Methodology of Seismic Hazard Analysis and Damage Assessment

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ABSTRACT

Since Taiwan locates at seismic active region, seismic hazard mitigation is always one of the most important parts in natural hazard mitigation plans. To have effective seismic hazard mitigation plans needs cooperation of experts in many fields. In order to accelerate the progress in seismic loss estimation and hazard mitigation, Haz-Taiwan adopts the methodology from HAZUS, which is developed by the scientists and engineers in the United States. However, Taiwan has its special construction environment and quality, seismic design codes, social and economic activities, etc., not only the inventory data but also the analytical models and the associated parameters need to be collected and calibrated.

This joint project investigates methodology of seismic hazard analysis and structural damage assessment from earthquake engineering viewpoint. The main objective is to provide technical support, to develop analytical models, and to calibrate associated parameters in Haz-Taiwan program. The research topics include estimation of ground motion intensities due to scenario earthquakes, estimation of permanent ground deformation due to soil-liquefaction, evaluation of site-dependent seismic demand of buildings, evaluation of capacity and fragility curves in damage assessment of reinforced concrete buildings and multi-span continuous bridges, and so on.

INTRODUCTION

The aim of Haz-Taiwan program is to provide a systematic approach, a unified inventory data classification system, and the state-of-the-art seismic scenario simulation technologies to simulate seismic hazard and to estimate structural damages and socio-economical losses. The outputs of Haz-Taiwan can
be used in planning seismic hazard mitigation and emergency response strategies. In order to obtain reliable results, there are many kinds of inventory and geological data to be collected and many site-dependent analysis parameters to be calibrated. The National Center for Research on Earthquake Engineering (NCREE) is in charge of the cooperative development of analytical models for seismic hazard analysis and structural damage assessment. NCREE also has the responsibility to maintain and update source codes of the application software.

There are four major analysis modules in the framework of Haz-Taiwan, i.e., potential earth science hazard analysis, structural damage assessment, induced physical damage evaluation, and socio-economical loss estimation. Only the first two modules are studied in this joint project, however, there are eleven sub-projects in this study. In order to clarify individual research objective and accomplishment, each sub-project will be summarized in the individual sections.

ATTENUATION LAWS OF PGA AND RESPONSE SPECTRA

The strong ground motion data obtained by the Taiwan Strong Motion Instrumentation Program (TSMIP) [1] are used to derive new attenuation laws for horizontal and vertical peak ground acceleration and response spectral accelerations for Taiwan and Chianan areas. More than 3,200 accelerograms, recorded from 39 shallow earthquakes in and around Taiwan, with moment magnitude $M_w$ ranging from 3.5 to 7.7, have been analyzed for investigating the behavior of attenuation relations with respect to magnitude, faulting style, wave propagation and site effect.

The following functional form is proposed for use in Taiwan:

\[
\ln Y = A \ln(X + h) + B X + C M + D + G (E \ln(X + F))
\]

where $Y$ is the ground motion parameter, $X$ is the shortest distance to the rupture plane, $M$ is the moment magnitude, $A$ is the geometrical spreading coefficient, $B$ is inelastic attenuation coefficient, $C$ is the magnitude coefficient, $D$ is a constant, $h$ is the close-in distance saturation term, $G$ takes on the value of zero at distances less than 100km and one at distances greater than 100km. $A, B, C, D, E, F, h$ are regression coefficients determined by the data. The results are given in Table 1 and Table 2.

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<th>A</th>
<th>B</th>
<th>C</th>
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<th>F</th>
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Note: $hv1 = 0.0493 \exp (0.740M_w)$, $hv2 = 0.00097 \exp (1.321M_w)$

$hh1 = 0.0312 \exp (0.855M_w)$, $hh2 = 0.0022 \exp (1.189M_w)$
Table 2  Attenuation relations of response spectra in Taiwan and Chianan areas

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Note: \( vh1 = 0.0824 \exp (0.813Mw), \) \( vh2 = 0.0510 \exp (0.920Mw), \) \( hh1 = 0.4278 \exp (0.610Mw), \)
\( vh3 = 0.2067 \exp (0.661Mw), \) \( hh3 = 0.3924 \exp (0.523Mw), \)
\( vh4 = 0.5179 \exp (0.433Mw), \) \( hh4 = 0.8898 \exp (0.325Mw), \) \( vh5 = 0.1733 \exp (0.622Mw), \)
\( vh6 = 0.1956 \exp (0.638Mw), \) \( hh5 = 0.4838 \exp (0.462Mw), \) \( vh7 = 0.4572 \exp (0.551Mw), \)
\( vh8 = 0.5948 \exp (0.492Mw), \) \( hh7 = 0.2374 \exp (0.602Mw), \) \( hh8 = 0.6808 \exp (0.427Mw), \)
\( vh9 = 0.7368 \exp (0.464Mw), \) \( hh9 = 0.3388 \exp (0.515Mw), \) \( hh10 = 0.2817 \exp (0.602Mw) \)

From analysis results in this study, the ratio of vertical peak ground acceleration to the average of peak ground accelerations in two horizontal directions is increased when the distance is decreased. So, it is necessary to evaluate separately attenuation coefficients of vertical and horizontal peak ground accelerations.

