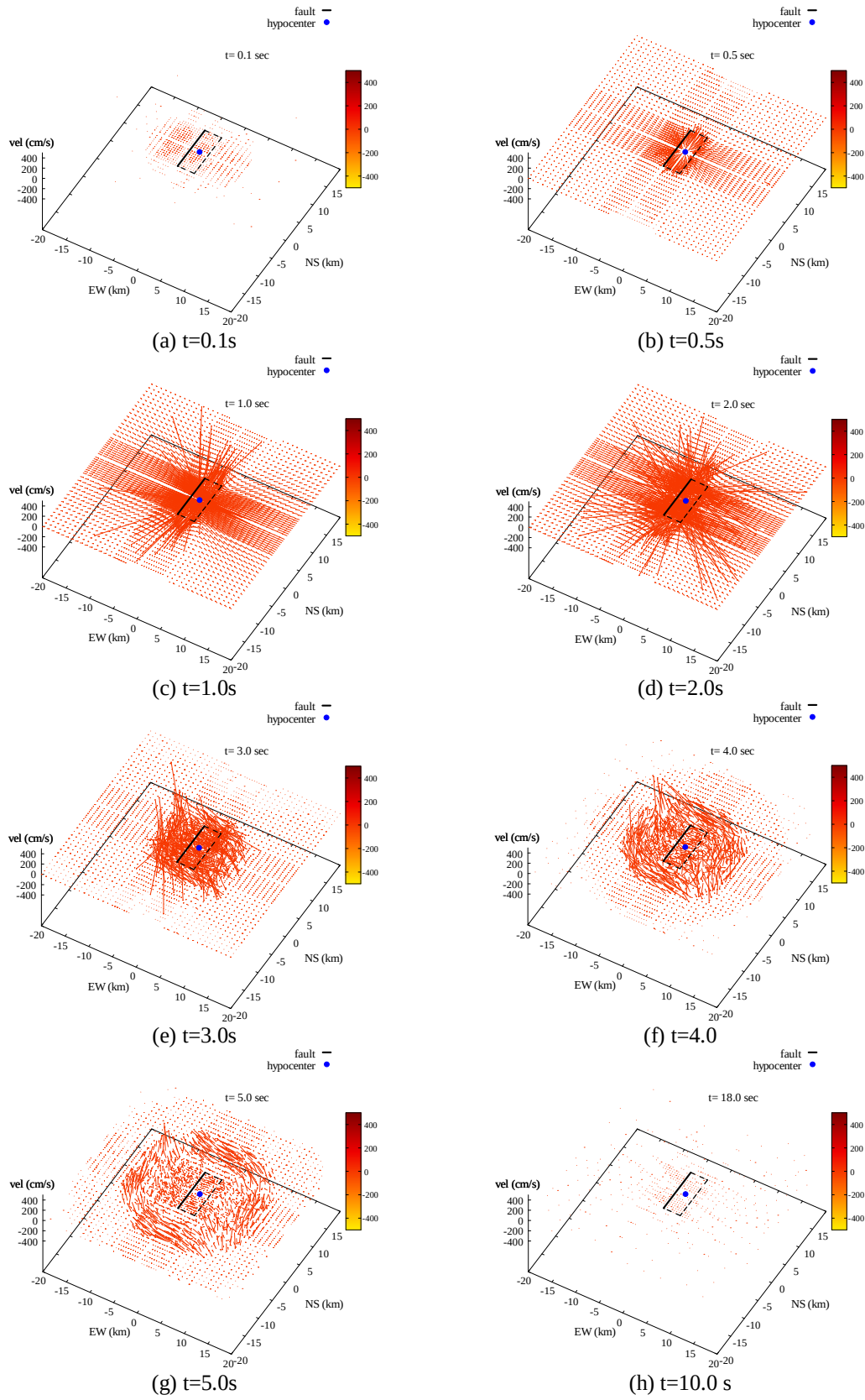


Figure 4-3 Time series of velocity at near-fault area

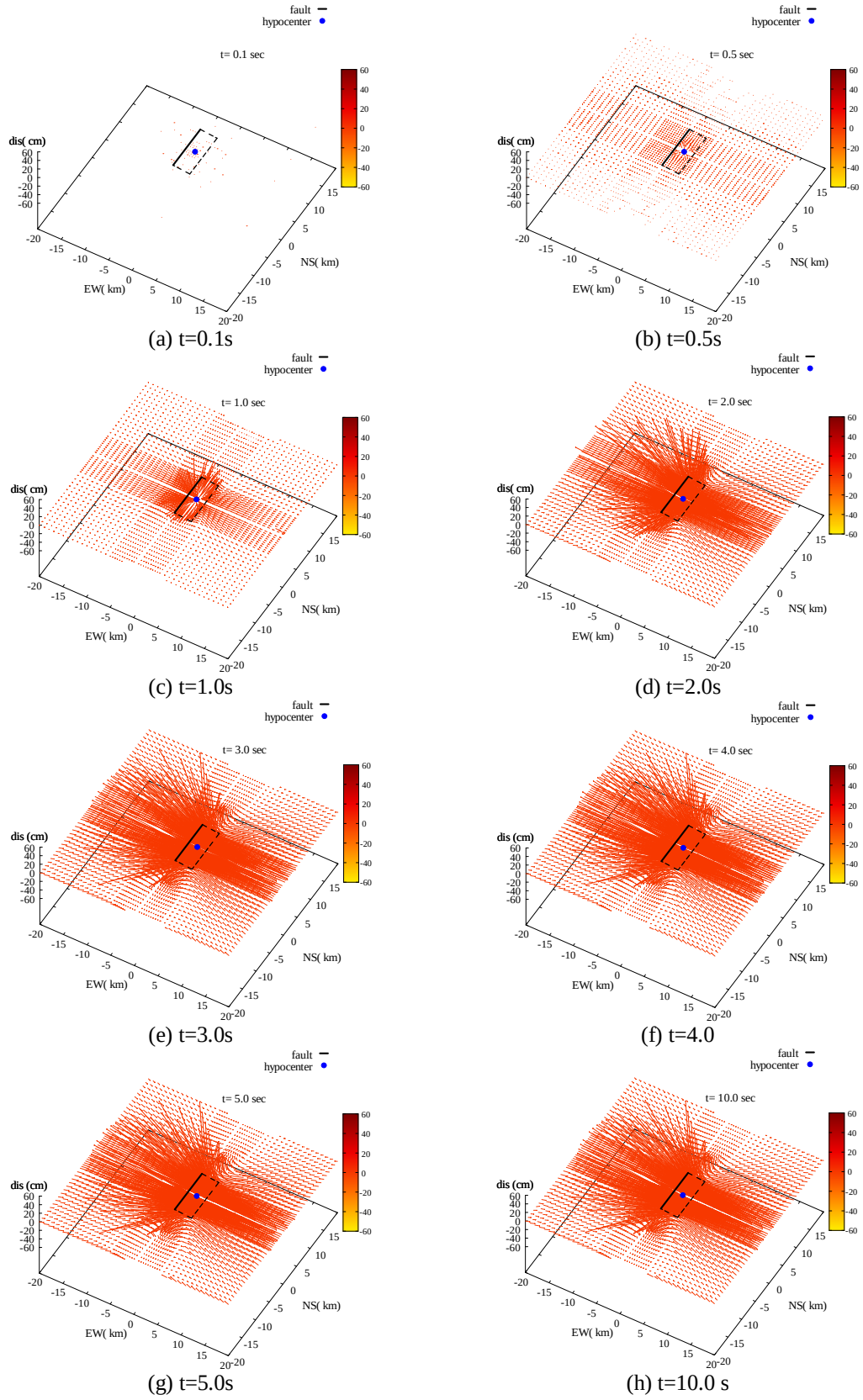


Figure 4-4 Time series of displacement at near-fault area

4.3 Effect of buried depth

4.3.1 Strike-slip fault model and cases

In order to check the influence factor of buried depth, take the dip-slip fault model in this research, we set five cases buried with 0.0 km, 0.1 km, 0.5 km, 1.0 km and 5.0 km listed in Table 4-1, with the fault model shown in Figure 4-5. We focused on two points: the first one is point A (0.1 km, 0) on the footwall, located very near the fault; and the second is AA (20.0 km, 0) on the hanging wall. Some other parameters of the fault plane are set the same as in section 3.6.2 above.

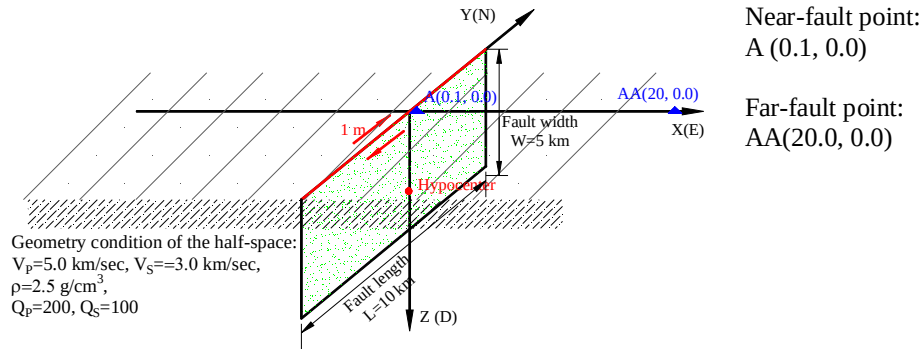


Figure 4-5 Receiver configuration for buried depth checking

Table 4-1 Calculation cases with various buried depth

Case No.	Case 1	Case 2	Case 3	Case 4	Case 5
Buried depth: D [km]	0.0	0.1	0.5	0.7	1.0

4.3.2 Results

Due to we are focusing on the permanent displacement, here we just check the displacement time histories of the 5 cases. The results are shown in the Figure 4-6 (a) for near-fault A and (b) for far-fault AA. It can be seen obviously, the large differences between the near-fault and far-fault displacement time histories. For the far-fault point AA, due to the attenuation with large distance, the dynamic responses are with time lag compared with the near-fault point A. due to this is a pure strike-slip fault, the permanent displacement along strike direction (NS) are obvious, while the dynamic response of ground along EW- and UD- directions are predominant. For the near-fault displacement, from surface fault (depth = 0.0 km) to 0.1 km, the fling-step displacement decreases from 32 cm to 15 cm, and from depth = 0.1 km to depth = 0.5 m, the fling-step displacement decrease from 15 cm to 3 cm; when the depth is larger than 0.5 km, the permanent displacement are less than 3 cm. For the far-fault point, the permanent displacements under all the cases are less than 1 cm along NS-direction.

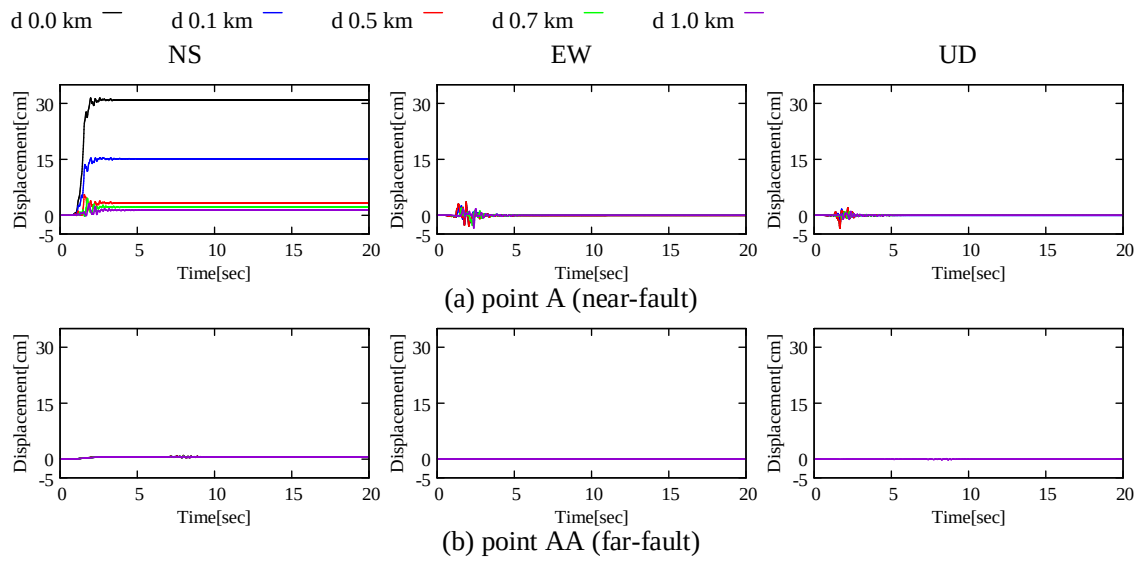


Figure 4-6 Displacement time-histories of near-fault and far-fault with different buried depth

4.4 Effect of the soil model

4.4.1 Fault model and cases

Due to the soil model is one of the most important influence factor for the ground motions simulation, in this section, we checked the soil model as the following 6 cases under two soil models (model 1 and model 2) for a dip-slip model, analysis models to the first three cases are shown in Figure 4-7, and the fault parameters are listed in Table 3-1 in Section 3.6.3.

We choose two different kinds of soil model, including hard rock (based on [1]) and soft rock (based on [2]) models, both of these two models contains 6-layers and the cases are varied with different layers which are listed in Tables 4-2 ~ 4-3.

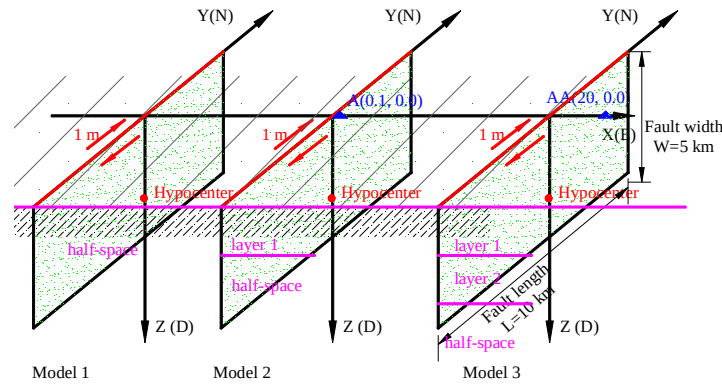


Figure 4-7 Receiver configurations for buried depth checking

Table 4-2 Hard rock for soil model (model 1)

Layer Number	Density (t/m ³)	Vp (m/s)	Qp0	Vs (m/s)	Qs0	Thickness (m)	Depth (m)
1	2.00	1560	200	900	100	290	0
2	2.05	2290	200	1320	100	420	290
3	2.30	2740	300	1580	150	1380	710
4	2.50	3780	500	2200	250	1910	2090
5	2.60	5040	500	3030	250	5000	4000
6	2.70	5710	500	3260	250	0	9000

Table 4-3 Soft rock model (model 2)

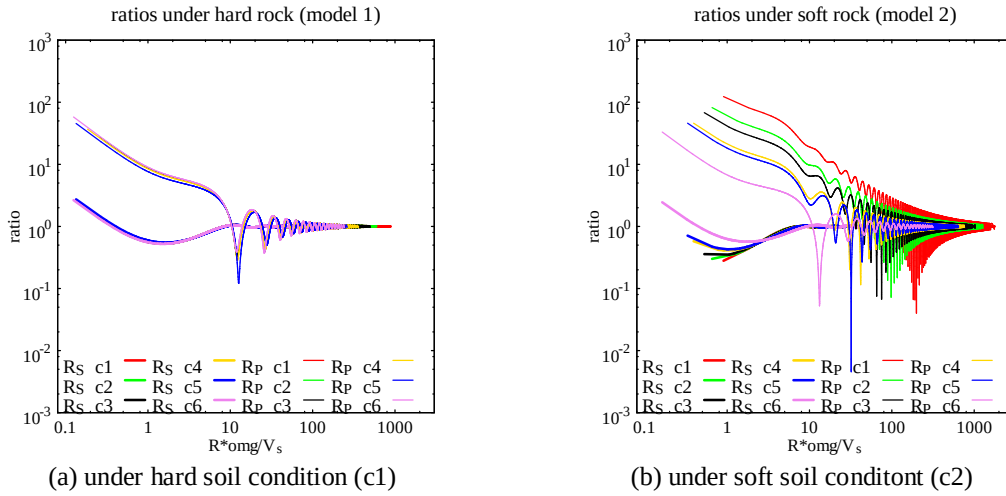
Layer Number	Density (t/m ³)	Vp (m/s)	Qp0	Vs (m/s)	Qs0	Thickness (m)	Depth (m)
1	1.70	1800	200	450	100	49	0
2	1.80	2000	200	620	100	130	49
3	1.80	2200	300	770	100	65	179
4	2.00	2400	500	1040	150	239	244
5	2.10	2700	500	1230	250	136	483
6	2.30	4000	500	2500	250	0	619

Table 4-4 Calculation cases with various layers in model 1 and model 2

Case No.	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Model 1: layers	6	5 ~ 6	4 ~ 6	3 ~ 6	2 ~ 6	1 ~ 6
Model 2: layers	6	5 ~ 6	4 ~ 6	3 ~ 6	2 ~ 6	1 ~ 6

4.4.2 The ratio for modify the dynamic terms under different soil conditions

Due to the ratio is not closely related to the focal mechanism as it was checked above, thus only the dip-slip fault model under different soil model was calculated. In this calculation, two kinds of soil conditions were considered, it is indicated that, the hard rock soil model did.t affect the ratios, small changes for both R_s and R_p as shown in Figure 4-8 (a), while under soft soil model schematic in Figure 4-8 (b), different tendencies for R_s and R_p , both the R_s and R_p are decreased along with increasing of soil model from soft to hard conditions, compared with the R_s , there is more significant changes for R_p and with larger oscillation even the increasing of $R\omega/V_s$, which suggested that more significant modification for P-waves should be modified for soft soil condition in the near-fault area.

Figure 4-8 Ratios (R_s and R_p) for modifying the dynamic terms under different soil conditions

4.4.3 Results

The results are shown in the Figure 4-9 for near fault A Figure 4-10 for far-fault AA, respectively.

From Figure 4-9 (a) for the near-fault point A, the NS-direction did vary less than 3 cm, while the dynamic response along all the directions under the three direction changed up to 10 cm especially along EW- direction under 3-layers model.

For the far-fault ground motion AA, it is with little change under both these 6 cases. This phenomena occurred mainly due to the bedrocks are very hard for all of the models, while for real situation, the structures, especially the long-span structures, might located in total different soil conditions, thus, for the affect of soil model, it needs further consideration.

For hard soil model, it caused longer shaking for surface sites along NS-, EW- and UD- directions, while large response along EW-direction under soft rock model, with the PGD up to 28 cm.

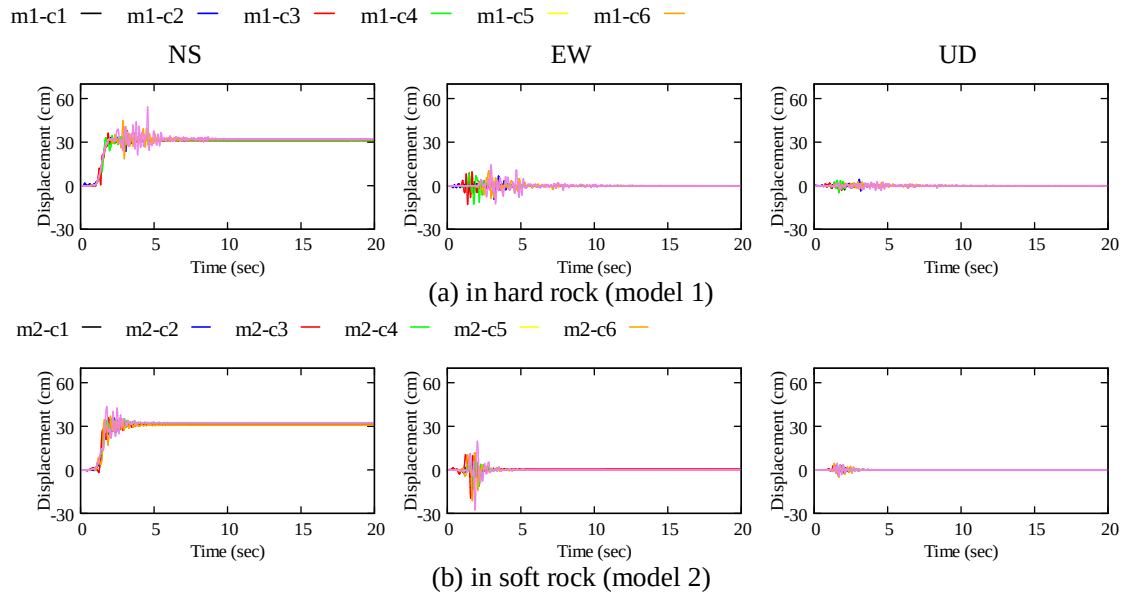


Figure 4-9 Effect of soil conditions on near-fault displacement (100m away form fault rupture trace)

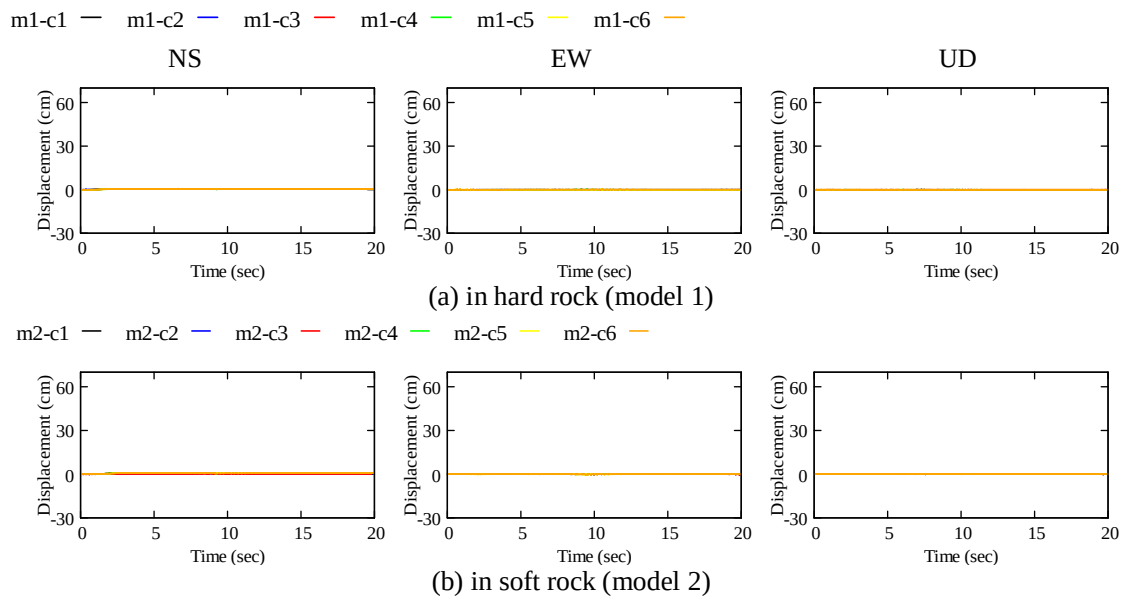


Figure 4-10 Effect of soil conditions on far-fault displacement (20km away form fault rupture trace)

4.5 Effect of M_w and distance

The PGD along different distance to the surface fault trace line under different M_w was also checked in this analysis. Base on potential fault in Japan (J-SHIS [3]), 8 potential faults with M_w from 6.3~7.0 were chosen to explore the PGD and final fling displacement under various M_w . The cases are shown in Table 4-5 below.

Table 4-5 Focal mechanisms of earthquake under different M_w

M_w	6.3	6.4	6.5	6.6	6.7	6.8	6.9	7.0
Strike angle ($^{\circ}$)	0	0	0	0	0	0	0	0
Dip angle ($^{\circ}$)	60	90	90	62.5	75	75	90	90
Rake angle ($^{\circ}$)	-90	-160	-160	0	0	0	0	0

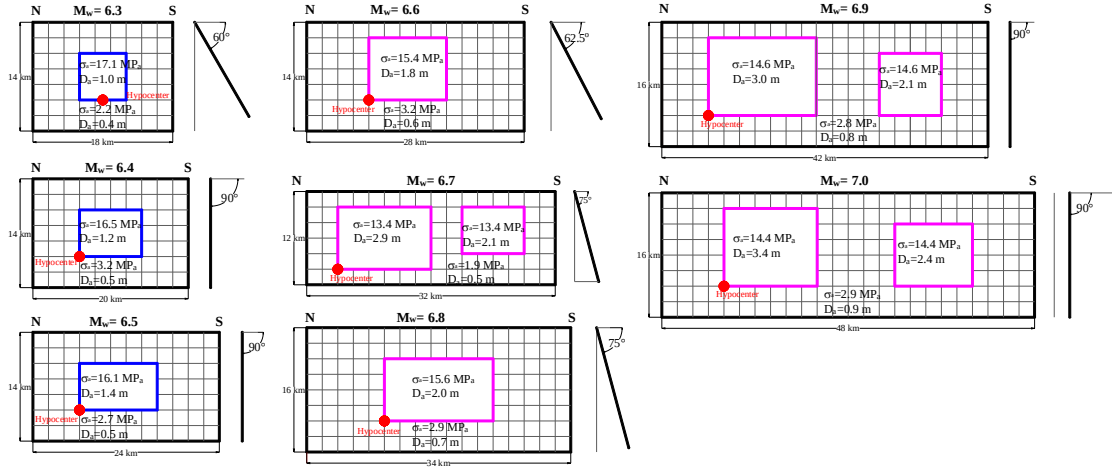


Figure 4-11 Fault plane model for different M_w

4.5.1 PGD along different distance under various M_w

The M_w – distance - PGD is also checked in this analysis, and the results are shown in Figure 4-12 along NS-, EW-, UD-directions, under different active faults with different rake angles (which means different focal mechanism) significantly affect the PGD. It should be noted that, we make assume all the strike angle is 0° for simplicity, thus along NS-direction is Fault parallel direction, and fault normal direction along EW-direction. And the location of hypocenter significant affects the results.

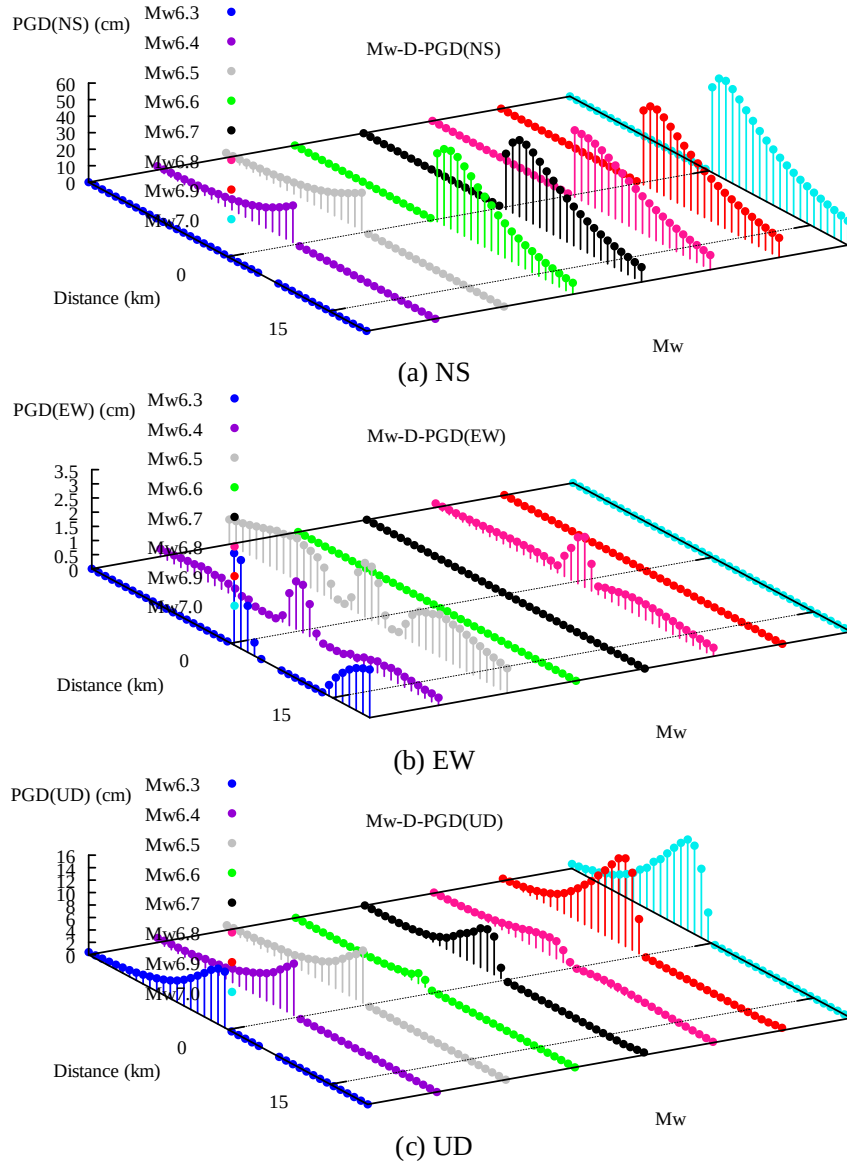


Figure 4-12 PGD along different distance and M_w
(distance: -20 km ~ 20km, M_w : 6.3 ~ 7.0)

4.5.2 Final displacement along different distance under various M_w

The final fling displacements along different distance to the surface fault trace was also checked, and the results are listed in Figure 4-13 below along NS-, EW- and UD-directions. $M_w = 6.3$ is a normal-slip fault with dip angle of 60° , thus, compared with oblique fault of $M_w = 6.4$ and $M_w = 6.5$, the NS-components are smaller, while EW- and UD- components are larger, which means the dip angle and focal mechanism significantly affect the final displacement.

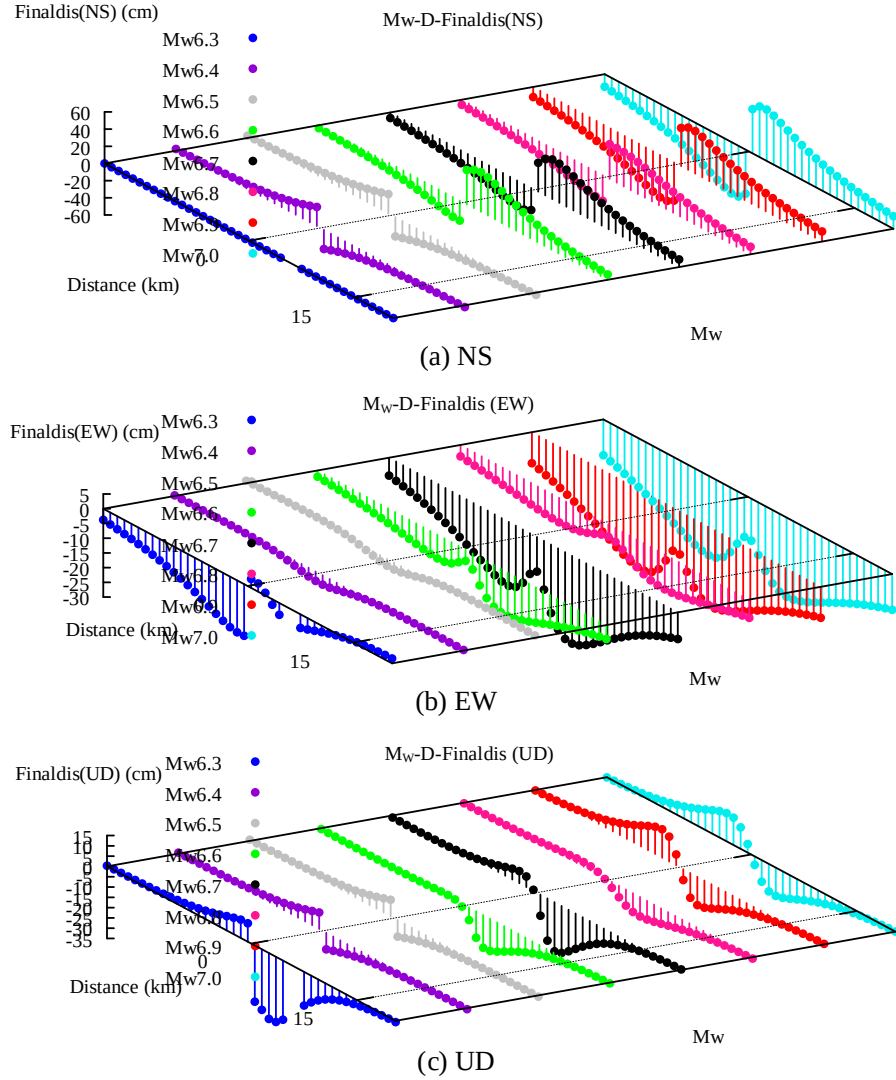


Figure 4-13 Final-displacement along different distance and M_w (distance: -20 km ~ 20 km, M_w : 6.3 ~ 7.0)

4.6 Conclusions

In Chapter 4, the evaluation on the near-fault grounds generated by the proposed hybrid synthesis method in Chapter 3, and the following achievements could be gained:

(1) The evaluations on the 3D spatial variation characteristics from 2500 points in near-fault area, mainly the the acceleration-, velocity- and displacement-field, plotted that, 3D vectors of acceleration field indicated that the near-fault accelerations are rather complicated. The velocity field showed the pulse-like during 0.5 s ~ 3.0 s, and the rupture propogation from the central area of the dislocation destruction is easy to seen; for the 3D vector of displacements, the hanging wall effect can be seen starting from 0.1s; from 0.1 s ~ 3.0 s, the vectors, including the direction and the values, changed a lot, and after 5.0 s, the permanent displacement remains the same, which means the rupture

of the surface are with 5.0 s. which well shown htat the complicated features is the phenomenon resulting form combination factors, while mainly due to the complexity dipole focal mechanism, the propagation characteristic of S-waves, and the directivity effects. dip-slip fault model, with 2500 observation sites located in the near-fault area (the source-distance is smaller than 20 km) is calculated.

(2) The effects of buried depth indicated that, the permanent displacements are not obvious (less than 3.0 cm) when the buried depth is more than 700m, which demonstrated that, the assumption on surface fault is resonable for numerical calculation under the consideration on fling-step displacements.

(3) The effects of two soil models (hard rock model and soft rock model) resulted that, in hard rock model, the near-fault ground motions shaken with longer duration and small amplitude, while with shorter duration and large amplitude in soft rock model, and attenuated rapidly due to energy are absorbed by softer soil with larger damping ratio, which is consistant with the theoretical analysis.

