Foreword

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Taiwan is an earthquake prone area, the government and communities on this island have become more aware of the seismic hazard. In respond to this strong urge from the public for a better seismic resiliency, the National Center for Research of Earthquake Engineering (NCREE) has prioritized state-of-the-art engineering and management tools for seismic mitigation, and hopes to eventually facilitate the authorities and communities with the know-how to confront future earthquakes.

This Research Progress and Accomplishments Report 2005 is the sixth of its series and is intended to highlight some of research tasks that are currently in progress at NCREE. A total of 42 articles covering a wide spectrum of research in 2005 have been collected. It spreads in 7 major categories:

- Development of performance-based seismic design methods to enhance earthquake resilience of new structures
- Development of structural diagnosis technologies and retrofit schemes in accordance with the urgent needs to upgrade earthquake resilience of existing structures
- Development of TELES decision support system to match the needs for earthquake emergency response and risk management
- Development of innovative structural technologies to deliver sustainable structural system
- Establishment of world-class experimental environment in support of continual improvement of experimental and numerical simulation capability
- Establishment of earthquake engineering knowledge bank for sharing and promotion of research findings
- Integration of earthquake engineering and earth science to facilitate the transformation of basic research into practical applications

Through this progress report, it is our sincere hope that the outcomes of these vibrant research efforts at NCREE could be evaluated and recognized by the researchers and engineers in the earthquake engineering community. Hopefully this information will create opportunities for exchange research findings as well as make contributions to the national coordination and international collaboration in this area.

This report is a compilation of research progress and accomplishments in 2005. Detailed research papers or reports of each individual task described in this report can be requested from the corresponding investigators. Most of the reports are available in both printed and electronic forms. The abstracts can be downloaded from NCREE website (http://www.ncree.org) in PDF format.
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III
Research on Performance-based Building Seismic Design Guideline (II)


Abstract

In addition to the performance objective, preliminary design procedure and analysis procedure that recommended in the research project of 2004. In this study, we enlarge the original guideline with the supplement of the mathematical modeling and the acceptance criteria for the steel structure. The designed example of a 12 story moment resisting frame steel structure was presented; the design procedures for this example building are linear static procedure and the nonlinear procedure. The detailed design methods for the beam and column members are presented.

Keywords: performance objective, moment resisting frame, steel structure

Introduction

In this research, we adopt the framework of SEAOC-99 as the methodology for performance-based design. The conceptual framework for performance-based design is defined as a full range of seismic engineering issues to be addressed in designing structures for predictable and definable seismic performance within established level of risk. The following design procedure issues are the ingredient parts in the methodology:

1. Select performance objective
2. Site suitability and design ground motions
3. Conceptual design
4. Preliminary design
5. Verification of preliminary design
6. Final design

In the following, we briefly describe the displacement based design steps in the preliminary design stage.

Step 1. Selection of target displacements
Step 2. Determine the effective period
Step 3. Determine the effective stiffness
Step 4. Determine the yield displacement and required system yield strength.
Step 5. Preliminary Design : System element sizes

In this study, we enlarge the original guideline draft with the supplement of the mathematical modeling and acceptance criteria for the steel structure. Finally, we give a design example of a 12 story moment resisting frame steel structure to illustrate the whole design procedure.

Design Example of the Steel Moment Frame Structure

A. Linear Static procedure

The building model selected for design is a
12-story steel frame structure. The plane view and top view of the building structure are shown in Fig. 1, the size of girder and column are shown in Table 1. The lateral force resisting system for the building is the special moment resisting frame. The story height is 4.2 m for the first story and 3.1 m from the second story to roof story. The seismic effective weight of the building is 7714 t, and the weight of each story is shown in Table 2.

The target seismic performance objective of the designed building is the basic safety objective (BSO). The BSO is the objective that achieves the dual design goals of life safety building performance level (3-C) for the BSE-1 (return period 475 years) earthquake hazard level and collapse prevention building performance level (5-E) for the BSE-2 (return period 2500 years) earthquake hazard level. The short-period (T=0.3 sec) response acceleration parameters $S_x^s=1.33$ g for the BSE-1 earthquake and $S_y^s=1.67$ g ($T_x$) for the BSE-2 earthquake, and the long-period ($T=1.0$ sec) response acceleration parameters $S_x^l=0.80$ g for the BSE-1 earthquake and $S_y^l=1.0$ g for the BSE-2 earthquake, respectively, then the characteristic period ($T_s$) of the response spectrum is 0.6 sec for both the BSE-1 and BSE-2 earthquake. The site class of the example is assumed as site 1(hard site), and the site amplification factors $F_s$ for the short period response and factor $F_v$ for the long period response are equal to 1.0.

According to the proposed regulation, the design base shear $V$ should be calculated as

$$V = C_1 C_2 C_3 C_m W$$

(1)

where

$C_1$: modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$C_2$: modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration.

$C_3$: modification factor to represent P-$\Delta$ effects.

$C_m$: modification factor to represent high mode effects.

For this example building, the fundamental period ($T$) by static analysis is 1.43 sec, the fundamental periods by dynamic analysis are 1.85 sec and 1.67 sec for the x-direction and y-direction vibration, respectively. The fundamental period ($T_s$) satisfy the condition $T\geq T_s$, then the modification factor $C_1$ is equal to 1.0, and the modification factor $C_2=1.0$ for linear static procedure, factor $C_3=1.0$ due to the post yielding stiffness is still in positive range of the example building.

<table>
<thead>
<tr>
<th>Story</th>
<th>Story weight (t)</th>
<th>$H_i$ (m)</th>
<th>$X$-direction (Fi/V)</th>
<th>$Y$-direction (Fi/V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>864.32</td>
<td>38.3</td>
<td>0.252</td>
<td>0.245</td>
</tr>
<tr>
<td>12FL</td>
<td>620</td>
<td>35.2</td>
<td>0.157</td>
<td>0.154</td>
</tr>
<tr>
<td>11FL</td>
<td>620</td>
<td>32.1</td>
<td>0.135</td>
<td>0.133</td>
</tr>
<tr>
<td>10FL</td>
<td>620</td>
<td>29</td>
<td>0.114</td>
<td>0.113</td>
</tr>
<tr>
<td>9FL</td>
<td>620</td>
<td>25.9</td>
<td>0.094</td>
<td>0.095</td>
</tr>
<tr>
<td>8FL</td>
<td>620</td>
<td>22.8</td>
<td>0.076</td>
<td>0.077</td>
</tr>
<tr>
<td>7FL</td>
<td>620</td>
<td>19.7</td>
<td>0.059</td>
<td>0.061</td>
</tr>
<tr>
<td>6FL</td>
<td>620</td>
<td>16.6</td>
<td>0.045</td>
<td>0.047</td>
</tr>
<tr>
<td>5FL</td>
<td>620</td>
<td>13.5</td>
<td>0.032</td>
<td>0.034</td>
</tr>
<tr>
<td>4FL</td>
<td>620</td>
<td>10.4</td>
<td>0.020</td>
<td>0.022</td>
</tr>
<tr>
<td>3FL</td>
<td>620</td>
<td>7.3</td>
<td>0.011</td>
<td>0.013</td>
</tr>
<tr>
<td>2FL</td>
<td>650</td>
<td>4.2</td>
<td>0.005</td>
<td>0.006</td>
</tr>
<tr>
<td>sum</td>
<td>7714.32</td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 1: member size of the steel building

<table>
<thead>
<tr>
<th>Story</th>
<th>C1 – C4</th>
<th>B1 - B4</th>
</tr>
</thead>
<tbody>
<tr>
<td>2F – 3F</td>
<td>70<em>70</em>4.0</td>
<td>H 60<em>30</em>1.3*2.4</td>
</tr>
<tr>
<td>4F – 6F</td>
<td>70<em>70</em>3.0</td>
<td>H 60<em>30</em>1.3*2.4</td>
</tr>
<tr>
<td>7F – RF</td>
<td>60<em>60</em>2.5</td>
<td>H 60<em>30</em>1.3*2.4</td>
</tr>
</tbody>
</table>

Fig. 1(a) Plane view of the building

Fig. 1(b) Elevation view of the building

Table 2: Story weight and seismic lateral force
Therefore, design base shear in x-direction \( (V_x) \) is
\[
V_x = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.80 \times W = 0.54W, \quad \text{for BSE-2 earthquake.}
\]
\[
V_x = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.80 \times W = 0.43W, \quad \text{for BSE-1 earthquake.}
\]
The design base shear in y-direction \( (V_y) \) is
\[
V_y = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.67 \times W = 0.60W, \quad \text{for BSE-2 earthquake.}
\]
\[
V_y = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.67 \times W = 0.48W, \quad \text{for BSE-1 earthquake.}
\]
\[h = \frac{1.10}{\sqrt{F_{ue}}} = 69.6\]

The design moment for the BSE-1 and BSE-2 earthquake, the design moment is governed by the BSE-1 earthquake, and the moment capacity of this beam element is
\[
M_d = \frac{(M_{dx} + M_{lx} \pm M_{ex})}{m} = \frac{(-14.2 - 3.1 \pm 492.7)}{8.0} = 59.4, -63.8 \quad \text{t-m}
\]

Comparing the design moment for the BSE-1 and BSE-2 earthquake, the design moment is governed by the BSE-1 earthquake, and the moment capacity of this beam element is
\[
M_p = \frac{SF_{ue} \times 2.5}{11477} = 114.8 \quad \text{t-m}
\]

Then the stress ratio for bending is 68.3/114.8 = 0.6 < 1.0 (OK)

The shear capacity of the beam element is
\[
\frac{87.4 + 9.1}{3.3} = 0.25 < 1.0 \quad \text{(OK)}
\]

B. Nonlinear Static Procedure

The static nonlinear procedure for the x-direction of the example building is stated as follow:

1. The preliminary design of the member size of the building structure is by the linear static procedure. The size of members is chosen to satisfy the drift limits, and the limits of reinforcement ratio as specified in the seismic design code.

2. After the preliminary design procedure, the yield moment and plastic rotation angle \( \theta_s \) of beams and columns should be determined for the performance check procedure. The yield moment and corresponding plastic rotation angle are calculated by the Table 5-5 of the guideline and show as follow:

\[
\theta_s = \frac{Z F_{ue} l_s}{6 E I_s} \quad \text{for beam element}
\]

\[
\theta_s = \frac{Z F_{ue} l_c}{6 E I_c} \left(1 - \frac{P}{P_{ue}}\right) \quad \text{for column element}
\]
(3) The structural seismic performance level is check by the nonlinear pushover analysis. The lateral load distribution, selection of control node, modeling parameters and numerical acceptance criteria for nonlinear pushover analysis are following the regulation of guideline. Figure 2 shows the analyzed capacity curve (base shear vs. roof displacement) and its corresponding effective yield strength of the building.

(4) The maximum displacement that building can resist under certain performance level is defined as: The roof displacement of control node at instance that the ductility demand of any structural member reaches its acceptance value (ductility capacity) of selected performance level. In the case of collapse prevention building performance level for the BSE-2 earthquake level, when the roof displacement of control node reaches 62.0 cm, the required ductility demand of column of second floor (base floor) approaches the acceptance value of collapse prevention performance level as regulated by guideline. Then the maximum displacement of the building for collapse prevention performance level is equal to 62.0 cm, at this instance, the maximum story drift occurred at 5th floor and its drift reach 2.57%, this drift value is less than the acceptance criteria (4%) by guideline. Figure 3 shows the distribution of plastic hinges at the collapse prevention performance level. By the same approach, the maximum displacement of the building for life safety performance level is equal to 54.0 cm.

(5) According to the regulation of guideline, the target displacement \( \delta_t \) is calculated as

\[
\delta_t = C_0 C_1 C_2 C_3 S \frac{T^2}{4\pi^2} \quad (3)
\]

where \( C_0 \) : modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system buildings, and equal to 1.3 in this example. The target displacements of life safety performance level (BSE-1) and collapse prevention performance level (BSE-2) are:

BSE-2: \( \delta_t = 1.3 \times 1.0 \times 1.0 \times \frac{1.0}{1.91} \times \frac{1.91^2}{4\pi^2} \times 980 = 61 \text{ cm} < 62 \text{ cm}, \text{OK} \)

BSE-1: \( \delta_t = 1.3 \times 1.0 \times 1.0 \times \frac{0.80}{1.91} \times \frac{1.91^2}{4\pi^2} \times 980 = 49.3 \text{ cm} < 54 \text{ cm}, \text{OK} \)

From above equation, in the collapse prevention performance level (BSE-2) and the life safety performance level (BSE-1), the target displacement is less than the corresponding maximum displacement. It is satisfy the requirement of target displacement requirement, the design result of this example building by linear static procedure is able to satisfy the requirement of nonlinear static procedure.

Conclusions

In this study, the designed example of a 12 story moment resisting frame steel structure was presented; the design procedures for this example building are linear static procedure and the nonlinear static procedure. The detailed design methods for the beam and column members are presented.

References


Applied Study of Nonlinear Seismic Demand Spectra and Seismic Performance Indices Considering the Properties of Strong Ground Motions

Yuan-Tao Weng, Wen-Yu Jean, Bo-Zhou Lin, and Sheng-Jhih Jhuang

Abstract

The characterization of earthquake demands for seismic analysis or design requires the specification of a level of intensity of the ground motion. Response history analyses require scaling of the ground motion records to a specified level of intensity. This study investigates the correlation between several ground motion intensity indices and four response variables: elastic and inelastic spectral ordinates, input energy and hysteretic energy, based on responses calculated for a specific set of earthquake records. An essential point of this study is that ground motion indices are relevant as long as they are a sign of the level of structural response. As expected, it is found that no index is satisfactory over the entire frequency range. Indeed, indices based on the ground acceleration history rank better in the acceleration-sensitive region of the spectrum, indices based on ground velocity are better in the velocity sensitive region, and correspondingly occurs in the displacement-controlled region. A rank of indices is presented, according to their correlation with response quantities. On the other hand, the structure’s ability to survive an earthquake may be measured in terms of the expected state of damage of the structure after the earthquake. Damage may be quantified by using any of several damage indices defined as functions whose values can be related to particular structural damage states. A number of available response-based damage indices are discussed and critically evaluated for their applicability in seismic damage evaluation. This study try to construct a new rational approach for damage assessment which provides a measure of the physical response characteristics of the structure and is better suited for non-linear structural analysis.

Keywords: damage index, vulnerability analysis, fragility curve, seismic demand

Introduction

In recent years, the development of structural design criteria for new structures and the renewed importance of the assessment of seismic vulnerability of existing under-designed buildings have broadened the objectives of seismic design. While safety against collapse is still the main goal, performance in terms of functionality and economy have assumed a central role in the design criteria. Hence, a great effort has been made to improve the current earthquake-resistant design methods in order not only to avoid collapse under a destructive earthquake, but also to limit the damage under moderate earthquakes. Furthermore, the new design philosophy is tending to multi-level probabilistic structural performance criteria, replacing completely the simple force strength approach. However, the implementation of all these new concepts requires the definition of a quantitative damage index and measures.

A key issue in the performance-based design is a...
reliable assessment of seismic damage potential. If the expected intensity of the earthquake is greatly underestimated, the cost of new construction and seismic rehabilitation of existing structures could be excessive. On the other hand, if the intensity is seriously underestimated, the results may be heavy damage and loss of life. To this end, a reliable definition of seismic intensity has to relate to the effect of damage on structural behavior in order to assess the potential seismic hazard and to classify the seismic input. Different reports have been published recently on the use of a damage index and damage measures in earthquake engineering. They aim to clarify the different approach methodologies and to detail different proposed formulations. This study will focus mainly on the cross-correlation between ground motion parameters used as damage potential measures and structural and nonstructural damage measures.

**Derivation of new vulnerability functions for Taiwan**

Damage indices may be defined locally, for an individual element, or globally, for a whole structure. Most local indices are cumulative in nature, reflecting the dependence of damage on both the amplitude and the number of cycles of loading. The main disadvantages of most local damage indices are the need for tuning of coefficients for a particular structural type and the lack of calibration against varying degree of damage. Global damage indices may be calculated by taking a weighted average of the local indices throughout a structure, or by comparing the modal properties of the structure before and after (and sometimes during) the earthquake. The weighted-average indices are prone to much the same problems as the local indices. The modal indices vary widely in their level of sophistication, those capable of detecting relatively minor damage requiring the accurate determination of a large number of modes of vibration. The development and application of damage indices has until now concentrated almost exclusively (and sometimes during) the earthquake. The proposed methodology for vulnerability curve derivation prescribes the analysis of a population of RC buildings subjected to a number of earthquake records with distinct characteristics. It is thus able to account for the effect of variability in seismic input and structural characteristics on the damage statistics simulated for the building class (“system”), and evaluate the associated uncertainty in the vulnerability prediction. The procedure may be regarded as consisting of four main steps. Step 1, “system definition”, consists in the selection and design of a single structure with material, configuration and seismic resistance characteristics that are representative of the building class being assessed. Deviation in seismic resistance of buildings within the class is considered through the analysis of a population of building models, generated from the general system design by varying its structural properties. Step 2, “definition of ground motion input”, involves the selection of suites of earthquake records for the analysis. Within the proposed methodology different suites of accelerograms are adopted in the derivation of each limit state curve. These “performance-consistent” record suites are selected in accordance with spectra that are characteristic of a seismic event with a return period, for which the structural damage state defining the curve is acceptable or is expected. Designs of experiment procedures are considered in the selection of the population and earthquake record suite sizes for the analyses. This is done in order to optimise computational effort and to guarantee convergence of results. Step 3, the “model evaluation”, is carried out using an innovative pushover analysis technique within a capacity spectrum framework of assessment. This Pushover analyses can account for the effect of ground motion characteristics on structural response and require reduced computational effort compared to time history analyses. The maximum inter-storey drift response of each structure within the population is assessed, for increasing intensities of ground motion, using a modified capacity spectrum method. This assessment method avoids the repetition of analyses for increasing ground motions, and further reduces the analysis number from several thousands to a few hundred. Step 4, the “statistical processing of analysis results”, involves the definition of response surfaces, relating the observed inter-storey drift response to the structural property and ground motion parameter values. Response surface equations are defined for each hazard scenario, through separate consideration of the analysis statistics resulting from each suite of performance-consistent records. A re-sampling technique is adopted to generate building damage statistics from the response surfaces, for a range of ground motion severities. Hence, vulnerability curves are plotted. The response surface equation used to develop each damage state curve is selected according to the hazard level associated with the satisfaction of a desired performance objective. The generated vulnerability curves may therefore be defined as being “performance consistent”. Uncertainty in the damage state prediction, and its variation with increasing ground motion intensity, is accounted for through vulnerability curve confidence bounds. These are determined from consideration of the fit of the analytical observations of maximum inter-storey drift to the response surfaces, within the damage histogram generation process.