The normalized response spectral accelerations of Taiwan and Chianan regions are similar to the Type S2 design spectra shape in Uniform Building Code (UBC). However, the normalized response spectral accelerations in the low-period region, i.e., 0.25 second to 0.5 second in the horizontal component, is above the corresponding UBC design spectra shapes.

The attenuation relationships in this study have taken into account the effects of faulting styles, geometrical spreading, inelastic attenuation, Moho discontinuity bounce, and local site response. Once earthquakes occur in the future, we can combine the attenuation laws and local site conditions to estimate the peak ground accelerations and response spectral accelerations, providing very important information to mitigate damage and to reduce life and property losses from the earthquakes.

**RAPID ESTIMATION OF PGA BY USING REAL-TIME STRONG MOTION NETWORK**

The objective of this study [2,3] is to predict the distribution of (horizontal) peak ground acceleration soon after the occurrence of earthquakes by utilizing
the data from real-time strong motion network, empirical attenuation relations of peak ground acceleration, and pre-determined site-dependent modification factors. After the occurrence of earthquakes, the real-time strong motion network can obtain seismic source parameter values and waveform data recorded at 75 real-time strong motion stations within one minute. The source location error is within 10km and the magnitude error within 0.2. So, it is a very time-effective method to cooperate the measured data and the empirical relations to estimate the distribution of ground motion intensity and also the extent of losses.

There were 269 seismic events of magnitude greater than 5, between 1995 and 1999, which had been recorded by both the TSMIP and the real-time strong motion network. All of the associated records were analyzed. The first step was to relocate the seismic sources and to calibrate magnitudes in order to reduce errors caused by the source parameters. Then, the empirical PGA attenuation relations were obtained by regression analysis, as shown in the following. When the source depth is less than 35km,

$$\text{PGA}_{\text{cal}} = \text{PGA}_{\text{att}} \cdot S \cdot \frac{\text{RTD}_{\text{obs}}}{\text{RTD}_{\text{cal}}}$$

where \(\text{PGA}_{\text{att}}\) is the PGA predicted by the attenuation relations, \(S\) is the site modification factor, and \(\text{RTD}_{\text{obs}}, \text{RTD}_{\text{cal}}\) are the observed and the calculated PGA at the nearest real-time strong motion station. Figure 1 compares four types of distribution of PGA at the 1999 Chi-Chi earthquake. As shown in the figure, the proposed method is very effective in predicting PGA soon after the occurrence of earthquakes, and thus can be provided as reference in emergency response actions.

**SIMULATION OF NEAR FIELD GROUND MOTION**

In this study, the quasi-dynamic rupture model \([4\sim6]\) is defined for a dip slip fault. Furthermore, combined with the Greens function of a double-couple line source that is expressed as an integral representation of complex wave-number, the ground surface displacement caused by the propagating dip-slip fault can be determined by the integration over the fault surface. The associated velocity response spectra due to the determined simple pulse-like ground motions are also determined to show the required spectral velocity demand to resist the threat of near-fault ground motions.

The following parameters are used in simulating the representative velocity pulse: the inclined angle of fault plane \(\delta = 40^\circ\), the velocity of P-wave \(C_p = 5.6\text{km/sec}\) and velocity of S-wave \(C_s = 3.2\text{km/sec}\). The slip function \(D(\xi, \phi)\) is defined as shown in Fig. 2, where \(\xi\) is the distance between ruptured point and the starting point. It is assumed that the rupture length is 1.5km and 6.0km,
respectively, above and below the starting point. The rupture velocity \( C_r = 2.4\text{km/sec} \) and healing velocity \( C_h = 3.2\text{km/sec} \). Rupture is healed at \( \xi = 5.25\text{km} \) and healing time is linearly decreased to zero. The total slip distance is set to 1.5m. In this study case, the sites belong to hanging wall when \(-4\text{km} \leq x \leq 34\text{km}\), and foot wall when \(15\text{km} \leq x \leq 34\text{km}\).