(4) Different M_w significantly affect the fling-step displacemetns, not only the size of M_w , but also the focal mechanism. In general, the larger the M_w , the larger the permanent displacement.

References

- [1] C. Wu, M. Takeo, and S. Ide, "Source process of the Chi-Chi earthquake: A joint inversion of strong motion data and global positioning system data with a multifault model," *Bull. Seismol. Soc. Am.*, vol. 91, no. 5, pp. 1128–1143, 2001.
- [2] A. Kitamura, K. Toki, T. Kuwahara, O. Kurimato and J. Ejiri "Soil strucutre explolation and earthquake prediction in Kyoto Prefecture," *Proc. JSCE Earthq. Eng. Symp.*, vol. 24, pp. 37–40, 1997. (in Japanese)
- [3] "J-SHIS Japan Seismic Hazard Information." [Online]. Available: <http://www.j-shis.bosai.go.jp/map/?lang=en>.

Chapter 5 Seismic response of near-fault bridges

5.1 Introduction

Near-fault ground motions are very different from far-field ground motions in that they often contain strong coherent dynamic long period pulses and permanent ground displacements, and cause serious damage or collapse of bridge due to both ground acceleration and fault ruptures (permanent displacement) as the situation shown in Figure 5-1, especially structures located near potentially active faults. Thus, seismic response to near-fault ground motion is important for structures and design. Also, the quantification of the response of a bridge to seismic disturbance in terms of the overall structural displacements, member forces, and local deformations is accomplished with the help of mathematical models and analysis techniques

Otsuka *et al.* [1][2][3][4] conducted many researches on the seismic response of near-fault bridges considering the fling-step displacement effects, and the multi-support excitation method was also adopted. However, the input ground motions mainly from the recordings of typical earthquake, i.e., Chi-Chi earthquake (TCU052, TCU078), the fling-step containing displacement time histories were obtained by baseline corrections [5] based on GPS data, which is not convent for design, nor not suitable for the specific bridge structure considering the site conditions.

Thus, in this chapter, based on the proposed hybrid synthesis method, it generated the fling-step displacement waveforms. And two other displacement waveforms, only containing the dynamic terms and the ground motions of Level 2 Type III ground (NS- and EW-components on the ground in Port Island, and it will use the JRA [6] for short in the following results analysis) were also conducted for comparisons. Then, by applying the simulated input displacement waveforms including the permanent displacement, seismic analysis (including linear and non-linear analysis) on a 5-span fault-crossing bridge was achieved; the comparisons between linear and nonlinear analysis results (mainly the seismic responses of girder, pier, bearing) could offer useful references for near-fault bridge designing.



(a) damage of girder: center line inclined



(b) relative displacement between girder and pier

Figure 5-1 Damage of near-fault Tawarayama Bridge during the 2016 Kumamoto Earthquake [7]

5.2 Procedure for near-fault bridge analysis

The present seismic design methods, which was mentioned in Chapter 1, mainly including the substitute structures approach, the pulse modeling approach, and displacement amplification approach, neither these method cannot always adequately estimate inelastic displacement demands due to the near-fault shaking, nor do not correctly predict the response of bridge that is strong enough to remain elastic and others have inconsistencies meaning that cannot easily be modified to describe inelastic demands caused by near-fault ground motions as well as far-fault shaking, or it is not convenient for bridge designing.

Based on JRA [6] and Otsuka et al. [3], the seismic analysis on a near-fault bridge was conducted, while it should be mentioned that, the response quantities considered in this investigation are the deck displacements, column drifts and the bearing behaviors (mainly the time histories of deformation and horizontal shear force - deformation hysteresis curve under nonlinear analysis). And due to the analysis did not consider the axial force changing effects on the seismic behavior of piers, the max strain or the normalized strain (max strain to the yield strain) was proposed to evaluate the seismic performance of piers.

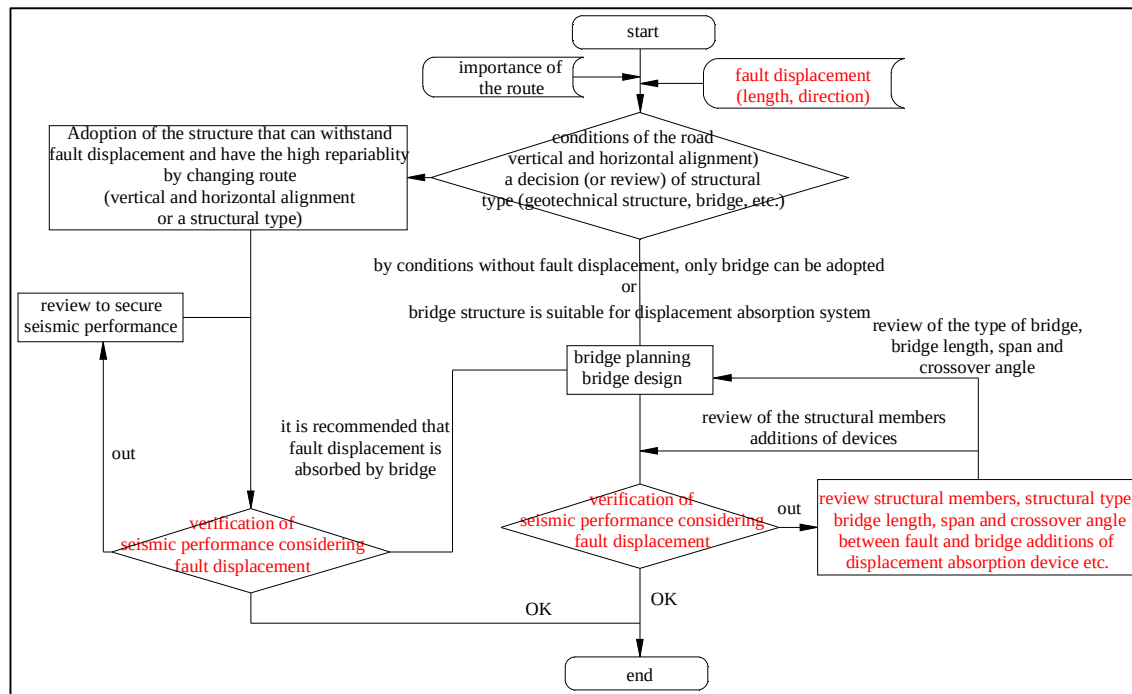


Figure 5-2 Procedure for near-fault bridges considering fling-step displacement [3]

5.3 Linear analysis on a simplified 5-span simply bridge model

5.3.1 Multiple excitation method for bridge engineering

The quantification of the response of a bridge to seismic disturbance in terms of the overall structural displacements, member forces, and local deformations is accomplished with the help of mathematical models and analysis techniques. In this paper, based on Chopra [8], the multiple excitation method is adopted. It should be noted that, the asynchronous input waveforms including the permanent displacements as shown in Figure 5-3, which are synthesized by the hybrid synthesis simulation method proposed in Chapter 3, are adopted as the earthquake excitations.

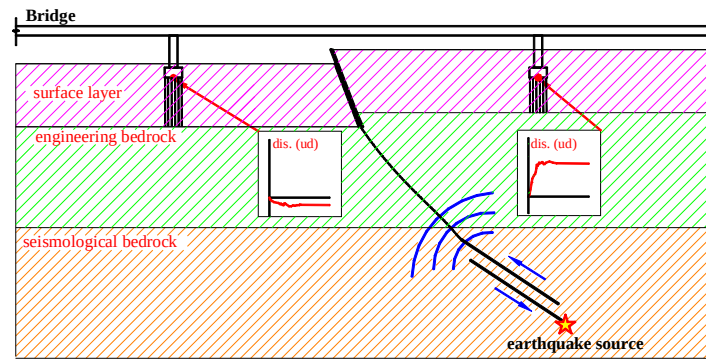


Figure 5-3 Multi-support excitation for fault-crossing bridge structures

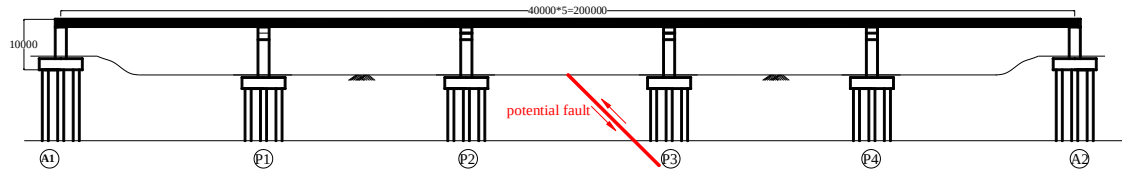
5.3.2 Seismic response of bridge with and without fling-step displacement

1) Bridge model

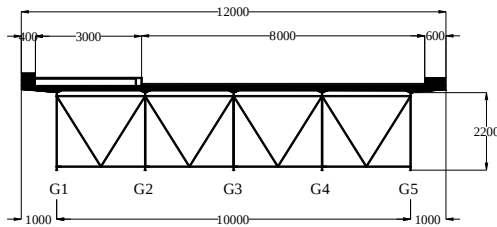
First of all, we examine the seismic linear behavior of the overall near-fault bridge, the lumped parameter model (LPM), in which bridge characteristics such as mass, stiffness, and damping are conveniently lumped or concentrated at discrete locations, is applied in this section. By inputting the waveforms of near-fault ground motion including the permanent displacement, seismic analysis on a 5-span fault-crossing bridge was calculated.

Base on [9], the bridge is 200m in length, comprising five 40m-long in spans. The middle span is assumed to cross a reverse surface fault. General view of this bridge is shown in Figure 5-3 (a), and the cross sections of girder, cross section and configuration of piers are shown in Figure 5-3 (b ~ d). The reverse surface fault and its parameters are presented in Figure 5-4 (e ~ g) and Table 5-1. The assumed bridge was analyzed through the structural analysis software Open System for Earthquake Engineering Simulation (OpenSees [10]). The girder and columns were modeled by linearly elastic beam elements and structural parameters of the bridge are listed in Table 5-2. A simplified model of a multi-degrees-of-freedom system is shown in Figure 5-5. All three translational movements are

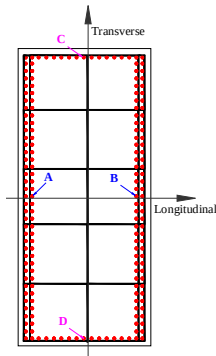
restrained at the model bases. And 3-dimensional directions of the bridge were considered. The bases of columns in the bents were fixed (restraint for all 6 degrees-of-freedom (DOFs)).



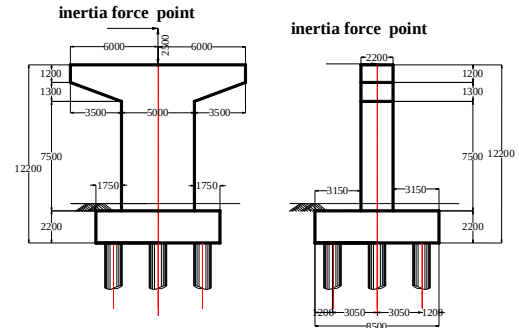
(a) bridge model (unit: mm)



(b) cross section of girder (unit: mm)

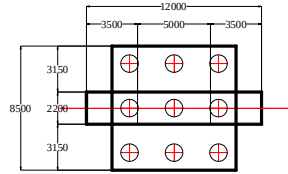


(c) configuration of piers (unit: mm)



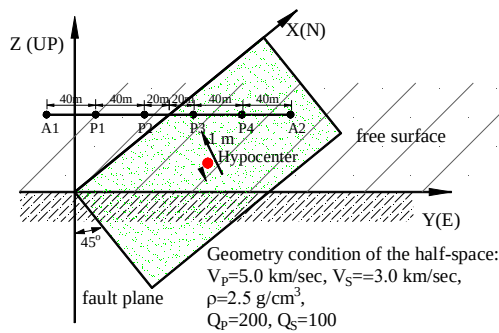
(a)

(b)

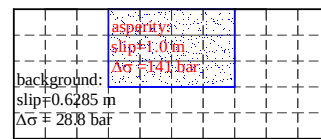


(c)

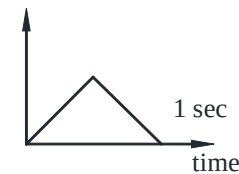
(d) cross-section of piers (unit: mm)



(e) the bridge-crossing fault plane



(f) Slip and stress drop



(g) slip-velocity

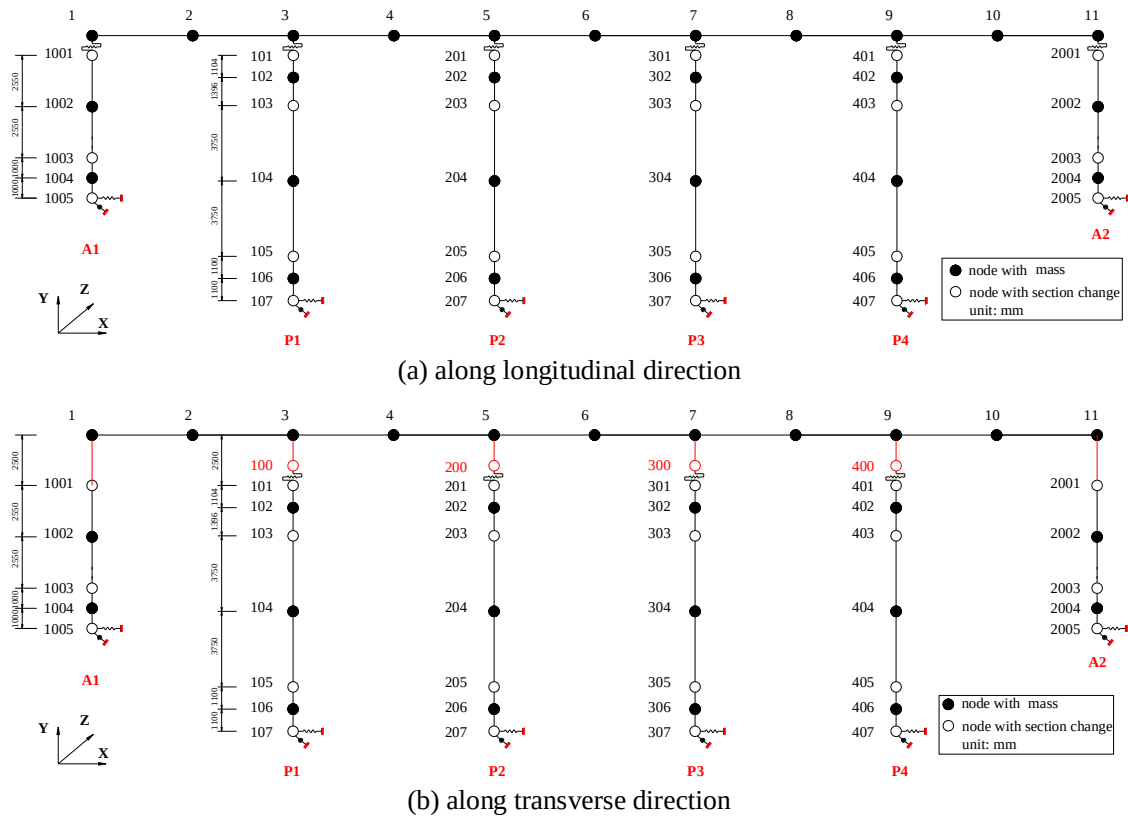
Figure 5-4 General view of the 5-span bridge crossing a dip-slip fault

Table 5-1 Fault parameters

Mo (dyne.cm)	Strike (°)	Dip (°)	Rake (°)	Length (km)	Width (km)	Hypocenter (km)			Rise time (s)
						x	y	z	
3.825×10^{25}	0	45	90	10.0	5.0	5.0	1.77	1.77	1.0

Table 5-2 Structural parameters of the bridge piers (along longitudinal direction)

	Items	unit	A1, A2	P1,P4	P2,P3
substructure	Equivalent stiffness	(tf/m)	4.25×10^5	4.25×10^5	4.25×10^5
	Equivalent damping ratio		5%	5%	5%
foundation	Stiffness	Horizontal	(tf/m)	1.798×10^5	1.425×10^5
		Rotational	(tf.m/rad)	1.760×10^5	1.760×10^5
	Damping ratio	Horizontal	(tf/m)	5%	5%
		Rotational	(tf.m/rad)	5%	5%
Seismic isolator system (lead rubber bearings)	Equivalent stiffness	(tf/m)	131	259	259
	Equivalent viscous damping ratio		0.25	0.25	0.25
	The design displacement	cm	26.8	23.1	23.1
	The effective displacement	cm	18.8	16.1	16.1



Note: both longitudinal- and transverse direction,
1) the nodes between 100-101, 200-201, 300-301, and 400-401 with the same coordinate;
2) the elastic support model: Y-axis-spring, X-axis-rotation fixed, Z-axis-rotation free.

Figure 5-5 Analytical bridge model (unit: mm)

2) Input ground motions

At present, seismic design of bridges takes into account only the shaking caused by ground acceleration. However, seismic design of bridges under earthquake excitation with fault displacement needs to consider the spatial variations and internal forces caused by the ground acceleration and the relative displacement between the bridge supports. Time histories which include fling displacements are required when conducting time-history analysis for such structures. Thus, the general seismic behaviors of bridge was checked both under the cases with and without the permanent displacements.

The synthesized near-fault ground motions for the two abutments (A1 and A2) and 4 piers (P1 ~ P4) along three directions of the bridge are shown in Figure 5-6, were generated using the proposed hybrid synthesis method in Chapter 3. It should be noted that, in order to save the calculation time, the simulated waveforms with the calculation duration of 10.24 s, while the ground motions from JRA [6] with calculation time of 20 s, and this hybrid method aims at engineering application; therefore, the data processing procedure, including a band-pass filter (0.01 ~ 20 Hz) and baseline correction. The considerations of asynchronous support motion are significant.

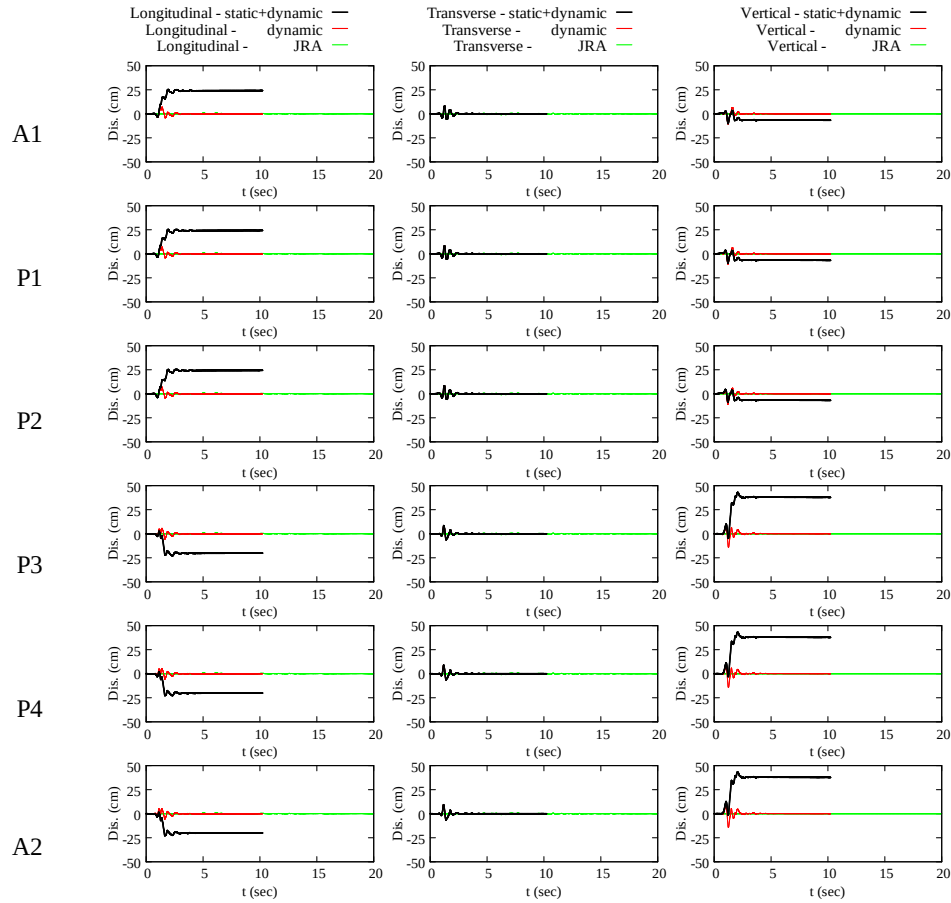


Figure 5-6 Input displacement time histories of GM with and without static displacement

Because the P2 and P3 are located close to the fault trace, it detailed the response spectra and Fourier spectra of P2 and P3 in this section. The Response spectra and Fourier amplitude of acceleration of P2 and P3 only are compared with the JRA are figured in the following Figure 5-7. It should be pointed out that, the vertical direction of input ground motion from JRA has not been considered in the numerical analysis, while the ground motions synthesis by the hybrid method considered the vertical components.

From Figure 5-7 (a), the response spectra showed that, when the period small than 0.6sec, along longitudinal and transverse directions, both simulated P2 and P3 are larger than the JRA ground motion, while when reverse results could be obtained when the period is larger than 0.6sec. The natural period of bridge is 1.05sec.

From Figure 5-7 (b), the same tendency could be obtained related to the response spectra of acceleration. In the high frequency parts when the frequency larger than 1Hz, the simulated results, both with and without considering the fling-step displacements, exhibited larger acceleration than the ground motion from JRA.

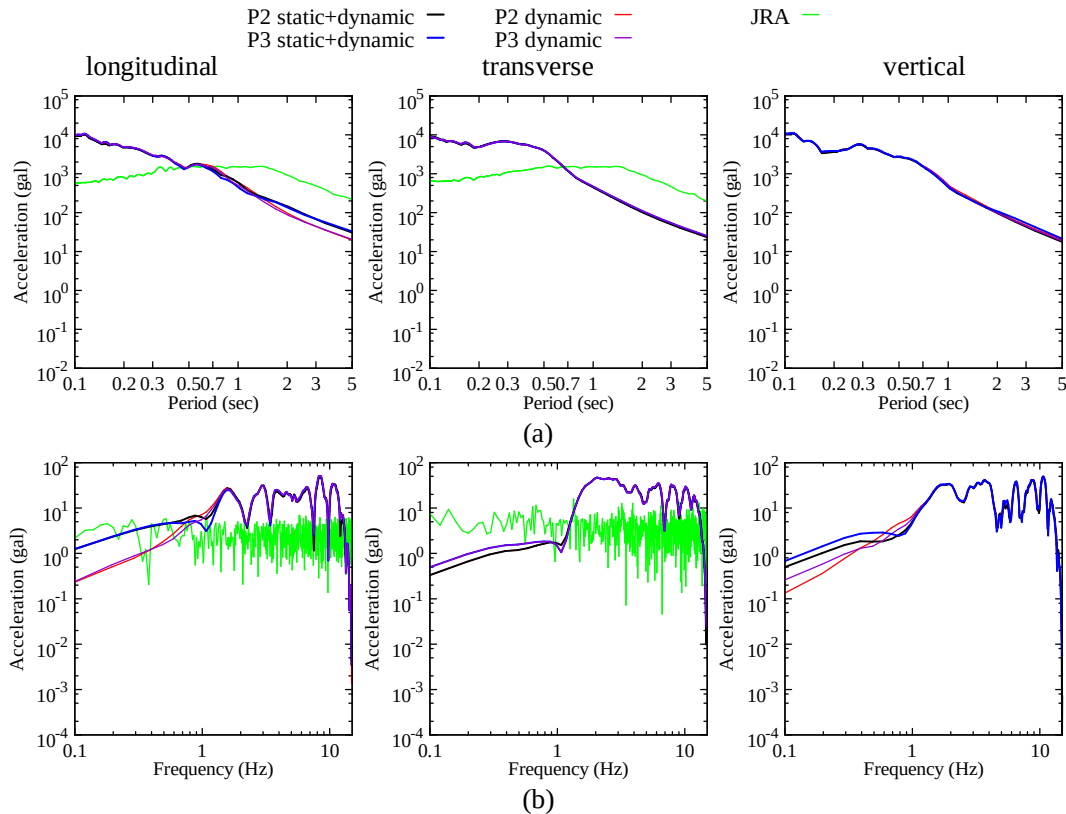


Figure 5-7 Response spectra (a) and Fourier amplitude (b) of acceleration of input ground motions

3) Results analysis

The response quantities considered in this investigation are the deck displacements, column drifts and the bearing behaviors. The numerical results are analyzed as the following.

(i) Deck displacements

The relative deck displacements are shown in Figures 5-8, which is used to estimate the relative displacement of the deck to the ground. Along longitudinal direction, due to large input ground motions, the relative displacement of deck are significantly large, up to around 20cm, which means the large deformation for bearing. Due to the ground motions are not large along transvers directions, the seismic response of the deck are also relative small; and mainly because of the enough stiffness of superstructures along the vertical direction, its relative deck displacements are also not obvious. With and without fling-step displacement, the remarkable difference between the deck displacements is the larger residual displacement of former than the latter, which might cause continuous damage following the shaking.

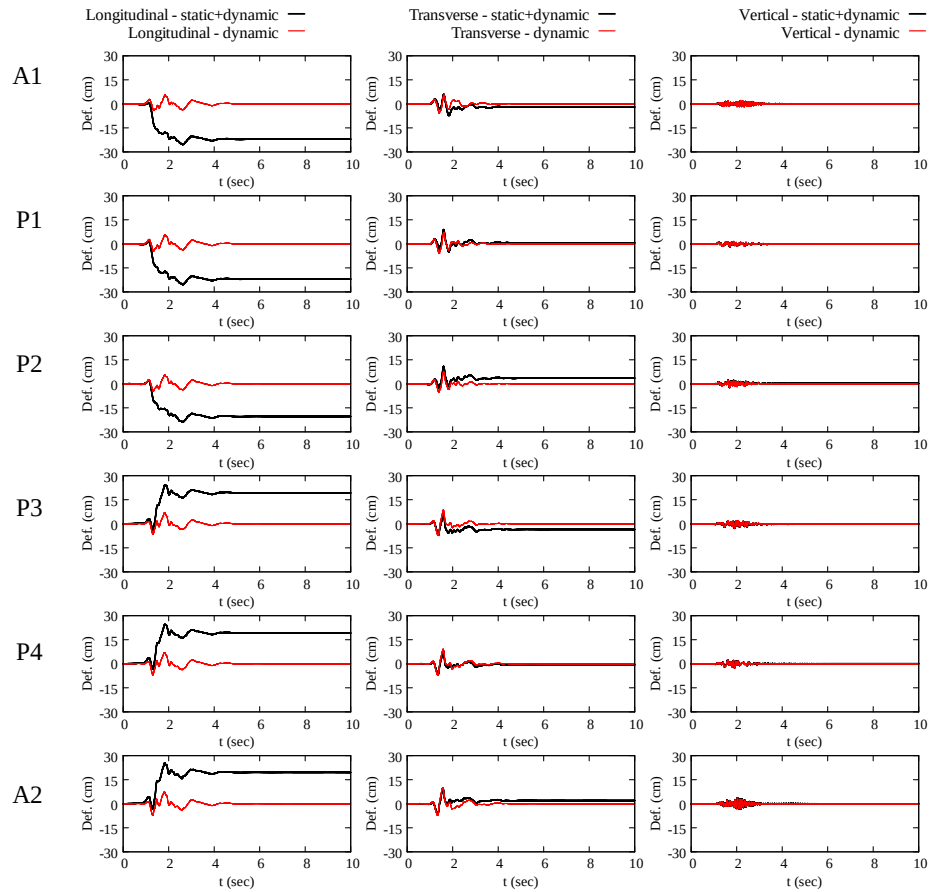


Figure 5-8 Deck displacement with and without static displacements

(ii) Column drifts

The column drift, which indicates deformation demand in the column, is defined as the displacement at top of the column relative to its base displacement. The time histories of column drifts are shown in Figure 5-9. Due to the enough stiffness along vertical directions, the responses of vertical column drifts are relatively small as compared with two other directions. For the sake of considering the permanent displacement of the near-fault ground motions, there are large differences between the abutments and the piers. Furthermore, since the complexities of near-fault ground motions, the deformation between P1 ~ P2 and P3 ~ P4 are different in residual deformations, which might cause the damage of substructures.

Because small deformations along the vertical direction, we just focus on orbit of column drift in the horizontal plan (xz-plane in this case) as shown in Figure 5-9. The hanging-wall effect is very obvious. Also the traces between these columns are different, P2 and P3 (very near the fault), the trace along longitudinal direction are predominant, while for P1 and P4, the trace almost uniform along both longitudinal- and transverse- direction.

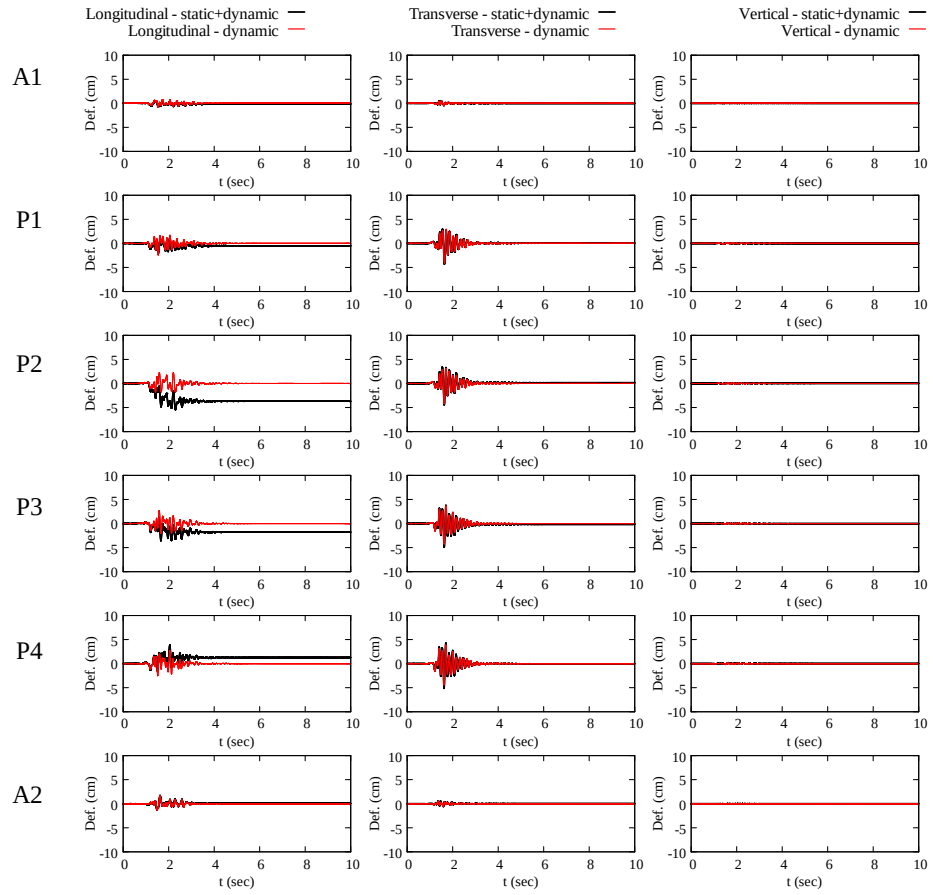


Figure 5-9 Column drifts with and without static displacements

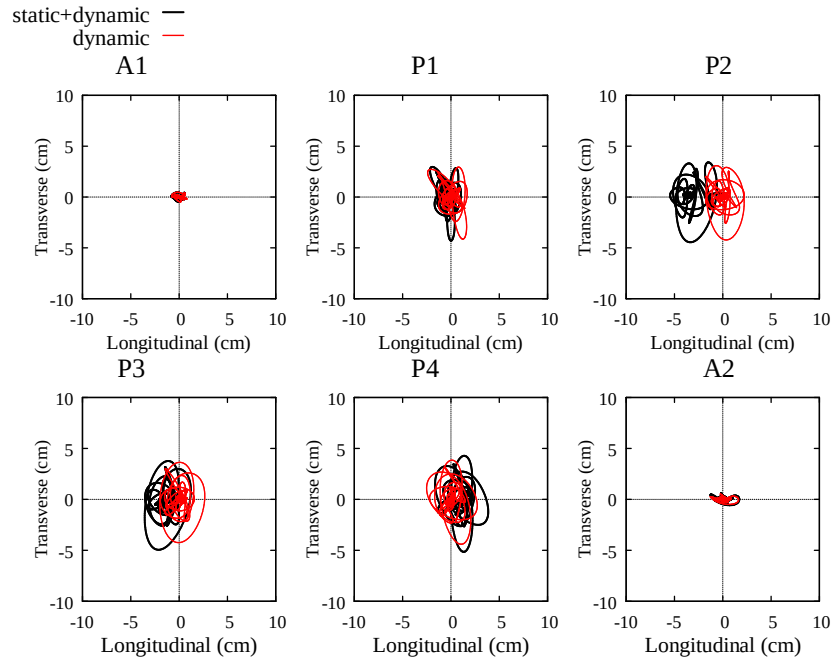


Figure 5-10 Orbit of column drift along horizontal plane (xz-plane)

(iii) Bearing behavior

The bearing behavior can be checked by the relative displacement between deck and pier top, as shown in Figure 5-11. The effective of hanging wall is significant, the displacement of A1, P1 and P2 located on the footing wall are in the opposite direction with the A2, P3 and P4 located on the hanging wall. Due to the permanent displacement, the deformations of piers are with larger reverse deformation than by the conventional method (only consider the dynamic terms). The effective displacement of lead rubber bearing are 18.8cm for A1 and A2, and 16.1cm for bearing on the P1~P4, while the results showed that, the deformation with permanent displacement of bearings are approximately 20.0cm, which indicated that the bearings are lose efficacy.

The same as the column drifts, it also focuses on the orbit of bearing displacements in horizontal plane (xz-plane in this simulation) shown in Figure 5-12. The seismic behaviors of bearings on the top of piers are more significant than the bearings on the abutments. The residual movements along longitudinal direction are predominant.