The choice of building design and material properties assumed for the model are dependent on the composition of the building stock in the assessed region. The buildings must be grouped into categories with similar lateral load resistance and different
vulnerability curves must be derived for each building class. The overall risk to the population for an earthquake of given size may then be obtained by combining the damage state exceedence probabilities for each structure class according to their relative proportions in the assessed region. It is emphasised that any structural typology may be assessed within the framework of the proposed methodology through an appropriate selection of structural model, structural properties for variation and their corresponding probability distributions, and the ground motion input.

Different from traditional format of fragility curve (e.g. Fig. 1) using PGA as earthquake intensity, this study try to construct a innovative format of fragility curve using specific damage index for damage assessment shown as Fig. 2, which provides a measure of the physical response characteristics of the structure and is better suited for non-linear structural analysis.

**Ground motion scaling methods for different site conditions and structure characteristics**

This study also investigates the effectiveness of seven ground motion scaling methods in reducing the scatter in estimated peak lateral displacement demands. Non-linear single-degree-of-freedom systems and non-linear multi-degree-of-freedom systems are considered with different site conditions (site soil profile and epicentral distance) and structural characteristics (yield strength, period, and hysteretic behavior). It is shown that scaling methods that work well for ground motions representative of stiff soil and far-field conditions lose their effectiveness for soft soil and near-field conditions for a wide range of structural characteristics.

Figures 3 show the COV-spectra for the scatter in nonlinear displacement $\triangle_{nlin}$ using the soft soil ground motion ensembles, respectively, for the bilinear hysteretic type with post-yield stiffness ratio $\alpha = 0.10$.

**Correlation between intensity indices and response**

Generally speaking, the intensity of motion cannot be satisfactorily characterized by a single parameter. As it will be shown herein, different intensity measures are suitable in the three characteristics spectral regions: short period (acceleration sensitive systems), intermediate period (velocity controlled responses), and long period (displacement sensitive systems). These three spectral regions were first identified by Newmark-Veletsos-Hall in their pioneer work on earthquake response amplification for the derivation of design spectra. Ground motion parameter could be divided two types: peak parameters and integral parameters. Riddell and Garcia (2001) found that the combined index

$$I_d = \frac{d_{max}}{T_d^{1/3}}$$

permitted to minimize the dispersion of hysteretic energy dissipation spectra for low frequency inelastic systems.

In the case of inelastic systems two response variables were considered: the maximum deformation $u_{max}$ and the hysteretic energy $EH$ dissipated by the oscillator, both of them for a response associated to a displacement response ductility $\mu = 3$. The specific value of $\mu$ chosen is not a limitation since similar conclusions are reached if other values are used. The total hysteretic energy dissipated per unit of mass is defined as:
To visualize the correlation among response quantities and intensity indices, plots like Figure 4 were made for $I_D$ and $E_H$. Figure 4 relates Riddell-Garcia’s index $I_D$ (Equation 1) with hysteretic energy $E_H$ for stiffness-degrading systems with response ductility associated to a response ductility factor $\mu=3$; there is no correlation at all at $f=5$ cps, extremely good correlation at $f=0.2$ cps ($\rho=0.967$).

Figure 4 Correlation between Riddell-Garcia’s index ($I_D$) and dissipated energy ($E_H$) for stiffness degrading systems with response ductility $\mu=3$.

**Conclusions**

This study has attempted to contribute to a better understanding of ground motion intensity indices used for specification of design ground motions or for normalizing or scaling ground motions for earthquake response studies. It was found that: a) No index shows satisfactory correlation with response in the three spectral regions simultaneously, indeed, acceleration related indices are the best for rigid systems, velocity-related indices are better for intermediate frequency systems, and displacement-related indices are better for flexible systems; b) The peak ground motions parameters ($a_{\text{max}}$, $v_{\text{max}}$, $d_{\text{max}}$) present very good correlation with elastic and inelastic spectral ordinates in their corresponding frequency ranges. Peak ground acceleration and displacement also present good correlation with input and hysteretic energies in their respective spectral regions, $v_{\text{max}}$ however does only moderately well. Since peak ground motion parameters can be established for future earthquakes with relative ease, on the basis of generally accepted available methodologies for earthquake hazard assessment, they are strongly recommended as intensity indices; c) The duration of motion is effective in increasing the correlation with energy responses when combined with other simple indices in product form. Compound indices proposed by Park et al., Fajjar, and Riddell and Garcia, rise to reach the top or nearly the top rank in their corresponding frequency ranges. In turn, root-square values of the ground motion histories always present better correlation with energy responses than root-mean-square values do, thus reflecting the effect of duration.

**References**


Performance-based Seismic Design and Retrofit of Bridges with Functional Bearing Systems(Ⅱ)

Kuo-Chun Chang

Abstract

This study is aiming at developing displacement based design methods for bridges with functional bearing systems and their life cycle behavior. Based on recent investigations of existing bridges damaged from major earthquakes, particular in the 1999 Chi-Chi earthquake, it is found that the bearing system of the bridge, including the movements of the bearing, shear keys and restrainers, plays a vital role for the behavior of the bridge during a major earthquake. This study proposes a three-year study to understand the effect of bridge bearing systems and to propose simplified and economical seismic retrofit methods for existing bridges based on the bearing systems. Basic mechanical behaviors and performance levels in each bearing component and the interrelationship among the different components will be tested to provide appropriate analytical models for numerical simulations. Besides, this project will also review and compare different pseudo-dynamic techniques and apply the discrete-time state-space numerical integration processes to build up a real-time bridge simulator.

Keywords: bridge, bearing system

Introduction

In Taiwan, the bridge type is different from America and Japan. Over the decades, a few large earthquakes stroke major cities in the world, causing tremendous losses on the transportation system. Therefore, the seismic design code is upgrading to prevent the likely damages in the future, and new design methods, such as ductile design, base-isolation design, and damper device design, are developing and applying in the construction practices. Nowadays, bridge retrofit puts more emphasize on the columns and the corresponding design criteria is developed. However, after enhancing the strength of the column, engineers should pay more attention to check the shear capacity of the foundation to prevent shear failure. It is likely to enlarge the footing size and add outside piles, only after the foundation is uncovered. With the limited financial support of the nation, this kind of design philosophy is not a well suggestion for all damaged bridges.

From the “The reconnaissance report of the Chi-Chi earthquake for bridges and transportation facilities[1],” over 1,094 investigated bridges, the supper structures suffer much damages than columns.

It is found that the bearing system of the bridge, including the movements of the bearing, shear keys and restrainers, plays a vital role for the behavior of the bridge during a major earthquake. To understand the reasons, one study[2] has a bridge model to simulate the seismic behavior in the Chi-Chi earthquake, and the results provide similar conclusions. Therefore, this project proposes a three-year study to understand the effect of bridge bearing systems and to propose simplified and economical seismic retrofit methods for existing bridges based on the bearing systems. In addition, combining with the performance-based design concept, this study will develop design methodologies to provide sufficient capacity on the existing bridges and limit the damage level to be acceptable under a given seismic demand. This study is aiming at developing displacement based design methods for bridges with functional bearing systems and their life cycle behavior.

Frame of the program

Total of one mother-project and 11 sub-programs are included and separated into three groups in this
program. Group1 carries out the mechanical tests of the bridge component, and provides probable mathematical models; Group2 is responsible for the nonlinear analyses, using verified models developed from group1; Group3 takes charge of the recommendation for the draft of the seismic performance-based design code, based on the experiments and studies from group1 and group2. Both group1 and group2 will share the same resource on a scale-down bridge model specimen and apply the discrete-time state-space numerical integration processes. The topics of 11 sub-projects are described below:

(1) Seismic analysis and design of naked-pile bridges with function bearing system
(2) Displacement-based design of bridges with functional bearing system
(3) Seismic behavior of pot-bearing
(4) Nonlinear analysis of bridges system with functional bearing system
(5) Seismic behavior of the restrainers
(6) Seismic behavior of the shear key and stoppers
(7) Nonlinear dynamic analysis and failure mechanism study of bearing system
(8) Study on the bridge failure mode and optical design of the bridge with bearing system
(9) Improved pseudo-dynamic techniques using fiber network controller
(10) Seismic behavior of the abutment
(11) Semi-control on the bridge with functional bearing system

Research of mother project

This project focuses on the study results on the mother-project. The followings are two main results: friction mechanism of the rubber bearings[3] and one scale-down single span bridge taking table test.

The behavior of steel-reinforced elastomeric bearings from friction coefficient tests and pseudo-dynamic tests are studied. The bearings, so called rubber bearing (RB), are widely used on the existing bridges in Taiwan. It was shown that the friction-sliding mechanic played a critical role against the seismic load passing to the substructures. Based on the fact of damages between bearing systems and columns, this test aims at understanding the friction behavior of RB. Friction coefficient tests are conducted first in a test frame at NCREE lab and followed by the SDOF pseudo-dynamic tests; finally, the whole structure is modeled in the program SAP2000 to verify the accuracy of RB element. The test setup is shown in the fig.1.

Fig. 1 Setup up of the friction test for the Rubber bearing specimen

In order to understand the behavior of the bridge with functional bearings, we perform a Scale-down shaking table test. The test specimen is scaled based on the “Bridge design drawing” of the Director General of Highway, MOTC. The prototype bridge is a simple support bridge with PCI girders and two single columns. The girder is 30m long, and 9m width. The bridge column is 11m height. The diameter of the column is 2 to 2.4m wide from top to bottom section. The weight of superstructure and substructure are 5062 and 1421.37kN, respectively. To put a scale-down model on the 5x5 meter shaking table, the scale factor 7.5 is chosen for the prototype bridge. The scaled deck is 5m long, 1.75 wide, and 0.138 in thickness. The scaled girder is 5m long, 0.2m wide, and 0.3m in height. The scaled column is a CFT column, with outside diameter about 0.216m, and inside diameter about 0.2m. The material for column is A36 steel. The design strength of concrete and steel reinforcement are 20.6MPa and 285.43MPa.

Fig. 2 Test setup of the scale-down bridge specimen

Fig. 2 shows the test setup of the scale-down bridge specimen. In order to understand the influence of different bearing system, both rubber bearing and rolling type bearing (RTB) are tested separately. Before regular tests of seismic input ground acceleration, the white noise is used first to determine
the resonance frequency. For rubber bearing, the resonance frequency is 2.5Hz; for rolling type bearing, the frequency is depended on the displacement it could move. The input time histories are El-Centro, Kobe, Northridge, TCU068 and TCU129.

From the preliminary tests results, the acceleration on the deck is limited in 100gal for RTB, but higher than the input for RB. In addition, the acceleration records on the cap-beam are higher than that at the bearing in both two types of bearing. It might be the reason that the inertial force induced by the mass of the cap-beam cause the high acceleration. The displacement of superstructure is also compared. There are no friction-sliding phenomena in the tests with RB, only shear deformation. It is because the horizontal stiffness is much higher than the designed scale-sown requirement. The smaller RB will be used in the future experiments to allow the friction displacement. On the other hand, when using RTB, the displacement is much large due to the very small friction coefficient 0.008. There is a need to add external damper devices to reduce the displacement.

**Research of the sub-projects**

A series of mechanical tests of the bridge component are performed. These tests include pot bearing test, restrainers test, shear key test. The test setup are shown in the fig.3 to fig.5. These test results will provide probable mathematical models

Summaries of those sub-projects are as follows:

**The Experiment and Analysis of Restrainers for Bridges:**

Determination of the appropriate strength for the restrainers and reduction in displacement caused by stiff restrainers are difficult unless dynamic inelastic analyses are carried out. We develop the design and analysis methodology or guideline of the restrainers for the usage in Taiwan. The ANSYS finite-element softwares will be used in this project to simulate the force in the restrainer under earthquakes.

**Nonlinear Numerical Analysis for the Performance Design of Bridge Bearing System:**

In this first year report of a three-year project, explicit finite element code is used to simulate the nonlinear behavior of the bridge bearing system. The nonlinear behavior of rubber bearing was investigated in detail. Three types of material models in the LS-DYNA coder were evaluated by comparing with the experimental results of the compression and shearing tests of rubber bearing. It turns out that the Hyperelastic Rubber model can simulate the performance better than the Kinematic Hardening Plastic Material model and the Mooney-Rivlin Rubber model. The material parameters obtained from those validation simulations will be used to conduct the numerical analysis of rubber bearing of other type of design of rubber bearing device. The comparison of the simulation and experiment for the compression test is shown in fig.6. The performance curves of the rubber bearing device in the local level will be linked with the global structural analysis of bridge under seismic loading.
Fig. 6 comparison of the simulation and experiment for the compression test

Analysis and design of seismic capacity enhancement of foundation scoured bridges with seismic performance bearings:

This sub-project intends to build up a process using the capacity spectrum analysis to determine the seismic capabilities. To model accurately the bridge studied, we have to determine the failure types of members at first. Afterwards, we set up the corresponding hinge properties. This study establishes the capacity curve of the bridge by carrying out pushover analysis, and obtains the corresponding capacity spectrum then. The seismic capacity is defined as the ground acceleration corresponding to the performance point of failure.

Fast Pseudo Dynamic Testing of Performance-Based Designed Bearing Bridge through SCRAM-Net Configuration:

With the advance in material science, a lot of high damping materials have already been utilized as vibration control devices to mitigate the dynamic response of civil infra-structures subjected to earthquake excitation. Among their complicated behavior, velocity dependence and highly nonlinearity are major mechanisms to provide larger energy dissipation capacities through their hysteretic characteristics. The traditional pseudodynamic testing methods are often unable to perform required tests to these devices before their fully implementation in real structures. In order to develop a real-time pseudodynamic testing apparatus, both simulation algorithms and required software/hardware are introduced and examined in this thesis. Especially, a real-time fiber-optic connected share memory network is proposed to perform the required parallel operation in real-time pseudodynamic testing. In order to investigate and develop this technology, a general hysteretic model is derived as a simulated specimen to interact with a state-space based simulation computer. Implementation related issues such as software/hardware integration, real-time signal communication, multi-sampling rates, model/specimen interface, and time delay effect are examined and discussed in detail. A linear steel frame with a diagonal bracing is used as the testing model to implement the proposed concept. The phase-shift compensation technique, commonly used in traditional control problem, is applied to overcome the effect of time-delay.

References


Study on Seismic Records of Building Structures and its Application to Design Codes

L.L. Chung¹, C.L. Lin¹, Y.K. Yeh¹, K.W. Chou¹, T.K. Chow¹, and S.J. Hwang¹

Abstract

The Central Weather Bureau has deployed seismic monitoring system on building and bridge structures. In case of earthquakes, structural responses can be recorded for further study. In this research project, structural characteristics are extracted from the seismic responses of structures by means of system identification. The identified results are compared with the regulations of design codes. Based on the design drawings of the structures, analysis and simulation are conducted. The analysis models are improved with the seismic records of the structures. The revised methods of analysis and simulation are transferred to engineers.

Keywords: system identification of structures; analysis and simulation of structures; seismic records of structures

Introduction

The purpose of research on earthquake engineering is to enhance the seismic performance of structures. The Central Weather Bureau launched the program for seismic monitoring system of structures in 1993 so that seismic records can be provided for research and development. In this research project, analysis and simulation study can be carried out based on the seismic responses of the building structures. Structural characteristics are extracted through system identification. The results can be fed back for the revision of seismic design codes. Analysis models are developed for the structures such that the accuracy of the models can be improved by comparing with the seismic responses.

Dynamic characteristics from system identification for diagnosis of structures have been applied in many fields. In civil engineering, utilizing structural responses, dynamic characteristics such as natural frequencies, damping ratios and mode shapes can be obtained through system identification. In addition, the nonlinear behavior of structures is investigated through time history analysis using structural analysis programs. The analyzed results are compared with the measurement such that the models of structures can be revised. The findings in the technique of structural analysis can be transferred to engineers so that they can master the seismic performance of structures much better.

Seismic Records of Building Structures

In order to immediately and accurately record the seismic responses of structures, the Central Weather Bureau launched to deploy seismic monitoring system on civil engineering structures. Up to now, there are 61 stations throughout Taiwan including 44 stations for building structures and 17 stations for bridge structures. A lot of seismic records for the structures have been accumulated. Using the program, XPLAYV, provided by the Central Weather Bureau, all the channels of all stations have been checked whether they are normally operated or not. The seismic records from the 2002/3/31 earthquake, the 2003/6/10 earthquake and the earthquake in the first quarter of 2004 have been checked. Among 44 building structures, 18 of them are with no abnormal channels, 20 with partial abnormal channels, 0 with majority abnormal channels, 4 with no data or abolished stations, and 2 newly established. Among 17 bridge structures, 6 of them are with no abnormal channels, 7 with partial abnormal channels, 1 with majority abnormal channels, and 3 with no data or abolished stations.