Figure 3 shows the simulated displacement, velocity and acceleration time histories at the site \( x = 12\text{km} \) in horizontal and vertical directions, respectively. Figure 4 shows the acceleration response spectra when the structure is subjected to the simulated near-fault velocity pulse.

Fig. 1 Distribution of PGA at the 1999 Chi-Chi earthquake by using different sets of data: (A) data of real-time strong motion network, (B) a point source model and site modification factors, (C) data of real-time strong motion network and site modification factors, (D) measured data at 650 stations
Fig. 2  Definition of slip function $D(\xi, t)$

Fig. 3  Simulated displacement, velocity and acceleration time histories at $x = 12$km in horizontal and vertical directions

Fig. 4  Acceleration spectra caused by near-fault velocity pulse
ELASTIC RESPONSE SPECTRA

Normalization of response spectrum is often carried out by scaling the time history to have the same peak ground acceleration. However, it is also well known that the peak values are very sensitive to different kinds of baseline correction methods, and seismically designed structures are often ductile. Thus, peak ground acceleration is not a good index for assessing damage potential of strong ground motions to engineering structures.

To seek another normalization parameter, the energy accumulation function [7] of a seismic record is defined as

\[ E(t) = \int_0^t [a_e^2(s) + a_n^2(s)] ds \]

where \( a_e(.) \) and \( a_n(.) \) are the accelerations in east-west and north-south directions, respectively. The average intensity, denoted by \( \bar{A} \), of the record is defined to be the largest values of energy difference within a 10-second period. The average intensity has the following characteristics: it is invariant with respective to coordinate transformation, and is insensitive to baseline correction methods. If the time function within a 10-second period is viewed as a stationary random process, the average intensity represents the standard deviation of the random process. Variances of response spectra are more uniform over the entire frequency range than those normalized by peak ground acceleration.

As an example, Fig. 5 shows the average response spectra at four CWB strong motion stations in Taipei basin. To differentiate the effects of ground motion intensity, the response spectra of seismic records are averaged in two groups, i.e., \( \bar{A} < 5 \) and \( \bar{A} > 5 \) cm/sec². It is noted that the dynamic characteristics at these four sites are very distinct, and the response spectra in low frequency range are amplified very much as the intensity becomes larger.

\[ \bar{A} < 5\text{cm/sec}^2 \]

\( \bar{A} > 5\text{cm/sec}^2 \)

Fig. 5 Comparison of normalized response spectra at four stations in Taipei area

INELASTIC SEISMIC DEMANDS OF BUILDINGS

The accomplishment in this sub-project is roughly divided into two parts, i.e., investigation of the dynamic characteristics of near-fault ground excitations, and evaluation of the nonlinear seismic demands of buildings subjected to the near-fault ground
excitations. Investigation of strong ground motion records in the 1999 Chi-Chi earthquake reveals that there exists long-duration, long-period, large velocity pulses in near-fault records, which have significant effects on the nonlinear response of high-rise buildings with large fundamental periods.

As shown in Fig. 6, the distances between sites and ruptured fault are less than 5km. The peak ground velocities of TCU068 and TCU052 were 383cm/sec and 254cm/sec, respectively. Most of the largest peak ground velocities occurred in the directions perpendicular to the ruptured fault.

Capacity spectrum method in ATC-40 [8] defines the performance point as the intersection of seismic demand curve and capacity curve. The performance points are obtained by an iterative method and are used to estimate the probable maximum response of the structure subjected to certain level of excitation. The nonlinear seismic demands can be obtained by using equivalent linear systems with certain effective damping ratio or by using $R-T(\text{period})-\mu(\text{ductility})$ formula. The Nassar and Krawinkler [9] $R-T-\mu$ parametric formula for far-field excitations is re-examined to see its validity in near-fault cases. As shown in Fig. 7, the loci of performance points (LPP) are significantly different by using the far-field $R-T-\mu$ formula and the nonlinear dynamic analysis, respectively. Because the two LPP have different trends, spectral displacement demands may be under-estimated by using far-field $R-T-\mu$ formula.

Based on the eight sets of near-fault records of the 1999 Chi-Chi earthquake, considering different post-yield stiffness ratio $\alpha = 0\%, 5\%, 10\%$ and $20\%$, new $R-T-\mu$ parametric formula for near-fault excitations is derived as
\[ R(\alpha, T, \mu) = \left[ \frac{\mu - 1}{\beta(\alpha, T)} + 1 \right]^{\beta(\alpha, T)} \]

\[ \beta(\alpha, T) = \alpha (1 - e^{-T}) + bT e^{-cT} \]

where \( a, b \) and \( c \) are the regression coefficients and are functions of \( \alpha \) and \( R \). Table 3 shows the coefficients for \( \alpha = 0\% \) and \( \alpha = 5\% \).