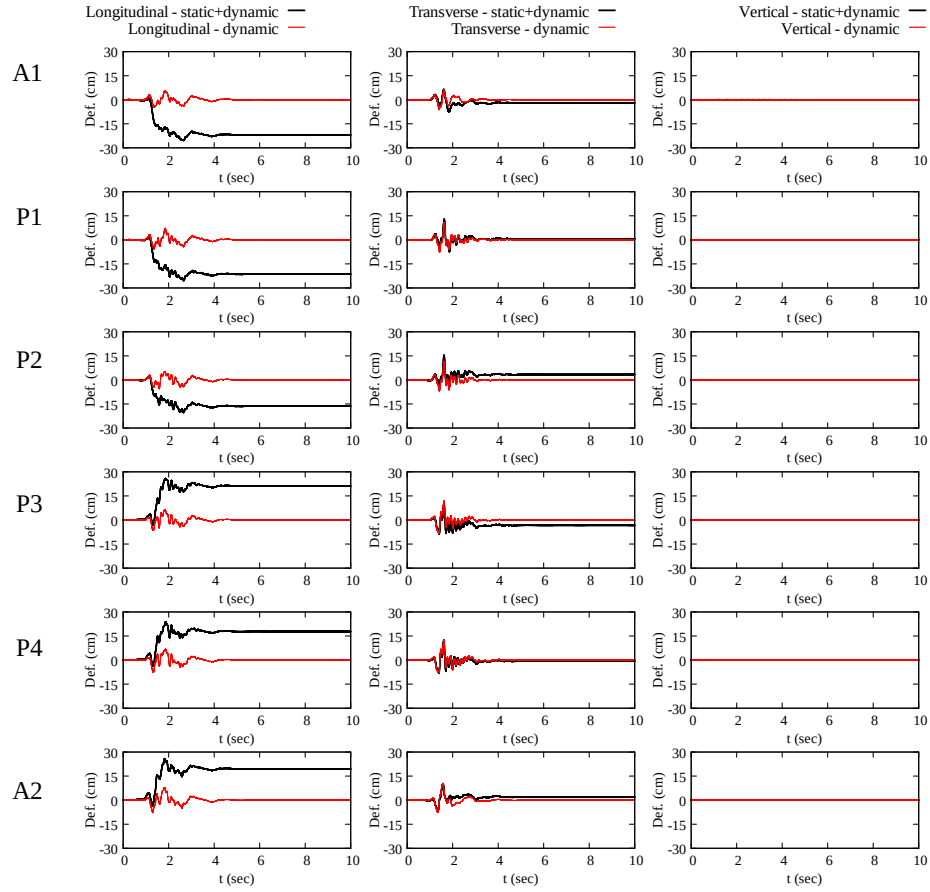


Figure 5-11 Bearing behavior with and without static displacements

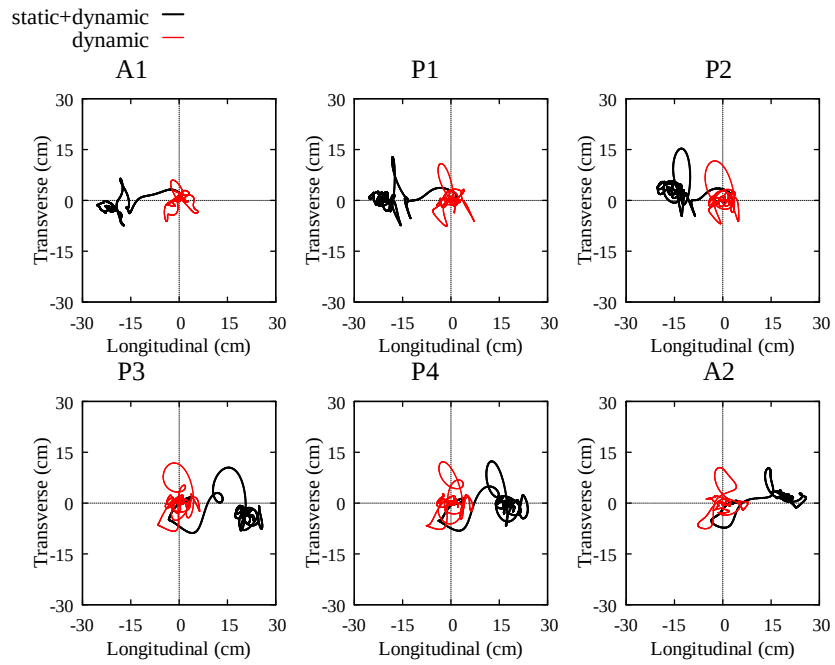


Figure 5-12 Orbit of bearing along horizontal plane (xz-plane)

5.4 Nonlinear analysis on bridge

Recent earthquakes have demonstrated the vulnerability of bridges that cross fault-rupture zones. While avoiding building bridges across faults may be the best practice, it may not always be possible to do so, especially in regions of high seismicity, such as Japan. Bridges crossing fault-rupture zones will experience ground offset across the fault and hence spatially-varying ground motion. And the nonlinear analysis is more realistic, thus in this section the nonlinear analysis on bridge is conducted to explore the general behavior of more real situations.

5.4.1 Bridge model

The previous 5-span simple bridge mentioned in section 5.3 (which was shown in Figure 5-4) has been adopted again as our objective bridge model. The deck and column section properties are typical of many bridges in Japan. A dip-slip fault was assumed to be located between Bents 2 and 3 of the selected bridges (Figure 5-4).

5.4.2 Input ground motion

The input ground motions considering the fling-step displacement are shown in Figure 5-6 in Section 5.3.2.

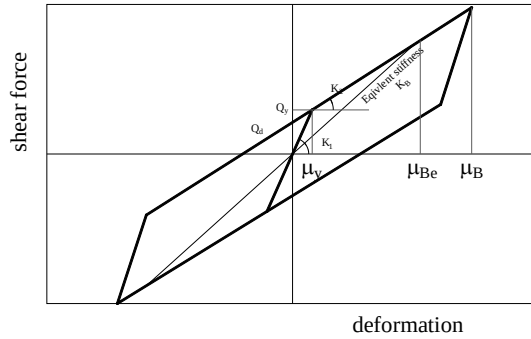
5.4.3 Analysis model

The non-linear responses of the selected bridge system were analyzed using the structural analysis software Open System for Earthquakes Engineering Simulation (OpenSees [10]) as mentioned in the linear analysis. The box girder was modeled by linearly elastic beam column elements. And the columns was model by for simplicity to capture the distribution of mass along the length of the box girder, two elements per span were used. The columns were modeled with single elements with the effective moment of inertia obtained from their section moment–curvature analysis. The isolated lead rubber bearing was modeled with linear springs in the transverse and longitudinal directions, the design was consistent with the design is stiffing the JRA [6]. The nonlinear behavior of bearing and its parameters are listed in Figure 5-13 below. It should be pointed out that, the vertical direction of bearing has not been considered in this numerical calculation, and the input ground motions from JRA are the uniform excitation for all abutments and piers.

Nonlinear time history analyses were carried out using the simulated ground motions to determine the dynamic responses of the selected bridge. It consisted of elastic 3D beam elements, nonlinear springs, 0-length link elements, and (nonlinear) fiber elements for plastic hinge range of piers.

1) Bearing model

The bearings of the isolation systems were modeled with the 0-length link elements in longitudinal and transverse directions, including 6-degree of freedom. And the bilinear model (the below Figure 5-13) along longitudinal and transverse direction was adopted in this numerical analysis.



Here, the parameters of the bilinear model of bearing are as following:
 $K_1=40023 \text{ kN/m}$
 $K_2=6154 \text{ kN/m}$
 $Q_y=1117 \text{ kN}$

Figure 5-13 Bilinear model of bearing (for five bearing)

2) Fiber model for piers and abutments

Because the fiber model is used to directly evaluate the nonlinear property of material at arbitrary point of cross section. Figure 5-4 (d) shows a division of the configuration of bars of piers for this objective bridge. The cross section is usually divided into small adequate concrete elements and reinforcement bar elements. Stress-strain curves for concrete and re-bar are consistent with the design from Figure 10.3.1 in JRA [6], the member is considered as assembled body from fiber elements, and cross sectional forces are calculated under the assumption of plane preservation. Thus, according to JRA [6], the fiber model for deck and piers are used here, and the plastic hinges are also considered and set as nonlinear properties.

It should be noted that, for the evaluations on seismic behavior of piers in this numerical calculation, the nonlinear model with constant axial forces ($M-\phi$ model) to evaluate the seismic responses of bridge structures and/or components is adopted, the moment-curvature relations are calculated by stress-strain curve of concrete defined in JRA [6]. Yield moment is obtained when tensile stress of reinforcement bar reaches yield strength of steel (namely the yield level in the following analysis); the limit state of seismic performance level 2 is the state where energy is expected to be absorbed stably; and the ultimate strength is obtained when concrete reaches compressive strength at the location where most outside reinforcement bar installed (namely the limit level 3), here the cover concrete spalls and longitudinal bars occur buckling.

5.4.4. Results analysis

In this section, it focuses on the responses of superstructures (including three items: the time history of displacement, time histories of bending moment of girder, and the maximum bending moment of girder), pier behaviors (column drift, bending moment curvature, the shear-deformation and the moment-curvature ($M-\phi$) hysteresis curves), and bearing behaviors (time histories of deformation and the shear-deformation hysteresis curves) are checked under the three input waveforms as plotted in Section 5.4.2).

1) Girder behavior

(i) Deck displacement (absolute displacement)

The absolute deck displacements are shown in Figure 5-14. Due to large permanents along vertical direction, it causes significant response of the bridge. Along longitudinal direction, we can also obtain the large relative displacements (the deck to the input motions).

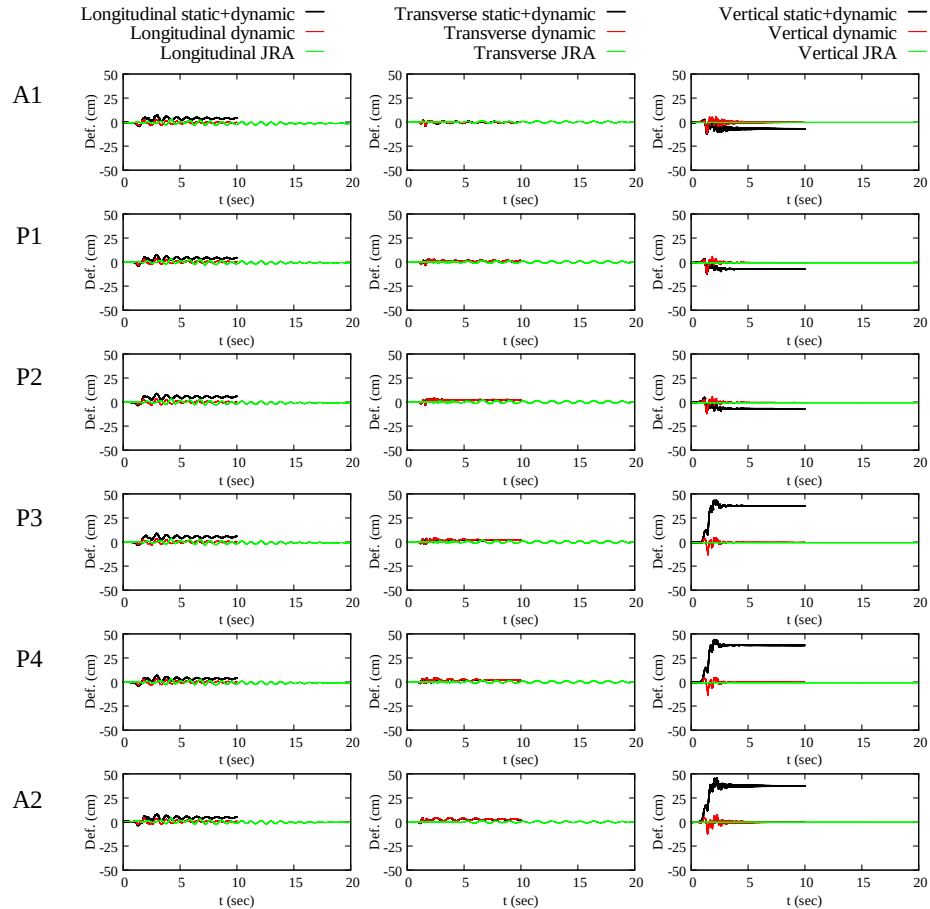


Figure 5-14 Deck displacement with and without static displacements

(ii) Bending moment of girder

The bending moment of girder along transverse and longitudinal directions are shown in Figures 5-15 ~ 5-16, respectively. Both the initial status ($t=0.0s$) and max bending moment are listed here. The comparisons between three input ground motions, only dynamic, both static and dynamic terms, and the ground motions from JRA, resulted that, the rather large responses for the former two cases are mainly triggered from the vertical components of ground motions.

For the M_y , mainly due to considerations of initial status (around 25000 kN.m from Figure 5-15 (a)), and for the max bending moments in Figure 5-15 (b), P1 is 120000 kN.m, and -100000 kN.m, 120000 kN.m and 120000 kN.m for P2, P3 and P4, respectively, additionally, there is a big changing of the max bending moment during the second-, third-, and fifth- spans. The specific difference between the third span is due to relative input ground motions.

For the M_z , there is also a significant changing in the third span (between P2 to P3), which is from 15000 kN.m to -7000 kN.m, it is mainly due to the large relative displacements along UD-directions.

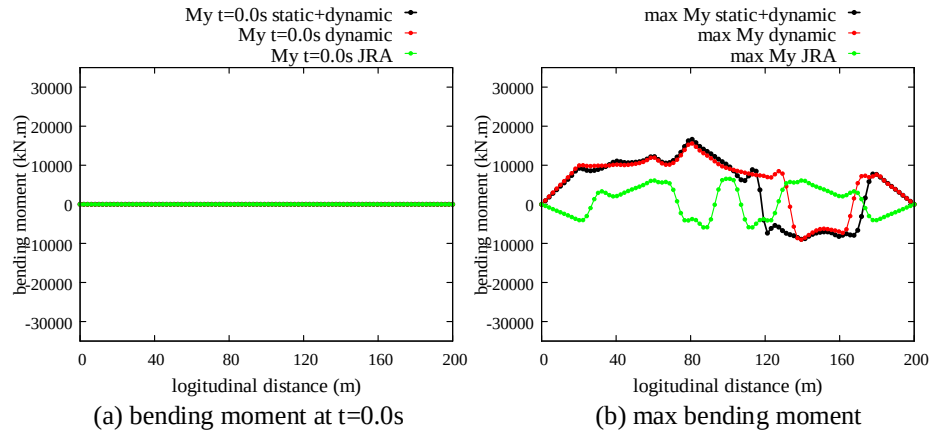


Figure 5-15 Bending moment of girder along transverse direction (M_y)

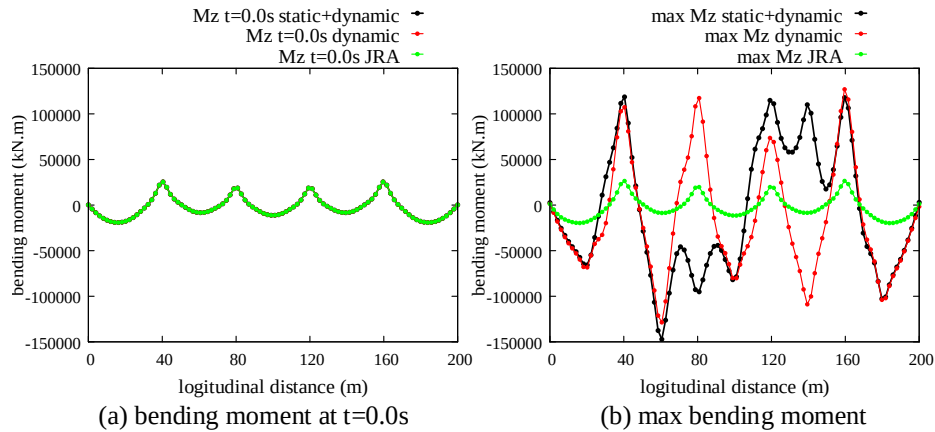


Figure 5-16 Bending moment of girder along longitudinal direction (M_z)

(iii) Time history of bending moment of girder

The time history of bending moment for the nodes connected to piers and girder along transverse and longitudinal direction are shown in Figure 5-17 (a) and (b), respectively. It can be seen obviously that, there are big differences for the response of bending moments both along transverse and longitudinal directions under the three excitations.

Along transverse directions as shown in Figure 5-17 (a), the max bending moments are about 120000 kN.m and 90000 kN.m for P1 and P2, and final bending moments of both P1 and P2 are less than 50000 kN.m. While the max bending moments for both P3 and P4 are over 100000 kN.m, and the final bending moment of P3 are larger than 50000 kN.m.

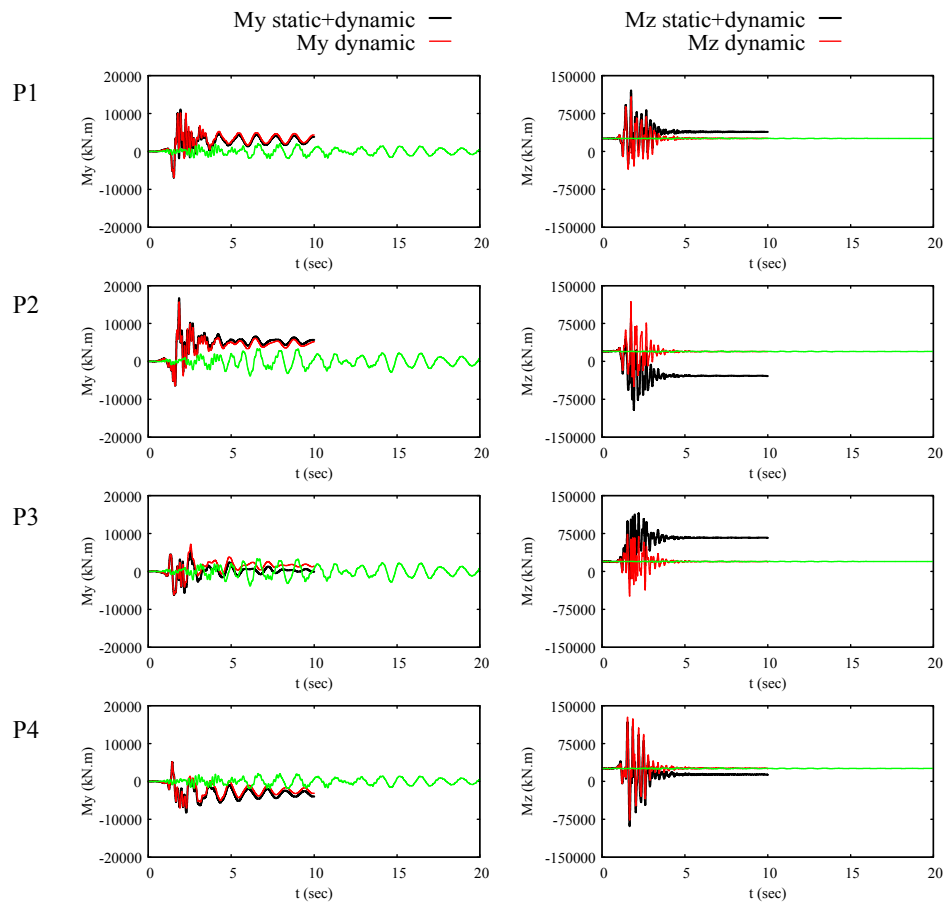


Figure 5-17 Time history of bending moment for girder contact to the pier location

2) Pier behavior

(i) Column drifts

The time histories of column drifts are shown in Figure 5-18.

Along longitudinal direction, the response of P1 ~ P2 are within yield level under all the three input excitations, but P3 ~ P4 are significant residual displacements than the yield level under the fling-step displacements, still behavior less than the limitations, while under the excitation from JRA, P1 ~ P4 are within the yield level,

Along the transverse direction, the response of all piers under the JRA excitation exhibit small residual displacements.

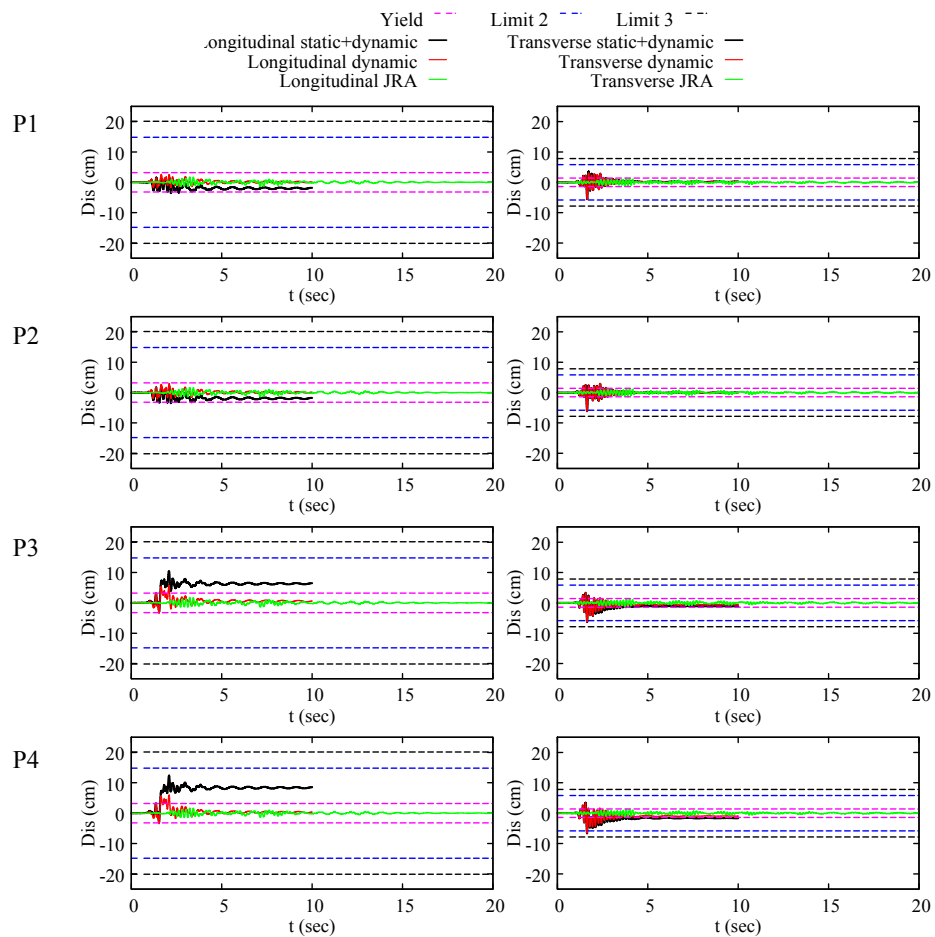


Figure 5-18 Time histories of column drift of piers

(ii) Bending moment

The numerical results of bending moments of piers for the initial status and max status are listed in Figures 4-18~4-19. The dashed lines are the yield resistance calculated by push-over analysis. From Figure 4-18 (b) and Figure 4-19 (b), all of the piers are within the yield limitations.

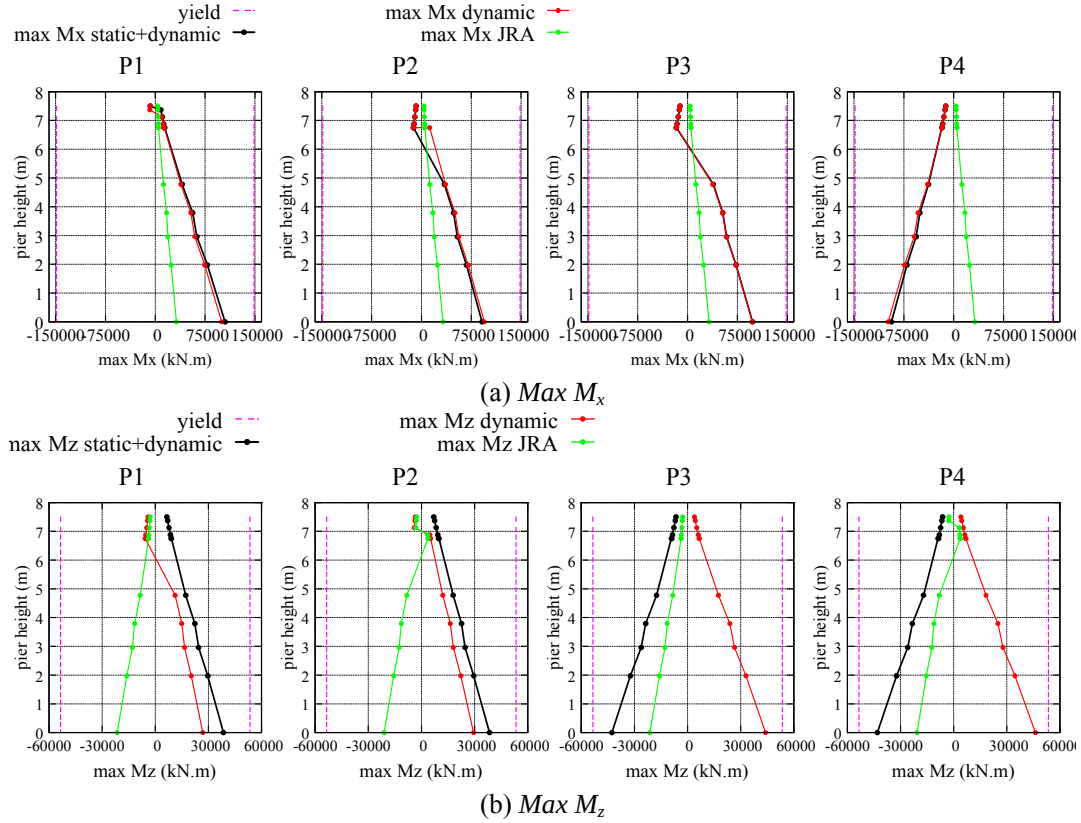


Figure 5-19 Max bending moment of piers

iii) Time history of bending moments

The time history of bending moments for the top and bottom of piers along transvers and longitudinal directions are shown in Figure 5-20 (a) and (b).

It can be seen apparently, along transverse direction Figure 5-20 (a), the small residual bending moment for P1 ~ P4 are consistent with the small fling-step displacement excitation. Along longitudinal direction Figure 5-20 (b), the residual bending moments under the ground motions only dynamic and from JRA are tended to zero, and with strong oscillation under JRA excitation; on the contrary, there remained the residual bending moment both for the top and bottom of piers, which were stem from the large static displacements input.

From the above results, dynamic analysis considering accelerations cannot estimate residual moments and displacements. This means the necessity of conducting dynamic analysis using displacement waveforms to obtain the more exact behavior of the bridge during earthquake.

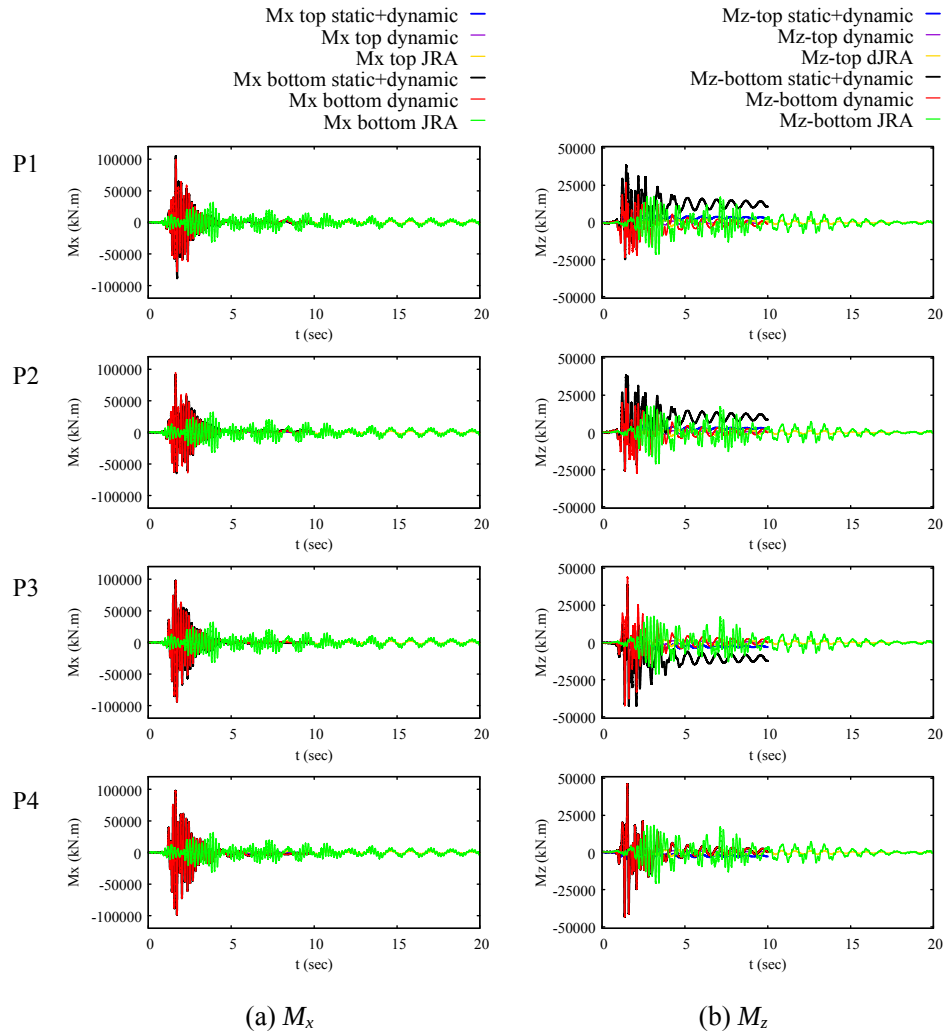


Figure 5-20 Time history of bending moment of piers (P1~P4)

iv) Max curvature

The curvature could be one of the most important factors for evaluation the seismic behavior of piers during earthquakes, because the curvature could reflect more realistic deformation from the local coordinate, which is different form the displacement from the global coordinate view point. But it is not convenient for engineering design due to the effect of changing axial forces might cause total different curvatures of piers. For simplify in this analysis, this effect has not been considered yet.

The max curvatures for all the piers along longitudinal and transverse directions are listed in Figure 5-21 (a) and (b), respectively. The components upper the plastic hinge are with the yield limitations obviously, but the parts lower plastic hinge are over the yield limitations, and the response for P3 and P4 along longitudinal and transverse directions are larger than P1 and P2. While the results of curvatures are contradiction with the judged results from bending moments, which means the push-over analysis are underestimated the yield and limitation levels in this calculation cases.

All the piers along longitudinal and transverse directions are yield under the ground motions generated by the hybrid synthesis method; on the contrary, all the piers remained the yield level under the earthquake excitation from JRA.

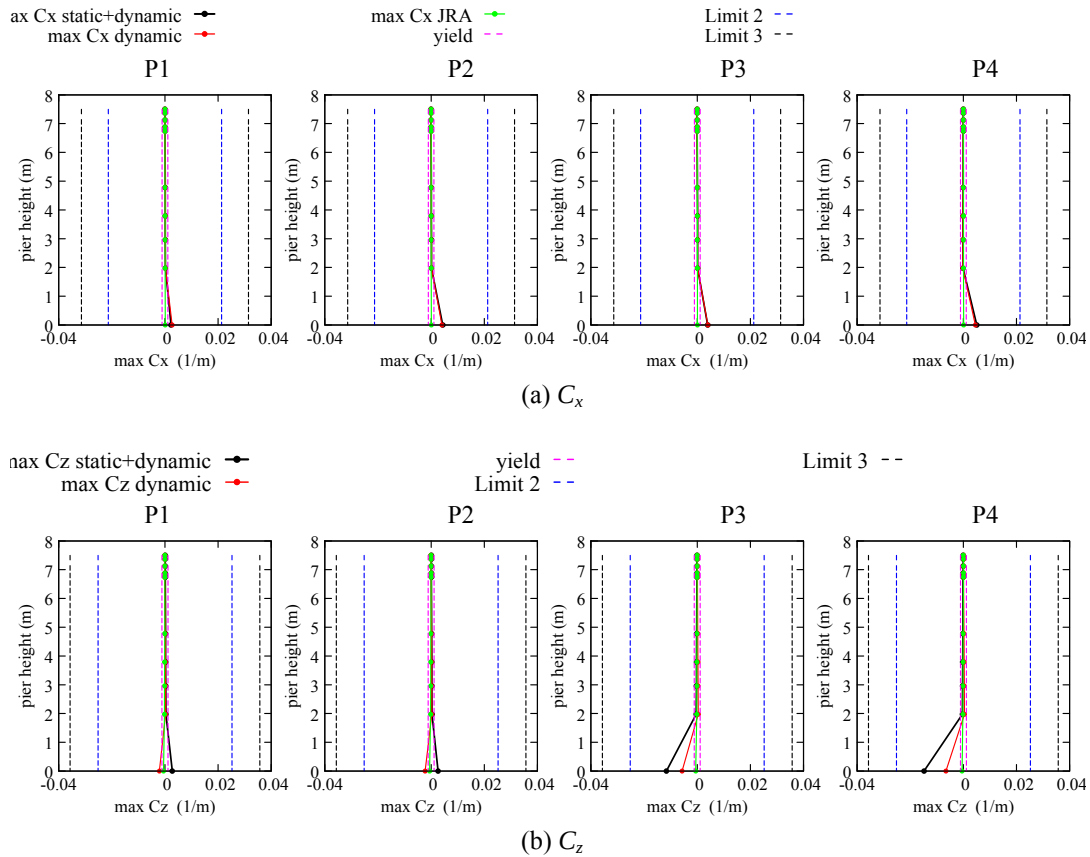


Figure 5-21 Maximum curvature of piers (P1~P4)

v) Time history of curvature

Accordingly, the time history of curvature for pier foot is listed in Figure 5-22, here only the longitudinal and transverse directions are summarized here. Along the longitudinal direction (Figure 5-22 (b) C_z), P1 and P2 are around the yield level under three input ground motions, but P3 and P4 are remarkably over the yield limitation but still within the level 2 (based on the stress and strain relationship in JRA mentioned before). Along the transverse directions (Figure 5-22 (a) C_x), P1 ~ P4 are slight over the yield level, which are different from the results by the displacement as above.