In this research project, design drawings of architects and structures are collected for the

¹ National Center for Research on Earthquake Engineering
System Identification

The building structures are considered as linear dynamic system. In discrete-time mode, the relationship between single input and single output (SISO) can be expressed using linear difference equation as:

\[ y(k) + a_1 y(k-1) + \cdots + a_n y(k-n_a) = b_1 u(k) + b_2 u(k-1) + \cdots + b_m u(k-n_b) \]  

(1)

where \( y(k) \) is the output; \( a_i \)'s are the coefficients of the output; \( n_a \) is the order of the output; \( u(k) \) is the input; \( b_i \) is the coefficient of the input; \( n_b \) is the order of the input.

According to the ARX (Auto-Regressive exogenous) model, the difference equation can be further expressed as:

\[ y(k) = \Psi^T(k)\theta + e(k) \]  

(2)

\[ \Psi^T(k) = [-y(k-1) \cdots -u(k-n_a)] \]  

(3)

\[ \theta = [a_1 \cdots b_m] \]  

(4)

where \( e(k) \) is the noise which is usually assumed to be white noise. The parameters can be obtained by using Recursive Prediction Error Method (RPEM). The solution is:

\[ \theta(k) = \theta(k-1) + L(k)[y(k) - \hat{y}^T(k)\hat{\theta}(k-1)] \]  

(5)

where

\[ L(k) = \frac{P(k-1)\Psi(k)}{\lambda(k) + \Psi^T(k)P(k-1)\Psi(k)} \]  

(6)

\[ P(k) = \frac{P(k-1) - \lambda(k)P(k-1)\Psi(k)}{\lambda(k) + \Psi^T(k)P(k-1)\Psi(k)} \]  

(7)

Usually, the initial condition for \( P(0) \) is assigned in the range of \( 10^3 I \) to \( 10^6 I \) in order to accelerate the rate of convergence.

Since structural characteristics are related to the coefficients, \( a_i \)'s. From the coefficients, \( a_i \)'s, the natural frequency \( f_j \) and damping ratio \( \xi_j \) can be computed as:

\[ f_j = \frac{\sqrt{(\ln r_j)^2 + \phi_j^2}}{2\pi\Delta t} \]  

(8)

\[ \xi_j = -\frac{(\ln r_j)^2}{\ln r_j \sqrt{(\ln r_j)^2 + \phi_j^2}} \]  

(9)

where \( \Delta t \) is the sampling period;

\[ r_j = p_j/\overline{p}_j \]  

(10a)

\[ \phi = \tan^{-1}(\text{Im}(p_j)/\text{Re}(p_j)) \]  

(10b)

\( p_j \) is the j-th complex root of the polynomial with coefficients, \( a_i \)'s.

Moreover, the parameters, \( \theta \), can be found by using recursive artificial neural network. The effectiveness is compared.

The parameters defined in equation (4) are defined as the weightings for the training of the artificial neural network. Ground acceleration is the input of the artificial neural network; floor acceleration is the output; and ARX is the model for estimation. The model can be expressed as:

\[ a = \text{purelin}(IW \ast p + b) \]  

(11)

In the above equation, \( a \) is the output of the artificial neural network, the floor acceleration. \( IW \) is the weightings, the parameters of ARX model. \( \text{purelin} \) is the transfer function which is called linear transfer function, \( f(x) = x \). \( p \) is the input, the ground acceleration. \( b \) is the bias. The weighting is searched such that the following error index \( EI \) is minimized:

\[ EI = \frac{1}{N} \sum_{k=0}^{N-1} [y(k) - \hat{y}(k)]^2 \]  

(12)

where \( N \) is the number of measurement samples; \( y(k) \) is the measured output; and \( \hat{y}(k) \) is the estimated output.

By comparing with equation (4), the weighting in the artificial neural network is the same as the parameter in the ARX after the noise and the bias are assigned to be zero. This model is called Auto Regressive eXogenous with Artificial Neural Network (ARX-ANN). The framework of the model is shown in Fig. 1.
In the first case, the building structure for system identification is the office building of NCREE and the earthquake event is the Chi-Chi earthquake occurred on September 21, 1999. X-axis is defined as the direction along Xinhai Road and y-axis is the direction perpendicular to Xinhai Road. For the RPEM, the input signal is the ceiling of the basement, and the output signals are the responses on the 1F, 3F and 6F, respectively. The orders of the ARX model are chosen as $n_x = n_y = 50$ for the output and input. SISO model is adopted for the identification of natural frequency and damping ratio. The natural frequencies identified from different floors are close to one another and the average value is 3.27 Hz. From the roof floor acceleration, the damping ratio is identified to be 7.0%. For the ARX-ANN, the input signal is the floor acceleration of the basement, and the output signals are the responses on the 1F, 3F and 6F, respectively. From the roof acceleration, the natural frequency and damping ratio are identified to be 3.67 Hz and 7.35%, respectively. The error index for the system identification is 0.2401.

In the second case, the seismic responses of Civil and Environmental Engineering Building at the National Chung Hsing University in the Chi-Chi earthquake are used for system identification. System identification is conducted in both x-axis and y-axis. 1F acceleration is considered as input while 4F and RF accelerations are considered as output. Since the feasibility of system identification has been illustrated in the first case, it is extended to Single Input Multiple Output (SIMO). The orders of the output and input in the ARX model are $n_x = n_y = 50$. From the identification results, in the x-axis, the first and second modal properties are, respectively, 2.14 Hz and 7.74 Hz in natural frequencies, and 10% and 9% in damping ratios. In the y-axis, the first and second modal frequencies are 2.18 Hz and 7.55 Hz, respectively. The first and second modal damping ratios are 8% and 9%, respectively.

**Numerical Analysis and Simulation**

The Office Building of NCREE is adopted for numerical analysis and simulation. It is a six-story building with basement of one story. 30 sensors are mounted on the basement floor, basement ceiling, third floor ceiling and sixth floor ceiling. The commercial program, SAP2000, is used to establish the mathematical model for the building in two ways without and with infilled walls (Figs. 2 and 3). Basement floor accelerations (channels CH04 and CH06) are considered as input. The numerical responses at the sixth floor ceiling are compared with the seismic records at the same location (channels CH16 and CH18).

According to the Seismic Design Codes of Building Structures, the empirical formula for the natural period $T = 0.070 h^{1/4}$ (unit: sec.) of reinforced concrete building is expressed as:

$$ T = 0.070 h^{1/4} \quad (13) $$

where $h$ is the height of the building in meters. Since the height of the office building is 23.2 m, the natural period computed from the seismic code is:

$$ T = 0.070(23.2)^{1/4} = 0.73 \text{ (sec.)} \quad (13) $$

Based on SISO model, the natural periods of the office building are identified to be 0.24 sec. (4.16 Hz) in the x-axis and 0.30 sec. (3.32 Hz) in the y-axis. If the infilled walls are neglected, the numerical results show that the natural periods are 0.75 sec. in the x-axis and 0.6 sec. in the y-axis. Based on SISO model, the natural periods of the Office Building of NCREE are identified to be 0.24 sec. (4.16 Hz) in x-axis. On the other hand, if infilled walls are taken into account, the natural periods from numerical simulation are decreased to 0.25 sec. in the x-axis and 0.26 sec. in the y-axis. With consideration of infilled walls, the natural periods from numerical simulation are close to those from system identification. Therefore, infilled wall cannot be neglected in the calculation of natural periods. When the infilled walls are neglected, natural periods from numerical simulation are close to those from seismic...
design codes.
Moreover, finite element models of the structures are also developed by means of MSC.Patran. Analysis of dynamic characteristics and seismic responses is carried out through MSC.Nastran. From the numerical results, the natural periods of the Office Building of NCREE are about 0.23 sec. in both x-axis and y-axis. The natural periods of Civil and Environmental Engineering Building at the National Chung Hsing University are 0.2 sec. in x-axis and 0.3 sec. in y-axis. However, from the empirical formula stated in the seismic design code, the natural period is 0.82 sec. since the height of the building is 26.8 m. The discrepancy is quite significant.

Conclusions
In this paper, system identification and simulation analysis of two building structures are illustrated. It is found that simulation of walls is so important to the dynamic analysis of building structures that they cannot be neglected. Design drawings of more building structures will be collected for research on system identification and simulation analysis. Moreover, data base of seismic records, design drawings and research accomplishments will be established. The platform will be available for engineering community.

Acknowledgements
The contributions from Central Weather Bureau, NCREE, National Chung Hsing University, Shi-Xian Primary School, National Taiwan University of Science and Technology, Sinotech Engineering Consultants, Inc., Professor C.L. Lee, Professor Steve Huang, Professor J.Y. Hwang, Professor J.Y. Ching to this research project are highly appreciated.
Numerical Simulation of a Collaborative Experiment on a Double-Skinned Concrete-Filled-Tubular Bridge

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Abstract

This paper presents numerical simulation results of a Taiwan-Canada collaborative experiment simulating a double-skinned concrete-filled-tubular (DSCFT) bridge subjecting to a series of bi-lateral earthquakes. Two numerical models based on different approaches are employed. These models are called Hinge Model and Fiber Model in this paper, and implemented and analyzed by PISA3D and OpenSees, respectively. Material and element properties are mainly based on uni-axial steel tensional tests and concrete compressive tests and then tuned based on a previously completed DSCFT cyclic experiment. The numerical results of the two models are reasonably close, and the differences between them are discussed. The estimated maximal shear forces and displacements are used to check that each laboratory has sufficient capacities to complete the experiment. Further studies on the DSCFT behaviors and the numerical models will be further studied after the experiment. This experiment not only to demonstrate the seismic performance of DSCFT piers, but also establishes a collaboration mechanism of earthquake engineering experiments between Taiwan and Canada.

Keywords: Double-skinned concrete-filled-tubular piers, numerical simulation, PISA3D, OpenSees, nonlinear dynamic finite element analysis

Introduction

A collaborative pseudo-dynamic experiment is being conducted by National Center for Research on Earthquake Engineering (NCREE), National Taiwan University (NTU) and Carleton University (CU). The experiment simulates the dynamic responses of a five-span bridge with four double-skinned concrete-filled-tubular (DSCFT) piers (see Fig. 1). This experiment is not only to demonstrate the seismic performance of DSCFT piers, but also establishes a collaboration mechanism of earthquake engineering experiments between Taiwan and Canada. Further details of the collaborative experiment can be found in other published papers (e.g., Yang et al., 2006 or Chang et al., 2005) and are not repeated in this paper.

Two Numerical Models

Two numerical models based on different approaches are employed. These models are called
Hinge Model and Fiber Model (Spacone et al., 1996) in this paper, and implemented and analyzed by PISA3D (Lin & Tsai, 2003; Hsu & Tsai, 2003; Tsai & Chuang, 2003) and OpenSees (McKenna & Fenves, 2000), respectively. Modal analyses, cyclic push-over analyses and transient dynamic analyses are conducted. Table 1 lists a rough comparison between the two models.

<table>
<thead>
<tr>
<th>Table 1 Comparison between two models</th>
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</thead>
<tbody>
<tr>
<td>Tools</td>
</tr>
<tr>
<td>Beam-column (BC) element used</td>
</tr>
<tr>
<td>Important assumption</td>
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<tr>
<td></td>
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<tr>
<td>Element internal force calculation</td>
</tr>
</tbody>
</table>

The widely used Newmark integration method is employed for transient dynamic analysis for both models. In the Fiber Model analysis, a modified Newton algorithm is used to decrease the unbalance forces; while in Hinge Model analysis, there is no iteration within time step and the unbalance force is taken by its next time step. Time increment sizes are selected for reasonable convergence.

**Models of a Completed S24 DSCFT Test**

A previously completed DSCFT cyclic push-over test, which specimen was numbered “S24” in Tsai & Lin (2002), is used to preliminary evaluate the aforementioned two models. Proper material parameters of the Fiber Model are tuned based on the experimental data (Filippou & Spacone, 1996).

The nonlinear behavior of the Hinge Model is based on an equivalent flexural stiffness based on the empirical formulation of concrete-filled-tubular piers (Mander et al., 1988). The beam-column element is modeled by homogeneous material with bi-linear stress-strain models. Further description of the Hinge Model can be found in Hsu & Lin (1997) and is not repeated here.

The Fiber Model employs the displacement-based beam-column element type implemented in OpenSees. The fiber sections within the beam-column elements are consisted with concrete and steel fibers, as shown in Fig. 2. The vertical line in Fig. 2 represents the DSCFT column, while the horizontal line is a uni-axial spring for displacement control analysis. The FEDEAS steel and concrete models (named steel02 and concrete02 in OpenSees, respectively) are selected to represent the nonlinear behavior of the moment-curvature properties of each fiber section in the beam-column elements. Mander’s concrete confinement formulation (Mander et al., 1988) is used to calculate the concrete strengths of confined by tubular steel tubes.

In order to match the S24 experimental result, the steel and concrete materials’ strengths of Fiber Model are reduced. The steel strengths are reduced to 90% of uni-axial tensional test strengths, and the initial Young’s modules are set to 180 MPa. The transition coefficient of the steel model is set to 10. The concrete strengths are reduced to 70% of Mander-confinement concrete strengths based on uni-axial compressive test results. Figure 3 shows the comparison among the numerical simulation results of Fiber Model and Hinge Model, and the testing result of the S24 specimen. The Fiber Model matches the testing results after the material strength tuning as described. The Hinge Model captures the strengths at 3% drift ratio but gives a lower initial stiffness. Both of the modeling methods are then used to generate the Fiber Model and Hinge Model of the DSCFT bridge system and to run the numerical simulations.

**Numerical Simulation of DSCFT Bridge**

The Hinge Model and Fiber Model are generated and used to perform the numerical simulation of the DSCFT bridge. Table 2 lists the uni-axial concrete compressive and steel tensional testing results. Figure 4 presents the first four pier-related natural modes of the Fiber Model. Natural modes related to only beam vibration are not shown here. It should be mentioned that the natural periods is based on the initial stiffness of the structure, which might be different to the
vibration period considering the nonlinearity of the structure. As shown in Fig. 5, the stress-strain relationship of the FEDEAS concrete material used in the Fiber Model is a curve, which has higher initial slope and can not represent the overall stiffness during vibration induced by earthquakes.

Table 1 Uni-axial material testing results of four piers

<table>
<thead>
<tr>
<th>Pier No.</th>
<th>P1 outer</th>
<th>P1 inner</th>
<th>P2 outer</th>
<th>P2 inner</th>
<th>P3 outer</th>
<th>P3 inner</th>
<th>P4 outer</th>
<th>P4 inner</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>f_y (MPa)</td>
<td>314.9</td>
<td>364.9</td>
<td>385.0</td>
<td>493.2</td>
<td>353.7</td>
<td>250*</td>
<td>250*</td>
</tr>
<tr>
<td>tubes</td>
<td>f_y (MPa)</td>
<td>489.0</td>
<td>493.2</td>
<td>385.0</td>
<td>493.2</td>
<td>353.7</td>
<td>250*</td>
<td>250*</td>
</tr>
<tr>
<td>Concrete</td>
<td>f_c' (MPa)</td>
<td>40.1</td>
<td>28.4</td>
<td>38.6</td>
<td>27.5</td>
<td>38.6</td>
<td>27.5</td>
<td>27.5</td>
</tr>
</tbody>
</table>

* Nominal strength

Table 2 lists the maximal shear forces of cyclic push-over analyses using Fiber Model and Hinge Model. The stiffness values in Table 2 are the least square approximated slopes between -0.25% and 0.25% drift ratios.

Table 2 Numerical cyclic tests using two models

<table>
<thead>
<tr>
<th>Ground motions</th>
<th>PGA*</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHY024 in Chi-Chi Eq.</td>
<td>X: 0.25g Y: 0.25g</td>
</tr>
<tr>
<td>Western Canadian Artificial</td>
<td>X: 0.39g Y: 0.23g</td>
</tr>
<tr>
<td>CHY024 in Chi-Chi Eq.</td>
<td>X: 0.74g Y: 0.74g</td>
</tr>
<tr>
<td>Western Canadian Artificial</td>
<td>X: 0.39g Y: 0.23g</td>
</tr>
</tbody>
</table>

* X: longitudinal direction; Y: translational direction

Figure 6 shows the hysteretic loops of numerical cyclic tests using Fiber Model (red curves) and Hinge Model (blue curves) of the four DSCFT piers (P1 to P4) of the bridge. Comparing the results using these two models, they give similar results on initial stiffness values of P1 and P4, and overall hysteresis loop of P2. In other cases, the Fiber Model tends to give higher initial stiffness values and lower maximal shear forces within 1.5%, 3% and 6% drift ratio.

The earthquake scenario to apply in the DSCFT pseudo-dynamic experiment is composed of four bi-lateral ground motion histories, as listed in Table 3. The durations of the four ground motions are 15 sec., 45 sec., 15 sec., and 40 sec., respectively, making the total scenario a 115-second earthquake. Vertical ground motion is not considered in this experiment due to the complexity and difficulties of applying tri-axial control in the three laboratories and the minor effects of vertical ground motion comparing to the horizontal ones. Vertical loads of the piers are applied on the tops of specimens by pre-stressed. Further details of the bi-lateral earthquake scenario and the vertical load calculation can be found in Weng et al. (2006) and are not described here.
second-order geometrical nonlinearity (P-Delta) effect emerges. The maximal drift ratios of P2 (installed at NCREE) and P3 (installed at NTU) are around 1%, while P4 is around 1.4%. The shear forces transformed to reduced scales of CU, NCREE and NTU laboratories are around 280kN, 800kN and 200kN, respectively, which are conservatively within the safe ranges in the laboratories.