Table 3 Coefficients \( a, b \) and \( c \) in \( R-T-\mu \) formula

<table>
<thead>
<tr>
<th></th>
<th>( \alpha = 0% )</th>
<th>( \alpha = 5% )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( a )</td>
<td>( b )</td>
</tr>
<tr>
<td>mean + ( \sigma )</td>
<td>1.02</td>
<td>0.94</td>
</tr>
<tr>
<td>mean</td>
<td>0.84</td>
<td>1.09</td>
</tr>
<tr>
<td>mean - ( \sigma )</td>
<td>0.55</td>
<td>0.42</td>
</tr>
</tbody>
</table>

**PERMANENT GROUND DISPLACEMENT INDUCED BY SOIL LIQUEFACTION**

Permanent horizontal displacement induced by soil liquefaction is one of the most destructive phenomena during strong earthquakes. In the 1999 Chi-Chi earthquake, soil liquefaction has induced severe damage in many areas. Therefore, it becomes very important to predict the permanent displacement with a degree of reasonable accuracy to provide a reference for hazard prevention.

This study documented and mapped the case histories of lateral spreading which occurred during the 1999 Chi-Chi earthquake. The liquefied soils were remolded and tested to obtain the steady-state strength by the triaxial undrained test. Relationships between normalized steady-state strength and relative density \( D_r \) or normalized standard penetration value (N1)\( _{60} \) were established for four kinds of liquefied soils. Newmark’s rigid block sliding model [10] is used to estimate the permanent horizontal displacement with the steady-state strength parameter obtained from laboratory. It was found that the estimated displacements were in acceptable agreement with those measured in the field after earthquake.

**DYNAMIC CHARACTERISTICS OF SCHOOL AND HOSPITAL BUILDINGS**

Many structural responses of buildings and bridges around Taiwan have been recorded by TSMIP when they are subjected to seismic excitations. At the beginning of this study, the ARX model is used to identify the frequency transfer functions of school and hospital buildings. Then, a nonlinear regression analysis based on mode superposition principle is performed to the identified frequency transfer functions in order to estimate the periods, damping ratios, and effective participation factors in the first three modes.

Since the budget of ambient vibration measurement system has been cut for this study, it is necessary to seek alternatives. Usually the pre-event time is set to be 5 to 20 seconds in CWB records. The record during the pre-event time can be viewed as the ambient signal of either free field or structure. By performing the random decrement technique [11,12] to the ambient response of structures, the free-vibration response can be obtained, and then the modal periods and damping ratios is identified. However, since the resolution of the pre-event records are not good enough, data using ambient vibration measurement system still plays an essential role in obtaining reliable results.
The identified modal parameters according to the strong-motion data and the ambient data are compared to estimate the scaling factors. Once the modal parameters are identified using the actual ambient vibration measurement system, they should be multiplied by the scaling factors in order to evaluate the seismic safety before and after the earthquake.

SEISMIC CAPACITY OF REINFORCED CONCRETE FRAMES

This study investigates and compares the dynamic response characteristics of reinforced concrete frames subjected to far-field and near-fault ground excitations, respectively. The ductility demands of reinforced concrete frames are calculated by sophisticated nonlinear dynamic analysis and by simplified method in ATC-40, and the results are compared with each other. Seismic parameters that will influence the behavior of structures are also investigated. Fragility parameters of reinforced concrete frames subjected to near-fault seismic excitations are evaluated.

All of the study cases use a five-story and a twelve-story reinforced concrete frame (Fig. 8) as the targets, which have been designed according to the seismic design codes in Taiwan. The near-fault records use four of those recorded in the 1999 Chi-Chi earthquake. The 12 far-field records, three for each site, were recorded at the same stations.

As shown in Fig. 9, the seismic demands of reinforced concrete frames increase a significant amount when subjected to near-fault excitations. The results from nonlinear dynamic analysis and the simplified method in ATC-40 do not have much difference, as shown in Fig. 10.

![Fig. 8 Side views of study cases for (a) a 12-story frames, and (b) a five-story frame](image)

![Fig. 9 Maximum story drift ratios versus PGA for the twelve-story frame](image)

![Fig. 10 Spectral displacement versus PGA for the twelve-story frame using different methods](image)
FRAGILITY ANALYSIS OF BURIED PIPELINES

Buried pipelines are essential components of various lifeline systems, such as potable water distribution systems, natural gas and oil transportation systems. They are vulnerable when subjected to excessive ground motion or ground failure. The objective of this study is to propose a method to assess the damage states of buried pipelines. There are many factors that influence the damage states of buried pipelines, including material properties, diameters of pipelines, shapes, connection types, depth, soil type, as well as ground motion intensity, permanent ground displacement, wave length of ground motion, and so on.