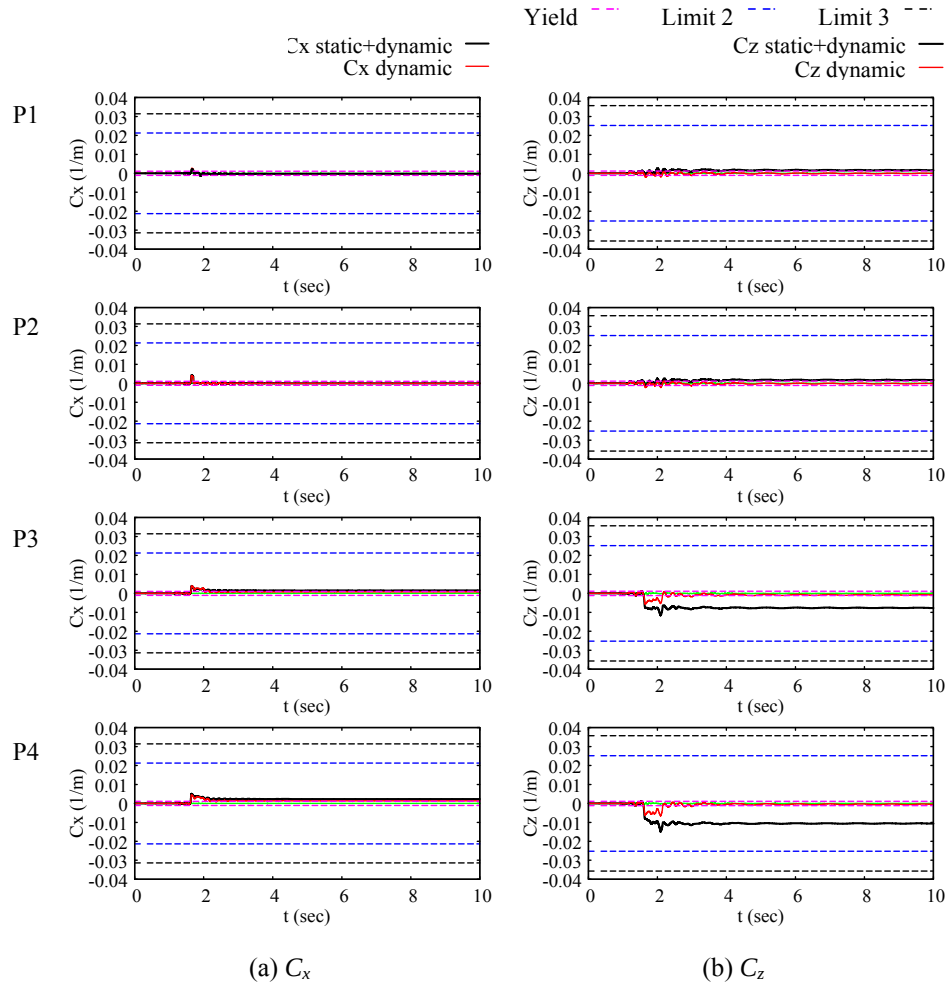


Figure 5-22 Time histories of curvature of piers (P1 ~ P4)

(vi) Shear force-deformation hysteresis curve

Due to the asynchronous input displacements, the input waveforms are not only different in the amplitudes, duration, but also the period, which is different from the conventional input ground motions assuming the time delay or the phase delay for different piers. Thus, the shear force-deformation hysteresis curves are significant from different piers and abutments as shown in Figure 5-23 below. For P3~P4, which are located in the hanging-wall with larger and stronger ground motions, due to the permanent displacement, they exhibit two cyclic, while for P1~P2 located in the foot-wall, their seismic responses are only one cycle.

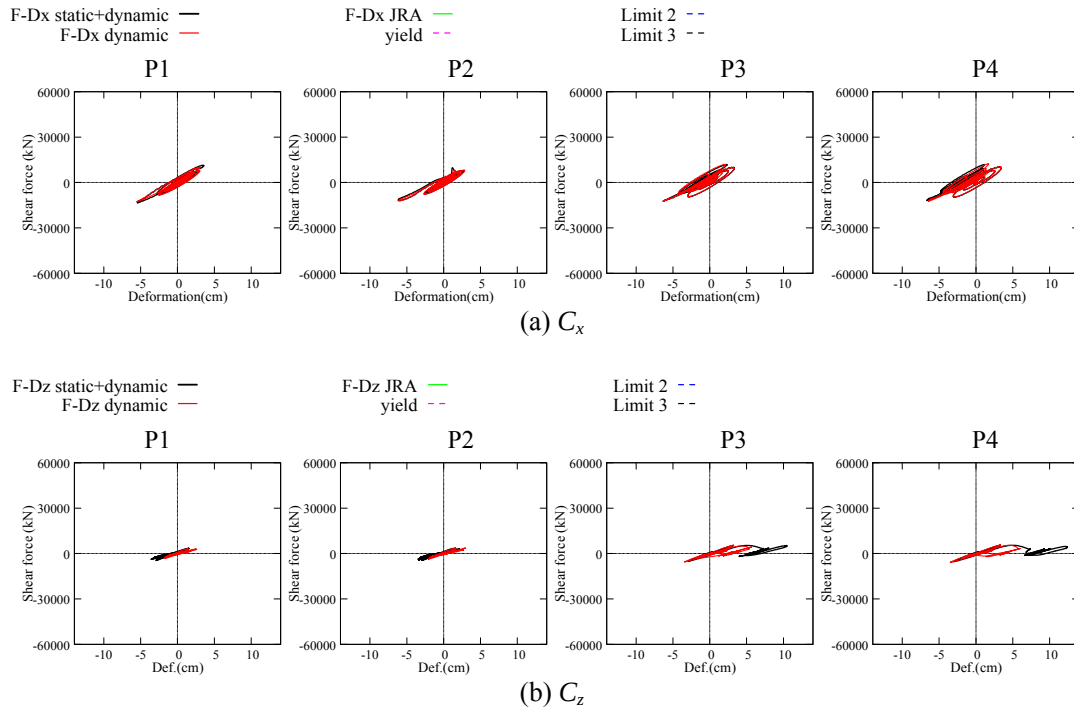


Figure 5-23 Shear force-deformation hysteresis curves of piers and abutments

(iv) $M-\phi$ hysteresis curve

$M-\phi$ hysteresis curve for piers and abutments are shown in Figure 5-24. It can be seen clearly, the seismic responses along longitudinal- and transvers- direction are totally different.

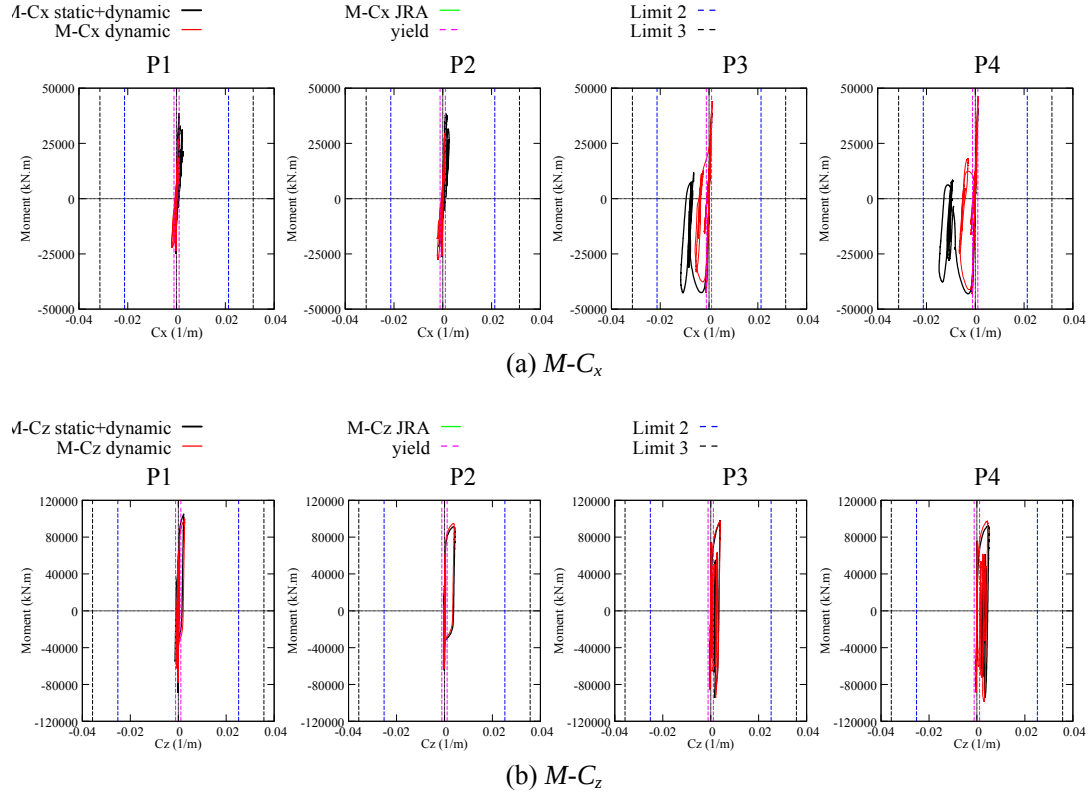


Figure 5-24 Moment-curvature hysteresis curves at the piers (P1~P4)

3) Bearing behavior

(i) Time histories of deformations

For the bearing, it was modeled by 0-length beam element; the big difference for the seismic behavior of bearing is along longitudinal direction as shown in Figure 5-25. Due to large relative displacement between piers and girder, the relative displacement (namely the bearing deformations) is large, especially along longitudinal direction in Figure 5-25 (a), because large differences of stiffness of pier and girder. The large relative displacements of P1 ~ P2 to P3 ~ P4, which was triggered by the earthquake excitation containing fling-step displacement along longitudinal direction, might give rise to serious damage of superstructures. Along the transverse direction in Figure 5-25 (b), the small static displacement waveforms result the small response of bearing.

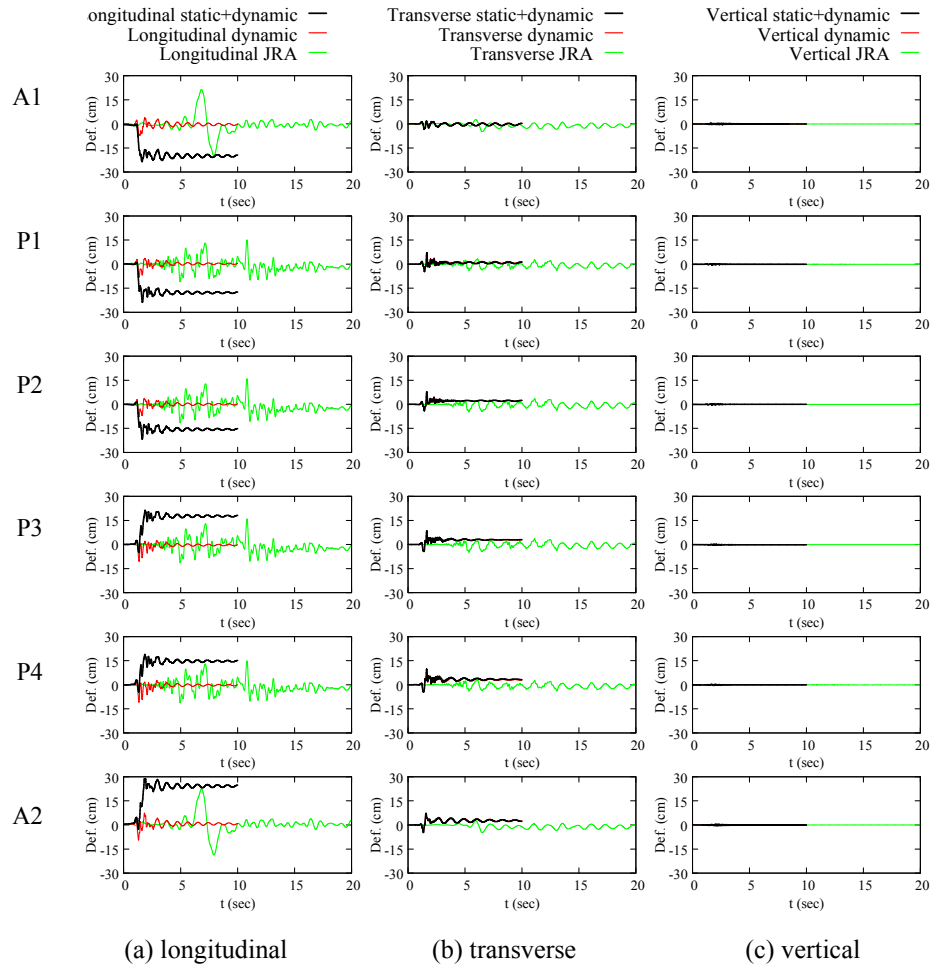


Figure 5-25 Bearing behavior of bridge under nonlinear analysis

It should be noted that, the excitation only consider the dynamic terms and waveforms from JRA exhibit oscillation near the original position. The uniform input for all piers and abutments under

the JRA excitation produced almost the same seismic response of bearing for P1 ~ P4 the same for A1 ~ A2, which strong reflect it is necessary to well consider multi-support excitations.

Compared with other components of bridge structures, i.e., deformation of piers and girder, the larger response values indicated that, the bearings are more sensitive to the permanent displacements in this case (one of the most commonly simplified bridge type).

(ii) Shear Force-deformation hysteresis curves

Force-displacement hysteresis curves for bearings along longitudinal and transverse directions are shown in Figures 5-26.

Along the longitudinal direction Figure 5-26 (a), it can be seen clearly, due to the permanent displacement and asytheros input ground waveforms (the black line), as well as the propaties of bridge, large deformation occurred in the bearing up to 25cm, which indicated that, in this case, the maximum displacements of piers are larger than 16.1cm, which means the bearings are broken in non-linear behavior. Along transverse direction in Figure 5-26 (b), mainly for the sake of enough stiffness of piers, the deformation along this direction did not exhibit significant behavior as compared with longitudinal direction.

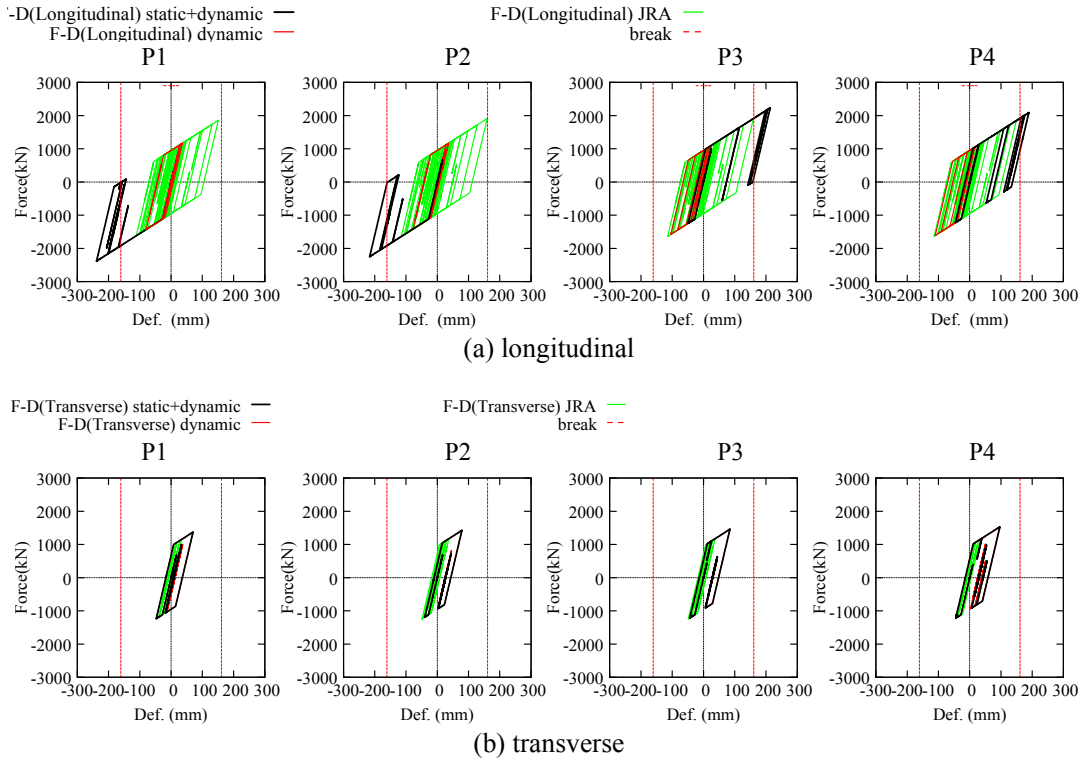


Figure 5-26 Shear force-deformation hysteresis curves of bearings

5.5 Evaluation on seismic behavior of the near-fault bridge

A ground displacement induced by fault activities is considered as the event that influences safety of the bridge. In the 1999 Kocaeli earthquake, Turkey, and the 1999 Chi-Chi earthquake, Taiwan, superstructures fell down due to extremely large fault displacements of 5 ~ 10 m. in order to directly incorporate characteristics of individual active faults into the design, active periods of the faults, and ground surface displacements caused by the faults, should be identified. Thus, based on [1][3], in this section, the seismic analysis was conducted to investigate the crossing angle between bridge and seismic fault is more sensitive, and which portion of members is suffered damage from fault displacements. Aimed at this objective, the surface fault located between P2 and P3 under various strike angle, dip angle and rake angle are explored for seismic analysis of bridge, and the general seismic response of bridge are focused on the bending moment of girder, bending moment of pier, and curvature of pier under all these cases mentioned before. The analytical model is shown in Figure 5-4. The numerical calculations on non-linear seismic behavior were conducted through the OpenSees following the same procedure as in Section 5.4.

It should be pointed out that, the response of peak value for bridge caught for each items might not occurred at the same time.

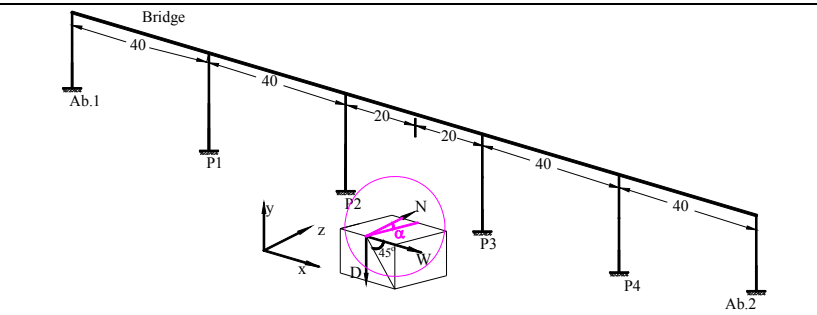
5.5.1 Cases setting and input ground motions

Analytical cases are shown in Tables 5-3~5-6, which change the strike angle for case 1, dip for case 2 and rake angle with strike angle 0° for case 3, and rake angle with angle 45° for case 4, are considered. It should be noted that, the time histories of input ground motions was simulated by the hybrid synthesis method proposed in Chapter 3, and in each case, there are two situations, only consider the dynamic terms and both the static and dynamic terms are taking into consideration in the seismic analysis.

1) Case 1: different strike angle

Changing the strike angle defectively means the crossing angle between the bridge and seismic fault, which will derive the longitudinal and transverse components of seismic excitation for bridge. Seismic fault models under the following various 6 strike angles (α) are listed in Table 5-3, based in the hybrid synthesis method on near-fault ground motions, the input ground motions are plotted in Figures 5-27 ~ 5-28.

Table 5-3 Different strike angle for cases study

No.	α ($^{\circ}$)	
Case 1-1	0	
Case 1-2	15	
Case 1-3	30	
Case 1-4	45	
Case 1-5	60	
Case 1-6	75	

From Figure 5-27 and Figure 5-29, there is slight change along vertical directions due to the strike angle varied. And the PGD under different strike angles are shown in Figure 5-28 and Figure 5-30, the following points could be obtained:

(a) From Figure 5-27 and Figure 5-29, along longitudinal direction, the ground motions increase along with the increasing strike angle, while the inverse tendency along transverse direction. Along longitudinal direction, when the strike angle is larger than 45° , the 2nd PGD change the diction; and along transverse direction when the strike angle is larger than 60° , the 1st and 2nd PGD change the directions. For the second situation in Figure 5-29, due to considering both the dynamic and fling-step displacements, the displacements are larger than the first situations.

(b) The various tendencies in hybrid method are larger than the conversional stochastic Green's function method as shown in Figure 5-28 and Figure 5-30. Along vertical directions, both the two method are without changing under different strike angles, mainly due to this is a pure strike-slip fault, and the dip angle is fixed as 45° .

(c) For the results calculated only considering the dynamic terms, the PGD exhibit complicated features, and all the absolute value of three components of PGD are less than 10cm.

(d) By hybrid synthesis method along with the increasing of strike angle, the longitudinal PGD decreases with different various rules, the PGD decreases when α changes in two situations (from 0° to 15° and from 15° to 30°) is smaller than when α is larger than 30° . It should be noted that, when α increase, the relative locations of asperity and the piers are changed, the hanging wall effects are not obvious in this cases.

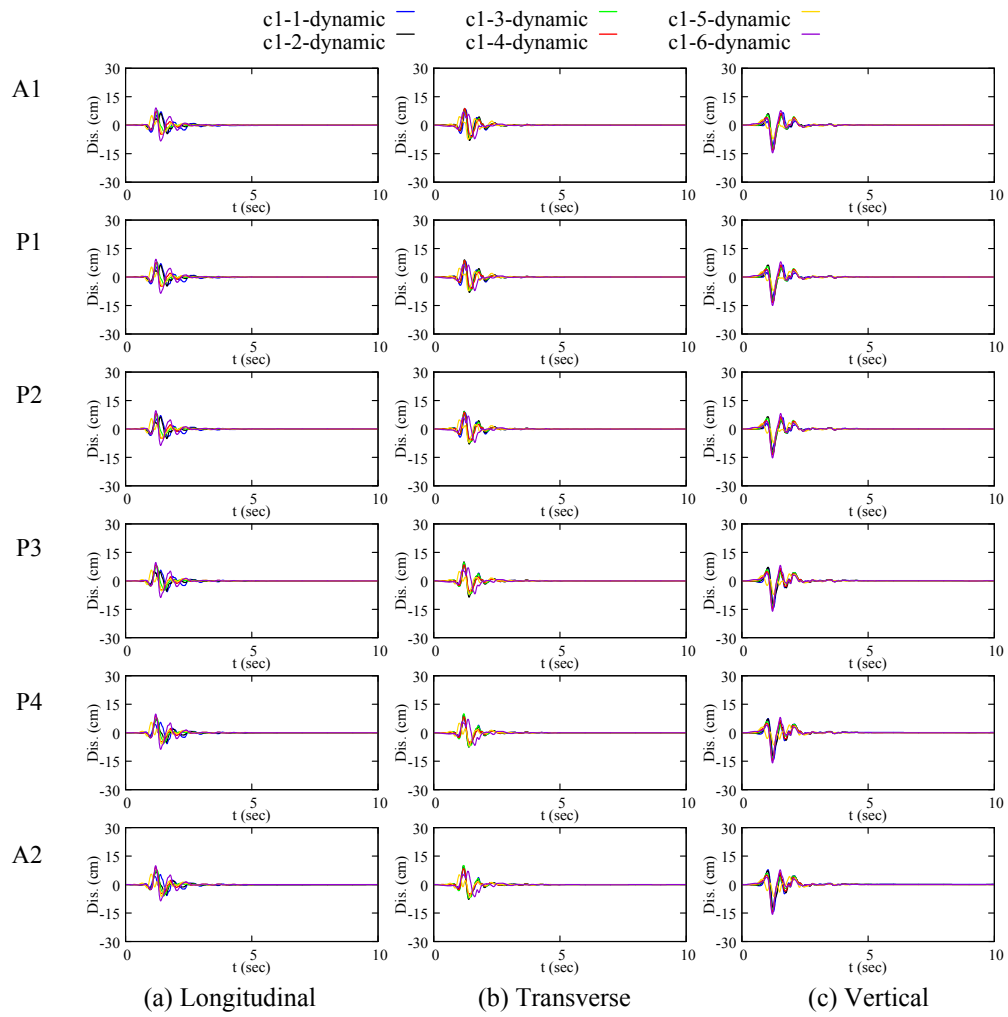


Figure 5-27 Input GM under different strike angle α (only dynamic)

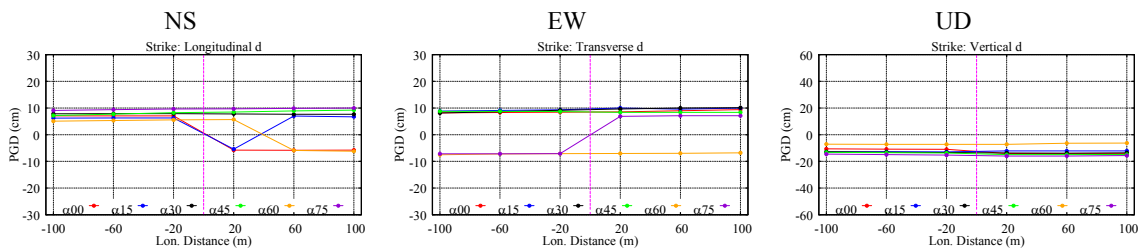


Figure 5-28 PGD under different strike angle (only dynamic terms)

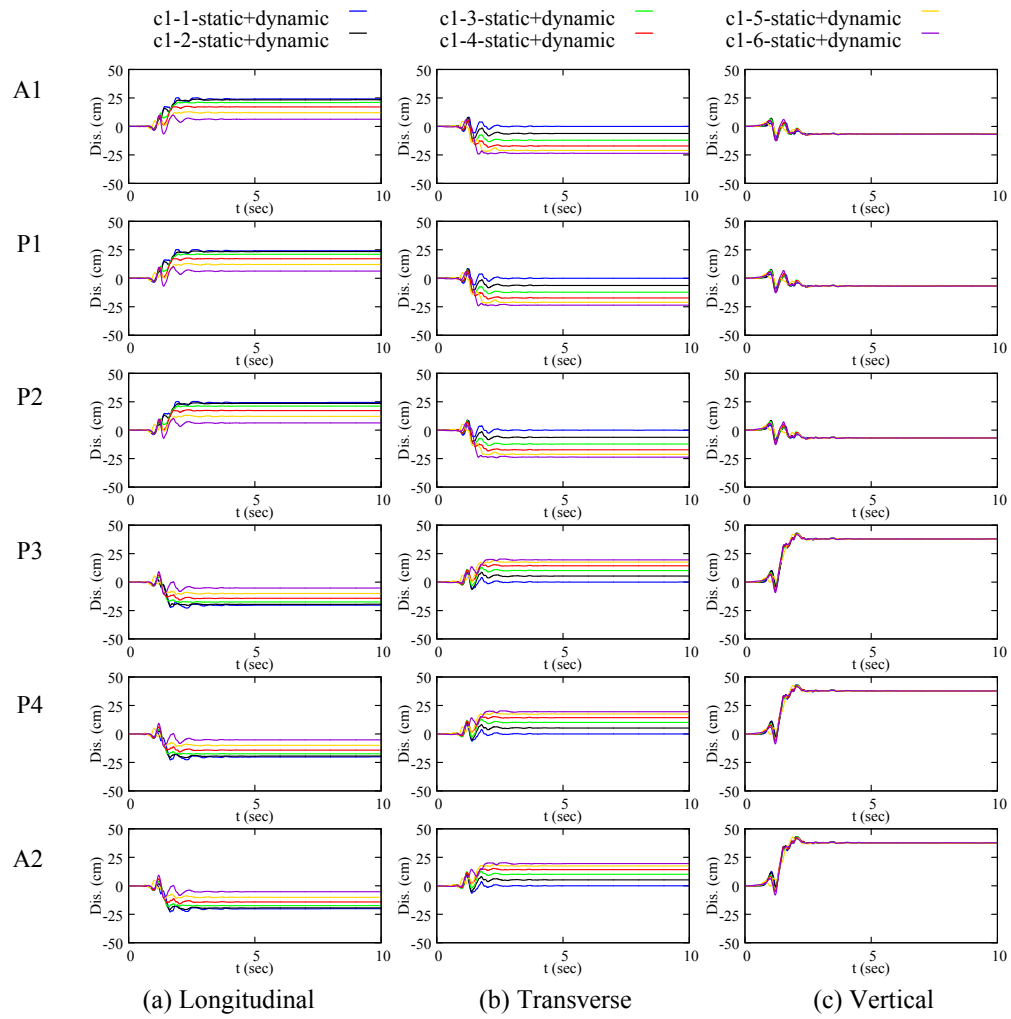


Figure 5-29 Input GM under different strike angle α (static + dynamic)

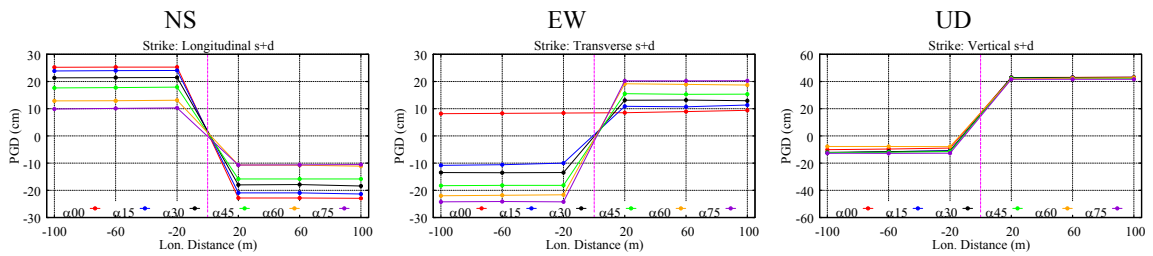
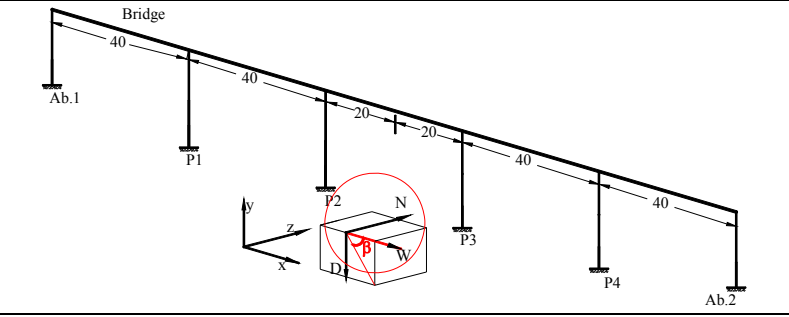


Figure 5-30 PGD under different strike angle (static + dynamic terms)

2) Case 2: different dip angle

Theoretically speaking, the dip angle might significant affect the vertical and longitudinal components for bridges. While based on the occurred earthquake and J-SHIS database, the dip-slip fault might difficult under the condition the dip angle is larger than 60° , (the rake angle is 90° in this fault model). Therefore, only 4 cases for various dip angles (β is 15° , 30° , 45° and 60°) are set in Table 5-4.

Table 5-4 Different dip angle for cases study

No.	β ($^\circ$)	
Case 2-1	15	
Case 2-2	30	
Case 2-3	45	
Case 2-4	60	

The simulated ground motion waveforms and its related PGD are listed in Figure 5-31 ~ 5-34, which indicated that,

(a) From Figure 5-31, only with dynamic items, with increasing of β , all the three components along longitudinal, transverse and vertical directions are changed, especially longitudinal components for P3 ~ A2. The ground motions along vertical direction with slight change, but the time delay is obvious. From Figure 5-33, with increasing of β , slight changes can be seen along transverse direction mainly due to this is a simple and pure reverse slip fault, while it exhibited different various tendency in hanging wall and foot wall. In foot wall, the longitudinal components increased from around 10 cm to 30 cm, and the vertical components are increased from approximated 0 cm to -15 cm; while in the hanging wall, about -50 cm to -5 cm along longitudinal directions and 20 cm to 40 cm along vertical directions, the obvious change might caused large responses of bridge;

(b) From Figure 5-32 and Figure 5-34, among the three directions, the various tendencies of PGD in the hanging wall are larger than the sites in the foot wall;

(c) By the hybrid synthesis method, from 15° to 30° , the longitudinal PGDs are with the most significant change (with up to 50cm decreasing) from A1~P2 (foot wall) to P3~A2 (hanging wall) in Figure 5-34, which might cause large relative seismic responses of the bridge. Due to this is a pure dip-slip fault, there is no large difference under different rake angles along transverse direction;

(d) For the PGD only dynamic terms, it can be seen, there are not large differences compared with the results by hybrid synthesis method, it emphasized that not real and exact seismic response of bridge can be obtained under the earthquake excitations considering only the dynamic terms.