![Graph](image-url)  
Fig. 7 Maximal bi-lateral drift ratios and shear forces of nonlinear transient dynamic analyses of Fiber Model and Hinge Model

**Conclusions**

In this work, the Hinge Model and the Fiber Model are implemented by PISA3D and OpenSees, respectively, to perform numerical simulations of the collaborative pseudo-dynamic experiment of a DSCFT bridge. The estimated maximal shear forces and displacements are conservatively within the safe ranges in the laboratories. Further studies on the DSCFT behaviors and the numerical models will be further studied after the experiment. This experiment not only demonstrate the seismic performance of DSCFT piers, but also establishes a collaboration mechanism of earthquake engineering experiments between Taiwan and Canada.

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Cable Force Analysis with the Constraint by Guide-Pipe—Vibration Measurement by Wireless Sensing Technology

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李政寬¹、林沛暘¹、張國鎮²、羅俊雄²

Abstract
Around the deck and the pylon of a cable-stayed bridge, there are two-to-three-meter-long guide-pipes protecting the cable tendons. With rubber damper inside, the guide-pipes reduce the vibration induced by wind, stem the flow of rain into anchors, and therefore increase the service life of the anchor system. From the viewpoint of cables, the guide-pipes constrain the free length of cables and affect the natural frequencies of the cables so that the engineers tend to overestimate or underestimate the true cable force by any theories or models without guide-pipe effect. To analyze the cable force more precisely, this paper utilize FEM method by considering both the mode shape and the frequency spectrum of a cable measured by wireless sensing technology. Through the inverse calculation process, the effective constraint imposed by guide-pipes and the precise cable force are quantitatively analyzed.

Keywords: cable-stayed bridge, guide-pipe, wireless sensing technology

Introduction
The main structure components of a cable-stayed bridge include pylons, decks, abutments, and cables. Through cables, the weight of the decks are transferred into the pylons and thereof the foundations. Therefore the tension force of the cables is one of the important health indexes of a cable-stayed bridge. To evaluate the cable forces, there are three methods applied in field: (1) applying a jack to pull out the anchor, (2) imbedding a load cell between the anchor and anchor seat, and (3) estimating the cable force by the natural frequencies of a cable. Practically, only method (3) could be applied in-situ during and after the completeness of a cable-stayed bridge construction. However method (3) is an indirect approach and this method has many uncertain parameters such as the rigidity of the cross section of the cables, gravity effect, and the constraint induced by the guide-pipes as illustrated in Fig.1. In general around the deck and the pylon of a cable-stayed bridge, there are two-to-three-meter-long guide-pipes for the cable tendons. With rubber damper inside, as shown in Fig.2, the guide-pipes reduce the vibration induced by wind, stem the flow of rain into anchors, and therefore increase the service life of the anchor system. However from the viewpoint of cables, the guide-pipes constrain the free length of cables and affect the natural frequencies of the cables so that the engineers tend to overestimate or underestimate the true cable force by any theories or models without guide-pipe effect. To analyze the cable force more precisely, taking Gi-Lu Cable-Stayed Bridge as example, this paper utilize FEM method by considering both the mode shape and the frequency spectrum measured by wireless sensing technology. Through the inverse calculation process, the effective constraint imposed by guide-pipes and the precise cable force are quantitatively analyzed.

Background of the Experiment-and-Analysis Cable
As the mentioned above, the guide-pipes constrain the free length of a cable to some extent. The effective cable length is uncertain so that the true cable force may be overestimated or underestimated. Regarding the problem, taking the example of Cable R13 of Gi-Lu Cable-Stayed Bridge (as illustrated in Fig.3), through experiment and analysis, the effective
constraint of the guide-pipes and the more precise cable force will be quantitatively analyzed in the following article.

**Application of Wireless Sensing Technology on the Vibration Measurement of Cables**

Conventionally to measure the vibration of cables, a recorder, signal transmission cables (STC), and velocimeters are applied in the field. Connected to a signal recorder through STC, velocimeters are tied up to the cables to-be-measured as illustrated in Fig.4. For the engineers who carry out the vibration measurement on a bridge, the measurement approach with STC is inconvenient for the following reasons: (1) If long distance between measurement points, it takes many man-hours to lay out the STC and long STC distort the original signals. (2) If many measurement points, the STC may be mis-connected to the sensors. (3) If the STC across the vehicle lanes, additional protective measures must be provided for the wheel press of the passing vehicles. (4) If work high above the ground, it is another burden to lay out the STC. (5) The longer the time to lay out the STC, the more the danger to the engineers in field. The purpose of this paper is to study the constraints of the guide-pipes on the deck and the pylon, so work high above the ground is inevitable. Therefore wireless sensing technology and a crane are adopted and applied as illustrated in Fig.5.

To study the guide-pipe effect on cable force estimation, the experiment is illustrated as Fig.5. Tied up on cable, four velocimeters measure the transverse vibration of Cable R13 in gravity plane. Fig.6 portrays the geometry in detail concerning the anchors, anchor seats, guide-pipes, rubber dampers, and velocimeters. On the cable near the two guide-pipes, respectively, two velocimeters are arranged to probe the constraints on the cable imposed by the guide-pipes. Fig.7 shows the Fourier Spectrum of the vibration signal measured by wireless sensing technology. Marked on Fig.7, the characteristic frequencies are 2.020, 4.040, 6.073, 8.129, and 10.19 Hz. Fig. 8 compares the amplitude for the first mode, in which B/A=7.2 and C/D=4.2. The two ratios will be applied to probe the constraint-stiffness on the cable induced by the two guide-pipes.
Cable R13 Analysis with the Constraints Imposed by Guide-Pipes

With the density, the angle of elevation, the cable length, the constraint lengths at the deck and at the pylon, and the experiment results in Fig.7 and Fig.8, four models to-be-analyzed in Fig.9 are considered and summarized in Table 1. After the optimization process (inverse calculation) for the above four models to match those experimental results in Fig.7 and Fig.8, Table 2 summarized the analysis results for comparison. Obviously from Table 2, the constraint conditions imposed by the guide-pipes strongly affect the analysis result. In this case study, the guide-pipe constraints behave more like hinge-support than free-support.

Fig.9: Four models considered for analysis

Conclusions

This paper applies wireless sensing technology to measures the vibration frequency spectrum and mode shapes to probe the constraints on the cable by the guide-pipes, the moment inertia and the tension force of the cable, through the experiment results and the FEM model analysis. The study shows the guide-pipe constraints strongly affect the analysis result. The precise cable force and the precise constraint conditions for the guide-pipes could be inversely calculated through delicate experiment design and FEM model analysis. Rough constraint assumptions for the guide-pipes could induce significant estimation error for the true cable force.

References

<table>
<thead>
<tr>
<th>Table 1. Four models in Fig. 9 summarized</th>
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<tr>
<td>(1) String Theory</td>
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<tr>
<td>Gravity</td>
</tr>
<tr>
<td>Geometric nonlinearity</td>
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<tr>
<td>Moment inertia of the cable's cross section</td>
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<tr>
<td>Constraint by guide-pipes</td>
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<tr>
<td>Parameters to be optimized</td>
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</table>

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<tr>
<th>Table 2. Analysis results for different constraint assumption imposed on R13 Cable by the guide-pipes</th>
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<tbody>
<tr>
<td>Experiment result*</td>
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<tr>
<td>Characteristic frequencies</td>
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<tr>
<td>1st mode</td>
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<tr>
<td>2nd mode</td>
</tr>
<tr>
<td>3rd mode</td>
</tr>
<tr>
<td>4th mode</td>
</tr>
<tr>
<td>5th mode</td>
</tr>
<tr>
<td>Amplitude ratio for 1st mode</td>
</tr>
<tr>
<td>B/A</td>
</tr>
<tr>
<td>C/D</td>
</tr>
<tr>
<td>Optimized physical parameters below</td>
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<tr>
<td>Cable Force</td>
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<tr>
<td>Moment inertia</td>
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<tr>
<td>Effective stiffness $K_A$</td>
</tr>
<tr>
<td>Effective stiffness $K_D$</td>
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An Innovative Optic-Fiber Health Monitoring System on the Cables of a Cable-Stayed Bridge

Zheng-Kuan Lee 1  Yung-Bin Lin 1  Kuo-Chun Chang 2 and Chin-Hsiung Loh 2

Abstract

Conventionally to evaluate the cable tensions of a cable-stayed bridge, piezoelectric sensors or force balanced sensors are applied to measure the vibration signal spectrum of cables. Connected to a signal analyzer (such as a PC) through parallel signal-transmission-cables, those sensors are attached to the to-be-measured cables. With limited sensors and signal channels, the measurements of all cables are carried out cable-by-cable or part-cables by part-cables. Therefore, measuring all cable vibration simultaneously becomes practically impossible. For overcoming the mentioned difficulties, an innovative optic-fiber health monitoring system on the cables of a cable-stayed bridge is invented in this article. Herein this paper will not only introduce the mechanism of the new sensing system but also the application to a real cable-stayed bridge. With such a sensing system, it becomes possible to monitor all cables of a cable-stayed bridge economically, simultaneously, and regularly.

Keywords: cable-stayed bridge, FBG sensor, cable vibration

Introduction

The structure components of a cable-stayed bridge include pylons, decks, abutments, and cables. Through cables, the weight of the decks are transferred to the pylons and thereof the foundations. Therefore the tension force of the cables is one of the important health indexes of a cable-stayed bridge. To evaluate the cable forces, three methods are generally applied in field: (1) using a jack to pull out the anchor, (2) imbedding a load cell between the anchor and the anchor seat, and (3) estimating the cable force by its natural frequencies. Practically, only the method (3) could be applied in-situ during and after the construction. To carry out the method (3), an engineer has to be equipped with vibration sensors together with a signal analyzer. Connected to a signal analyzer (such as a PC) through signal-transmission-cables, in parallel those sensors are attached to those to-be-measured cables. Unfortunately a typical cable-stayed bridge has more than one hundred cables; with limited sensors and signal channels, the measurements of all cable vibration can only be carried out cable-by-cable or part-cables by part-cables, as illustrated in Fig.1. This way really takes many man-hours to finish the work. Furthermore the surrounding environment may vary dramatically during the long-hour measurement, such as temperature, wind and rain, so the measured data are not in the same environmental condition. Besides, suppose the conventional piezoelectric sensor system is installed permanently for all cables, with so many sensors and signal-transmission-cables, the system will cost a fortune and affect the aesthetic view of the bridge. For overcoming the mentioned difficulties or drawbacks, an innovative optic-fiber health monitoring system is invented in this article. Herein this paper will not only introduce the new sensing system but also the application to a real cable-stayed bridge. With such a new measuring system, it is possible to monitor all cables of a cable-stayed bridge economically, simultaneously, and regularly.

Fiber Bragg Grating Sensor

Fiber Bragg grating (FBG) sensors are highly

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attractive these years for their inherent wavelength response, immune from electrical wave or magnetic wave, and their multiplexing capability for the distributive sensing network in a series of arrays along a single optical fiber. The Bragg phase-matching condition determines the Bragg wavelength of a fiber grating. The wavelength shifts of a fiber Bragg grating subjected to physical disturbance can be expressed as followed:

\[
\frac{\Delta \lambda_B}{\lambda_{B,0}} = (1 - p_e) \Delta \varepsilon + (\alpha + \xi) \Delta T
\]

in which \(p_e\), \(\Delta \varepsilon\), \(\alpha\), \(\xi\), \(\lambda_{B,0}\) and \(\Delta T\) are the effective photoelastic constant, axial strain, thermal expansion coefficient, thermal optic coefficient, the initial wavelength of fiber Bragg grating, and temperature shifts, respectively. These coefficients generally depend on the type of optical fibers and the wavelengths at which they are written and measured. For typical FBG sensors, the effective photoelastic constant is about 0.21.

Field Application on Gi-Lu Cable-Stayed Bridge

Gi-Lu Bridge, crossing the longest river in Taiwan, is a modern design pre-stressed concrete cable-stayed bridge. The bridge has a single pylon, two-row harped cables, and a streamline-shape main girder with 2.75 meters in depth and 24 meters in width. The girder rigidly connects with the pylon and spans 120 meters to each side bent as shown in Fig.2. For monitoring the cable forces and studying the wind-rain induced vibration in the future, the authors applied the above mentioned FBG sensing system to the three longest cables, R33, R31 and R29 as shown in Fig.4. In Fig.4 two FBGs connect R33-R31 and R31-R29 respectively, while two velocimeters independently measure the vibration of Cable R33 and Cable R31. Fig.5 is the first 10-second data within the 6-minute measurement. Both Fig.5 (a) and Fig.5 (b) indicate the wavelength variation of the two FBGs, while Fig.5 (c) and Fig.5 (d) show the velocity history of Cable R33 and Cable R31 at the two points connecting the pretension wire between R33 and R31. Marked with characteristic frequencies, Fig.6 shows the Fourier Spectrum of those 6-minute measured signal.
Fig. 4: FBG sensing system together with Velocimeters on Cable R33, R31, and R29

Fig. 5: The first 10-second data within the 6-minute measurement data

Fig. 6: The Fourier Spectrum of those signals in Fig. 5 (all 6-minute measurement data)

Analysis of the measurement signals

In Fig. 6(c) and Fig. 6(d), which are measured by velocimeters, not only the characteristic frequencies of R33 and R31 could be identified respectively, but also the natural frequencies of the deck could be recognized. On the other hand, Fig. 6(a) and Fig. 6(b) are the relative displacement signal concerning R33-R31 and R31-R29, respectively.

Conclusion

From the above simple field application on Gi-Lu Cable-Stayed Bridge, it's verified the proposed “optic-fiber health monitoring system on the cables of a cable-stayed bridge” works. Unafraid of raindrop, permanent installed, low cost, and other important features listed in Table 1, the suggested system is superior to the conventional vibration sensor system with signal-transmission-cables. In the future, the proposed optic-fiber monitoring system could not only be applied to studying the cable vibration induced by wind and rain, but also to monitoring a cable-stayed bridge under construction.

References

Table 1: Advantage Comparison between the FBG Sensing System and Conventional Sensors

<table>
<thead>
<tr>
<th>FBG sensing system</th>
<th>Traditional Sensor system</th>
</tr>
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<tbody>
<tr>
<td>Simple installation</td>
<td>High Resolution</td>
</tr>
<tr>
<td>Not afraid of raindrop</td>
<td>Technology mature</td>
</tr>
<tr>
<td>Not afraid of lightning</td>
<td>Independent measurement</td>
</tr>
<tr>
<td>Low equipment cost / few man-hours needed</td>
<td>Being equipped with wireless transmission</td>
</tr>
<tr>
<td>Permanent installation</td>
<td>Applicable under bridge construction</td>
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<tr>
<td>Wavelength encoded</td>
<td></td>
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<tr>
<td>Low/no transmission lost</td>
<td></td>
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<tr>
<td>High sampling rate</td>
<td></td>
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<td>Applicable under bridge construction</td>
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<tr>
<td>Immune from electrical wave or magnetic wave</td>
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<tr>
<td>All cable vibrations are measured under the same environmental condition.</td>
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<tr>
<td>Little influence on the aesthetic scenery of a cable-stayed bridge</td>
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</table>
Analysis and Design of Rectangular RC Columns with Lap-spliced Longitudinal Reinforcement

K.C. Chang¹  S.B. Chang²  K.Y. Liu³  P.H. Wang⁴
張國鎮¹ 、張順賓² 、劉光晏³ 、王柄雄⁴

Abstract

Many studies had showed that RC columns lap-spliced at the plastic hinge zone exhibit poor ductility, even directly applied with CFRP or steel jacket. In order to solve the problem, a new retrofit method, the Carbon fiber-Steel plate retrofit method (CS retrofit method), is proposed to improve the confinement effect while keeping the original cross section almost intact. According to the experimental and analysis results, design formulas for the CS retrofit method are proposed. Beside, for a rectangular column confined by multi-spirals, a simplified finite element analysis procedure is proposed to analyze the axial compressive behavior. Good agreement is found between the analysis and experimental results.

Keywords: lap-spliced, rectangular RC column, seismic retrofit, confinement

Introduction

Many existing low-rised reinforced concrete buildings in Taiwan were designed without ductile details and suffered moderate to major damages, even collapse in the 1999 Taiwan Chi-Chi Earthquake. Based on the reconnaissance report, the poor quality of lap spliced longitudinal reinforcements at the plastic hinge zone and insufficient confinement by transversal reinforcements are considered fatal factors that lead to structures collapse in a brittle manner.

Reinforced concrete is composed of steel bars and concrete. While using lap-spliced design, engineers should make sure that the bonding force transferred between concrete and reinforcement is sufficient. Many studies had showed that RC columns lap-spliced at the plastic hinge zone exhibited poor ductility, even directly applied with CFRP or steel jacket. In order to solve the problem, the Carbon fiber-Steel plate retrofit method (CS retrofit method (Figure 1)) is proposed to improve the confinement effect. The CS retrofit method combines the advantages of steel and CFRP jackets to obtain more efficient confinement.

This research continues the experimental research project “Seismic Retrofit for RC Column Lap-Spliced at the Plastic Hinge Zone” carried out in NCREE, further analysis are progressing for the specimens retrofitted by CS retrofit method. A cross-sectional analysis program RESPONSE 2000 is employed in this research to calculate the lateral force and displacement. Then, according to the experimental and analysis results, this research proposed a design formula for the CS retrofit method. On the other hand, another topic of this research is to investigate the confining behavior of rectangular RC columns confined by multi-spirals. Based on the experimental

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results of axial compressive test performed by Yin et al. (2004), a simplified finite element analysis procedure is proposed to analyze the axial compressive behavior of the column confined by multi-spirals.