In this study, the Aoki spectrum quasi-static solution of buried structures is used to estimate stress, strain and curvature of pipeline along longitudinal and transverse directions, considering the relative rigidity of pipeline and the surrounding soil. The equation of motion along longitudinal direction of the pipeline can be expressed as

$$
\frac{\partial^2 u_s(z, t)}{\partial t^2} + K_z u_s(z, t) - E A \frac{\partial^2 u_s(z, t)}{\partial z^2} = K_z u_s(z, t)
$$

where \(\mu\) is the mass of unit length, \(u_s\) and \(u_g\) are the displacements of pipeline and ground, respectively, \(E\) is the elastic modulus of pipeline, \(A\) is cross-sectional area of pipeline, \(K_z\) is the rigidity of soil. Let \(\alpha\) be the relative rigidity of pipeline and soil, and be expressed as

$$\alpha = \frac{2\pi}{V} \frac{E\Lambda}{K_z}$$

where \(V\) is shear velocity of soil.

Regarding the external vibration as arbitrary combination of sinusoidal waves of different amplitudes and frequencies, the maximum longitudinal strain of the pipeline can be obtained as

$$\varepsilon_{\text{max}}(z, t) = \frac{\partial u_s}{\partial z} = V \left| \sum_{n} \frac{T_n}{2\pi} D_{L_n} \bar{u}_{gn} \right|_{\text{max}}$$

$$S_{\text{max}}(z, t) = |V \times \varepsilon_{\text{max}}(z, t)|_{\text{max}}$$

The maximum longitudinal strain is used as a damage index in this study. Other quantities, such as maximum longitudinal stress or curvature, can be used as damage indices, too.

APPLICATION OF BAYESIAN LAW IN BRIDGE FRAGILITY ANALYSIS

The objective of this study is to combine the investigated bridge damage data, expert opinions and theoretical results in order to improve the accuracy of fragility parameters [13,14]. The Bayesian approach provides an effective way to update the fragility parameters when more data are available in the future. Expert opinions are served as prior distribution of fragility curves of a specific type of bridges. With the aid of investigated damage data and theoretic analysis results to form likelihood functions, Bayesian law is applied to update the posterior distribution of fragility curves of the specific type of bridges. Figure 11 shows the schematic diagram of Bayesian approach to update the fragility parameters of bridges.
DAMAGE IDENTIFICATION OF SEISMICALLY DESIGNED CONTINUOUS BRIDGES

The objective of this study is to develop a quasi-static pushover analysis to identify bridge damages due to seismic ground motion. The quasi-static pushover test will be performed in both transverse and longitudinal directions of the bridge. The modal analysis has been conducted to evaluate the coupling effects of bridge responses in the transverse and longitudinal directions. The bridge deck is assumed as a rigid body in longitudinal direction, while in the transverse direction it is treated as an elastic beam element. Because the bridge columns are the most vulnerable components among the entire system, only columns are modeled as inelastic elements. No soil-structure interaction is considered. Both fixed and updated force profiles are used in quasi-static pushover analysis. Comparison of results from both approaches is also made in this study.

Capacity curves are expressed in terms of deck displacement and total base shear, or in terms of modal displacement and effective modal mass. Inelastic seismic demand is obtained by equivalent viscous damping method. The target displacement is the intersection of capacity curves and demand curves, which is obtained by an iteration scheme. The equivalent viscous damping ratios accounted for bridges in this study are different from those adopted in the buildings. Instead of treating the entire bridge system as a whole unit, the equivalent viscous damping ratio is calculated for each individual pier. The equivalent viscous damping is combined with inherent elastic viscous damping to become effective damping.

Nau-Lan River Bridge, which is one of the highway bridges located near Hsin-Chu, is selected to develop the demand-capacity curves. This bridge is a typical multi-span continuous highway bridge in Taiwan. The bridge is constructed with pre-stressed concrete post-tensioned box
girder and monolithic rigid connection to the multi-cell box columns. Several seismic ground motions have been recorded and are used to calculate the demand curves of bridges and to evaluate the possible damages of the bridge columns. As an example, Fig. 12 shows the seismic capacity and demand curves in longitudinal direction of Nau-Lan River Bridge assuming stiff ground conditions.

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Fig. 12 Capacity and demand curves in longitudinal direction of the Nau-Lan River Bridge on stiff ground

REFERENCES