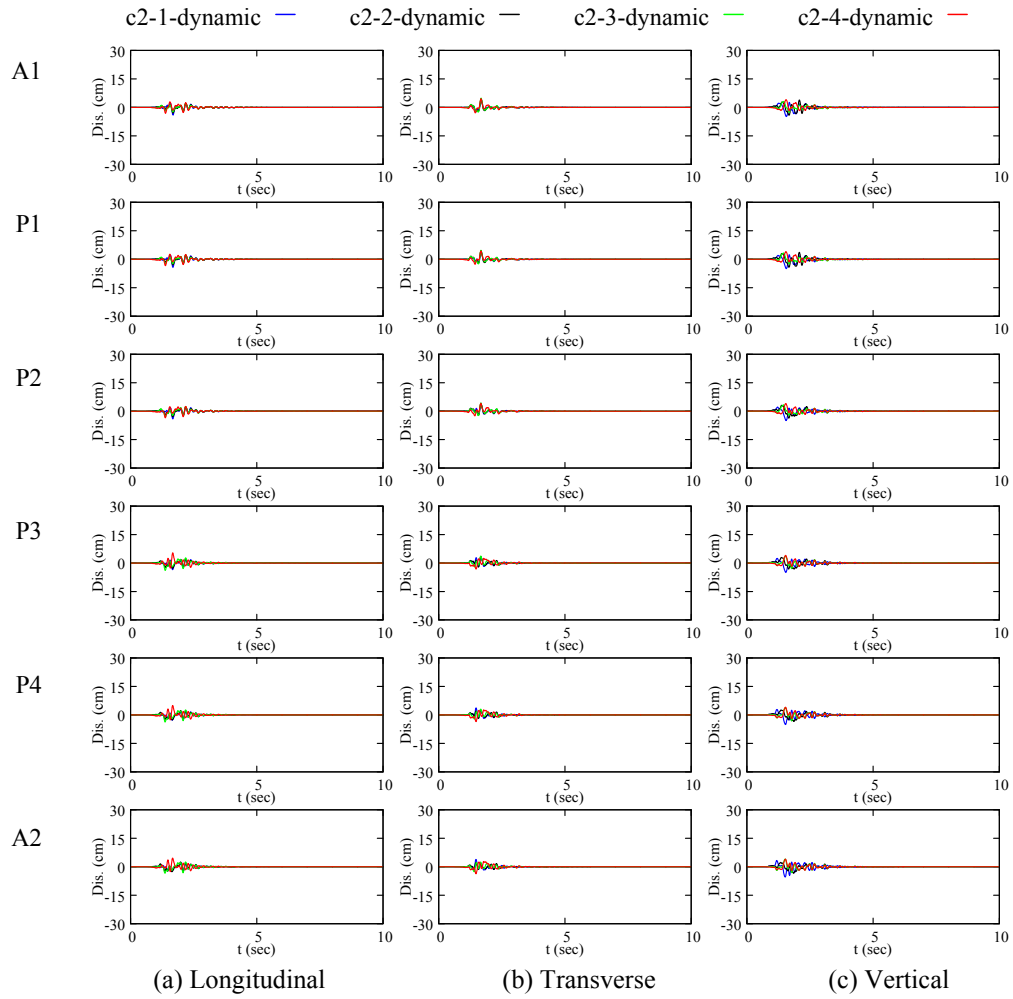


Figure 5-31 Input GM under different dip angle β (only dynamic terms)

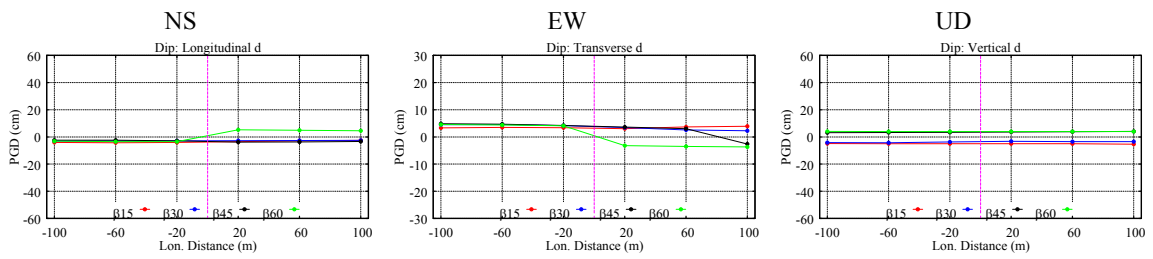


Figure 5-32 PGD under different dip angle (only dynamic terms)

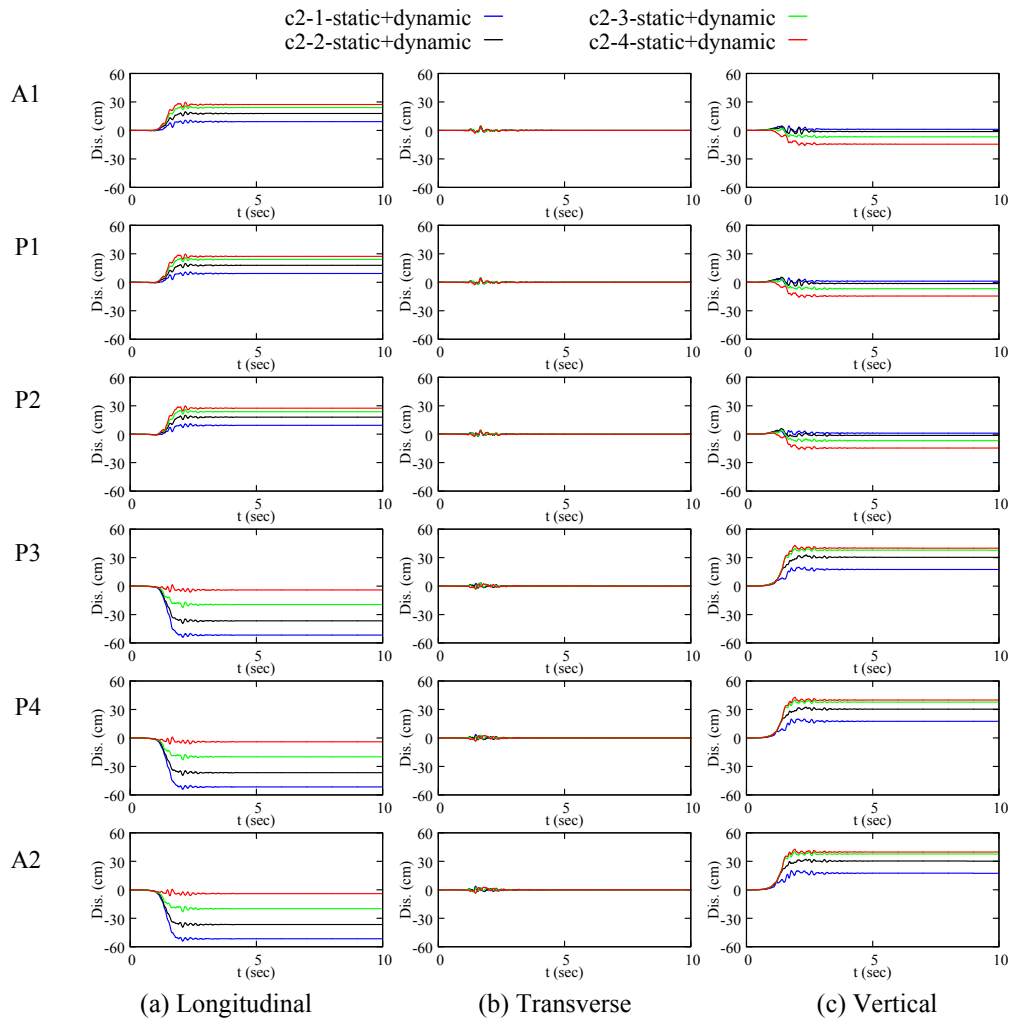


Figure 5-33 Input ground motion under different β (static + dynamic)

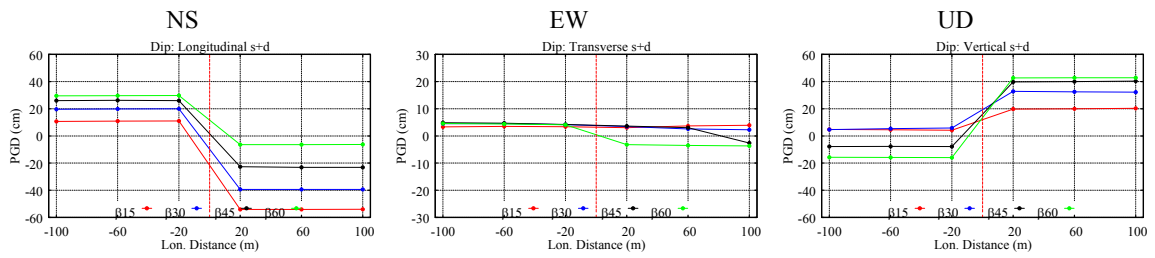


Figure 5-34 PGD under different dip angle (static + dynamic)

3) Case 3: different rake angle with strike angle 0°

The rake angle is one of the important factors of focal mechanism. The rake angle equals to -90° means the normal fault, and 0° means the left-lateral slip fault, and 90° is for dip-slip fault. Case 3 including 7 changing rake angles are as listed in Table 5-5 with strike angle equals to 0° .

Table 5-5 Different rake angle with strike angle ($\alpha=0^\circ$) for cases study

No.	$\gamma (^{\circ}), \alpha=0^{\circ}$	
Case 3-1	-90	
Case 3-2	-60	
Case 3-3	-30	
Case 3-4	0	
Case 3-5	+30	
Case 3-6	+60	
Case 3-7	+90	

Figures 5-35 ~ 5-38 showed the input ground motions and PGD under these 7 situations, which clearly produced that:

(a) The differences between these two methods are the results by hybrid method are larger than considering only dynamic terms.

(b) For hybrid method, (i) Along longitudinal direction, the anti-symmetrical various tendencies for hanging wall and foot wall sites can be obtained in the same situations in (a) except the rake equals to 0° ; due to this is a strike-slip fault, different focal mechanism with the other calculation cases. (ii) The transverse components are affect by rake angle and exhibit different tendencies in hanging wall and foot wall as shown in (b), larger various are obtained in the former than the latter one. And because the focal mechanism with rake angles for -90° and 90° , -60° and 60° , -30° and 30° , the transverse components are the almost the same except for -90° and 90° with approximately the same PGD but in reverse mark. And the maximum transverse PGD is under the rake angle of 0° . (iii) Due to different focal mechanism, the vertical PGD performed the most significant various. For $\gamma=60^\circ$, the PGD for P2 is out of the other sites and is around 13cm due to the static terms made the negative displacement smaller than the positive components.

(c) For stochastic Green's function method, the longitudinal components changed from approximated 8cm to -8cm when $\gamma=60^\circ$, and when $\gamma=0^\circ$, they are various from around -8 cm to 8 cm during the spans of P2 to P3, and from around -12 cm to 12 cm during the spans of P3 to P4. And it should be noted that the PGD in positive signs and negative signs are with small differences when we checked the calculations in details.

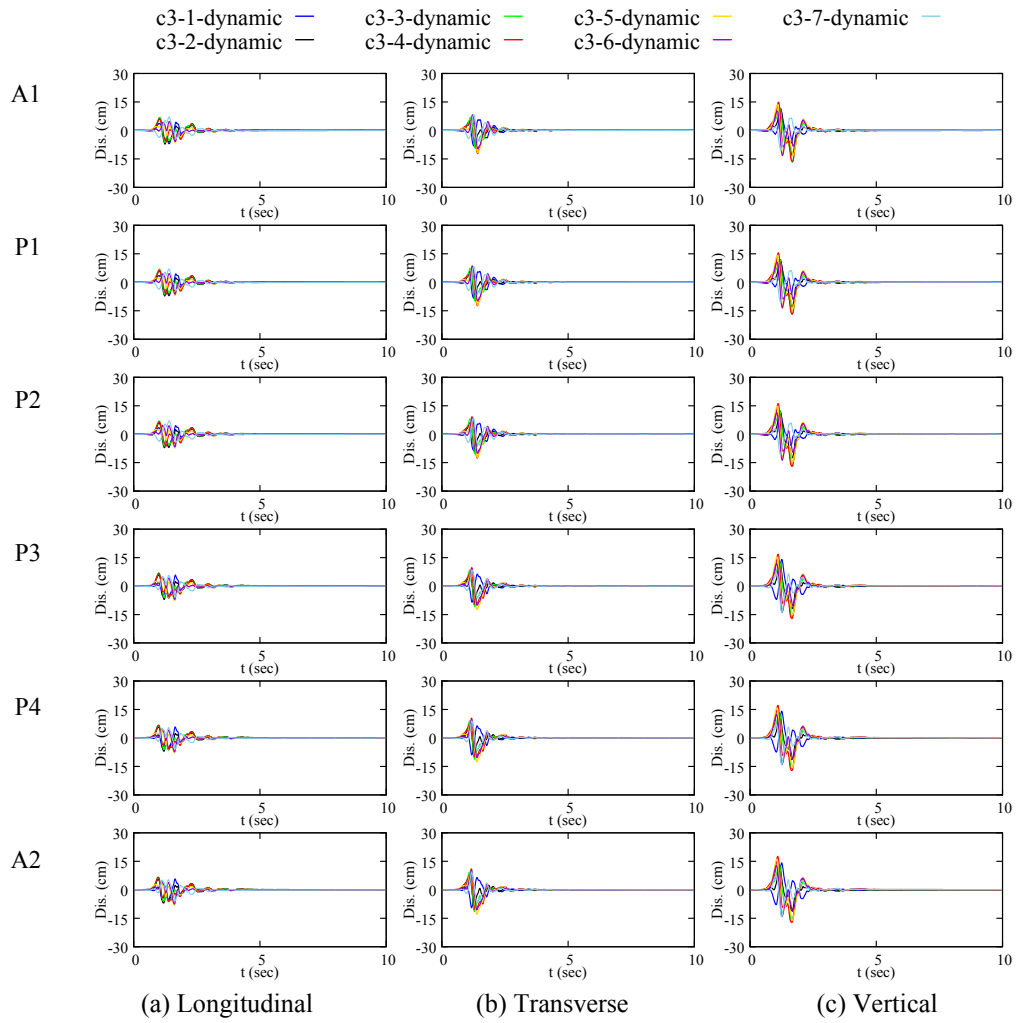


Figure 5-35 Input GM under different rake angle γ with strike angle 0° (dynamic terms)

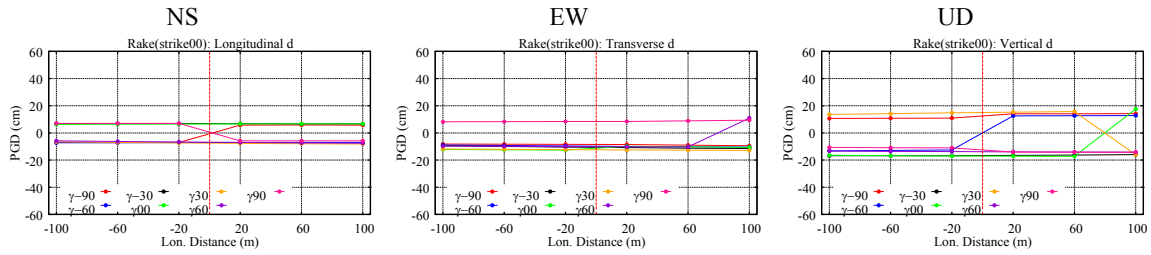


Figure 5-36 PGD under different rake angle (Strike 0°) (only dynamic terms)

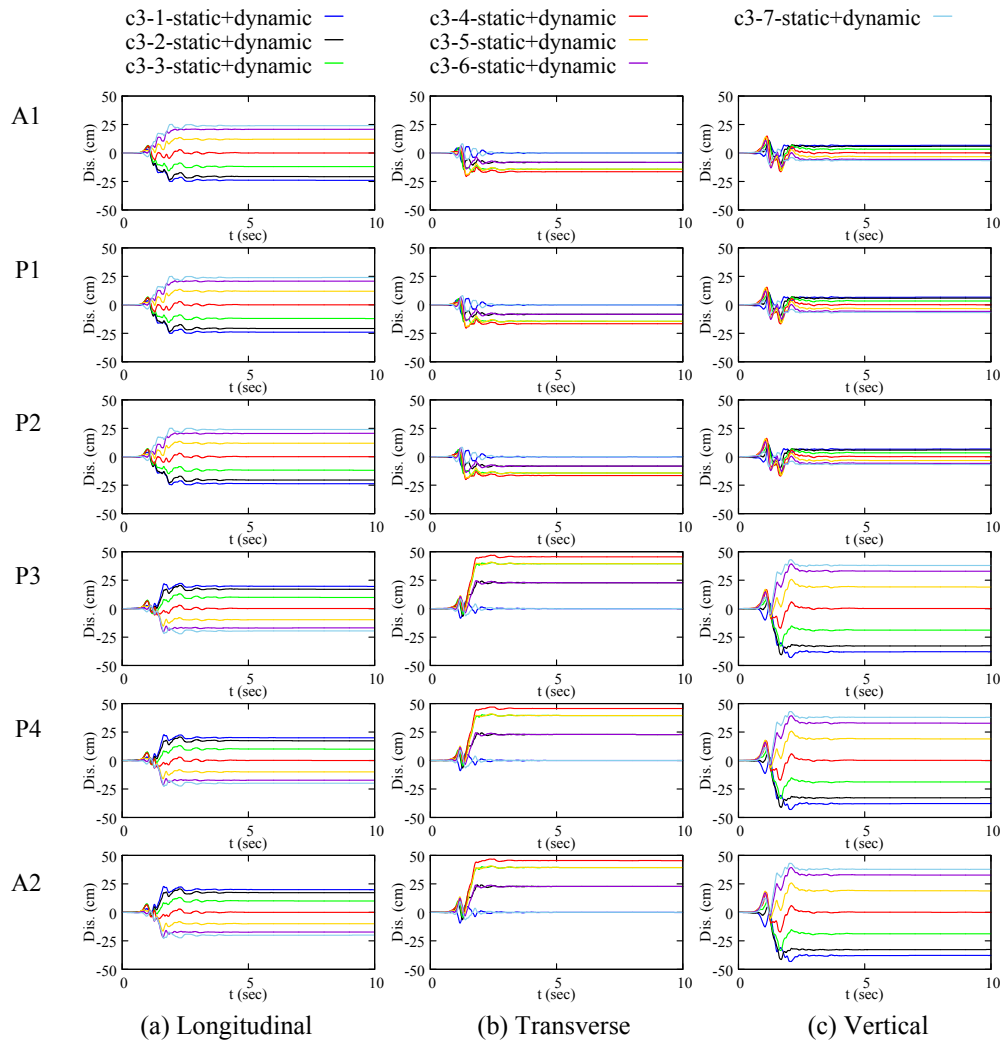


Figure 5-37 Input GM under different rake angle γ with strike angle 0° (static + dynamic)

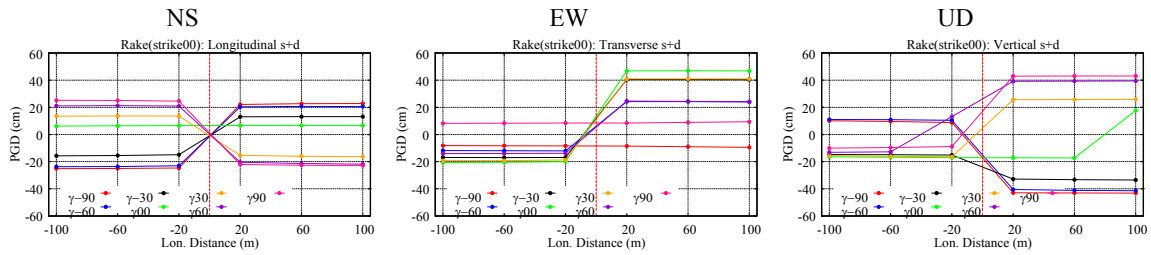
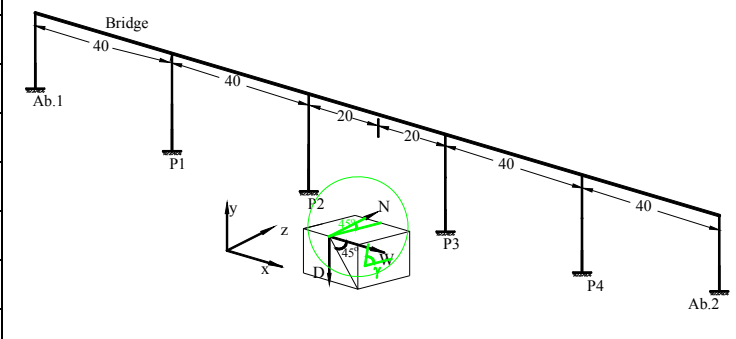


Figure 5-38 PGD under different rake angle (Strike 0°) (static + dynamic)

4) Case 4: different rake angle with strike angle 45°

Case 4 is the same considering as case 3, but with strike angle 45° , which means the components along longitudinal and transverse direction are different.

Table 5-6 Different rake angle with strike angle ($\alpha=45^\circ$) for cases study

No.	γ ($^\circ$), $\alpha=45^\circ$	
Case 4-1	-90	
Case 4-2	-60	
Case 4-3	-30	
Case 4-4	00	
Case 4-5	+30	
Case 4-6	+60	
Case 4-7	+90	

Figures 5-39 ~5-42 showed the ground motions and its related PGD. Compared with the former case 3, due to the strike angle also significantly affect the ground motions, the PGD changed more significant than the calculation results in case 3, especially the longitudinal and transverse PGD.

In this calculation, the dip angle and rake angle significantly affects the PGD, the PGD varied in the range of approximately -45 cm to 45 cm. while the strike angle also affects the longitudinal and transverse PGD, but within the range of -25 cm to 25 cm.

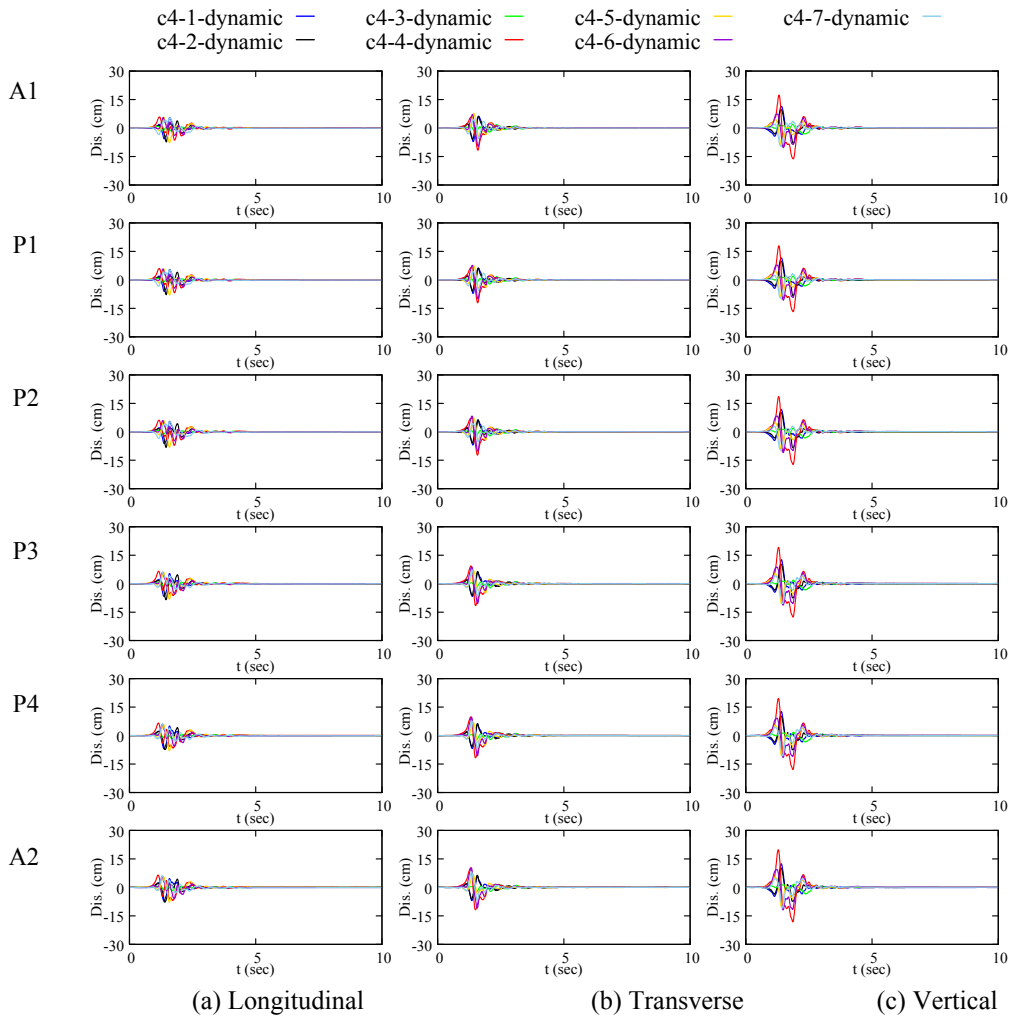


Figure 5-39 Input GM under different rake angle γ with strike angle 45° (only dynamic)

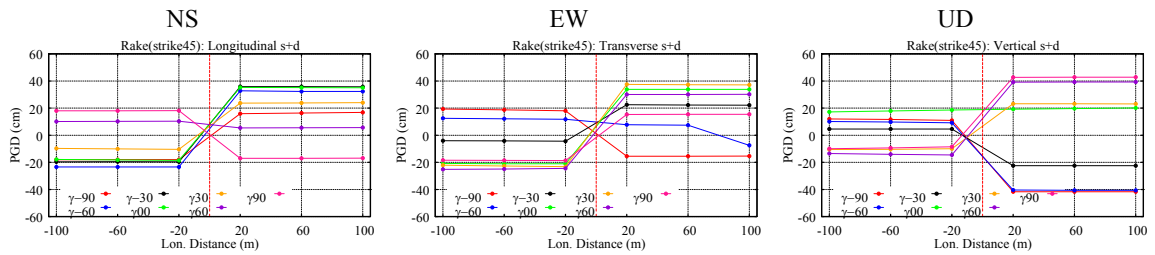


Figure 5-40 PGD under different rake angle (Strike 45°) (static + dynamic terms)

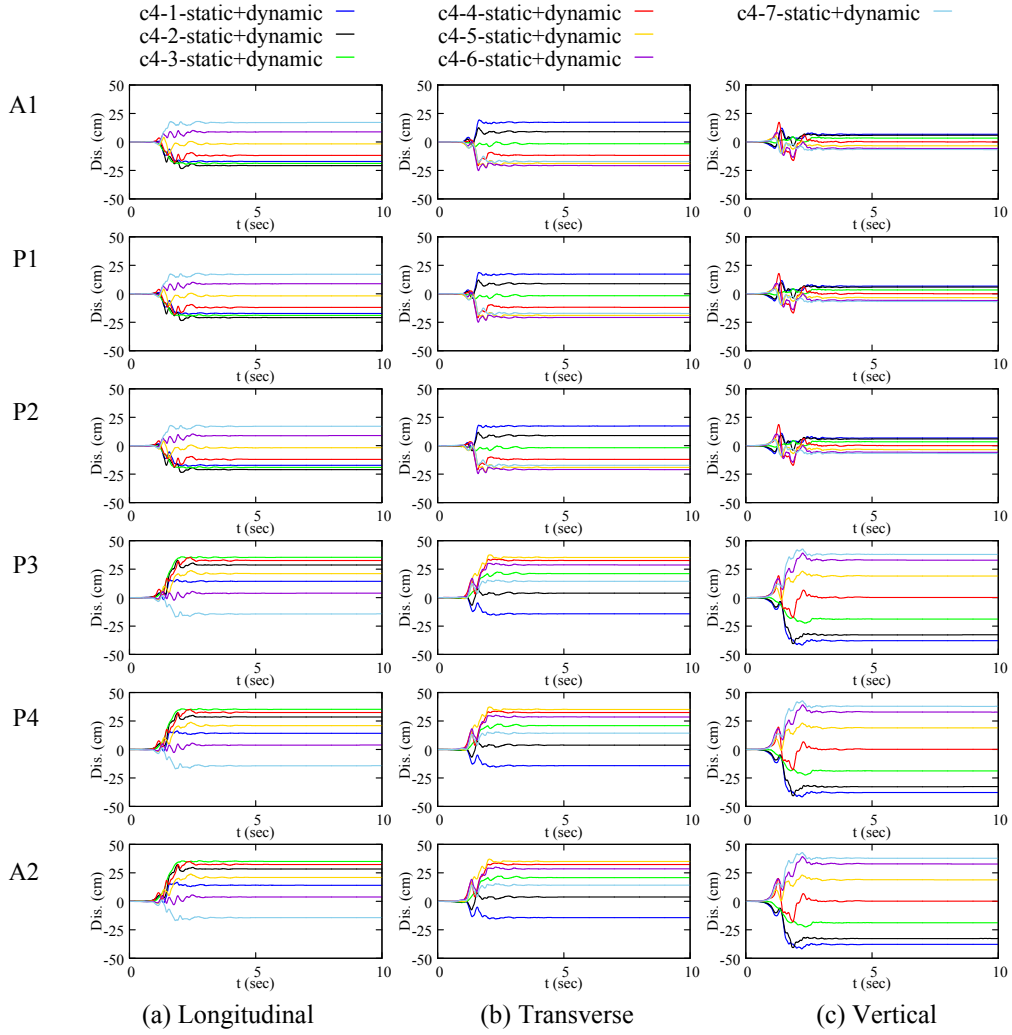


Figure 5-41 Input GM under different rake angle γ with strike angle 45° (static + dynamic)

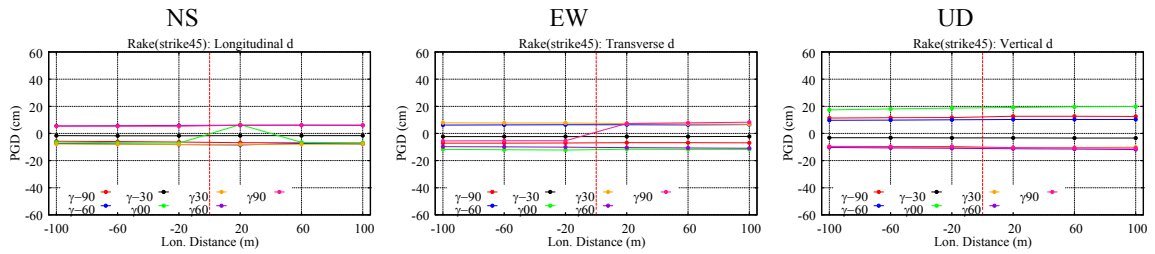


Figure 5-42 PGD under different rake angle (Strike 45°) (only dynamic terms)

5.5.2 Girder: bending moment

The bending moment of girder are analyzed under the 4 cases earthquake excitations mentioned above.

The case 1 is the case study focusing on the effect of different strike angle. The results are summarized in Figures 5-43 ~ 5-46. In all these figures, the left results are the bending moments of girder only consider the dynamic terms, and the right listed the results with considering both the static and dynamic terms. It is seen clearly, the strike angle significantly affect the bridge response both for only dynamic and the dynamic + static terms. The following various could be obtained in this analysis:

(a) It can be seen clearly, the responses of bridge under the excitation considering fling-step displacements are significant different from the response only considering the dynamic terms, and the former are larger than the latter ones, especially the response of piers (P2 and P3) located very near the fault due to this is a dip-slip fault, and the large relative surface displacements cause more severer damages, which is closer to the real situations.

(b) In Figure 5-43, along with the increasing of α (strike angle), the response of M_y also increasing, while the M_y varied larger when α changing from 0° to 30° than the α from 45° to 75° , which means the larger the crossing angle between the longitudinal direction of bridge and the strike angle of fault, the larger the response of M_y . And the varied strike angle slight affect the response of M_z in this analysis due to this is a pure-dip-slip fault, thus the increasing α didn't induce significant changes on the vertical components.

(c) In Figure 5-44, the dip angle β extremely attributed to the large responses of both M_y and M_z .

(d) Rake angle γ under two situations (strike angle $\alpha = 0^\circ$ and $\alpha = 45^\circ$) also yield dramatic different response of both M_y and M_z as shown in Figures 5-45 ~ 5-46, particularly the relative bending moment between P2 and P3 due to the large relative permanent displacements occurred.

From the above, the dip angle and rake angle are the factors which extremely affect the bending moment of girder compared with the strike angle.