**Analysis of experiment results in rectangular RC column lap-spliced at the plastic hinge Zone**

To investigate the seismic behavior of rectangular RC columns with lap-spliced problem, in this paper, the proposed method is applied in the experimental program and compared to other methods, such as CFRP jacketing and steel jacketing with adhesive anchor bolts. Test specimens are classified according to the specimen size to five groups, namely, B, W, S, M and F, respectively. The smallest dimension of the specimens (B group) is 30×50×110cm³, and the largest dimension of the specimens (F group) is 100×120×380cm³. For all specimens, the lap-splice length of longitudinal reinforcement is about 40 times of the longitudinal bar diameter in the plastic hinge zone. Specimen named B1L21-BM is considered a benchmark.

![Specimen B1L21-BM](image1)

![Specimen B2L17-C8](image2)

![Specimen B3L21-B30S6](image3)

![Specimen B5L21-C12S5](image4)

**Fig. 2** Hysteretic hoops of test specimens

Figure 2.(a) shows the hysteretic hoop of the specimen without retrofitting. When the drift ratio reaches 2%, the strength of the specimen rapidly decreases and the pinching behavior is obvious. The major reason for large force reduction is bond failure between longitudinal reinforcements and lower strength concrete in the plastic hinge zone. Even directly retrofitted by CFRP in Specimen B2L17-C8 (Fig. 2(b)), the performance is as poor as benchmark. For Specimen B2L21-B30S6 (Fig. 2(c)), using steel jacket with a small number of adhesive anchor bolts, the result is satisfactory. In addition, the proposed CS method in B5L21-C12S5 (Fig. 10(d)) shows high energy-absorption capacity, as the drift ratio goes to 7%, lateral strength remains the same at the last cycle.

The performance of column after retrofitting by CS method, as well as steel jacketing, is analyzed by a program named RESPONSE- 2000. Selected specimens are B4, B5, F1, S1, S2, M1, W1 and W2, respectively. Based on the design details and material properties, the analysis model can be build up easily, except for taking account the plastic hinge length as “yielding penetration length”, and confined concrete stress-strain relationship of CS method. This relation is simplified as a bilinear curve according to the axial compression tests. Points at (0.005, 1.5f'c) and (0.025, 2f'c) are defined as the turning and ultimate point. For the specimen with larger cross section, such as F1, points are relocated at (0.005, 1.0f'c) and (0.025, 1.5f'c) to consider the size effect. Moreover, the vertical spacing of confinement steels is rearranged to a shorter length; say 10mm, to reflect the extra confinement force from CS or steel jacketing. Compared to the experimental results, the analysis results in Figure 3 show well accuracy. The proposed CS retrofit method and analysis approach can benefit definition of hinge properties in the pushover analysis.

![Comparison between experimental and analysis result](image5)
Retrofit Design of CS retrofit method

According to the experimental and analysis results, design formulas for the CS retrofit method are proposed in this research. Because the maximum cross section of test specimens is 100 cm × 120 cm, the proposed design formula is applied to the RC columns having a width in less than 100 cm. Besides, in order to ensure that the bonding force transferred between concrete and reinforcement is sufficient, the strength of concrete is limited in excess of 19 MPa. The design formula of CS retrofit method is showed as follows:

(1)Volumetric ratios of CFRP(ρj)
   (A)Ductility retrofit:
   \[ ρ_j \geq 0.4 \% \]
   (B)Retrofit for lap-spliced longitudinal steels:
   \[ ρ_j \geq 0.6 \% \]
   (Note: The length of retrofit for lap-spliced longitudinal steels is within 50 cm from the bottom of the column, exceeding of that is regarded as ductility retrofit.)

(2)Dimension of the steel plate
   (A)Thickness of the steel plate(t):
   \[ t \geq 0.01B \cdot 0.01 D \]
   where, B: width of column
   D: depth of column
   (B)Width of the steel plate(Sw):
   \[ Sw \geq B-10 \cdot D-10 \]
   (C)Height of the steel plate(Sh)
   \[ Sh \geq L_1 + 10(\text{cm}) \]
   where, L1: lap-spliced length
   (Note: If the lap-spliced length is unknown, replacing that by plastic hinge length.)

(3)Radius of the corner(r):
   \[ r \geq 5 \ (\text{cm}) \]

Numerical analysis model for rectangular RC column confined by multi-spirals cages

For a rectangular column confined by multi-spirals cages in this research, a simplified finite element analysis procedure is proposed to analyze the axial compressive behavior. The concept of the proposed simplified analysis procedure originated from the confined concrete model proposed by Mander et al. (1988). To obtain the compressive strength of confined concrete, the lateral confining pressure on concrete and confinement effectiveness coefficient have to be calculated. For simple and regular shape of confinement such as traditional spirals and hoops, the confining pressure can be easily calculated by simple mechanical model. For confinement with complicated shape, it is usually difficult to obtain the magnitude and distribution of the confining pressure using simple mechanical model. Therefore, finite element analysis is employed in this research to obtain the confining pressure under confinement with complicated shape. Once the confining pressure is obtained, Mander’s confined concrete model is applied to calculate the axial compressive stress-strain curve.

Two types of multi-spiral confinements are considered in this research (Fig. 5). They are referred to as type 4S, formed by 4 spirals and type 5S, by 5 spirals. Due to the symmetry of the geometry and the loading condition, only 1/8 and 1/2 of the model columns with type 5S and 4S were analyzed, respectively. Boundary conditions in the plane of symmetry were assigned. The application of axial loading in both types of columns is realized using displacement control on the top of the column with a monotonic axial displacement. In the modeling of steel reinforcement, this research utilized the discrete method. The steel elements were explicitly constructed and attached to the concrete element. For simplicity, circular hoops are used to approximately represent the spirals (Fig. 5).
Validation against experimental results for multi-spirals

Fig. 6 and 7 show the comparisons in terms of axial stress-strain curves between the analysis and experimental results for type 4S and 5S, respectively. Analysis results consider the individual contributions of confined concrete, cover concrete and longitudinal reinforcement to the axial compressive strength. By summing the individual contributions, two analysis results with and without considering the cover concrete can be obtained. As axial deformation increases, the cover concrete tends to spall off the confined core due to their different level of lateral expansion. Thus, the cover concrete is usually unable to achieve maximum strength. This was also observed during the tests. Besides, it is found from the analysis curves that the cover concrete has significant influence on the initial stiffness and the spalling of cover concrete will affect the point where significant nonlinearity begins. In general, good agreements are found between the analysis and experimental results. For the test specimens investigated, the maximum percentage of error in the axial compressive strength is within 5%. After the maximum strength has been reached, the rate of decrease in strength is similar between analysis and experimental results.

![Fig. 6 Axial stress versus axial strain for type 4S](image)

![Fig. 7 Axial stress versus axial strain for type 5S](image)

Conclusion

The Carbon fiber-Steel plate retrofit method proposed in this research is proved that the specimens with lap-spliced longitudinal reinforcement retrofitted by the CS method can effectively improve the ductility and energy dissipation capacity. Besides, for the rectangular RC column confined by multi-spirals cages, a simplified finite element analysis procedure is proposed to analyze the axial compressive behavior. The experimental results show that type 5S confinement is superior in terms of confinement effectiveness and simplicity in construction. Thus, parametric studies are performed on type 5S and then provided some design suggestions in application.

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A Neural-network-based System for Bridge Health Monitoring

T.K. Lin¹ K.C. Chang² C.C. Chen³ C.Y. Chen⁴ I.J. Tsai⁵

Abstract

A bridge health monitoring system based on neural network technology is proposed in this research. Nowadays, accompanying with the aging of the existing bridges all over the world, how to effectively identify the health condition of the bridges has become an important issue. The method should offer a rapid and reliable result immediately after major strikes without using lots of labor and time. The demand of this health monitoring system grows rapidly and research on this topic has been widely discussed. Meanwhile, neural networks, commenced from artificial intelligence, have also shown their outstanding performance in complex problems. For this reason, a monitoring system using neural network is developed. As commonly known, the strong motion recording network of structures and bridges in Taiwan has offered an excellent database for health monitoring. Analytical result of different methods including transfer function, Autoregressive with Exogenous (ARX) model, and the proposed neural-network-based system are used to evaluate the efficiency in bridge health monitoring. The result has shown that the proposed neural-network-based system can be successfully used in bridge health monitoring after major earthquakes.

Keywords: Neural network, Bridge health monitoring, System identification

Introduction

Earthquake has been the most threatening disaster to civil structures. The damage of infrastructures not only causes the loss in economic activities but the life and properties of people. For that reason, how to apply instrumentation on structures to monitor their characteristic during and after earthquakes has become an important issue. By the development of system identification technology in buildings and bridges, some theories and methods has gradually been proposed in the last few decades.

In order to collect earthquake information in Taiwan, the Central Weather Bureau (CWB) of the Ministry of Transportation and Communication (MOTC) has instrumented sensors in free field, buildings and bridges among Taiwan to record their response during major earthquakes. Sixteen bridges have been chosen as the demonstration examples and a safety monitoring and seismic evaluation system has been established. The database collected by the system can offer abundant information for researchers. The practical situation of bridges can be reflected by the system and a proper modification of design code can be determined.

Traditionally, the identification methods were commonly developed under frequency domain. However, two close frequency modes may not be effectively separated by frequency-domain-based method when noise is contained in the measured data. To solve this problem, the discrete-time-domain-based identification technique has been applied to civil engineering in the last two decades. In most methods, structures are considered time-invariant. Namely,
parameters of structures are assumed to be constant during the whole time history. However, structures might be damaged or behave nonlinearly during earthquake. To solve this problem, neural networks, proved to have outstanding performance in dealing complex problems, was integrated into identification systems. The adaptability and fault tolerance of neural networks have made them good candidates in dealing with data of uncertainty and incompleteness and identification of nonlinear systems under major earthquakes may be implemented by neural networks. (Adeli et al. 1995) (Masri et al. 1992) (Masri et al. 1993)

A neural-network-based method is proposed in this report. The neural network is constructed under the multi-input-single-output structure. Similar to the ARX model, which has been widely used in basic system identification, the Nonlinear Autoregressive with Exogenous (NARX) method has offer a different viewpoint in evaluating the behavior of structure. The adaptability and fault tolerance of neural networks have made them good candidates in dealing complex problems, was integrated into identification systems. The first order response of point \( j \) can be described as

\[ y_j(t) = \sum_{\eta=0}^{\infty} h_1(\eta) x_j(t-\eta) \]  

where \( x \) and \( y \) are the input and output values of the identification system ; \( n_x \) and \( n_y \) are the maximum time delay steps of \( x \) and \( y \). \( g \) represents a linear or nonlinear function for \( y(t) \).

The relationship between the input and the output can be shown as

\[ y^{(j)}(t) = s + \sum_{i=1}^{n_x} w_i \tan(\sum_{k=1}^{n_y} (y_{ik}^{(j)} (t-k)) + \sum_{m=0}^{\infty} (u_{im}^{(x)} x^{(x)} (t-m) + u_{im}^{(y)} x^{(y)} (t-m) + ... + b_i)

where \( y_{ik}^{(j)} \) is the \( k \)th weighting between the \( l \)th node in the hidden layer and the input node \( y^{(j)} \). And \( u_{im}^{(y)} \) is the \( m \)th weighting between the \( l \)th node in the hidden layer and the input node \( x^{(j)}(\mu = a \cdot b \cdot ...); w_i \) is the weighting between the \( l \)th node of hidden layer and the output node; \( n_h \) is the node number of the hidden layer, \( s \) is the threshold of the output nodes, \( b_i \) is the threshold of the \( l \)th node in the hidden layer.

The output of point \( j \) in a multi-degree-of-freedom system can be expressed as (Worden et al., 1997)

\[ y^{(j)}(t) = y_1^{(j)}(t) + y_2^{(j)}(t) + y_3^{(j)}(t) + \ldots (3) \]

where \( y_n^{(j)}(t) \) is the \( n \)th order component of point \( j \).

The first order response of point \( j \) is the sum of each first order response from the input node and the relationship between \( y_1^{(j)}(t) \) and the input nodes can be described as

\[ y_1^{(j)}(t) = \sum_{\eta=0}^{\infty} \int_{-\infty}^{\infty} h_1(\eta) x_j(t-\eta) \]  

Where \( h_1(\eta) (\eta = a \cdot b \cdot \cdot \cdot) \) represents the impulse response function and the corresponding kernel transformation is shown as follows

\[ H_1^{(j)}(\omega_1) = \int_{-\infty}^{\infty} h_1(\eta) (\eta = a \cdot b \cdot \cdot \cdot) e^{-i\omega_1 \tau} d \tau \]  

Similarly, the corresponding core transformation of the second-order direct-kernel and cross-kernel can be written as

\[ H_2^{(j/\eta)}(\omega_1, \omega_2) = \int_{-\infty}^{\infty} h_2(\eta) (\eta = a \cdot b \cdot \cdot \cdot) e^{-i\omega_1 \tau} d \tau \]  

The kernel transformation of a single-input-single-output system with an NARX weighting can be calculated by using harmonic detection method. The response of the system can be checked by utilizing simple harmonic inputs. For example, if the input signal of point \( a, b \) are (Chance et al. 1998)

\[ x^{(a)}(t) = e^{i\omega_1 \tau} \quad x^{(b)}(t) = 0 (7) \]

then

\[ H_1^{(j/a)}(\omega_1) = \sum_{l=0}^{n} \sum_{m=0}^{n} \sum_{k=0}^{n} u_{lm}^{(a)} e^{-i\omega_1 \delta} \]

PRACTICAL APPLICATION—A CASE STUDY

The identification object is a bridge located in the
second southern freeway in Taiwan. Distribution of sensors on the bridge is shown in figure 1. By data collected under large ground excitation, a NARX neural network system with a structure of two inputs and one output is established to evaluate the characteristic of the bridge. Channel 14 and 21 which are the signals from the pile-cap are chosen as the input channel and channel 16 which is the response of the web on the middle span is the output channel. All the data are collected in the tranverse direction.

The established NARX neural network has 212, 1, and 1 node in its input, hidden and output layer, respectively. Variables of the input nodes are \( y^{(j)}(t-1) \cdot \ldots \cdot y^{(j)}(t-70) \cdot x^{(a)}(t) \cdot \ldots \cdot x^{(a)}(t-70) \cdot x^{(b)}(t) \cdot \ldots \cdot x^{(b)}(t-70) \) and the output is \( y^{(j)}(t) \). The kernel transformation and the characteristic of the structure can be obtained by the weighting after successfully training the network.

The first-order kernel transformation of channel 14 to channel 16 is shown in figure 2 after training by the 921 earthquake time history with sampling rate of 200 Hertz. The fundamental frequency of the structure is approximately 3.4 Hz in figure 2. The comparison of the two NARX systems is depicted in Figure 3. The result has proved that the structure characteristic of the first-order kernel transformation is very close with choosing each support as the input signals.

The plan view and the contour of the second-order kernel transformation of channel 14-channel 16 is shown in figure 4 and 5. The interaction phenomena can be easily observed during \( f_1=3.4 \) Hz (21.35 rad/sec), \( f_2=3.4 \) Hz (21.35 rad/sec) and \( f_1+f_2=3.4 \) Hz (21.35 rad/sec). Namely, the bridge would be resonated by an excitation or summation of 3.4 Hz in channel 14.

The NARX system is also trained by the 1022 earthquake time history with sampling rate of 200 Hertz. The first-order kernel transformation of channel 14 to channel 16 and channel 21 to channel 16 is shown in figure 6. The fundamental frequency of the structure is approximately 3.7 Hertz in figure 6. The result has once again proved that the structure characteristic from the first-order kernel transformation is very close by choosing each support as the input signals.
In order to demonstrate the performance of the proposed NARX system, two different methods including the transfer function and ARX model are also used here to offer the reference value for comparison. Researches have shown that these two methods can be easily applied to identify the basic property of linear stru cture. The evaluation consequences under two different major earthquakes are shown in Table 1. For the 921earthquake, the fundamental frequency of the bridge is approximately 21.4 Hz. Results from these three methods are very close. Moreover, the identified frequency during the 1022 earthquake is about 22.3Hz. Due to the smaller seismic intensity, the frequency has raised slightly in the second earthquake. Though all the results are alike, however, there are two specific characteristics of the new proposed method. The nonlinear behavior can not be accessed by these traditional methods. Moreover, structures with multi-point inputs such as bridges can not be completely monitored with conventional techniques. By the illustration of the higher-order kernel transformation, the resonant phenomenon of the bridges can be carefully avoided.

Table 1 Fundamental frequency under different methods

<table>
<thead>
<tr>
<th>Frequency (rad/sec) /Method</th>
<th>921 earthquake</th>
<th>1022 earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transfer Function</td>
<td>21.5 Hz</td>
<td>22.37 Hz</td>
</tr>
<tr>
<td>ARX</td>
<td>21.43 Hz</td>
<td>22.32 Hz</td>
</tr>
<tr>
<td>NARX</td>
<td>21.35 Hz</td>
<td>22.28 Hz</td>
</tr>
</tbody>
</table>

Conclusions

A bridge health monitoring system based on neural network technology is proposed in this paper. In order to identify the nonlinear behavior of structures, a NARX system is trained from data collected in major earthquakes. The relationship between the input and output channel can be reflected by the weighting of the neural network and the fundamental period of the structure can then be derived. By applying the system to bridges, the multi-support characteristic can be analyzed and the combination of specific frequencies causing resonant phenomena can also be obtained. The result would be an important basis for verifying the source of damage behavior on structures.

To demonstrate the performance of the proposed system, a bridge of the second southern freeway in Taiwan is used. By data collected from two large ground excitations, the NARX system with a structure of two input nodes and one output node is established to evaluate the characteristic of the bridge. The input channels are signals from the pile-cap and the output is the response of the web on the middle span. Analytical results of different methods including transfer function, ARX model are also compared with the proposed neural-network-based system to evaluate their efficiency in health monitoring.

The result has shown that besides identifying the fundamental frequency of structure, the proposed neural-network-based system can also be successfully applied in bridge health monitoring after major earthquakes. The combination of specific frequencies causing resonant phenomenon is clearly shown in the kernel transformation diagrams and damage on structures may be avoided during design process. More information can be studied from the complex high-order kernel transformation.

The capability of the NARX system in dealing with nonlinear structure would be another research focus. By the proposed method, bridges with nonlinear bearing such as lead rubber bearing or visco-elastic dampers can be precisely monitored during major earthquakes. The behavior of these elements under specific time history can be used to assess the performance of these equipments and the bridge design code can be revised basing on the practical condition of bridge structures.

References


Behavior of Tenon and Mortise Connections in Traditional Taiwan Timber buildings

Shyh-Jiann Hwang¹, Chin-Lu Lin², Min-Lang Lin³

Abstract
Among the traditional buildings in Taiwan, most of them consist with the timber frame. The connections of the traditional timber frame are various kinds of tenon and mortise. There has been little research and experimentation focused on the mechanical behavior of traditional tenon-mortise connections. Based on that, a total of 15 full-scale specimens were created to test six common tenon-mortise connections in this research. From the result of cyclic loading tests, it was found that the timber connections with tenon-mortise joints have high deformation capacity. The depth of the tenon influenced the moment capacity of the joints. The moment capacity of the joint can be improved by increasing the depth of the tenon. The stepped dovetail is effective in improving the performance of the straight tenon.

Keywords: traditional building, timber structure, tenon and mortise, straight tenon, stepped dovetail

Introduction
Most of Taiwan’s traditional buildings were constructed by woods and bricks. For those important historical buildings, such as large-scale residences and historic temples, the ratio of employment of the timber frame is therefore soaring. This kind of construction was certified for most of the historical buildings. The 921 earthquake caused the serious damage to many of the historical buildings in central Taiwan. In the wake of the quake, the government spent a great deal of manpower and money to restore the damaged historic spots. The agenda of quake influence and seismic retrofit for the historic spots is gradually catching the attention of the government.

Before researching into the retrofit strategy, we have to be aware of the seismic resistant capacity of the traditional buildings. For the traditional timber frame, besides its material characteristics, the most obvious trait is using the plug-slot type of beam-to-column connection without the use of metal nails or bolts. The merit of this method is the simplicity of component exchange, but it is foreseeable that its structure would be looser. And the correlation between its looseness degree and the merit and demerit in seismic resistance need to be explored further.

In the research by King et al. (1996), the one third scale specimens were used to study the rotational stiffness. Chang and Hsu (2005) also studied the behavior of traditional timber joints in Taiwan by using 30 sets of specimens with rectangular pillars and beams. Both experiments were limited to the testing equipments and therefore cannot completely represent the information (like hysteretic curve) about seismic resistance of timber joints. This study focuses on the research of mechanics for mortise-tenon joint. Full-scale specimens with round columns and beams are used, which are the closest to the traditional buildings.

Scheme of specimens
To study the rotational performance of traditional timber joints, a total of 15 specimens (classified as 6

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types) with 130cm in column length and 90cm in beam length (tenon not included) were tested. The specifications of all specimens are listed in Table 1. Fig. 1 is for the appearances of some specimens. The first letter of the specimen name represents the type of column (R for rectangular column and C for circle column). The second letter of the specimen name represents the type of beam (R for rectangular beam, C for circle beam, H for straight tenon, T for through tenon and S for stepped dovetail tenon).

The full-scale specimens were fabricated from Chinese fir (*Cunninghamia lanceolate*), dried and processed at a wood plant. The first batch of specimens is delivered to NCREE after the installation of the beam-to-column joints. For the second batch of specimens, the mortises and tenons were connected after the experiment was set up.

Table 1 Specification of specimens

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Tenon type</th>
<th>Column section (cm)</th>
<th>Beam section (cm)</th>
<th>Mortise size (cm)</th>
<th>Tenon length (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RR</td>
<td>straight</td>
<td>20x20</td>
<td>30x12</td>
<td>30x10</td>
<td>10</td>
</tr>
<tr>
<td>CR</td>
<td>straight</td>
<td>φ 30</td>
<td>30x13</td>
<td>30x10</td>
<td>10</td>
</tr>
<tr>
<td>CC</td>
<td>straight</td>
<td>φ 30</td>
<td>φ 32</td>
<td>30x10</td>
<td>10</td>
</tr>
<tr>
<td>CH</td>
<td>straight</td>
<td>φ 30</td>
<td>φ 30</td>
<td>28.5x10</td>
<td>15</td>
</tr>
<tr>
<td>CT</td>
<td>through</td>
<td>φ 30</td>
<td>φ 30</td>
<td>28.5x10</td>
<td>50</td>
</tr>
<tr>
<td>CS</td>
<td>dovetail</td>
<td>φ 30</td>
<td>φ 30</td>
<td>32x10</td>
<td>15</td>
</tr>
</tbody>
</table>

The relationship among various specimens is shown in Fig. 2. The types of RR, CR and CC are all straight tenons, and which tenon lengths are all 1/3 of column diameter (10cm). The difference is about the section shape of column and beam. Circle column and circle beam (CC) are the common style seen in traditional temples. The designs of circle column/rectangular beam (CR) and rectangular column/rectangular beam (RR) are for checking whether the simplified designs of column and beam can affect the performance of timber joints. The round section can make full use of the whole tree. But the rectangular section is easier in specimens making (lower cost) and it is also easier to make sure of size accuracy. CC, CH and CT are all circle column and circle beam and the types of tenons used were are all straight tenons. The only difference is about the length of tenons (through tenon is the straight tenon which gets through the column body). The tenon lengths of CH and CS are all 15cm. What this combination should compare to is the difference of tenon type. The swallow-tailed tenon (or named as dovetail tenon) is famous for its similar shape to swallow tail. Its trapezoid jointing of mortise and tenon is helpful to pulling restraint between column and beam. Considering the process for construction, the mortise hole must be enlarged. After pushing the swallow-tailed tenon in place, the remaining holes then can be inserted into a wedge for firm matching. (Fig. 3)

**Experimental Setup**

The test frame shown in Fig. 4, the column was laid horizontally and the beam was erected. The lateral actuator (250 kNs) connecting to the end of the beam is displacement controlled. Apply cyclic load with displacement in sequence ±0.25%, 0.5%, 0.75%, 1%, 1.5%, 2%, 3%, 4%, 5%, 6%, 8%, 10%, 12% and 15% for 2 cycles individually. After completion of the stage of 15% displacement, apply to specimens the
displacement of ±12%, 10% and 8% for one cycle each to check the residual strength of timber joint after large displacement increments. Refer to Fig. 5. Besides the test of repeated side pushing for 2 sets each type of the samples, conduct the mono-pushover test for 3 sets of samples (CH-3, CT-3 and CS-3). The largest lateral displacement can be up to 30% (225mm) to check the potential largest strength of joints.

Results and Discussions

Fig. 6 shows the moment-rotation diagrams for each type of specimens. Although wood is anisotropic material with a large variation in characteristics, there were still some phenomena observed under the limited tests.

1. Although the section of the column and beam combinations in RR-1, CR-1 and CC-1 are different, their strength of rotation are all about 4kN-m. When the lateral displacement exceeds 10%, the development of strength becomes stable. According to the findings, the shape design of columns and beams has little influence on the test results with the identical conditions for mortise and tenon.

2. The strength development of CH-1 is slower at the initial stage in push direction. It is because the mortise holes of this batch are made larger than the tenon for installation at the laboratory so the fitting of joints is not better than the first batch of specimens (RR, CR and CC), and the joint has a larger gap in the push direction.

When the positive displacement is larger than 10%, the strength still keeps increasing, and the largest moment reaches 5kN-m which is even larger than CC-1. Upon observation of the curves for all the specimens, the performance of the through tenon (CT) is the most outstanding. When its lateral displacement reaches 15% (rotation angle 8.63°), the moment strength can be 18–21 kN. It is obvious that when the tenon sections are identical, the deeper the depth for inserting, the larger the rotational strength of timber joints with larger engagement area for mortise and tenon in movement.
Specimen CC-2 and CS-2. The joints develop similar strength in the first and second cycle under the same displacement stage. But after 2% lateral drift, the strength degrades at second cycle obviously. The hysteretic loops of the last 3 cycles (repeated 12%, 10% and 8% lateral drift ratio) were shown in Figure 8. The enveloping area is much smaller than that in the first cycle of the same drift ratio shown in Fig. 6. For the traditional timber joints, it is evident that the capacity of energy dissipation would degrade significantly after undergoing a severe earthquake.

![Graph showing moment-rotation relations (drift ratio ≤ 2%)](image1)

Fig. 7 Moment-rotation relations (drift ratio ≤ 2%)

![Graph showing moment-rotation relations (residual stiffness)](image2)

Fig. 8 Moment-rotation relations (residual stiffness)

5. The results of pushover test (displacement reach 30%) have been shown in Fig. 9 and Photo 1. It proves again that the strength of through tenon is obviously superior to the other two sets. The strength of stepped dovetail joint would gradually decline after reaching the highest point. It is mainly because the part of dovetail end is worn out to make the restraint effect decline.

![Graph showing pushover curve](image3)

Fig. 9 Pushover curve

Conclusions

The test results show: 1) In regard to the dynamic test for the joint, the shapes of column and beam have little influence on the test results. If we would like to test various types of joint in a large quantity, we may adopt the simpler types of column and beam such as rectangular column and rectangular beam. 2) With the same tenon type, the tenon length (depth of inserting) would decide the rotational strength. 3) The rotational strength of stepped dovetail joint has a large difference in two ends. The strength of dovetail end is superior to the straight tenon’s, but the strength of wedge end is inferior to the straight tenon’s.

Generally speaking, the bending resistant capacity of traditional timber joints is generally lower but they allow the largest deformation. In the testing process, neither the frame body nor the tenon has the serious damage caused. As to the whole structure under earthquake, the merits and demerits of such joint characteristics still need to be researched further.

References


Dynamic Collapse Simulation of RC Building Frames under Extreme Earthquake Loadings

Chiun-lin Wu¹, Yuan-Sen Yang¹, Shyh-Jiann Hwang², Chin-Hsiung Loh³, Wu-Wei Kuo⁴, Kenneth J. Elwood⁵, Jack P. Moehle⁶

Abstract

Following the 2004 dynamic collapse tests of low-ductility RC portal frames, NCREE has moved one step further to gain insight into both flexural and flexural-shear failure modes through international collaborations with PEER Center (USA) and the University of British Columbia (Canada). A total of 4 single-story-3-bay RC frames and 2 ductile portal frames was tested in 2005 using near-fault records from the 1999 Chi-Chi Taiwan earthquake to collect structural characteristics in the vicinity of shear failure and post-peak behavior in gravity load collapse. OpenSee was employed as the analysis testbed to compare pros and cons of different numerical models. It was observed that the current strength estimate formulae are able to predict the general structural behavior before incipient shear failure within engineering accuracy, but further research is required in the near future to comprehensively capture post-peak structural behavior and, thereby empirical formulae can be made available to practicing engineers.

Keywords: gravity load collapse, shear failure, low ductility, RC frame, shake table test

Introduction

It has become a custom in industry that before a new product is made available in the consumer market, a series of fatigue and/or failure tests will be conducted to ensure its serviceability fulfills exactly its original design concept, and more importantly complies with its corresponding industry standards. For instance, a car maker will conduct a large number of collision tests for a prototype vehicle to ensure life safety of its driver and passengers even with an understanding of the relatively low probability of extreme collision accidents in real life. Recently, collapse simulation in both experimental and analytical aspects in the fields of earthquake engineering is getting much popular worldwide more or less in view of the following needs:

- To ensure energy dissipation capacity and collapse prevention: old seismic design code documents consider only 10%/50yrs. earthquakes (i.e. an average return period of 500 years) and a performance objective of life safety is set accordingly, which imposes a single control point over the structural nonlinear skeleton curve. This arrangement lessens computational efforts and effectively simplifies seismic design procedure, but can not ensure sufficient structural ductility during extreme earthquake events. To eliminate this drawback, performance-based design adds an extra control point at 2%/50yrs. earthquake level to ensure the ultimate performance objective of collapse prevention can be satisfied. Collapse analysis should be capable of revealing information that, during such extreme events, local collapse (or, component failure) is acceptable, but system collapse should not occur.

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To offer house owners an option for custom-made buildings: Performance-based design framework enables a building structure performs according to owner-specified objectives under 10%/50yrs. and 2%/50yrs intensity levels. An enterpriser may set up a higher seismic standard for his headquarters to alleviate earthquake-induced loss due to interrupted operation.

To reduce probability of casualties: A return period of 2500 years earthquake indicates an occurrence rate of 2% in 50 years. If collapse prevention cannot be guaranteed at this hazard level, then it means a 2% probability in 50 years that residents could die in earthquake attacks.

To distinguish unique characteristics of innovative structural systems from conventional ones: In current engineering practice, a building is seismically evaluated through its strength capacity and maximum interstory drift under design earthquakes. These years, innovative structural systems adopt smart material and advanced design to be equipped with cutting edge self-centering devices to reach the goal of seismic isolation and/or energy dissipation to minimize residual deformation. These structural systems may have comparable maximum drift as traditional buildings, but permanent deformation is considerably reduced. To classify substantial difference from conventional structural systems in seismic performance, a combined consideration of permanent deformation together with maximum drift may be required in the future evaluation framework. In this regard, consideration of collapse will help advancing accurate prediction of these indices.

During the September 21 (local time) 1999 Chi-Chi Taiwan earthquake, a large number of older buildings built before 1982 sustained severe damage and many others suffered from complete failure. Most older structures are prone to shear type of failure in a low ductility manner. A large portion of the elementary and high school buildings falls into this category. The Chi-Chi earthquake hit the central part of Taiwan at 1:47am, so these collapsed school buildings did not cause tremendous tragedy of students’ death. However, it becomes main concerns of the governments how to retrofit old school buildings that are identified in high risk of structural collapse to prepare for future earthquakes. To reach this goal, dynamic nonlinear behaviors of these low ductility columns must be first thoroughly understood.

Finally, according to solid mechanics theory, development of negative slope stems from P-Δ effects and/or material fractures. Although in the literature there are plenty of studies on how to incorporate P-Δ effects in response analysis, few are able to successfully simulate fracture-induced collapse, especially in brittle shear failure. Accurate fracture-based numerical approach is technically sophisticated and might be economically unaffordable to most design firms. As such, collection of experimental data on structural collapse, in global and local manner, becomes very informative in developing computationally affordable macroscopic models. This study, by performing shake table tests on typical building columns designed according to past Taiwanese practice, expects to develop a reliable phenomenon-based hysteretic model with consideration of material post-peak behavior. This model, when combined with P-Δ effects, will be capable of predicting structural dynamic response under code-defined maximum considered earthquakes.

Specimen Design and Experimental Setup

Both flexural-shear and flexural failure modes were investigated. Specimen frames built were:

- 1/3-scale single-story-3-bay RC frames (see Fig. 1): 2 prototype frames without lap-spliced connections at the bottom of the columns, one frame with lap-spliced connections at the bottom of the columns, and one frame with lap-spliced connections but retrofitted with wing walls. The 2 columns on the north side of the frame were designed without ductile details and failed in flexural-shear mode, while the 2 south columns showed ductile behavior and failed in flexural mode. Load redistribution is permitted to study both local and global collapse. Prototype frames...
also serve as a counterpart of UC Berkeley’s 3-story-3-bay frame.

- 1/2-scale RC portal frames (Fig. 2): This type of ductile frames was composed of 2 ductile columns interconnected by a strong beam. This is a follow-up test after the flexural-shear failure tests conducted in 2004. 2 frames were built to be tested under near-fault records TCU082ew and TCU076ns of the 1999 Chi-Chi earthquake.

  Supporting steel frames was installed to prevent the test frame from excessive out-of-plane movement, and to catch the frame after collapse. Load cells, inclinometers, accelerometers, strain gages, and Temposonics displacement transducers were applied to monitor structural hysteresis.

  Fig. 3 Group photo of 2 NCREE PIs, Director Jack Moehle of PEER, and 3 US exchange graduate students after collapse test was successfully conducted.

### Shake Table Test Results

Selected snapshots from global collapse of frame specimens are presented in Figs. 5 and 6, through which failure mechanism and development of plastic hinge and cracking can be observed with clarity. Figs. 4 and 7 give base shear vs. interstory drift hysteresis, which provide most valuable information on both hysteretic backbone skeleton and dynamic stability.

  Fig. 4 Base shear vs. interstory drift of ductile portal frame subjected to TCU082ew record of the Chi-Chi earthquake at 2g PGA level.

  Fig. 5 Selected snapshots of progressive global collapse of ductile portal frame subjected to TCU076ns record (Nov. 3, 2005).

  Fig. 6 Selected snapshots of flexural-shear collapse of the non-ductile column of 1-story-3-bay frame subjected to TCU076ns record (Oct. 3, 2005).
Fig. 7 Base shear vs. interstory drift of 1-story-3-bay frame without lap-spliced connections subjected to TCU082ew record of the Chi-Chi earthquake.