1) Effects of strike-angle

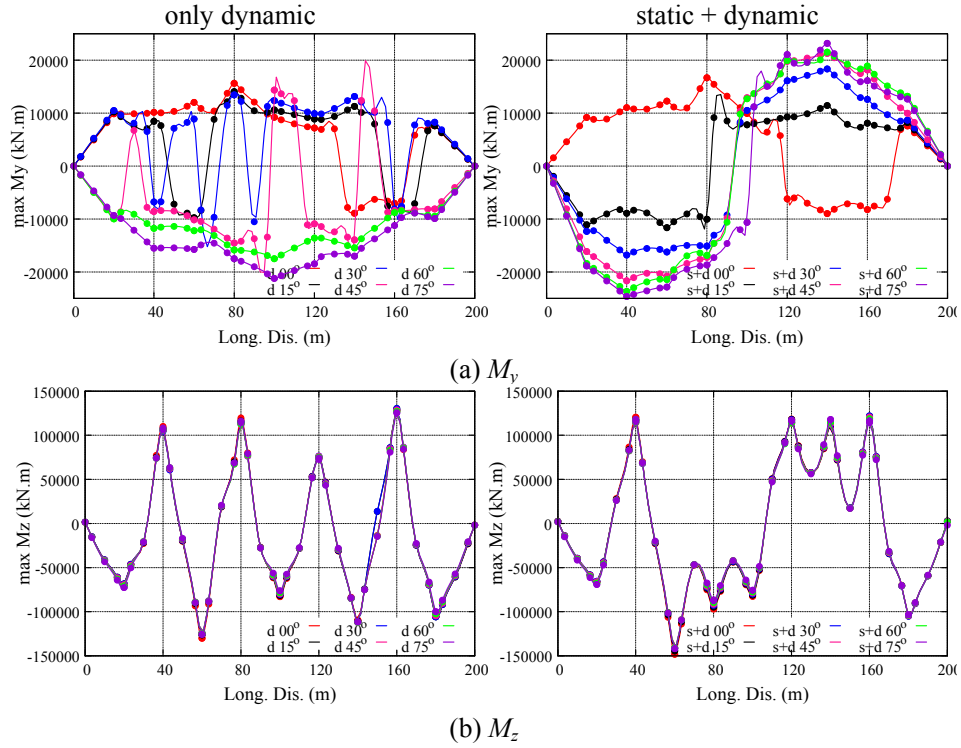


Figure 5-43 Case 1: Bending moment of girder under different strike angle α

2) Effects of dip-angle

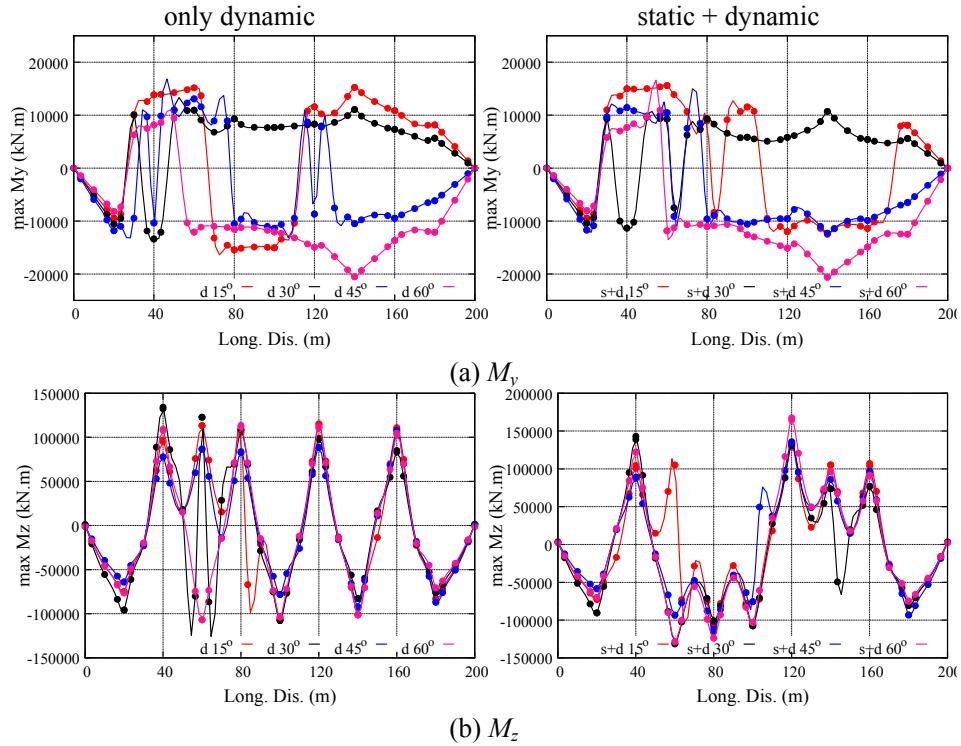


Figure 5-44 Case 2: Bending moment of girder under different dip angle β

3) Effects of rake-angle (1)

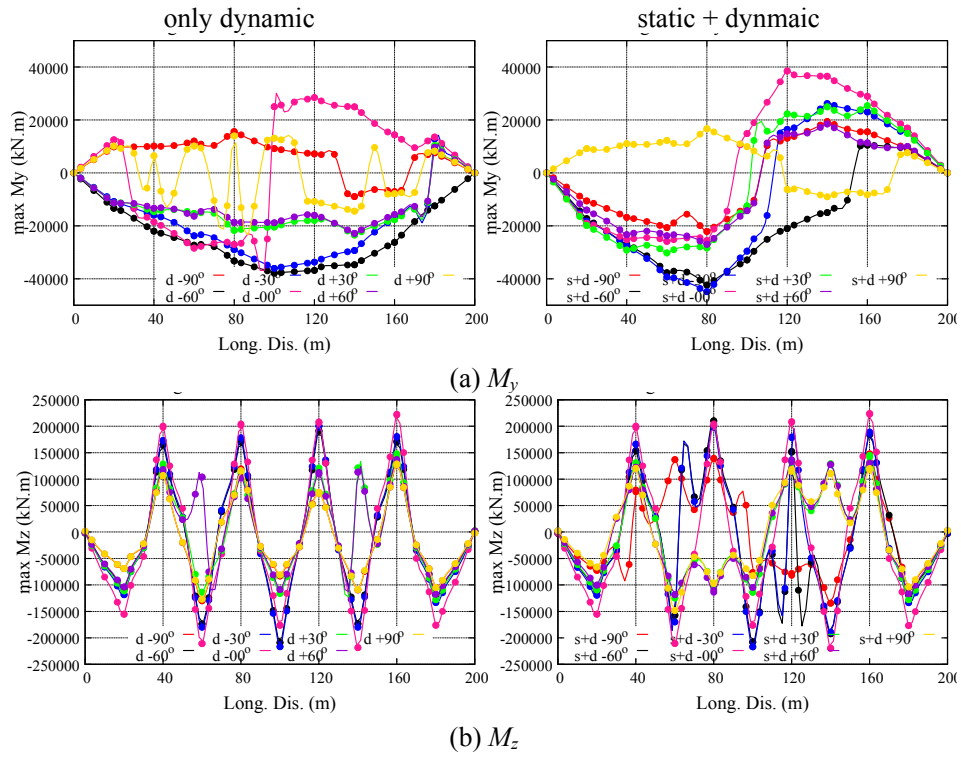


Figure 5-45 Case 3: Bending moment of girder under different γ with strike angle 0°

4) Effects of rake-angle (2)

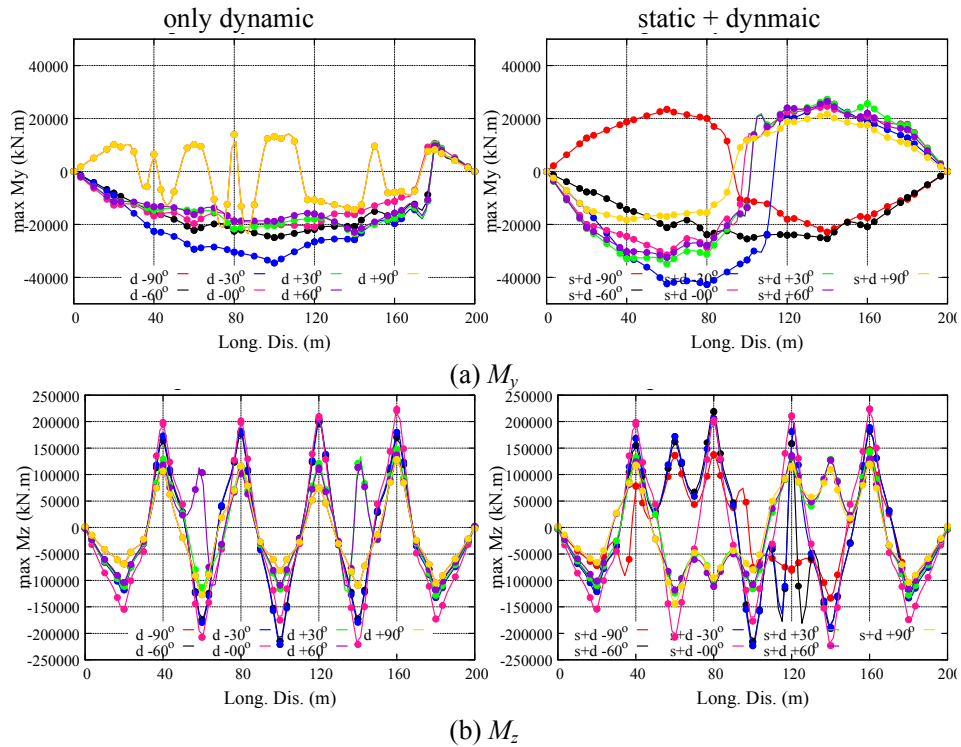


Figure 5-46 Case 4: Bending moment of girder under different γ with strike angle 45°

5.5.3 Pier: bending moment

The bending moments of the four piers (P1 ~ P4) under these 4 cases both considering only the dynamic and both static and dynamic displacements are summarized in this calculation, and the results are presented in Figures 5-47 ~5-54 below, which can be found that:

(a) The maximum response of each pier under the situations only the dynamic and both static and dynamic terms are not large, but it should be noted that the direction of bending moment are different when considering the combination of static and dynamic terms, particularly along the transverse direction, the large relative response of bending moment might cause very relative displacements and deformations, and further to trigger several damage of bearings and superstructures, or even the collapse of bridge. It should be noted that the maximum values might not occurred at the same time.

(b) Accordingly the M_x is much larger than M_z due to its larger stiffness along transverse direction.

1) Effects of strike-angle

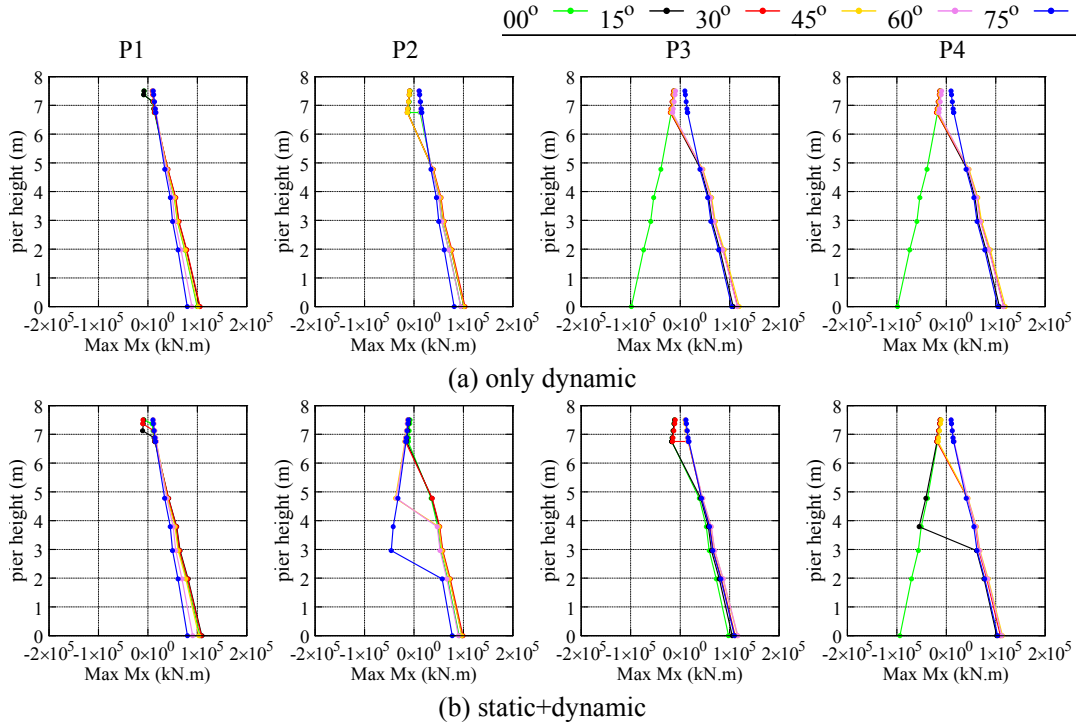


Figure 5-47 Case 1: Bending moment (M_x) of pier under different α

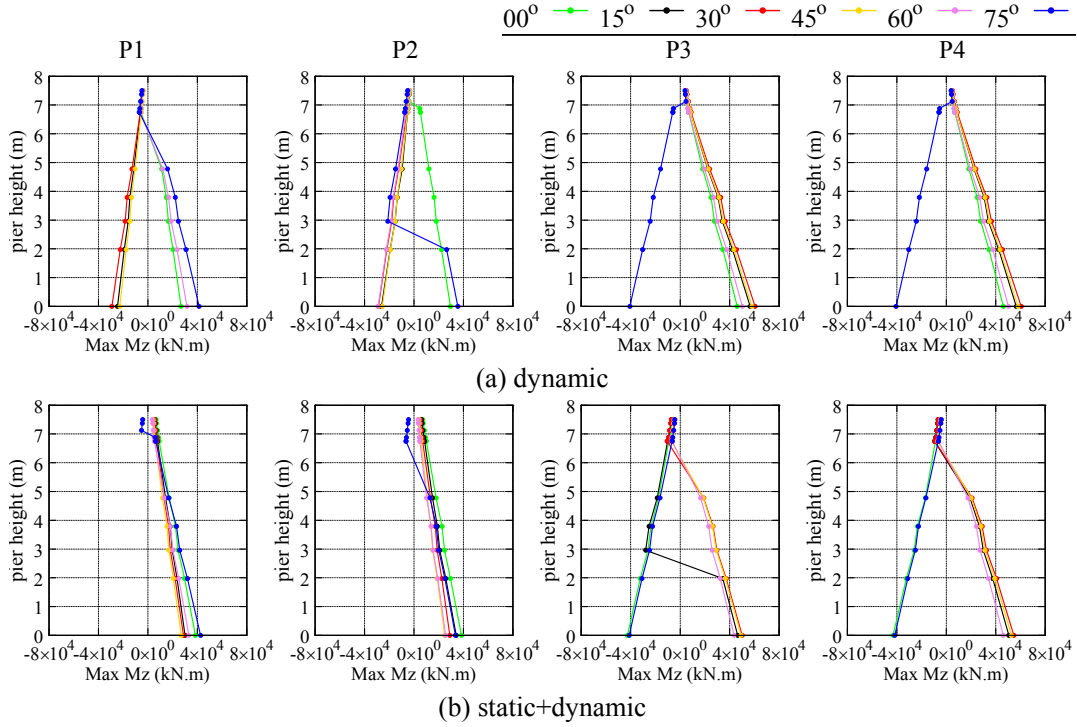
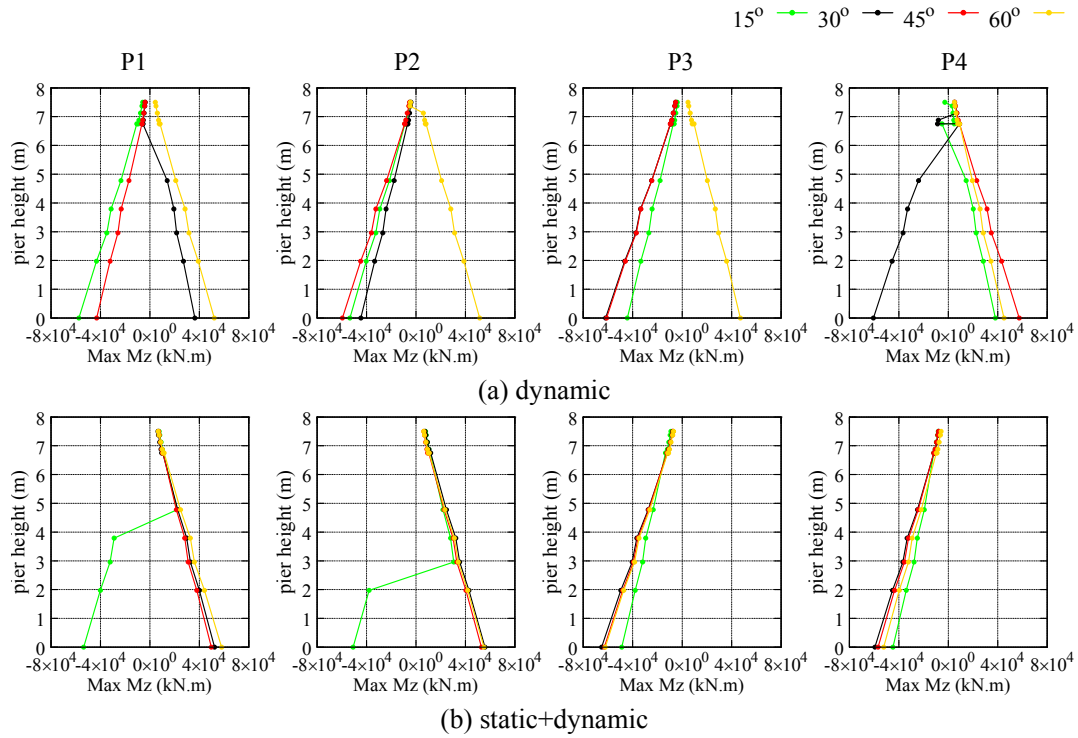
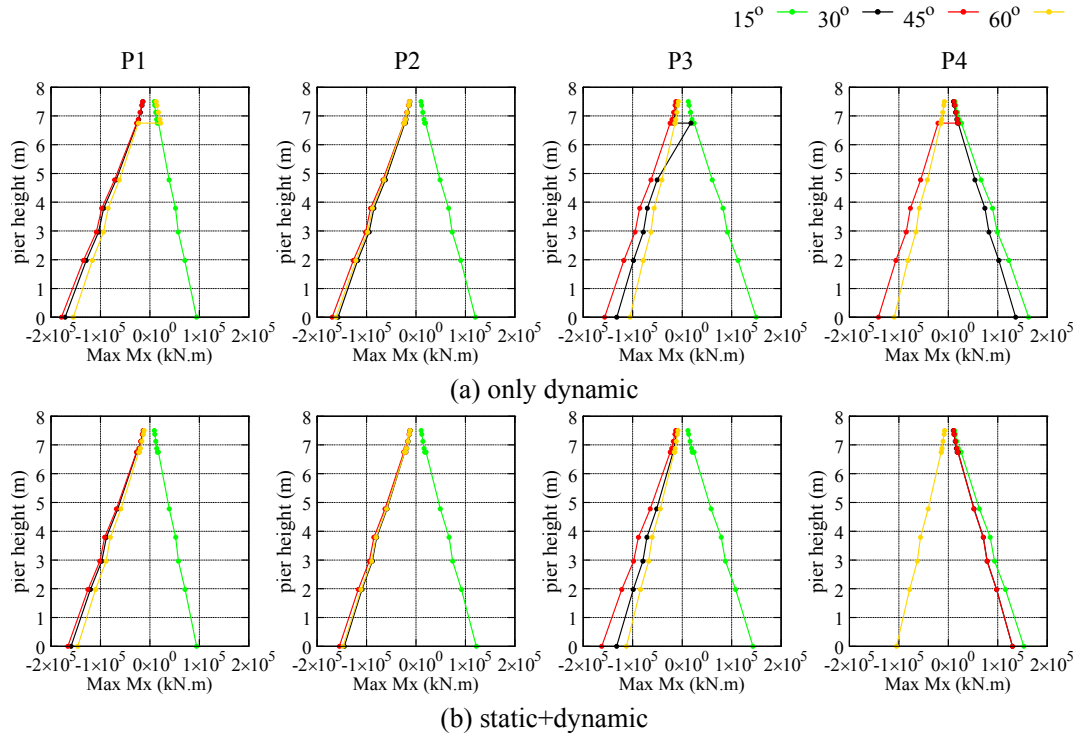


Figure 5-48 Case 1: Bending moment (M_z) of pier under different α

2) Effects of dip-angle



3) Effects of rake-angle (1)

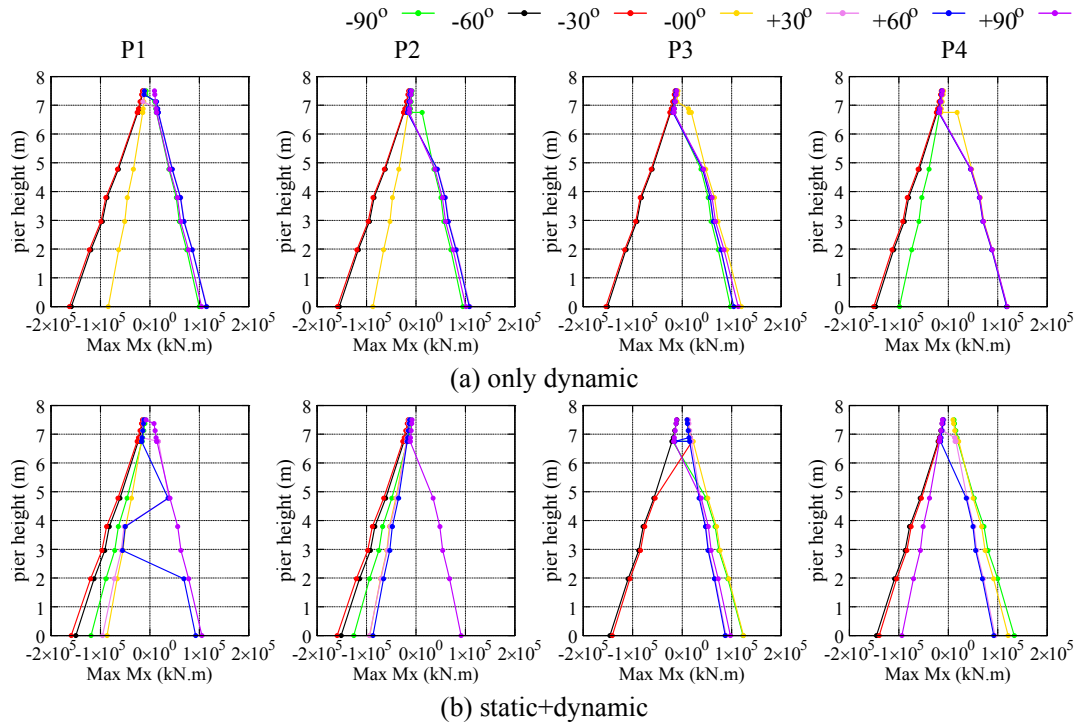


Figure 5-51 Case 3: Bending moment (M_x) of pier under different γ

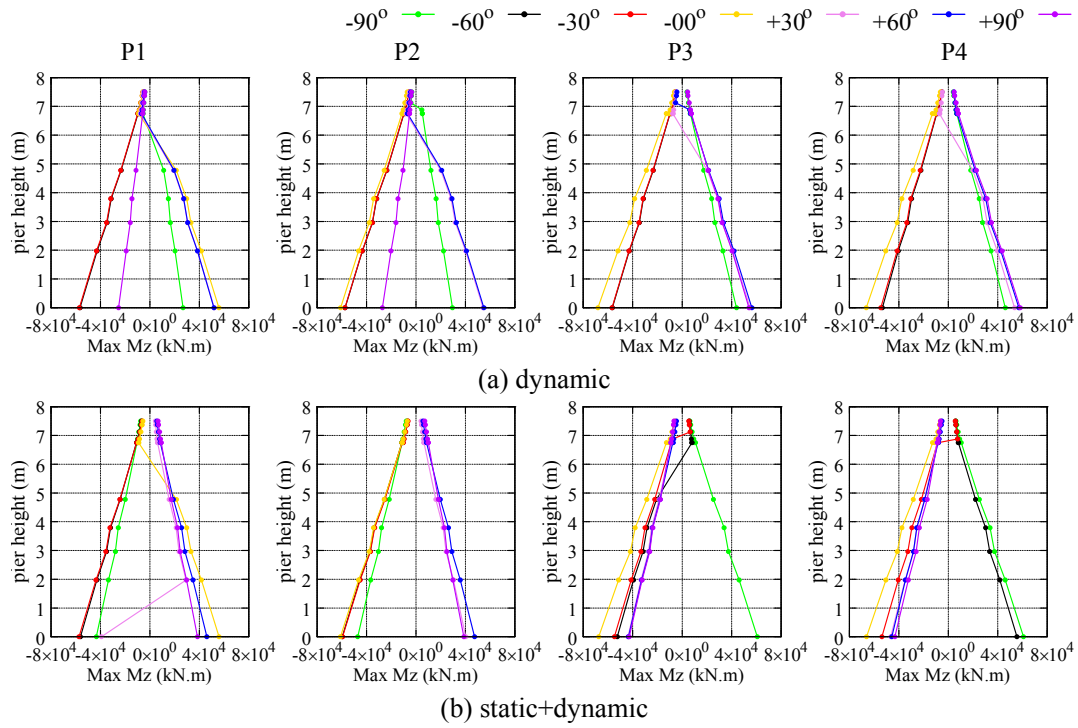


Figure 5-52 Case 3: Bending moment (M_z) of pier under different γ

4) Effects of rake-angle (2)

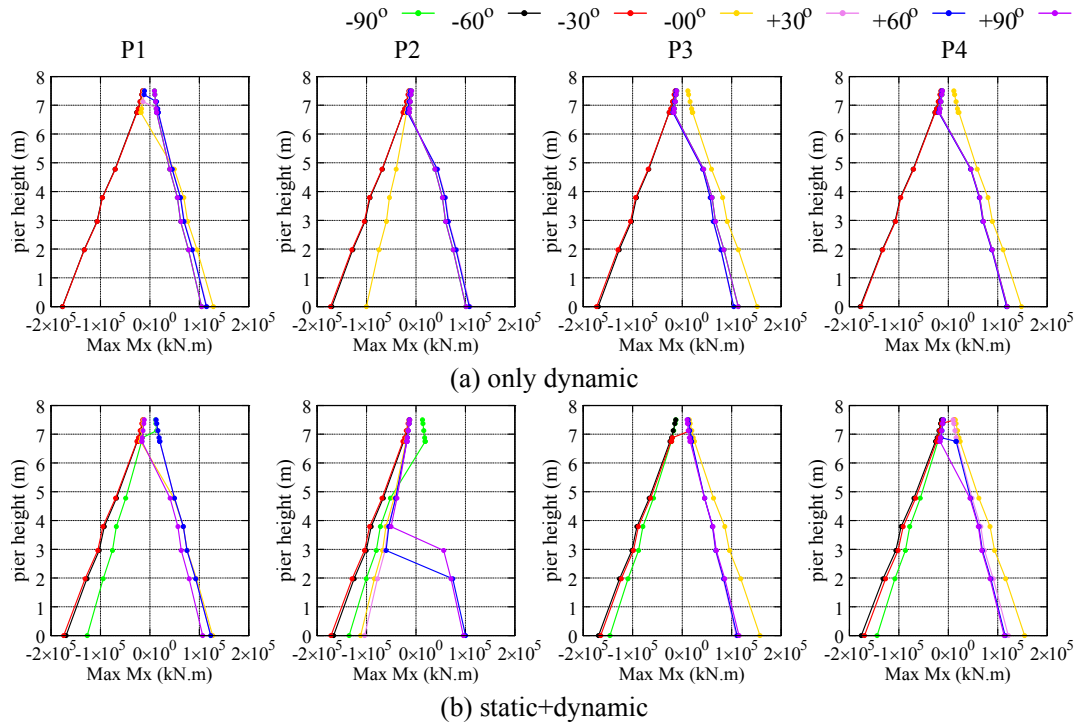


Figure 5-53 Case 4: Bending moment (M_x) of pier under different γ

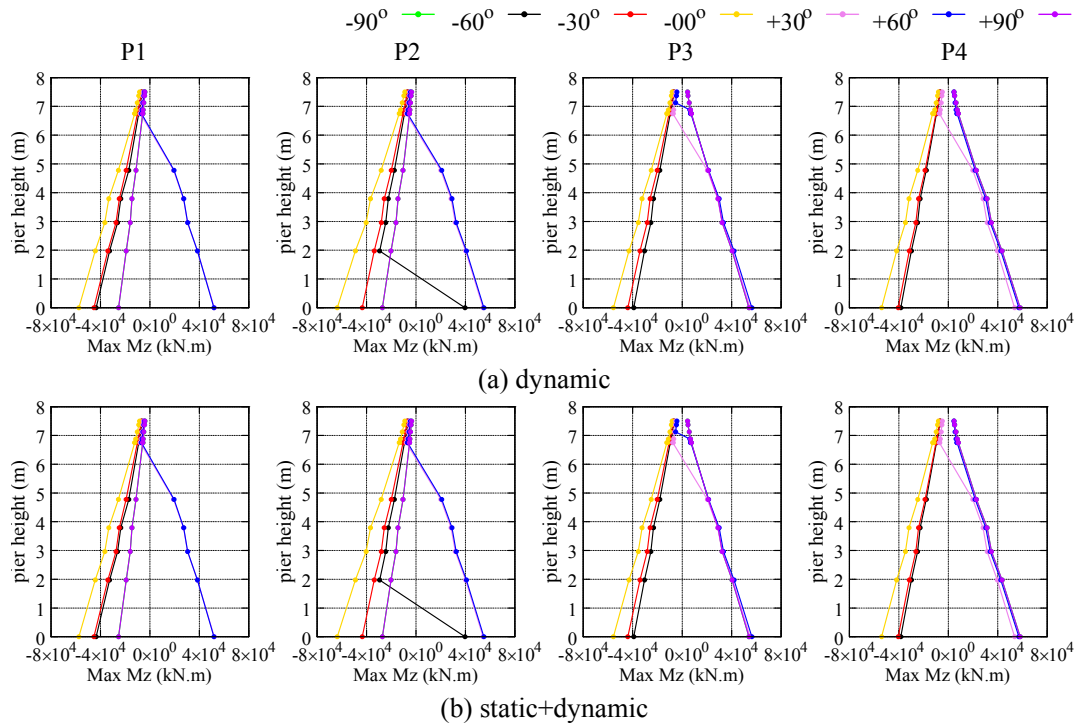


Figure 5-54 Case 4: Bending moment (M_z) of pier under different γ

5.5.4 Pier: curvature

The effects of strike angle, dip angle and rake angle on seismic response of pier curvature are conducted in this example, and the results are illustrated in Figures 5-55 ~ 5-62 as the following. And we can obtain the rules as below:

(a) The effects of the strike angle, dip angle and rake angle significant differences along maximum C_z (curvature along longitudinal directions) are larger the maximum C_x (curvature along transverse direction of bridge) mainly due to the relatively large stiffness along transverse directions. Thus, the effect of various strike angle, dip angle and rake angle slight affect the response of C_x (curvature along transverse direction). The same tendencies as the bending moment along the longitudinal direction, the large relative response between P1 ~ P2 and P3 ~ P4 can be obtained, especially the plastic hinge area;

(b) The differences between the two calculations methods, only the dynamic terms and by the hybrid synthesis method cause the significant differences on C_z , generally, only considering the dynamic terms, the maximum curvature of plastic hinge are within 0.01/m, while by the hybrid synthesis method, the C_z is closer to or larger than 0.01/m and even over 0.02/m in several situations, for example, when $\beta=15^\circ$, and when $\gamma=-30^\circ + \alpha=45^\circ$.

1) Effects of strike-angle

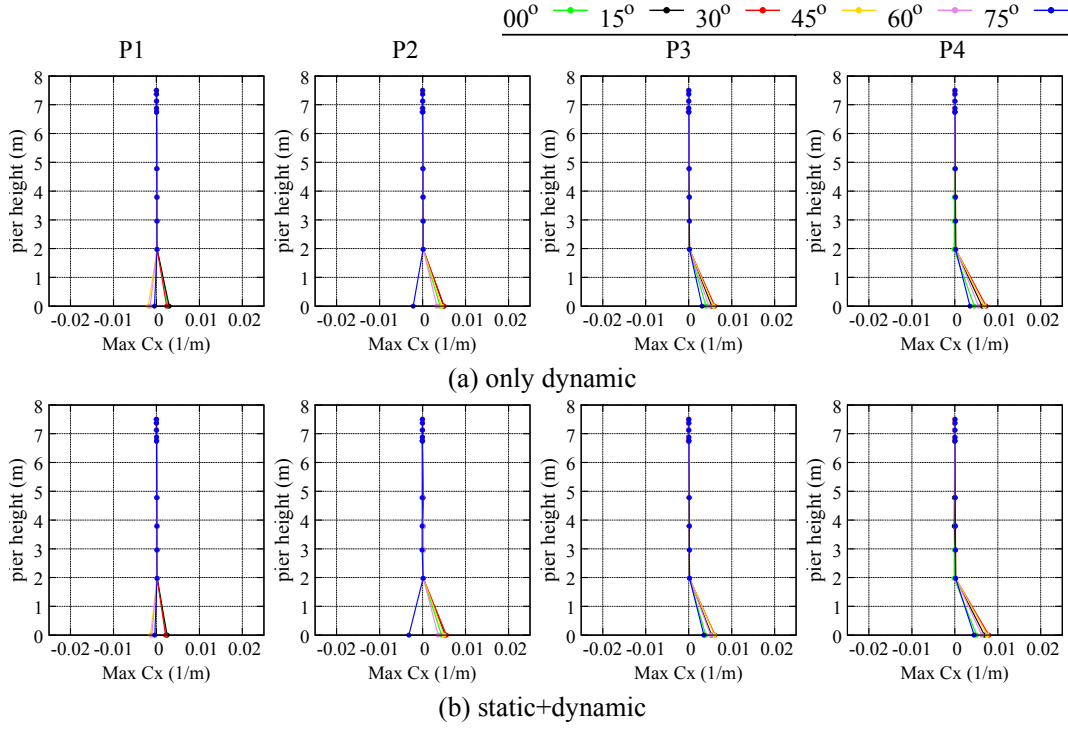


Figure 5-55 Case 1: Bending moment (C_x) of pier under different α

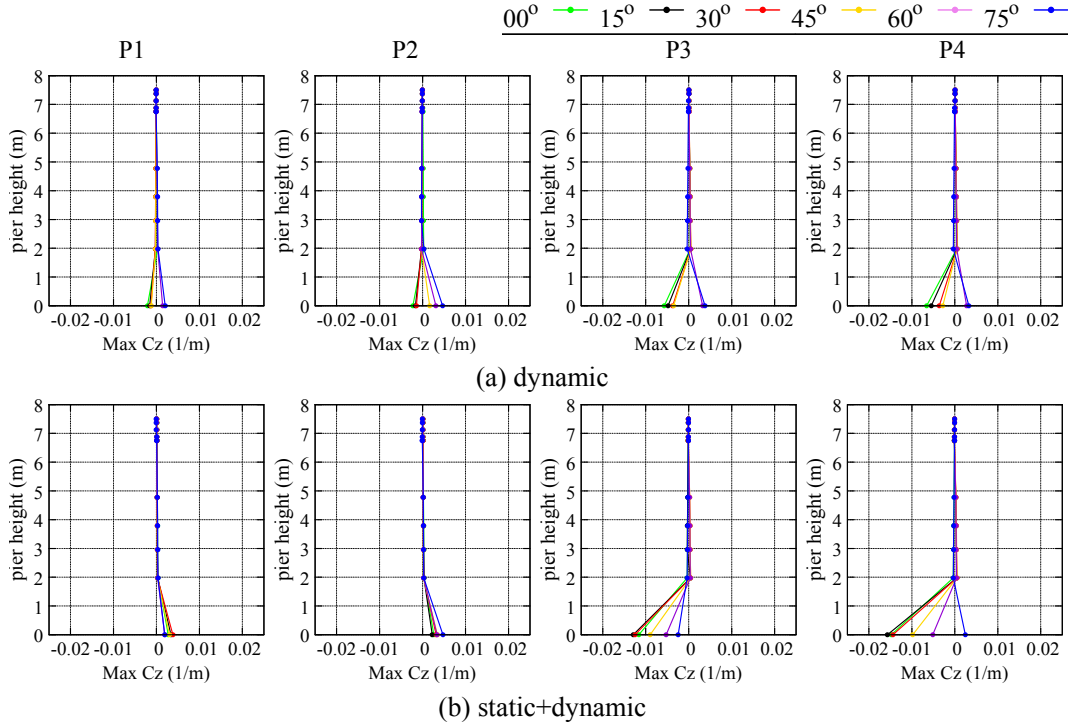


Figure 5-56 Case 1: Bending moment (C_z) of pier under different α

2) Effects of dip-angle

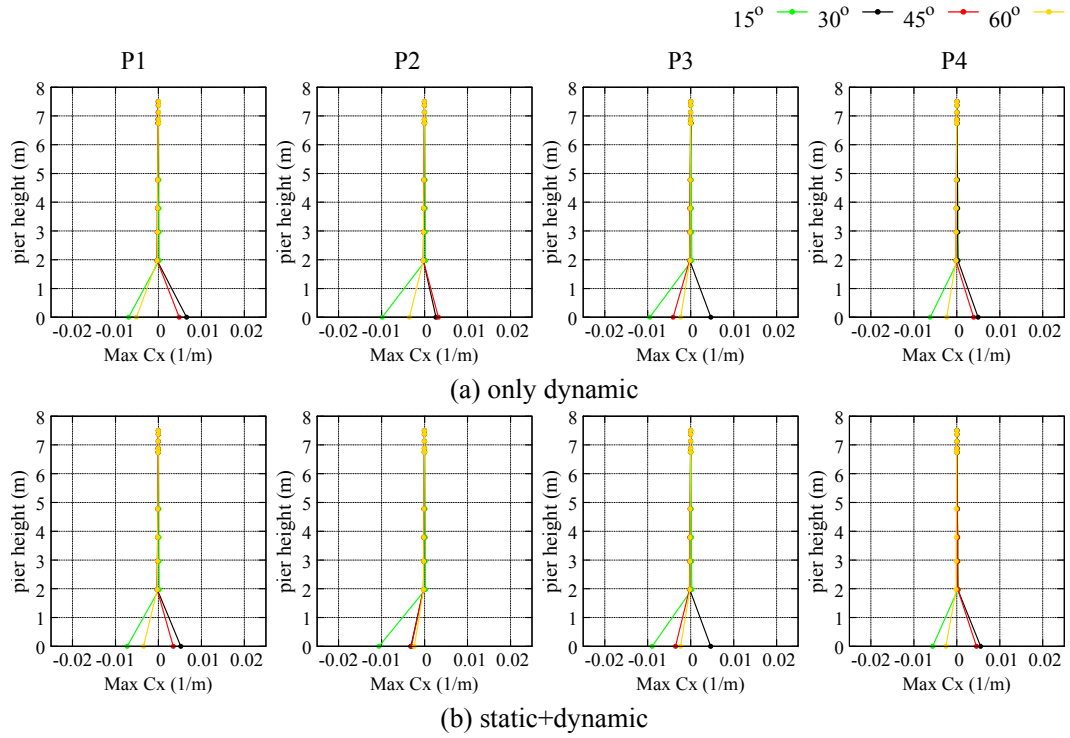


Figure 5-57 Case 2: Bending moment (C_x) of pier under different β

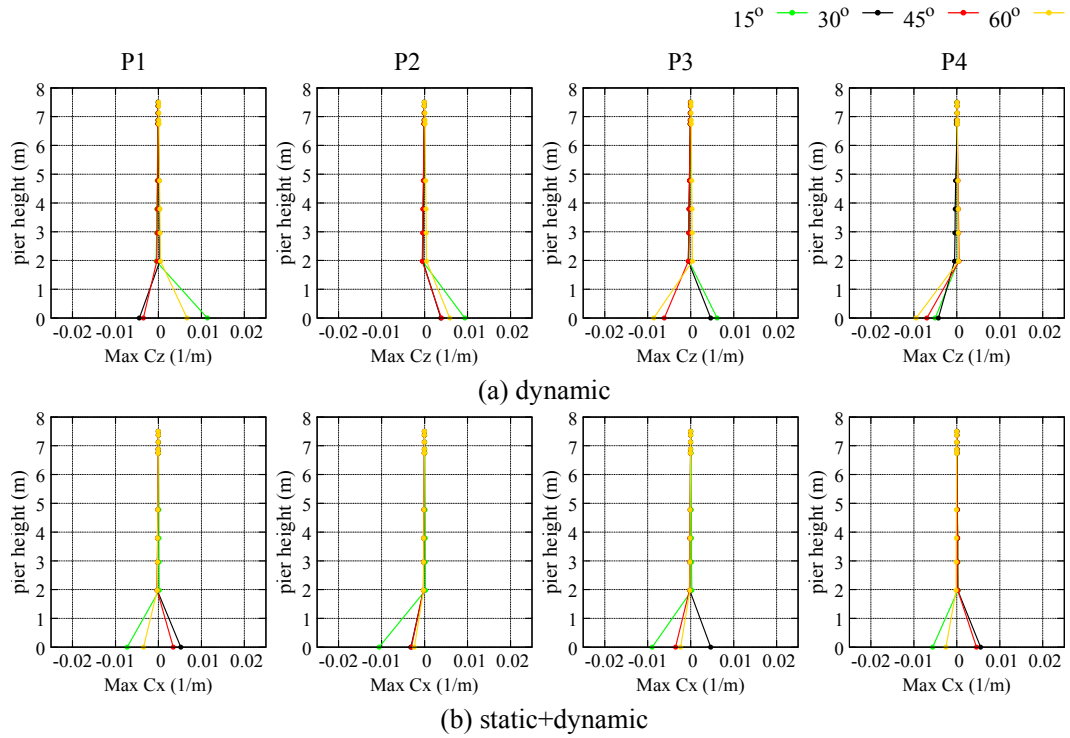


Figure 5-58 Case 2: Bending moment (C_z) of pier under different β

3) Effects of rake-angle (1)

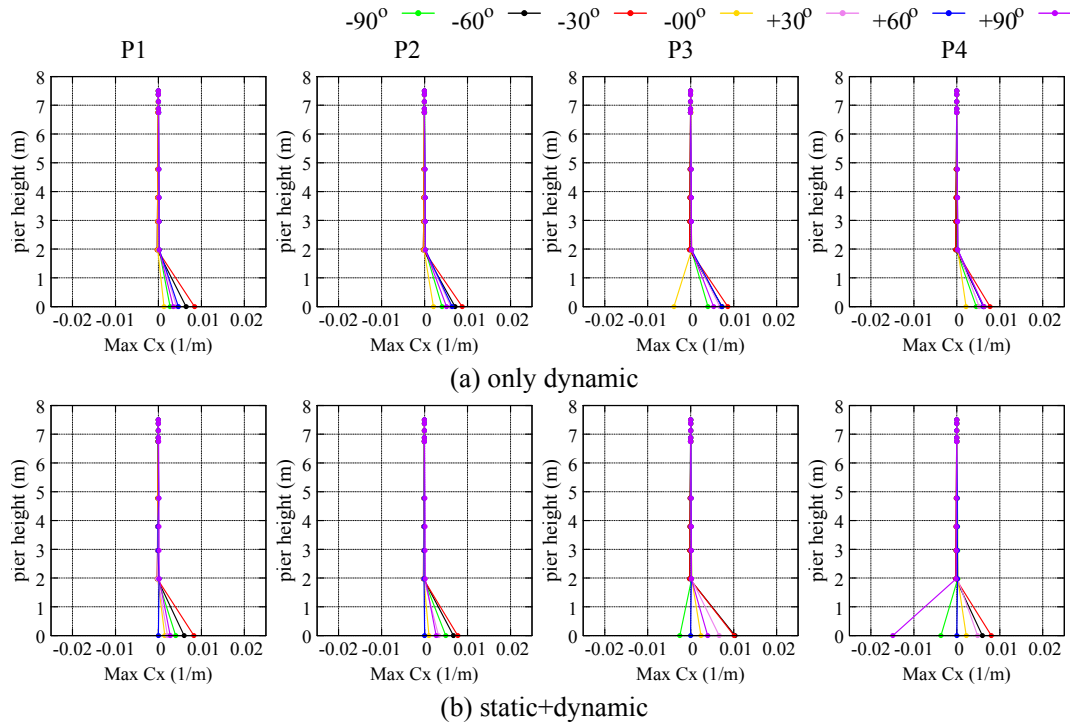


Figure 5-59 Case 3: Bending moment (C_x) of pier under different γ

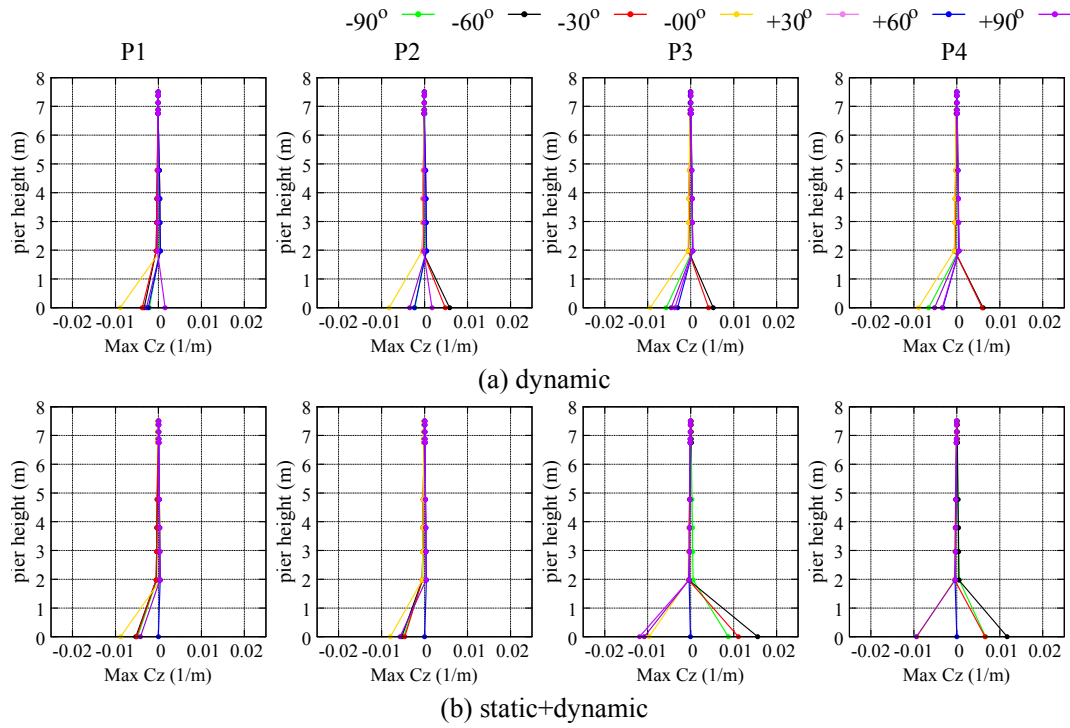


Figure 5-60 Case 3: Bending moment (C_z) of pier under different γ

4) Effects of rake-angle (2)

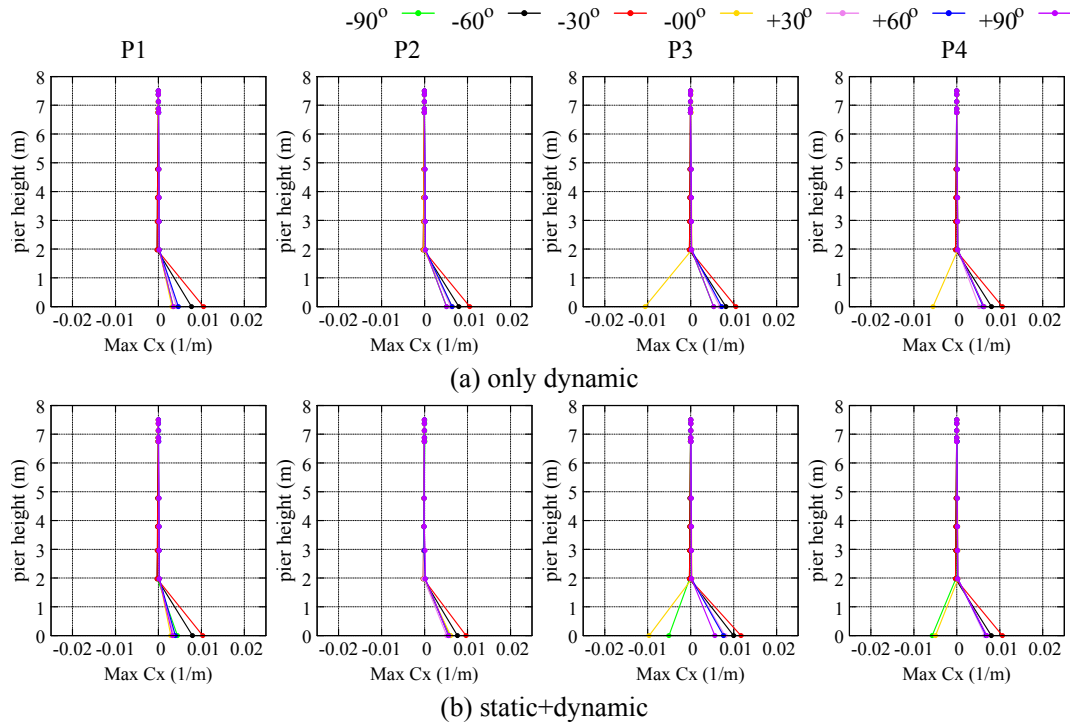


Figure 5-61 Case 4: Bending moment (C_x) of pier under different γ

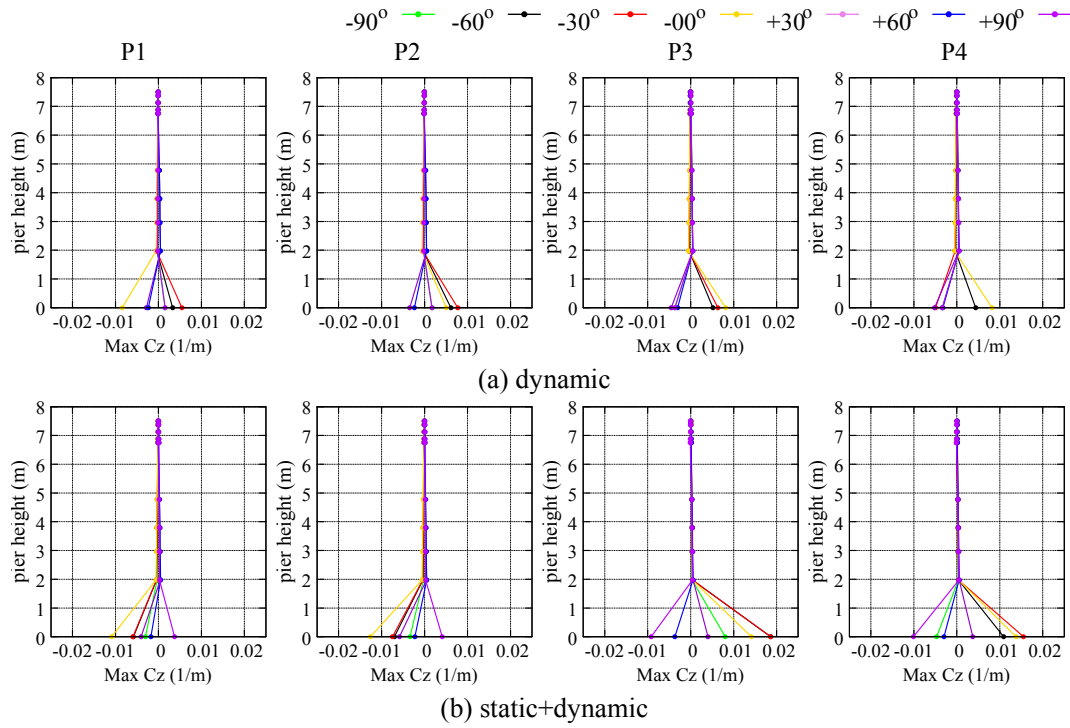


Figure 5-62 Case 4: Bending moment (C_z) of pier under different γ

5.5.5 Maximum strain of piers under 4 cases

The earthquake excitation might cause significant different seismic behaviors of bridge structures and/or the components, especially the nonlinear behavior of piers, if considering the axial forces might change during the earthquake. Due to this effect of axial forces changing on curvature has not been considered in this numerical calculation, which was mentioned in Section 5.5.3, thus the strain or normalized strain (the ratio to yield level strain) was recommend to evaluate the seismic behavior of piers, which is the most objective factors for evaluating the seismic response of piers, and its strain distribution within initial yield limit and the limit state of seismic performance Level 2 or 3 is shown in Figure 5-63.

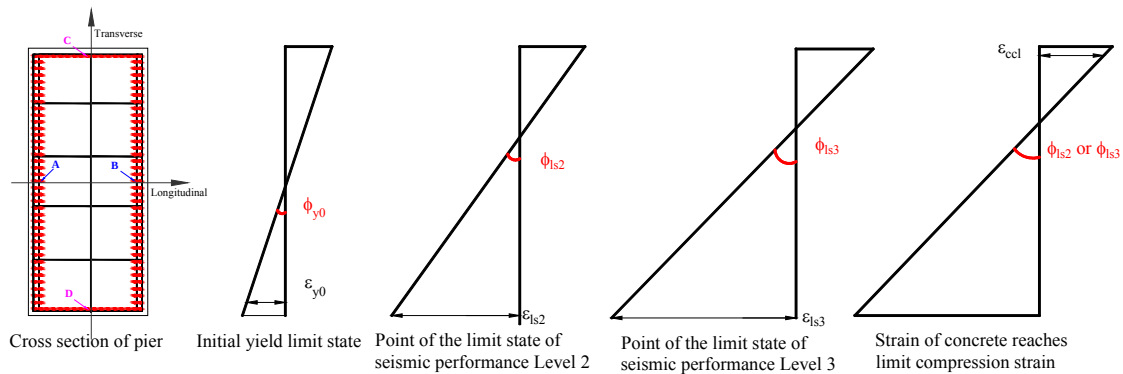


Figure 5-63 Strain distribution within initial yield limit and the limit state of seismic performance Level 2 or 3 [6]

In this analysis, under the four cases (the various strike-, dip-, and rake-angle), the strains of four points in the cross section of piers: A and B, C and D along longitudinal and transverse direction have been monitored, and the maximum normalized strain have been obtained listed in Figure 5-64 ~ 5- 67 (a) and (b), in the following, and the related time history of maximum strain of the points (A and B or C and D) also plotted in Figure 5-64 ~ 5-67 (c) and (d), it should be mentioned, mainly due to P2 and P3 are very located very close to the fault, thus the plotted results are focused on these two piers. From Figure 5-64 ~ 5-67, the following items can be obtained:

(a) The effect of dip angle and rake angle affected the results remarkably: the dip angle caused the Point A of pier 2 under β equals to 15° with static and dynamic terms even beyond the performance Level 3, and Point A of pier 3 are close to the performance level 3, this is the most serious situation, which could not be obtained when evaluated by curvature or displacement as mentioned above.

(b) Except the results in (a), there are also some cases over the performance level 2, e.g., in Figure 5-64 (a) in case 1, under strike angle 75° , pier 2 under both only dynamic and considering

static and dynamic terms are over or closed to performance level 2; and in Figure 5-67 (a) in case 4, under rake angle of 0° and strike angle 45° (strike-slip fault), the normalized strain of P3 is slightly over the performance level 2 under excitation considering static and dynamic terms; and in Figure 5-67 (c), under rake angle of 60° and strike angle 45° (strike-slip fault), the normalized strain of P2 is close to the performance level 2 also under the earthquake excitation with static and dynamic terms.

Base on the above results, the evaluation on the seismic behavior of piers are different by curvature and normalized strain, the performance level for former are calculated by push-over analysis, and the changing of axial-forces has not been considering in the above analysis. Thus, it resulted the push-over analysis overestimated the seismic response of piers. On one hand, the axial-force changing effect should be well considered for seismic analysis of structures. On the other hand, due to the axial changing force is not convenient to practical engineering design, and the strain (or normalized strain) of concrete exhibited liner behavior as compared with its stress, the strain (or normalized strain) is more objective and recommend in evaluations or judgements of bridge behavior.

1) Effects of strike-angle

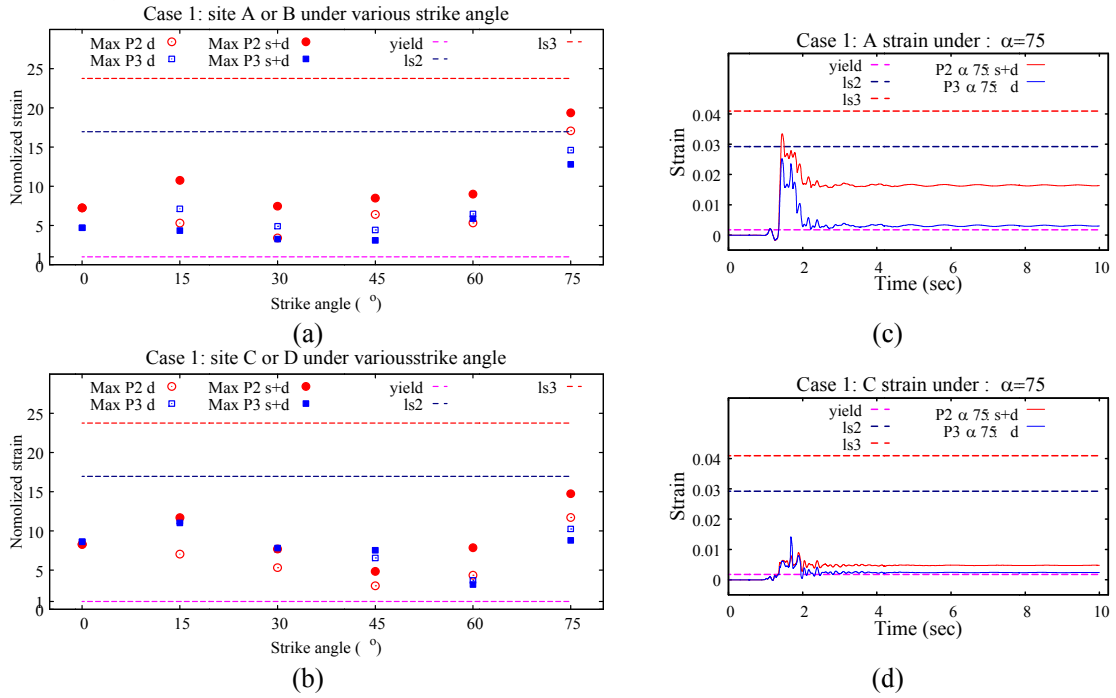


Figure 5-64 Case 1: For, Pier 2 and Pier 3, maximum normalized strain distribution (a) and (b) and related time history of maximum strain (c) and (d)

2) Effects of dip-angle

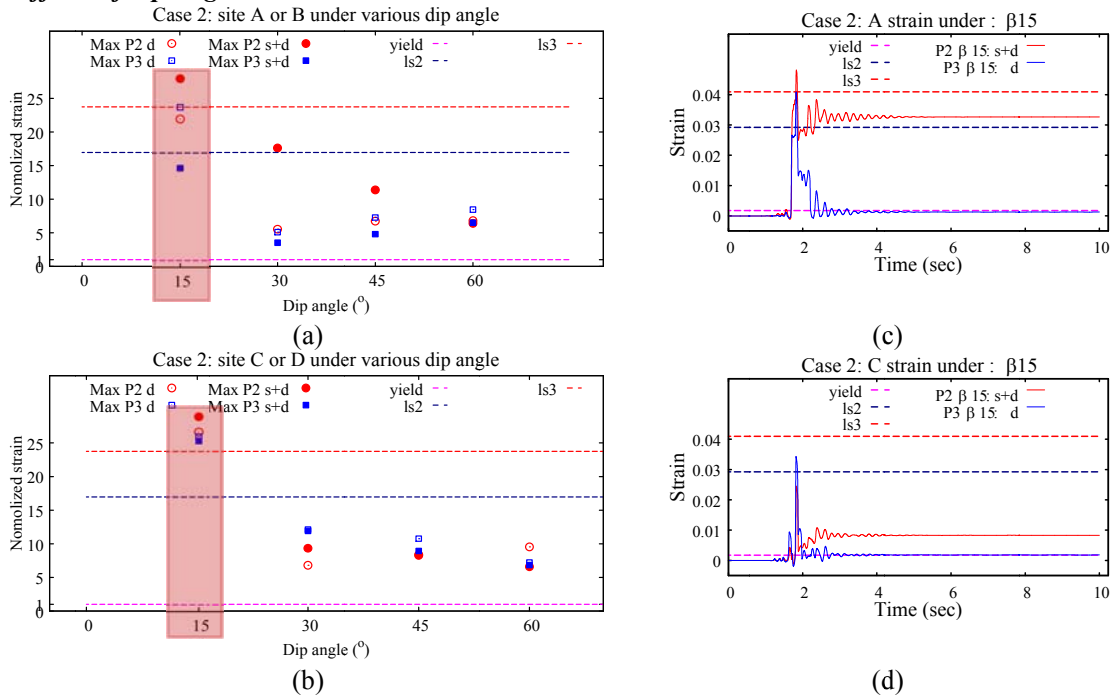


Figure 5-65 Case 2: Pier 2 and Pier 3, maximum normalized strain distribution (a) and (b) and related time history of maximum strain (c) and (d)

3) Effects of rake-angle (1)

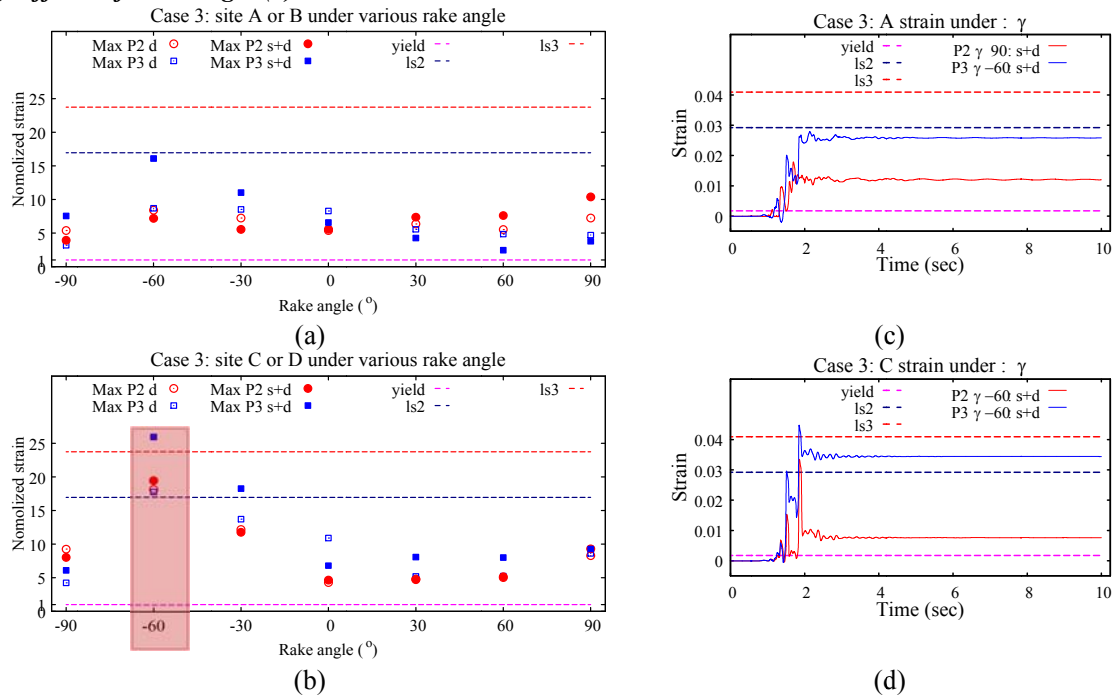


Figure 5-66 Case 2: Pier 2 and Pier 3, maximum nomorlized strain distribution (a) and (b) and related time history of maximum strain (c) and (d)

4) Effects of rake-angle (2)

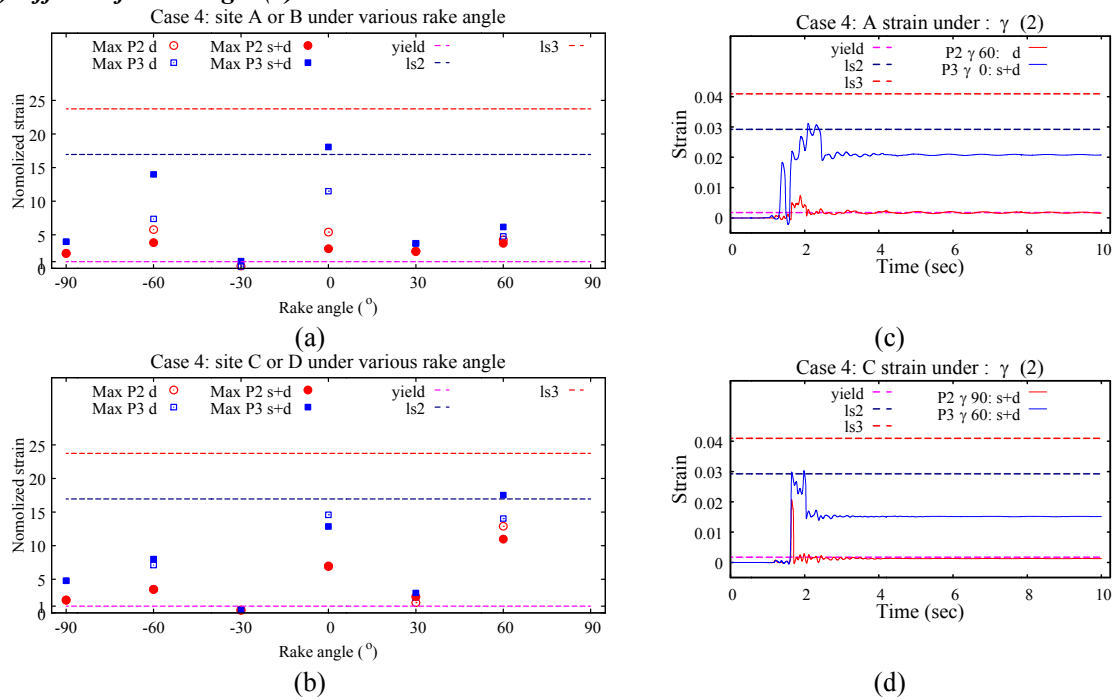


Figure 5-67 Case 2: Pier 2 and Pier 3, maximum nomorlized strain distribution (a) and (b) and related time history of maximum strain (c) and (d)

5.6 Conclusions

Chapter 5 described the seismic design procedure considering the fling-step (permanent or static) displacement on a simplified 5-span bridge crossing an active fault based on multi-support excitation method, and the OpenSees was performed as the important analysis tool for seismic analysis of this bridge. The linear analysis was achieved under the earthquake excitation with and without fling-step displacements, and non-linear analysis was conducted under three input waveforms (both the static and dynamic terms, only dynamic terms, and the ground motion from JRA), a complete comparison on the seismic response of bridge components (superstructure, pier and bearing) among these three cases was conducted. The linear and non-linear analysis could acquire the following important points:

(1) Firstly, the linear analysis on this 5-span simplified bridge to check the probabilistic of the displacement time histories input, the bridge model located near fault exhibits large response values, especially under the input containing fling-step static terms, which should be carefully considered when designing bridge. Based on the results, the proposed hybrid synthesis method in Chapter 3 can be employed to simulate the near-fault displacements, and comparisons between with and without considering the static displacement results that, the non-linear analysis is necessary for such near-fault bridge structures.

(2) Secondly, a non-linear analysis, mainly the non-linear behavior of bearing and plastic hinge range of piers, on this 5-span bridge was calculated by OpenSees, which illustrated that large displacement on the bearing caused its break; for evaluation on behaviors of piers, the push-over method might underestimate the performances of piers under the evaluation by deformation as compared with judged by curvatures; the significant differences are bearing behavior along longitudinal direction, which indicated that the isolation system is more sensitive than the members of bridges.

(3) Based on the linear and non-linear analysis on the 5-span fault-crossing bridge, it obviously demonstrated that, the residual response (deformation, curvature) for girder, pier and bearing are larger when considering the fling-step displacement than by the conventional method, also dynamic analysis considering only accelerations cannot estimate residual moments and displacements, and it is necessary to conduct dynamic analysis using displacement time histories to gain more exact seismic behaviors of bridge structures during earthquake.

(4) Furthermore, the effects of dip-, strike-, and rake-angles on the fault-crossing bridge provided that, the strike and rake angle extremely affect the seismic behaviors, etc., bending moment of girder, maximum curvature of piers, which indicated the large crossing angle (bridge and fault) should be avoided in designing, the dip angle has a significant affect on the vertical components of the

ground waveforms and the seismic response of the bridge; if the dip angle is smaller than 15° , the vertical components are slightly smaller or even equal to the horizontal components, while when the dip angle is larger than 45° , they become larger than the horizontal terms. Therefore, the vertical responses are correlated with the dip angle and also need special attention.

(5) Due to limit state of performance level for curvature didn't consider the changing axial force effect; the normalized maximum strain for piers is good option for seismic performance evaluation.

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Chapter 6 Conclusions and recommendations

6.1 Conclusions

The main objective of this paper is to make clear the characteristics of near-fault ground motions. Additionally, it is to propose a new hybrid synthesis method (considering the pulse-like velocity and fling-step displacements, complete waveform solutions) on near-fault ground motions. Also, based on the generated near-fault ground motions, an analytical procedure to conduct the seismic analysis and designing on near-fault structures, especially the long-span bridges, was proposed. The results well showed that, this analytical procedure are applicably and can make good reference for the near-fault structures designing.

6.1.1 Clarify the characteristics of near-fault ground motions

Based on the recording database and analysis, due to the complex of near-fault ground motions is very different from the far-fault ground motions, it detailed the characteristics of near-fault ground motions, mainly contains the concentration effects, the pulse-like velocity effects, the permanent displacement (fling-step) effect, the hanging-wall effect, and directivity effects. Even though not all these effects might occur during one earthquake, but these effects can generate the spatial varied near-fault ground motions and caused damage on near-fault bridge engineering.

The field survey on the 2016 Kumamoto Earthquake further clarified the damage of near-fault ground motions on structures and makes clear its characteristics, and the evaluations resulted that, under the high frequency, high amplitude, and especially the impulse-like and permanent displacement ground motions, among the 138 buildings, 36 houses without damage, 13 houses with slight damage, 6 with moderate damage, 7 with heavy damage, and 76 houses are totally collapsed. The building exhibited collapse, rotation with large cline angle, shear failure of columns, large relative rupture when cross the fault, which showed the complicated characteristics of near-fault ground motions, and the collapsed exhibit the direction related to the near-fault radiation pattern.

6.1.2 Hybrid synthesis method on near-fault ground motion

Numerous studies have focused on this topic. While up to now, no rational seismic design philosophy has been established for structures crossing active faults. Based on previous researches, this research proposed a hybrid synthesis method on near-fault ground motions; it combines the most commonly used stochastic Green's function method and the theoretical Green's function method.

To evaluate the effectiveness of our approach, the proposed method is applied to two simple fault mechanisms, the strike- and dip-slip faults. Finally, the 1999 Chi-Chi earthquake in Taiwan is simulated to further validate our method. The calculations resulted that, this method advantages that it can synthesize near-fault ground motions faster and more efficient than other methods. And the

displacements obtained could provide the input displacement for seismic design of road bridges across active faults.

6.1.3 Evaluation on near-fault ground motions by the hybrid synthesis method

The evaluation on near-fault ground motions generated by the proposed hybrid synthesis method was achieved. The 3D vectors of near-fault acceleration field, velocity field, and displacement field well demonstrated the characteristics of near-fault ground motions (i.e., hanging wall effect, pulse-like velocity, and permanent displacement), wave propagation or radiation pattern by this hybrid synthesis method.

Then, the evaluations on buried depth of fault plane, effects of soil models and focal mechanism and M_w , resulted that, the permanent displacements are not obvious (less than 3.0 cm) when the buried depth is more than 700m; , in hard rock model, the near-fault ground motions shaken with longer duration and small amplitude, while with shorter duration and large amplitude in soft rock model, and attenuated rapidly due to larger damping ratio; generally the larger the M_w , the larger the permanent displacement.