**Preliminary Numerical Simulation**

A pre-test challenge was called on numerical simulation with participation from UBC, NTU and NCREE. A macroscopic model, based on fiber section technique, was employed to simulate dynamic collapse of RC frame specimens as seen in Figs. 8 and 10, considering that it is computationally inexpensive and successful in predicting structural response. To do so, OpenSees (2004) developed by PEER Center was used as the analysis platform. The table achieved motion was applied to excite the numerical FEM model, in which OpenSees built-in material models (steel02 & concrete02) and Elwood-developed LimitState material models are both used, with the laboratory material strength test results as input parameters. The obtained numerical analysis results are plotted in Figs. 9 and 11. It is observed that numerical simulation provided satisfactory agreement with experimental observations prior to shear failure.

**Concluding Remarks**

This study investigates structural collapse in experimental and numerical simulation under a trilateral collaboration with USA and Canada.

Near-fault ground motion records from the Chi-Chi earthquake were used to test the specimens. Matlab code of image-based displacement measurement software ImPro was upgraded into C language to improve computational efficiency. This study also serves as a joint project of the school building rehabilitation plan to provide criteria for collapse judgment and numerical models for collapse analysis.

**References**

Elwood K. (2002), Shake table tests and analytical studies on the gravity load collapse of reinforced concrete frames, PhD dissertation, Department of Civil and Environmental Engineering, University of California, Berkeley (adviser: J.P. Moehle).

OpenSees (2004), Open system for earthquake engineering simulation, (opensees.berkeley.edu) University of California, Berkeley.
Numerical Simulation of a Dynamic Gravity Load Collapse Test of a Non-Ductile RC Frame
Chin-Hsiung Loh¹, Yuan-Sen Yang², Chiun-lin Wu², Chia-Hong Lin³ and Shu-Hsien Chao³

Abstract

This work employs three numerical modeling approaches to represent different failure modes observed in an RC frame collapse shake table experiment. The first model, called FAM Model in this paper, consists of linear beam-column elements with nonlinear zero-length hinges at two ends of each column. The second model, called Fiber-Section Model, uses nonlinear beam-column elements with fiber sections to simulate column flexural behavior and its failure. The third model, called Limit-State Model, extends the Fiber-Section Model by adding a nonlinear spring between each column top and the strong beam to simulate the shear limit-state failure. The springs at the top of the column use a limit-state shear failure model. The details of the above modeling approaches for the RC frame are introduced. This paper briefly summaries possible future study topics of the three models.

Keywords: RC frame collapse, numerical simulation, force analogy method, fiber section, limit-state shear failure model

Introduction

The numerical simulation of structural dynamic collapse analysis has been getting important due to the increasing awareness of researches on the global/local collapse consideration of civil structures. Structure nonlinear behaviors are typically much more complicated and hard to predict than the linear ones, and it would be even harder to simulate structure collapse under dynamic responses. It leads to not only high nonlinearity and inelasticity responses of structures, but also material cracks and inter-material interactions. Many approaches have been proposed to simulate the complicated mechanisms of structure collapse. However, there is no general solution being able to satisfy all requirements on simulating all failure mechanisms for all types of structures.

Besides the capability of simulating complicated behaviors, the simplicity is the same important to a numerical simulation approach of dynamic collapse analysis. Numerical simulation strategies employed by civil engineering industry are different from those in manufacturing industries such as automobile or aviation industries. Instead of modeling entire structures in refined finite element meshes of shell and solid elements, practical structural engineers in civil engineering industry commonly prefer coarse meshes and simple analysis approaches. The simple numerical models are typically composed of springs, trusses, beam-column elements, or other simplified element types. Simple and reliable numerical approaches help the promotion of next generation performance based design concepts.

RC Portal Frame Tests

Shake table tests of two non-ductile RC portal frames are numerically simulated in this work using three numerical modeling approaches. The RC portal frames are based on the same structural design (see Fig. 1) and are numbered Frame 2 and Frame 3, respectively. The so-called Frame 1 was not a collapse test and is not discussed in this paper. Table 1 lists three shake table tests to discuss in this paper. The Test 2A test was applied on Frame 2, while the Tests 3A and 3B were applied on Frame 3 in sequence. It should be mentioned that the table achieved ground motion measured by accelerometers are typically
slightly different from the ones we demanded. Three numerical models, namely the FAM Model, Fiber-Section Model and Limit-State Model, will be introduced in the following sections. More details about the shake table tests can be found at Wu et al. (2006).

### Table 1 Ground motions of three tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test2</th>
<th>Test 3A/3B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demand motion</td>
<td>Chi-Chi Eq. TCU076 N-S Direction</td>
<td>Chi-Chi Eq. TCU082 E-W Direction</td>
</tr>
<tr>
<td>PGA of achieved motion</td>
<td>1.29g</td>
<td>0.63g</td>
</tr>
<tr>
<td>Collapsed</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

#### The FAM Model

In the Force Analogy Method (FAM) model in this work, a new algorithm based on the FAM proposed by Wong & Yang (1999) is employed and is expanded by Chao & Loh (2005). In the new algorithm, linear beam-column elements with zero length hinges at two ends was used to simulate the results of the RC frame collapse tests. The P-Delta effect was considered by geometric matrix, and nonlinear behavior of the reinforced concrete element was considered by deteriorating plastic hinges at two ends. The deteriorating moment vs. plastic rotation relationship, including stiffness degradation, strength deterioration, and pinching effects, was used to simulate the nonlinear behavior of the reinforced concrete element. The shear plastic hinge was not considered in this work.

In the simulation of Test 2A, the moment vs. plastic rotation relationship was adjusted to get the best simulated result. Figure 2 shows the results of experiment and simulation. The dynamic collapse response of the RC frame can be satisfyingly simulated by the modified FAM method. However, how to determine the moment vs. plastic rotation relationship is a major problem while one estimating the response of the structures.

#### The Fiber-Section Model

The fiber section model (see Fig. 3) is constructed to simulate the failure modes observed in dynamic collapse tests Test 2A, Test 3A and Test 3B. OpenSees displacement-based nonlinear beam-column elements were used to model the strong beam and two low-ductility columns. Mass blocks were rigidly constrained with the beam. A protective beam system to catch the beam with mass blocks was installed at laboratory, and was numerically simulated by two vertical beam-column elements at both sides. The based beam column element employed in this work follows the fiber beam-column model (Spacon et al., 1996). It requires a section model to represent flexural and axial mechanical properties on the integration points along an element. In this work, the section model for the two columns was aggregated by a fiber section model and a high torsional stiffness. The fiber section model was composed of steel fibers and concrete patches. Figure 4(a) shows the reality section of an RC column in the tests; while 4(b) presents the ideal numerical fiber-section configuration. The torsion part was given a high stiffness and its behavior can be ignored due to the in-plain movement during the entire test.

![Fig. 2 FAM simulation and experimental results](image-url)

![Fig. 3 Elevation of the Fiber-Section Model](image-url)
Both pushover static numerical simulation and dynamic numerical simulation are performed using the detailed fiber section model in this work. Figures 5 and 6 show the inter-story drift vs. shear force relationships of the pushover numerical simulation (red curves) and the shake table tests (dark blue curves). The pushover numerical simulation and shake table tests roughly match at the maximal base shear forces and their corresponding inter-story drift, but they present different shear force degradations after the shear force peaks. Shear failure and flexural failure backbone curves are plotted in these figures based on the limit-state calculation suggested by Elwood & Moehle (2005). The shear failure backbone curves will be used and described in next section.

Figures 7 and 8 show the shear-drift curves of Test 2A and Test 3A/3B of experiment result and Fiber Model transient dynamic analyses. It is observed that Frame 2 was dominated by its flexural failure (see Fig. 9a), and the numerical analyses using Fiber Model well predicted base shear and roof drift time history. On the other hand, Frame 3 demonstrated more shear deformation in its structural failure (see Fig. 9b), making larger difference between experimental result and Fiber Model numerical result. The fiber section model takes into account only the flexural contribution such that roof drift prediction still needs to be improved. Generally speaking, the Fiber Model reasonably predicted maximum base shear force. This is because shear force does not vary significantly in the nonlinear plateau.

The Limit-State Model

Shear failure mechanism and its simulation approach are important to numerical simulation of
dynamic collapse. Shear failure modes are observed in the aforementioned shake table tests, especially in the Test 3B. The nonlinear beam-column elements with fiber sections and geometrical nonlinear analysis approaches provide good potential to simulate flexural failure and P-Delta effects. Theoretically, however, they are not sufficient to simulate shear failure modes due to fiber section’s plane-remains-plane assumption.

A limit-state model proposed by Elwood & Moehle (2005) is employed to extend the Fiber Model so that the shear limit capacity can be considered. In Limit-State model, a nonlinear shear spring is added between the rigid beam and the top of each RC column (see Fig. 10). The shear spring is simulated by a zero-length element provided in OpenSees; while its uni-axial material is a limit-state material with a three-point backbone curve. The red curves in Figures 11 and 12 show the pushover analysis results using the detailed fiber section with shear limit state models of Frames 2 and 3, respectively. The numerical analyses did not proceed before they reach the shear failure backbone curves. More study work is needed to solve the divergence problems in these cases.

**Summary**

Three numerical modeling approaches are employed to simulate of RC frame collapse shake table Tests 2A, 3A and 3B. The FAM Model consists of linear beam-column elements with nonlinear zero-length hinges at two ends of each column matches the Test 2A result after tuning parameters of the nonlinear hinges in a try-and-error manner. The Fiber-Section Model leads to good matches with the three tests on maximum base shear forces, as well as the displacement histories before structure collapses, but does not represent the shear failure mode due to its plain-section assumption. The Limit-State Model constructed in this work is extended from the Fiber Model and considers shear failure mode using a shear-limit-state spring but does not converge well when the limit point is reached.

More research works are still needed to simulate the collapse behavior of RC frames. The parameter setting of the FAM model needs to be studied, so that it can be employed without parameter tuning with experimental results. The Fiber Model needs further improvement on the shear failure simulation, especially for the simulation of non-ductile RC frames, which collapse behavior is probably dominated by shear failure. Shear-Limit Model needs further study to explore more potential to simulate shear failure modes of RC columns. Other limit models considering axial failure or flexural/shear/axial composite failure modes should be tried in the future.

**References**


A Study on Seismic Evaluation and Retrofit of Fire Department Buildings

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Fu-Pei Hsiao² Te-Kuang Chow⁴ Chin-Lu Lin⁴

Abstract

While an earthquake causes disasters, the fire department must be at the position of front line. The fire department will be the command post. Therefore, the seismic capacity of the fire department buildings must be higher than common general buildings. As the new technology is developed, the seismic design code is revised time after time. Some existing fire department buildings are designed according to the old code. These existing buildings may not satisfy the requirement of the new design code. The structural system of fire department buildings is very unique for the purpose of mobility of fire trucks. The fire trucks must quickly leave the buildings to execute their duty after the earthquake. Their structural systems may cause the weakness in seismic capacity. The fire department buildings not only need structural safety but also performance. Because earthquakes occur frequently in Taiwan, the seismic evaluation and retrofit technique of fire department buildings are very urgent topics.

This project has completed the structure investigation of ten fire department buildings in Taipei. The preliminary seismic evaluation method was developed and used to evaluate the ten fire department buildings in Taipei. A detailed seismic evaluation method was also developed based on the push-over analysis. Two virtual fire department buildings were used as the examples of the detailed seismic evaluation method. According to the needs of performance of fire department, the damage ground acceleration and floor drift ratio are used as the numerical indexes of retrofit effect evaluation. One virtual fire department building used steel plate retrofit, wing wall retrofit, and extended column retrofit, and its seismic capacity after retrofit was evaluated.

Keywords: fire department buildings, seismic evaluation, seismic retrofits, push-over analysis.

Introduction

While an earthquake causes disasters, the fire department must be at the position of front line. The fire department will be the command post. Therefore, the seismic capacity of the fire department buildings must be higher than common general buildings. According to the seismic design code of buildings, the importance factor is 1.5 for the fire department buildings. In other words, the fire department buildings are 1.5 times the seismic design force of the general buildings. The seismic capacity of the fire department buildings is really important. As the new technology is developed, the seismic design code is revised time after time. Some existing fire department buildings are designed according to the old code. These buildings may not satisfy the requirement of the new design code. The structural system of fire department buildings is very unique for the purpose of mobility of fire trucks. The fire trucks must quickly leave the buildings to execute their duty after the earthquake. Their structural systems may cause the weakness in seismic capacity. The fire department buildings not only need structural safety but also performance. Because earthquakes occur frequently in Taiwan, the seismic evaluation and retrofit technique of fire department buildings are very urgent topics.

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Keywords: fire department buildings, seismic evaluation, seismic retrofits, push-over analysis.

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buildings are designed according to the old code. These existing buildings may not satisfy the requirement of the new design code. The structural system of fire department buildings is very unique for the purpose of mobility. The fire trucks must quickly leave the buildings to execute their duty after the earthquake. Their structural systems may cause the weakness in seismic capacity. The fire department buildings not only need structural safety but also performance to reduce the secondary disaster of earthquakes. Because earthquakes occur frequently in Taiwan, the seismic capacity of all fire department buildings must be examined. If the seismic capacity of any buildings is not enough, a proper retrofit must be taken. So the seismic evaluation and retrofit technique of fire department buildings are very urgent topics.

**Structure Investigation of Fire Department Buildings**

This study has investigated the structural systems of Jingmei, Bade, Chengzhong, Longshan, Shezi, Tianmu, Guangming, Shuangyuan, Guting, and Huashan fire departments in Taipei. The major investigation items included floor area, number of stories, story height, column dimensions, number of columns, wall dimensions, number of walls and the performance.

After the investigation, we found some common characteristics: (1) high-raised ground floor, ground floor is used as garage of fire trucks; (2) the direction, which is vertical to the traffic line of fire truck, is the weak axis; (3) cantilever structural system to increase the space of office.

**Preliminary Seismic Evaluation of Fire Department Buildings**

This study has collected structure data of seventeen fire department stations as the parameter base to develop the table of preliminary seismic evaluation. This first step of this preliminary seismic evaluation method is to measure the dimensions of existing buildings such as the dimensions of columns and walls, floor area and number of stories. The second step is to count the strength of every horizontal-force-resisting element then to sum all acquired data. According to the seismic force equations of existing seismic design code, the seismic performance of the entire structure can be obtained. From the calculation, we have the basic seismic index E which is multiplied by the investigation factor Q to get the final seismic index I. The seismic index can be used to evaluate the seismic capacity of buildings. The used parameters include: (1) unit area weight 1200 kg/m², (2) unit area strength of three-side constrained brick walls 1.5 kg/cm², (3) unit area strength of four-side constrained brick walls 5 kg/cm², (4) unit area strength of RC walls 24 kg/cm², (5) unit area strength of RC columns 8 kg/cm², (6) horizontal acceleration spectrum factor (C) 2.5, and (7) seismic force reduction factor (F₀) 1.53.

The purpose of this evaluation table is to find out the fire department buildings which may collapse under the design seismic force. The standard seismic index value is 100. If the investigated seismic index value is lower than 100, it means this building may collapse under the earthquake whose return period is 475 years. This kind of buildings needs to proceed to the detailed seismic evaluation stage in the first priority. If the investigated seismic index value is between 100 and 150, it means the safety factor of this building is less than the needs of such important buildings. This kind of buildings needs to proceed to the detailed seismic evaluation stage in the second priority. If the investigated seismic index value is higher than 150, it means the seismic capacity of this building is enough.

**Detailed Seismic Evaluation of Fire Department Buildings**

Up to now, the detailed seismic evaluation method used by engineers was developed by Prof. I.C. Tsai of Taiwan University. The method named seismic performance static method does not consider the deformation of the structure. Its calculation does not include the nonlinear behavior of the members. So its analysis result is different from the actual response of the structure. And its ductility factor is obtained from weighting method, which is arguable.

In this study, the suggestion of ATC-40 is adopted. We need to construct the relationship curve of the base shear and the roof displacement of fire department buildings comparing with the non-elastic response spectrum. By setting of the performance point and the equivalence damping, we can determine the damage ground acceleration of the fire department building. If this value is higher than design ground acceleration, the building’s seismic capacity is enough. The relationship curve of the base shear and the roof displacement can be obtained from push-over analysis. The analysis preserves the force and deformation relationship of structures. The seismic performance of structures is not decided from strength only. The strength and ductility are together to effect the seismic performance of the structure.

This study used a commercial software ETABS as the analysis tool for the detailed seismic evaluation. ETABS-Nonlinear is a version of ETABS, which includes the nonlinear analysis. The program provides a nonlinear static analysis, also named as push-over analysis, which can be used to calculate the relationship curve of the base shear and the roof displacement. This curve is also named as capacity curve. The nonlinear behavior of structures is presented by the specific plastic hinges of the structure.
members. The types of plastic hinges are: (1) axial hinges, no specific position and presenting the axial failure of members, (2) shear hinges, no specific position and presenting the brittle shear failure, and (3) moment hinges, setting at the both ends of members and presenting the ductile flexural or flexural-shear failure. The parameters of plastic hinges are provided by the plastic part of loading-displacement curve of the member. According to the failure mechanism, the loading-displacement relationship of the beam or column has three kinds: (1) flexural-shear failure, as shown in Fig. 1, (2) shear failure, as shown in Fig. 2, and (3) flexural-shear failure, as shown in Fig. 3. In these figures, \( V_m \) is the flexural strength; \( V_n \) is the shear strength; \( \Delta_s \) is the displacement at shear failure; and \( \Delta_a \) is the displacement at which the element loses its ability to carry axial loading.