6.1.4 Seismic design method on fault-rupture crossing bridge

Near-fault seismic ground motion is of great importance in the design of structures in seismic areas. Relayed on the artificial near-fault ground motions generated by the proposed hybrid synthesis method in this research, the multiple-support excitation method was used to examine the seismic behaviors of the bridge subjected to three-directional ground motions (longitudinal, transverse, and vertical components). Through the numerical simulation under the ground excitations with and without permanent displacements by OpenSees, the linear analysis illustrated that, larger residual displacement and deformation can be obtained due to the permanent displacement than the conventional method.

The non-linear analysis was conducted under three input displacement waveforms (with static and dynamic terms, only dynamic terms generated by the hybrid synthesis method, and ground motion form JRA), the comparison on seismic responses demonstrated that, large displacement on the bearing caused the its break; for evaluation on behaviors of piers, the push-over method might underestimate the performances of piers under the evaluation by deformation as compared with judged by curvatures; the significant differences are bearing behavior along longitudinal direction, which indicated that the isolation system is more sensitive than the members of bridges.

Furthermore, the effects of dip-, strike-, and rake-angles on the fault-crossing bridge provided that, the strike and rake angle extremely affect the seismic behaviors, etc., bending moment of girder, maximum curvature of piers, which indicated the large crossing angle (bridge and fault) should be

avoided in designing, the dip angle has a significant affect on the vertical components of the ground waveforms and the seismic response of the bridge; if the dip angle is smaller than 15° , the vertical components are slightly smaller or even equal to the horizontal components, while when the dip angle is larger than 45° , they become lager than the horizontal terms. Therefore, the vertical responses are correlated with the dip angle and also need special attention.

Because the limit state of performance level for curvature didn't consider the changing axial force effect in this analysis, it recommend the normalized maximum strain for piers to evaluate the seismic performance of near-fault bridge structures.

6.2 Recommendations

It should be noted that, due to the time-histories waveforms are required both for seismic analysis and design of near-fault structures, while the real surface soil conditions have not been considered for engineering designing in this research, either a more detailed procedure for evaluation on the near-fault bridge structures considering the permanent displacements has not been established, thus, by the proposed hybrid synthesis method and combined with the site amplification factors are required, and two main items should be clarified and detailed in the future research:

(1) For the hybrid synthesis method, the real surface soil conditions, especially the nonlinear response of ground motions should be well considered to obtain the near-fault ground motions which are closer to real situations. Furthermore, the sensitive analysis on affected parameters is needed. Empirical relationship should be pointed out for engineering application.

(2) For the seismic analysis on near-fault bridge structures, the phenomena of the slides, separation of the joint of deck and collapse in details has not been checked yet, additional factors should be included in future research. A more realistic situation (e.g., more realistic soil conditions) should be considered in further studies to represent the seismic behavior of a prototype bridge with more details, structural component models or finite-element models are required. Some retrofitting measures for near-fault structures designing under the seismic analysis combined numerical simulations with damage evaluation and experiments (if it is possible) also it will explore systematically optimized methods for such engineering designing.

Appendix A Dynamic and static displacement-stress vectors

We show here the dynamic and static displacement-stress vectors in equations (3-48) and (3-49), which are completely free from the exponentials growing with wavenumber and/or frequency. We derive them using the generalized R/T coefficient method (Luco and Apsel, 1983) and the stress discontinuity (Harkrider, 1964) to express the boundary and source conditions, respectively.

The dynamic displacement-stress vectors of the j th layer (v_{pq}^j and H_{pq}^j) are expressed by the down-going and upgoing P and S waves in the following matrix form:

$$\begin{Bmatrix} D^j(z;h) \\ S^j(z;h) \end{Bmatrix} = \begin{bmatrix} E_{11}^j & E_{12}^j \\ E_{21}^j & E_{22}^j \end{bmatrix} \begin{bmatrix} \Lambda_d^j(z) & 0 \\ 0 & \Lambda_u^j(z) \end{bmatrix} \begin{Bmatrix} C_d^j(h) \\ C_u^j(h) \end{Bmatrix} \quad (\text{A-1})$$

$$\text{Where } D^j(z;h) = \begin{Bmatrix} V_{1q}^j(z;h) \\ V_{2q}^j(z;h) \end{Bmatrix}, S^j(z;h) = \begin{Bmatrix} V_{3q}^j(z;h) \\ V_{4q}^j(z;h) \end{Bmatrix} \quad (q=x,y, \text{ or } z) \quad (\text{A-2})$$

$$C_d^j(h) = \begin{Bmatrix} c_{d\alpha}^j(z;h) \\ c_{d\beta}^j(z;h) \end{Bmatrix}, C_u^j(h) = \begin{Bmatrix} c_{u\alpha}^j(z;h) \\ c_{u\beta}^j(z;h) \end{Bmatrix} \quad (\text{A-3})$$

$$\left\{ \begin{array}{l} \begin{bmatrix} E_{11}^j & E_{12}^j \\ E_{21}^j & E_{22}^j \end{bmatrix} = \begin{bmatrix} -1 & \gamma_\beta^j & -1 & \gamma_\beta^j \\ -\gamma_\alpha^j & 1 & \gamma_\alpha^j & -1 \\ 2\mu^j v_\alpha^j & -\mu^j \chi^j & -2\mu^j v_\alpha^j & \mu^j \chi^j \\ \mu^j \chi^j & -2\mu^j v_\beta^j & \mu^j \chi^j & -2\mu^j v_\beta^j \end{bmatrix} \\ \Lambda_d^j(z) = \begin{bmatrix} \exp\{-v_\alpha^j(z - z^{(j-1)})\} & 0 \\ 0 & \exp\{-v_\beta^j(z - z^{(j-1)})\} \end{bmatrix} \end{array} \right. \quad (\text{A-4})$$

$$\left\{ \begin{array}{l} \Lambda_u^j(z) = \begin{bmatrix} \exp\{-v_\alpha^j(z^{(j)} - z)\} & 0 \\ 0 & \exp\{-v_\beta^j(z^{(j)} - z)\} \end{bmatrix} \\ v_\alpha^{j2} = k^2 - k_\alpha^{j2} \\ \left(\text{Re}\{v_\alpha^j\} \geq 0, k_\alpha^j = \omega/\bar{\alpha}^j, \bar{\alpha}^j = \alpha^j(1 - i/2Q_\alpha^j) \right) \\ v_\beta^{j2} = k^2 - k_\beta^{j2} \\ \left(\text{Re}\{v_\beta^j\} \geq 0, k_\beta^j = \omega/\bar{\beta}^j, \bar{\beta}^j = \beta^j(1 - i/2Q_\beta^j) \right) \end{array} \right. \quad (\text{A-5})$$

$$\gamma_\alpha^j = v_\alpha^j/k, \gamma_\beta^j = v_\beta^j/k, \chi^j = 2k - k_\beta^{j2}/k \quad (\text{A-6})$$

For the P-SV wave, and

$$D^j(z;h) = H_{1q}^j(z;h), S^j(z;h) = H_{2q}^j(z;h) \quad (q=x,y, \text{ or } z) \quad (\text{A-7})$$

$$C_d^j(h) = c_{d\beta}^j(h), C_u^j(h) = c_{u\beta}^j(h) \quad (\text{A-8})$$

$$\begin{bmatrix} E_{11}^j & E_{12}^j \\ E_{21}^j & E_{22}^j \end{bmatrix} = \begin{bmatrix} 1 & 1 \\ -\mu^j \nu_\beta^j & \mu^j \nu_\beta^j \end{bmatrix} \quad (\text{A-9})$$

$$\begin{cases} \Lambda_d^j(z) = \exp\left\{-\nu_\beta^j(z - z^{(j-1)})\right\} \\ \Lambda_u^j(z) = \exp\left\{-\nu_\beta^j(z^{(j)} - z)\right\} \end{cases} \quad (\text{A-10})$$

For the SH wave.

In equation (A-6), α^j and Q_α^j are the P-wave velocity and the quality factor of the P wave of the j th layer, respectively, and β^j and Q_β^j are those of the S-wave. Note that the definition of (A-4) is different from those of Luco and Apsel (1983) or equation (7.55) in Aki and Richards (1980), because of the difference of the normalization of the down/upgoing coefficients (A-3).

In the case of the static displacement-stress vectors ($\omega = 0$), we need to modify equation (A-4) through (A-10). Following the same procedure from equations (53) to (60) in Luco and Apsel (1983), we obtain:

$$\begin{bmatrix} E_{11}^j & E_{12}^j \\ E_{21}^j & E_{22}^j \end{bmatrix} = \begin{bmatrix} - & 1 & 1 & 1 \\ -(\kappa^j - 1) & 1 & (\kappa^j - 1) & -1 \\ k\mu^j(\kappa^j - 3) & -2k\mu^j & -k\mu^j(\kappa^j - 3) & 2k\mu^j \\ k\mu^j(\kappa^j - 1) & -2k\mu^j & k\mu^j(\kappa^j - 1) & -2k\mu^j \end{bmatrix} \quad (\text{A-11})$$

$$\text{Where } \kappa^j = \frac{1 + (\bar{\beta}^j / \bar{\alpha}^j)^2}{1 - (\bar{\beta}^j / \bar{\alpha}^j)^2}$$

$$\begin{cases} \Lambda_d^j(z) = \begin{bmatrix} 1 & 0 \\ -k(z - z^{(j-1)}) & 1 \end{bmatrix} \exp\left\{-k(z - z^{(j-1)})\right\} \\ \Lambda_u^j(z) = \begin{bmatrix} 1 & 0 \\ -k(z^{(j)} - z) & 1 \end{bmatrix} \exp\left\{-k(z^{(j)} - z)\right\} \end{cases} \quad (\text{A-12})$$

For the P-SV wave, and

$$\begin{bmatrix} E_{11}^j & E_{12}^j \\ E_{21}^j & E_{22}^j \end{bmatrix} = \begin{bmatrix} 1 & 1 \\ -\mu^j k & \mu^j k \end{bmatrix} \quad (\text{A-13})$$

$$\begin{cases} \Lambda_d^j(z) = \exp\left\{-k\left(z - z^{(j-1)}\right)\right\} \\ \Lambda_u^j(z) = \exp\left\{-k\left(z^{(j)} - z\right)\right\} \end{cases} \quad (\text{A-14})$$

For the SH-wave.

In equation (A1), the down/up-going coefficients C_d^j and C_u^j are determined from the source and boundary conditions. In the following, we divide our formulation into three cases.

Case 1: the down/up-going coefficients for $h \neq 0$ and $h \leq z^{(N-1)}$

We first present the down/up-going coefficients, in the case of $h \neq 0$ and $h \leq z^{(N-1)}$, (the source is located neither on the free surface nor in the underlying half-space). In this case, we divide the source layer (*sth layer*) into the upper (S^-) and lower (S^+) layer at the source depth h , and consider them as two individual layers (see Fig. A - 1).

First, since the down-going waves only exist in the underlying half-space ($j=N+1$), we get

$$C_u^{N+1} = \{0\} \quad (\text{A-15})$$

Second, we define the modified R/T coefficients $T_d^{(j)}$, $R_d^{(j)}$, $T_u^{(j)}$, and $R_u^{(j)}$ (Luco and Apsel, 1983) for the j th boundary between the j th and $(j+1)$ th layers

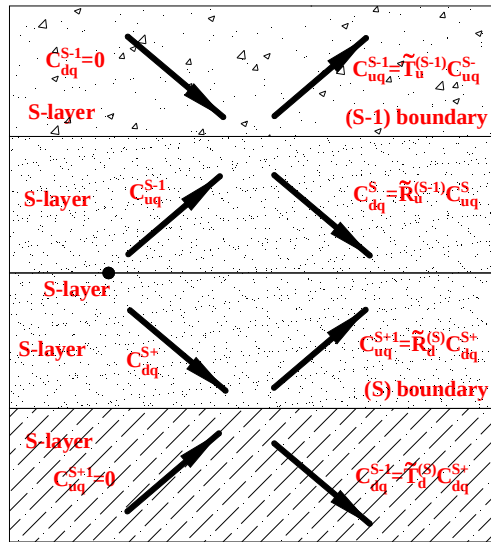


Figure A-1 Asymptotic down/upgoing coefficients in the $S-1$, S^- , S^+ , and $S+1$ layers for case 1: $Z^{(S-1)} \leq h \leq Z^{(S)}$, $S \neq 1$, and $S \neq N+1$

$$\begin{cases} C_n^j = R_d^{(j)} C_d^j + T_u^{(j)} C_u^{j+1} \\ C_d^{j+1} = T_d^{(j)} C_d^j + R_u^{(j)} C_u^{j+1} \quad (\text{for } j=1,2,\dots,N) \end{cases} \quad (\text{A-16})$$

For the free surface ($j=0$), since the reflection coefficients only exists, we define

$$C_d^j = R_d^{(0)} C_u^1 \quad (\text{A-17})$$

Using equation (A-1) for the j th and $(j+1)$ th layers and considering the continuity conditions of displacement and stress, we obtain the modified R/T coefficients defined above as follows:

$$\begin{cases} R_u^{(0)} = -\left(E_{21}^1\right)^{-1} E_{22}^1 \Lambda_u^1(0) \\ \begin{cases} T_d^{(j)} & R_u^{(j)} \end{cases} = \begin{bmatrix} E_{11}^{j+1} & -E_{12}^j \\ E_{21}^{j+1} & -E_{22}^j \end{bmatrix}^{-1} \begin{bmatrix} E_{11}^j & -E_{12}^{j+1} \\ E_{21}^j & -E_{22}^{j+1} \end{bmatrix} \begin{bmatrix} \Lambda_d^j(z^{(j)}) & 0 \\ 0 & \Lambda_u^{j+1}(z^{(j)}) \end{bmatrix} \\ (\text{for } j=1,2,\dots,N-1) \\ \begin{bmatrix} T_d^{(N)} \\ R_d^{(N)} \end{bmatrix} = \begin{bmatrix} E_{11}^{N+1} & -E_{12}^N \\ E_{21}^{N+1} & -E_{22}^N \end{bmatrix}^{-1} \begin{cases} E_{11}^N \Lambda_d^N(z^{(N)}) \\ E_{21}^N \Lambda_d^N(z^{(N)}) \end{cases} \end{cases} \quad (\text{A-18})$$

Third, we define the generalized R/T coefficients $\tilde{T}_d^{(j)}$, $\tilde{R}_d^{(j)}$, $\tilde{T}_u^{(j)}$, and $\tilde{R}_u^{(j)}$ (Luco and Apsel, 1983) as follows:

$$C_u^j(h) = \tilde{T}_u^{(j)} C_u^{j+1}(h), C_d^{j+1}(h) = \tilde{R}_u^{(j)} C_u^{j+1}(h) (\text{for } j=1,2,\dots,S-1) \quad (\text{A-19})$$

$$C_d^{j+1}(h) = \tilde{T}_d^{(j)} C_d^j(h), C_u^{j+1}(h) = \tilde{R}_d^{(j)} C_d^j(h) (\text{for } j=N-1,\dots,S) \quad (\text{A-20})$$

Where S is the source layer. We can derive the generalized R/T coefficients from the modified R/T coefficients using the following recurrence relations:

$$\begin{aligned} \tilde{R}_u^{(0)} &= R_u^{(0)}, \quad (\text{for } j=0) \\ \tilde{T}_u^{(j)} &= \left(I - R_u^{(j)} \tilde{R}_u^{(j-1)}\right)^{-1} T_u^{(j)}, \\ \tilde{R}_u^{(j)} &= R_u^{(j)} + T_d^{(j)} \tilde{R}_u^{(j-1)} \tilde{T}_u^{(j)}, \quad (\text{for } j=1,\dots,S-1) \end{aligned} \quad (\text{A-21})$$

For the layers upper than the source, and

$$\begin{aligned} \tilde{T}_d^{(N)} &= T_d^{(N)}, \tilde{R}_d^{(N)} = R_d^{(N)}, \quad (\text{for } j=N) \\ \tilde{T}_d^{(j)} &= \left(I - R_u^{(j)} \tilde{R}_d^{(j+1)}\right)^{-1} T_d^{(j)}, \\ \tilde{R}_d^{(j)} &= R_d^{(j)} + T_u^{(j)} \tilde{R}_d^{(j+1)} \tilde{T}_d^{(j)}, \quad (\text{for } j=N-1,\dots,S) \end{aligned} \quad (\text{A-22})$$

For the layers lower than the source, I is the identity matrix.

Fourth, in order to obtain the down/up-going coefficients of the S th layer, we introduce “the stress discontinuities” (Harkrider, 1964) at the boundary between the S^- and S^+ layers to express the source conditions

$$\begin{bmatrix} D^{S+}(h) \\ S^{S+}(h) \end{bmatrix} = \begin{bmatrix} D^{S-}(h) \\ S^{S-}(h) \end{bmatrix} + \begin{bmatrix} 0 \\ \Delta Q_q \end{bmatrix}, \quad (q = x, y, \text{ or } z) \quad (\text{A-23})$$

Where

$$\Delta Q_x = \begin{bmatrix} -\frac{k}{2\pi} Q_x \\ 0 \end{bmatrix}, \quad \Delta Q_y = \begin{bmatrix} -\frac{k}{2\pi} Q_y \\ 0 \end{bmatrix}, \quad \Delta Q_z = \begin{bmatrix} 0 \\ \frac{k}{2\pi} Q_z \end{bmatrix} \quad (\text{A-24})$$

For the *P-SV* wave, and

$$\Delta Q_x = -\frac{k}{2\pi} Q_x, \quad \Delta Q_y = -\frac{k}{2\pi} Q_y \quad (\text{A-25})$$

For the *SH* wave.

Using equation (A-1) for the S^- and S^+ layers and considering (A-23), we finally obtain the down/up-going coefficients of the two layers

$$\begin{aligned} C_u^{S-}(h) &= (B^{S+} D^S - B^{S-})^{-1} \Delta Q_q, \quad (q = x, y, \text{ or } z) \\ C_d^{S+}(h) &= D^S C_u^{S-}(h), \\ C_d^{S-}(h) &= \tilde{R}_u^{(S-1)} C_u^{S-}(h), \\ C_u^{S+}(h) &= \tilde{R}_d^{(S)} C_d^{S+}(h) \end{aligned} \quad (\text{A-26})$$

Where

$$\begin{aligned} A^{S-} &= E_{11}^S \Lambda_d^{S-}(h) \tilde{R}_u^{(S-1)} + E_{12}^S, \\ A^{S+} &= E_{11}^S + E_{12}^S \Lambda_u^{S+}(h) \tilde{R}_d^{(S)}, \\ B^{S-} &= E_{21}^S \Lambda_d^{S-}(h) \tilde{R}_u^{(S-1)} + E_{22}^S, \\ B^{S+} &= E_{21}^S + E_{22}^S \Lambda_u^{S+}(h) \tilde{R}_d^{(S)}, \\ D^S &= (A^{S+})^{-1} A^{S-} \end{aligned} \quad (\text{A-27})$$

Finally, we obtain the down/up-going coefficients of all layers by substituting (A26) into (A19) and (A20).

Case 2: the down/up-going coefficients for $h=0$

When the source is located on the free surface ($S=1$ and $h=0$), we need to slightly modify our formulation. We first derive all modified and generalized R/T coefficients using equations (A-18) and (A-22), respectively. Second, instead of (A-23), we use the following source condition:

$$S^1(h=0) = \Delta Q_q, \quad (q = x, y, \text{ or } z) \quad (\text{A-28})$$

Substituting (A-28) and (A-20) in equation (A-1) of the first layer, we get the down-going coefficient in the first layer,

$$C_d^1 = \left(E_{21}^1 + E_{22}^1 \Lambda_u^1(0) \tilde{R}_u^{(1)} \right)^{-1} \Delta Q_q, \quad (q = x, y, \text{ or } z) \quad (\text{A-29})$$

All the other coefficients are obtained by (A20).

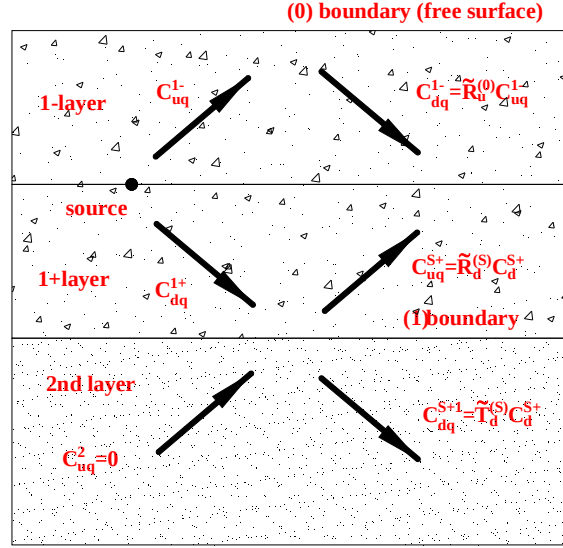


Figure A-2 Asymptotic down/upgoing coefficients in the 1-, 1+, and 2nd layers for case 2: $Z^{(0)} \leq h \leq Z^{(1)}$ and $S=1$

Case 3: the down/up-going coefficients for $h \geq z^{(N-1)}$

When the source is located in the underlying half-space ($S = N + 1$, and $h \geq z^{(N)}$), we first divide the underlying half-space into the upper layer ($N+1^-$) and the lower half-space ($N+1^+$) at the source depth, and then derive the modified and generalized R/T coefficients for all boundaries between adjacent layers using equations (A-18) and (A-21). Second, we introduce “the stress discontinuities” at the boundary, as done in (A-23).

$$\begin{bmatrix} D^{N+1^+}(h) \\ S^{N+1^+}(h) \end{bmatrix} = \begin{bmatrix} D^{N+1^-}(h) \\ S^{N+1^-}(h) \end{bmatrix} + \begin{Bmatrix} 0 \\ \Delta Q_q \end{Bmatrix}, \quad (q = x, y, \text{ or } z) \quad (\text{A-30})$$

In the lower half-space ($N+1^+$), since only the downgoing waves exist,

$$C_u^{N+1} = \{0\} \quad (\text{A-31})$$

Using equation (A-1) for the ($N+1^-$) and ($N+1^+$) layers and considering (A-30) and (A-31), we finally obtain the down/up-going coefficients of the two layers:

$$C_u^{N+1^-}(h) = \left(B^{N+1^+} D^{N+1^-} - B^{N+1^-} \right)^{-1} \Delta Q_q, \quad (q = x, y, \text{ or } z) \quad (\text{A-32})$$

$$C_u^{N+1^+}(h) = D^{N+1} C_u^{N+1^-}(h)$$

Where

$$\begin{aligned}
A^{N+1-} &= E_{11}^{N+1} \Lambda_d^{N+1-}(h) \tilde{R}_u^{(N)} + E_{12}^{N+1}, \\
A^{N+1+} &= E_{11}^{N+1}, \\
B^{N+1-} &= E_{21}^{N+1} \Lambda_d^{N+1-}(h) \tilde{R}_u^{(N)} + E_{22}^{N+1}, \\
B^{N+1+} &= E_{21}^{N+1}, \\
D^{N+1} &= \left(A^{N+1+} \right)^{-1} A^{N+1-}
\end{aligned} \tag{A-33}$$

The all other coefficients are obtained from (A-19).

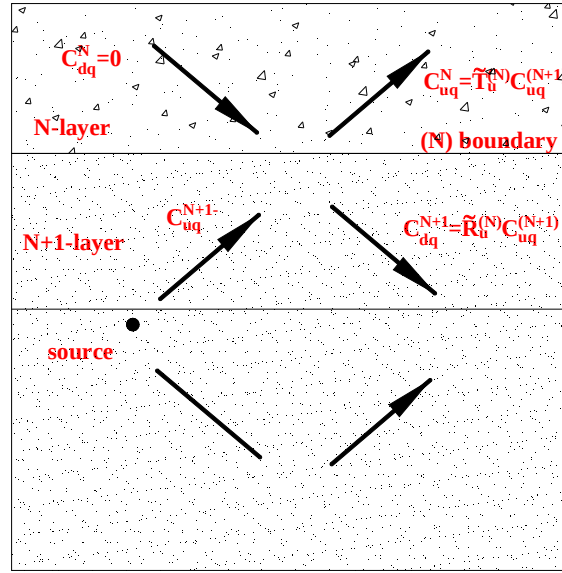


Figure A-3 Asymptotic down/upgoing coefficients in the N, N+1-, and N+1+ layers for case 3:
 $Z(N) \leq h$ and $S = N+1$

Appendix B Asymptotic solutions of the displacement-stress vectors

We derive the asymptotic solutions of the displacement-stress vectors at large wavenumbers, using the equations given in Appendix A. since the dynamic displacement-stress vectors converge to the static ones with increasing wavenumber displacement-stress vectors in the following.

As k tends toward infinity, the highest orders of k in equation (A-11) and (A-13) are:

$$\begin{aligned} E_{11}^j \text{ and } E_{12}^j &\rightarrow o\{k^0\}, \\ E_{21}^j \text{ and } E_{22}^j &\rightarrow o\{k^1\}, \end{aligned} \quad (\text{B-1})$$

Those in (A12) and (A14) are:

$$\begin{aligned} \Lambda_d^j &\rightarrow o\{k^0\} \left(\text{for } z = z^{(j-1)} \right), \\ &\text{or } 0 \left(\text{for } z \neq z^{(j-1)} \right) \\ \Lambda_u^j &\rightarrow o\{k^0\} \left(\text{for } z = z^{(j)} \right), \\ &\text{or } 0 \left(\text{for } z \neq z^{(j)} \right) \end{aligned} \quad (\text{B-2})$$

Case 1: the down/up-going coefficients for $h \neq 0$ and $h \leq z^{(N-1)}$

From (B-1) and (B-2), we find the modified R/T coefficients given in (A-18) converge to zero with increasing wavenumber:

$$\begin{aligned} R_u^{(0)} &\rightarrow 0, \\ T_d^{(j)}, R_u^{(j)}, T_u^{(j)} \text{ and } R_d^{(j)} &\rightarrow 0, (j = 1, 2, \dots, N-1) \\ T_d^{(N)} \text{ and } R_d^{(N)} &\rightarrow 0. \end{aligned} \quad (\text{B-3})$$

Therefore, the generalized R/T coefficients given in (A-21) and (A-22) also:

$$\begin{aligned} \tilde{R}_u^{(0)} &\rightarrow 0, \\ \tilde{T}_d^{(j)}, \tilde{R}_u^{(j)}, \tilde{T}_u^{(j)} \text{ and } \tilde{R}_d^{(j)} &\rightarrow 0, (j = 1, 2, \dots, N-1) \\ \tilde{T}_d^{(N)} \text{ and } \tilde{R}_d^{(N)} &\rightarrow 0. \end{aligned} \quad (\text{B-4})$$

This indicates that the amplitudes of all the reflected and transmitted waves from boundaries converge to zero with wavenumber.

On the other hand, the highest orders of k of the source term in (A-24) and (A-25) are:

$$\Delta Q_q \rightarrow o(k^1), \quad (q = x, y, \text{ and } z) \quad (\text{B-5})$$

Using (B-2) and (B-4), those of the coefficients in (A-27) are:

$$\begin{aligned}
A^{s-} &\rightarrow E_{12}^S \rightarrow o(k^0), \\
A^{s+} &\rightarrow E_{11}^S \rightarrow o(k^0), \\
B^{s-} &\rightarrow E_{22}^S \rightarrow o(k^1), \\
B^{s+} &\rightarrow E_{22}^S \rightarrow o(k^1), \\
D^s &\rightarrow o(k^0)
\end{aligned} \tag{B-6}$$

Therefore, the orders of the down/up-going coefficients of the S^- and S^+ layers in (A-26) are:

$$\begin{aligned}
C_u^{s-} &\rightarrow o(k^0), \\
C_d^{s+} &\rightarrow o(k^0), \\
C_d^{s-} &\rightarrow 0, \\
C_u^{s+} &\rightarrow 0.
\end{aligned} \tag{B-7}$$

Substituting (B-1) and (B-7) into (A-1) of the S^- and S^+ layers, we get the highest orders of k of the displacement-stress vectors in the source layers:

$$\begin{aligned}
\begin{Bmatrix} D^{s-}(z;h) \\ S^{s-}(z;h) \end{Bmatrix} &\rightarrow \begin{bmatrix} o(k^0) & o(k^0) \\ o(k^1) & o(k^1) \end{bmatrix} \begin{bmatrix} \Lambda_d^{s-}(z) & 0 \\ 0 & \Lambda_u^{s-}(z) \end{bmatrix} \begin{Bmatrix} 0 \\ o(k^0) \end{Bmatrix} \rightarrow \begin{Bmatrix} o(k^0)\Lambda_u^{s-}(z) \\ o(k^1)\Lambda_u^{s-}(z) \end{Bmatrix} \\
\begin{Bmatrix} D^{s+}(z;h) \\ S^{s+}(z;h) \end{Bmatrix} &\rightarrow \begin{bmatrix} o(k^0) & o(k^0) \\ o(k^1) & o(k^1) \end{bmatrix} \begin{bmatrix} \Lambda_d^{s+}(z) & 0 \\ 0 & \Lambda_u^{s+}(z) \end{bmatrix} \begin{Bmatrix} 0 \\ o(k^0) \end{Bmatrix} \rightarrow \begin{Bmatrix} o(k^0)\Lambda_u^{s+}(z) \\ o(k^1)\Lambda_u^{s+}(z) \end{Bmatrix}
\end{aligned} \tag{B-8}$$

Case 2: the down/up-going coefficients for $h=0$

Similarly, we obtain the highest orders of k of the displacement-stress vectors in the source layer (the first layer):

$$\begin{Bmatrix} D^1(z;h) \\ S^1(z;h) \end{Bmatrix} \rightarrow \begin{bmatrix} o(k^0) & o(k^0) \\ o(k^1) & o(k^1) \end{bmatrix} \begin{bmatrix} \Lambda_d^1(z) & 0 \\ 0 & \Lambda_u^1(z) \end{bmatrix} \begin{Bmatrix} 0 \\ o(k^0) \end{Bmatrix} \rightarrow \begin{Bmatrix} o(k^0)\Lambda_d^1(z) \\ o(k^1)\Lambda_d^1(z) \end{Bmatrix} \tag{B-9}$$

Case 3: the down/up-going coefficients for $h \geq z^{(N-1)}$

Similarly, we obtain:

$$\begin{aligned}
\begin{Bmatrix} D^{N+1-}(z;h) \\ S^{N+1-}(z;h) \end{Bmatrix} &\rightarrow \begin{bmatrix} o(k^0) & o(k^0) \\ o(k^1) & o(k^1) \end{bmatrix} \begin{bmatrix} \Lambda_d^{N+1-}(z) & 0 \\ 0 & \Lambda_u^{N+1-}(z) \end{bmatrix} \begin{Bmatrix} 0 \\ o(k^0) \end{Bmatrix} \rightarrow \begin{Bmatrix} o(k^0)\Lambda_u^{N+1-}(z) \\ o(k^1)\Lambda_u^{N+1-}(z) \end{Bmatrix} \\
\begin{Bmatrix} D^{N+1+}(z;h) \\ S^{N+1+}(z;h) \end{Bmatrix} &\rightarrow \begin{Bmatrix} o(k^0)\Lambda_d^{N+1+}(z)o(k^0) \\ o(k^1)\Lambda_d^{N+1+}(z)o(k^0) \end{Bmatrix} \rightarrow \begin{Bmatrix} o(k^0)\Lambda_d^{N+1+}(z) \\ o(k^1)\Lambda_d^{N+1+}(z) \end{Bmatrix}
\end{aligned} \tag{B-10}$$

Using the equations from (B-8) to (B-10) and (A-12) and (A-14), we can summarize the asymptotic solutions of the displacement-stress vectors for the source layer in the following forms:

$$\begin{aligned}
\begin{Bmatrix} V_{1 \text{ or } 2}^{S-or+} \\ V_{3 \text{ or } 4}^{S-or+} \end{Bmatrix} &\rightarrow \begin{Bmatrix} \bar{A} + k|z-h|\bar{B} \\ k\bar{C} + k|z-h|\bar{D} \end{Bmatrix} \exp\{-k|z-h|\} \quad \text{for } P-SV \text{ wave.} \\
\begin{Bmatrix} H_1^{S-or+} \\ H_2^{S-or+} \end{Bmatrix} &\rightarrow \begin{Bmatrix} \bar{E} \\ k\bar{F} \end{Bmatrix} \exp\{-k|z-h|\} \quad \text{for } SH \text{ wave.}
\end{aligned} \tag{B-11}$$

Where $\bar{A}$ through $\bar{F}$ are constant values.

From (B-11), we see that the displacement vectors converge and the stress vectors increase linearly with k , in the case of $z=h$, while all displacement-stress vectors slowly converge to zero as exponential functions in the case z close to h . on the other hand, when the receiver z is located in a other than the source layer, the displacement-stress vectors converge more quickly to zero. This is because the all R/T coefficients converge to zero, as seen in (B-3) and (B-4).

Appendix C Parameters for differentiation

In order to obtain equations (3-52) and (3-53), we use the following relations. Most of those are easily derived by differentiating equation (3-42) with respect to z and/or r .

$$S_1 \equiv \frac{1}{R} = \int_0^\infty \left[\exp\{-k|z-h|\} J_0(kr) \right] dk \quad (C-1)$$

$$S_2 \equiv \frac{|z-h|}{R^3} = \int_0^\infty \left[\exp\{-k|z-h|\} k J_0(kr) \right] dk \quad (C-2)$$

$$S_3 \equiv \left\{ 3 \frac{|z-h|}{R^2} - 1 \right\} \frac{1}{R^3} = \int_0^\infty \left[\exp\{-k|z-h|\} k^2 J_0(kr) \right] dk \quad (C-3)$$

$$S_4 \equiv \frac{R-|z-h|}{r} = \int_0^\infty \left[\exp\{-k|z-h|\} \frac{J_1(kr)}{k} \right] dk \quad (C-4)$$

$$S_5 \equiv \frac{R-|z-h|}{rR} = \int_0^\infty \left[\exp\{-k|z-h|\} J_1(kr) \right] dk \quad (C-5)$$

$$S_6 \equiv \frac{1}{R^3} = \int_0^\infty \left[\exp\{-k|z-h|\} k J_1(kr) \right] dk \quad (C-6)$$

$$S_7 \equiv 3 \frac{r|z-h|}{R^5} = \int_0^\infty \left[\exp\{-k|z-h|\} k^2 J_1(kr) \right] dk \quad (C-7)$$

where $R = \sqrt{k^2 + (z-h)^2}$