![Fig. 1 Loading-displacement curve of flexural-shear failure](image)

\[ \text{Shear strength curve} \]

\[ V_m \quad V_n \quad V=2M/L \]

\[ \Delta_s \quad \Delta_a \quad \Delta \]

This study used equivalent columns to simulate reinforced concrete walls and equivalent struts to simulate brick walls. Two fire department buildings were used as the examples for the detailed seismic evaluation.

![Fig. 2 Loading-displacement curve of shear failure](image)

\[ \text{Shear strength curve} \]

\[ V_m \quad V_n \quad V=2M/L \]

\[ \Delta_a \quad \Delta \]

**Strategy and Design Principle of Seismic Retrofit of Fire Department Buildings**

To improve the seismic capacity the structure whose seismic capacity is not enough to the requirement of the modern seismic design code by some methods is called the seismic retrofit. The seismic retrofit methods include: (1) to strengthen the structure elements, (2) to improve the structural system, and (3) to reduce the seismic force. In the seismic retrofit design, we must consider the performance of the retrofitted structure, the loading path of the seismic force, the stability of the foundation, and the cost of the seismic retrofit. The three important principles for the seismic retrofit include increasing ductility, increasing strength and preventing large displacement.

![Fig. 3 Loading-displacement curve of flexural failure](image)

**Extended column retrofit of fire department station A**

Based on the characteristics of fire department buildings and the needs of mobility of fire trucks, the seismic retrofit can not influence the available space of the fire department buildings. This study used extended column retrofit (as shown in Fig.4), wing wall retrofit (as shown in Fig.5) and steel plate retrofit (as shown in Fig.6) as the examples. A virtual fire department station A was used as the example of these three kinds of seismic retrofits. In general, the
extended column retrofit and wing wall retrofit were used to increase the strength of the buildings. The extended column retrofit is to increase the cross section area. This strength retrofit increases the loading capacity and stiffness of structure elements to improve the seismic performance of the building. The wing wall retrofit is to set wing walls in the weak direction of the structure. This strength retrofit increases the strength along which the seismic capacity of the structure is not enough. The steel plate retrofit is to cover the structure element with steel plates. This way is not to increase the cross dimensions of the structure element but increase the ductility of the structure.

Fig. 5 Wing wall retrofit of fire department station A

Fig. 6 Steel plate retrofit of fire department station A

Retrofit Effects of Seismic Retrofits

From the push-over analysis to get the capacity curve and damage ground acceleration of the building, we need to specify a performance point in the capacity curve. This performance point can be used as the numerical index to evaluate the effects of seismic retrofits. The decision of the performance point must obey the following requirements:

(1) Strength reduction is lower than 20%: In the push-over analysis, plastic hinges will be observed at part of structure members and the structure has the nonlinear behavior. As the procedure is going on, the strength of the structure will decay. When the base shear strength is lower than 80% of the maximum strength, the structure may be considered as unstable.

(2) The vertical elements keep their axial load carrying ability: When any element of the structure loses its axial load carrying ability, this structure may collapse.

(3) The maximum displacement of the first floor can not exceed 25 cm: Many fire trucks parks are parked in the typical fire department buildings. The buildings cannot deform too seriously to influence the mobility of fire trucks to execute their duty. According to the investigation, we estimate the clear distance between trucks and columns is nearly 25 cm.

Fig. 7 Capacity curves of fire department station A

The design ground acceleration of the seismic design code is the standard to the decision of retrofit and the goal of the seismic retrofit. The example fire department station is at the Taipei Zone Two of the seismic design code. Its damage ground acceleration need to be larger than 0.36 g. Fig. 7 is shows the relationship curves between the base shears and the roof displacements of three retrofitted buildings and one original building. The extended column retrofit and wing wall retrofit can largely increase the strength of the building to match the requirement of seismic capacity. The steel plate retrofit did not increase the strength but largely increase the ductility of the building. Because the maximum displacement of the fire department buildings is restricted, the steel plate retrofit is not suitable for the fire department station A.

Conclusions

This study provides the numerical performance indexes according to the performance needs of fire department buildings. These indexes are used to set the performance point on the capacity curve. It is also used to check the effects of the seismic retrofit of the fire department buildings.
The Construction and Application of Database on Potential Seismic Geotechnical Hazards (II)

Ming-Chun Ke¹, Min-Yann Hsieh¹, Chin-Hsun Yeh²

柯明淳¹、謝旻諺¹、葉錦勳²

Abstract

Safety against earthquake hazards has two aspects: first, structural safety against potentially destructive dynamic forces and secondary the safety of a site itself related with geotechnical phenomena such as liquefaction and landslide. Soil liquefaction has been a major cause of damage to lifelines, facilities and building foundations in past earthquakes and clearly poses a significant threat to the integrity of structures and facilities during future earthquake. If the liquefaction potential areas could be predicted in advance, it would be possible to plan for earthquake disasters mitigation program. This study is mainly to develop the module of liquefaction potential and landslide. In the module of liquefaction potential, the geotechnical database will be constructed in TELES for predicting the liquefaction potential and settlements induced by liquefaction. In the module of landslide, the gradient of the slope can be calculated according to the digital topography model and the geological map. The landslide potential is classified as several categories in consideration of earthquake acceleration. The threshold of acceleration triggering landslide for each category is defined. Mapping with the road map, the impact of landslide to the road in mountain area can be evaluated once the peak ground acceleration of the site is predicted by TELES. The analytical results from the modules of liquefaction potential and landslide potential could be a basic reference for emergency responses.

Keywords: TELES, liquefaction potential, landslide potential

Introduction

People live in limited areas which is usually located on alluvial soil in Taiwan. Severe damages on structures induced by soil liquefaction during some major earthquakes caused losses of people’s lives and properties. Besides, the city development was gradually expanded to the mountain areas since the urban areas were highly developed. However, mountains in Taiwan are steep and with lots of joints. Landslide was frequently induced by earthquake and the traffic in mountain areas was interrupted. Food and material could not be delivered to the mountain area and it becomes a stumbling block in the emergency response system.

The objective of this research is to develop the liquefaction and landslide potential module in Taiwan Earthquake Loss Estimation System, TELES.

Soil Liquefaction Potential

In the analysis of soil liquefaction potential, the mainly research of this year can be categorized into several parts as below.

(1) Updating database of borehole

The database of latest version of borehole database in Geo2002 provided by Central Geological Survey, MOEA are collected this year. Due to the goals of collecting and installing database are not focusing on the analysis of soil liquefaction potential. Therefore, it is necessary to transfer undefined data into formatted data defined by the research prior to saving the data, then the integrated borehole database which are supplied for the analysis of soil liquefaction potential are installed in the TELES.

(2) Integrate soil liquefaction module into TELES

Since the soil liquefaction potential analysis code

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used in the research in last year belongs to constant value analysis method which is recent and common applied. It is not include the concept of probability risk, so it has lower reference value for seismic hazard analysis. In this research, through the combination of Earthquake intensity model, site effect, borehole database, and soil liquefaction potential module in Taiwan Earthquake Loss Estimate System (TELES), then we can consider earthquake intensity in the analysis of soil liquefaction potential under the earthquake event simulation and establish soil liquefaction potential and settlement system that provides real seismic hazard. Considering Taipei as the research area, Jean (2001) and magnitude 7.5, Fig1 is liquefaction potential map in Taipei.

(3) The update of liquefaction susceptibility map

Combining the liquefaction potential result evaluated by module through current boreholes and this year added boreholes, geological maps, hydrology maps and history, this research updates the soil liquefaction susceptibility map in TELES. The amount of boreholes classified by different liquefaction susceptibility in the Geological database of TELES integrated from the latest version of borehole database in Geo2002 was shown on the Table1. The boreholes with \( P_L > 15 \), when the peak ground accelerations are 0.15g, 0.2g, 0.25g, 0.35g and 0.45g, belong to “very high”, “high”, “moderate”, “low” and “very low” susceptibility categories, respectively. Other boreholes with \( P_L < 15 \), when the peak ground acceleration is 0.45g, is classified as “none” sensitive to soil liquefaction. The earthquake magnitude and the ground water depth are assumed to be 7.5 and 1.5 meters, respectively, in the previous classification scheme. The modified Seed method (1997) was employed to estimate the liquefaction safety factor at each depth of the site in this research.

Next, according to the borehole position of very high, high, moderate, low, very low, and none liquefaction potential, draw the radius of the circle area 200, 300, 400, 600, 800, and 1000m respectively. We will give different liquefaction potential area different weighting value. From very high to very low, we will give them 6, 5, 4, 3, and 2 respectively. Besides, the liquefaction potential of the deposit layer for 1:500000 geological maps is given 2. The area for none liquefaction potential is given 1. The liquefaction potential of other areas classified as none deposit layer and no liquefaction is set 0. If some of the areas overlap, these overlapping areas are regarded as higher liquefaction potential. For example, the area overlapped by high liquefaction potential area and low liquefaction potential area is regarded as high liquefaction potential area. Taipei as an example is expressed as Figure 2.

(4) The village liquefaction susceptibility map

Regarding the borehole positions of various liquefaction susceptibility categories as the centre of a circle separately, we draw the each round region in accorded with the proportion of 4:9:16:36:64:100 in area. All the regions were individually assigned the weighted relation of 6, 5, 4, 3, 2 and 1 when we took a village as a basic unit in drawing the liquefaction susceptibility map. In this study, it is found that the most computational results are controlled by the area of each region.

To consider the effects which were produce by the proportion of area, this study used an inverse proportion of area as new weighting to calculate the classification of susceptibility of soil liquefaction that the village is a unit. To give the weighted relation: 25, 11.1, 6.3, 2.8, 1.6, and 1 separately. Take all villages of Taipei as example, we rank the different soil liquefaction susceptibility value of classificatory area from maximum to minimum, and set 25, 11.1, 6.3, 2.8, 1.6, and 1 separately. Overlapping them with the border of the village, we can get the liquefaction value of the village by using the area weighted averages method. The liquefaction value above 18.05 is regarded as very high liquefaction potential level. The liquefaction value between 8.7 and 4.55 is regarded as moderate liquefaction potential level. The result of Taipei was showed as figure 3.

Comparing the result of old weighting and the new one, we discover that new weighting can make the influence of higher classificatory area of susceptibility appear significantly.

Table 1 The classification of soil liquefaction susceptibility categories (Geo2002)

<table>
<thead>
<tr>
<th>Category</th>
<th>Conditions</th>
<th>Number of borehole</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>PGA=0.15g then P_L&gt;15</td>
<td>28</td>
</tr>
<tr>
<td>High</td>
<td>PGA=0.15g~0.2g then P_L&gt;15</td>
<td>109</td>
</tr>
<tr>
<td>Moderate</td>
<td>PGA=0.2g~0.25g then P_L&gt;15</td>
<td>207</td>
</tr>
<tr>
<td>Low</td>
<td>PGA=0.25g~0.35g then P_L&gt;15</td>
<td>439</td>
</tr>
<tr>
<td>Very Low</td>
<td>PGA=0.35g~0.45g then P_L&gt;15</td>
<td>239</td>
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Fig. 1 The liquefaction potential map of Taipei

Table1

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<td>207</td>
</tr>
<tr>
<td>Low</td>
<td>PGA=0.25g~0.35g then P_L&gt;15</td>
<td>439</td>
</tr>
<tr>
<td>Very Low</td>
<td>PGA=0.35g~0.45g then P_L&gt;15</td>
<td>239</td>
</tr>
</tbody>
</table>
Landslide Potential Module

The major job in this year has been keeping on objective at last year. We have defined the potential dangerous levels of landslides occurred by earthquakes and have combined with the database of slopes and the database of villages, Taiwan. Because of 14 million grids, we selected Nanto country for a testing model to avoiding mistakes.

We know that it’s suddenly and fast when earthquake happens. If it will be made hazards, we have needed a tool with efficiency and high precision when earthquake happened. So Remote Sensing is the best choice for inquiring into disaster. By FORMOSAT-2 had successfully launched on May 21, 2004, this technique has been more careful than before.

(1) Combining grids with databases that are about landslides occurred by earthquakes.

Combining grids with databases that are about landslides occurred by earthquakes is the major job in this year. First, we have combined 14 million grids with geologic database and database of villages by location of grids. Because there a lot of grids must be analyzed and we must keep mistakes off, we choose Nanto country to be a model for this job.

Compared this result with researches on 921 Chi-Chi earthquake, we can revise many data, like sums of grids, to accord with true (Fig4).

(2) The potential dangerous grades of landslides occurred by earthquakes.

The conditions for the potential dangerous grades of landslides occurred by earthquakes is by slope and rock formations. Landslides occurred by earthquakes are seated in 45°~60°, 30°~45°, 60°~90°, 10°~30° and 0°~10° in turn. We have matched four parts with rock types, with scale of 1/250,000 geological map produced by the Central Geological Survey. We have granted points of slope types and rock types, that points of the highest are 10. We have gotten the product of multiplication of rock types and slopes types on formula.1. We have divided the potential dangerous grades into 15 groups by \( P \). (table 2)

\[
P = \sqrt{S^2 \times R^2} \quad (\text{form.1})
\]

\( P \) : the point of the potential dangerous grades
\( S \) : the point of slope types
\( R \) : the point of rock types

Then we have been revising points of all types, we will get the parameter which we develop landslide
potential module from of each village.

Table 2  the table of the potential dangerous levels

<table>
<thead>
<tr>
<th>Slope</th>
<th>Rock grade</th>
<th>slope</th>
<th>Rock grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>46~60</td>
<td>c</td>
<td>61~90</td>
</tr>
<tr>
<td>8.9</td>
<td>46~60</td>
<td>b</td>
<td>31~45</td>
</tr>
<tr>
<td>8</td>
<td>31~45</td>
<td>c</td>
<td>11~30</td>
</tr>
<tr>
<td>7.7</td>
<td>45~60</td>
<td>a</td>
<td>11~30</td>
</tr>
<tr>
<td>6.9</td>
<td>61~90</td>
<td>c</td>
<td>4.4</td>
</tr>
<tr>
<td>6.3</td>
<td>31~45</td>
<td>a</td>
<td>11~30</td>
</tr>
<tr>
<td>6.9</td>
<td>61~90</td>
<td>b</td>
<td>0~10</td>
</tr>
<tr>
<td>6.3</td>
<td>46~60</td>
<td>w</td>
<td>3.4</td>
</tr>
<tr>
<td>11~30</td>
<td>c</td>
<td>2.8</td>
<td>0~10</td>
</tr>
</tbody>
</table>

(3) Study of using the technology of Remote Sensing on hazards by earthquake.

The other important work in this year is study of using the technology of Remote Sensing on hazards by earthquakes. According to the researches on Remote Sensing, there three kinds of Remote Sensing are used. Those are Synthetic Aperture Radar (SAR), Airborne MSS and Unmanned Aerial Vehicle (UAV, Fig5).

Because we need images that must be high dpi and efficiency to estimate damages on earthquake, we suggest that Unmanned Aerial Vehicle (UAV) is the best tool for inquiring into damages on earthquake. Beside high dpi and efficiency, the cost of UAV is cheaper than the cost of SAR and Airborne MSS. But it a defect of this tool that UAV is limited for flight altitude, the covering areas of images, catching by UAV, are always smaller than are catching by SAR and Airborne MSS.

![Fig. 5 Remote control copter with a camera is one of Unmanned Aerial Vehicle.](image)

Conclusions

The information of borehole susceptibility categories can be extended to obtain the susceptibility map and can be applied in earthquake loss estimation methodology, such as TELES. The liquefaction susceptibility map of Taiwan was updated by coupling the data obtained from this study and the information from the geological maps. Once the influence factors in estimating liquefaction potential and the amount of settlement during strong earthquakes, such as susceptibility category, peak ground acceleration, earthquake magnitude and ground water depth, are studied in detail, the results and the associated parameter values can then be incorporated in TELES to assess the liquefaction probability and settlement during scenario earthquakes.

Because grids in each village have been checking with researches on 921 Chi-Chi earthquake, the mean of slopes in each village will be revised more exact than before. If we use the technology of Remote Sensing on hazards by earthquake, we could step rate of transmit messages for distribution of hazards by earthquake up. But the technology and software about Remote Sensing must be needed for professionals. We would must appraise cost of drill and purchase in minute or find partners with professional specialty if we have deal with damage after earthquakes happened.

References


Application of GIS Technologies to Setup a Seismic Disaster Simulation Website

Wei-Chang Chen¹, Chin-Hsun Yeh²

Abstract

Taiwan Earthquake Loss Estimation System (TELES) has been developed to integrate research accomplishments on seismic hazard analysis, structural damage assessments and socio-economic impacts. The software is run on personal computer and is based on the commercial GIS software MapInfo. After putting lots of efforts, it has been successfully used in proposing disaster mitigation plans and emergency response affairs by governments and cooperative institutions. However, there are some constraints to make the software popular, for example prerequisite of expensive software MapInfo and insufficient number of well-trained people to run TELES. Now, based on the internet technologies, it is possible to access, query and display spatial data over the world-wide Web. In this paper, we discuss the advantage of using Web-GIS technology to construct a Web-based Seismic Disaster Simulation System, which has close relationship with TELES.

Keywords: Taiwan Earthquake Loss Estimation System, Seismic Disaster Simulation, Web Geographic Information System

Background

The main target of Seismic Disaster Simulation Division in National Center for Research on Earthquake Engineering is to promote the earthquake loss estimation system and to reduce the seismic losses or disasters. To achieve this goal, we have devoted lots of energy and time to develop GIS-based software: Taiwan Earthquake Loss Estimation System (TELES) since 1999. It has been installed in some government agencies to be a decision-making support system while strong earthquakes occur. It also has been applied by local governments for proposing disaster mitigation plans.

There are some troubles for us to promote TELES. First of all, the commercial software MapInfo Professional is the prerequisite to install TELES. However, its price is not affordable for the majority who want to experience TELES. Secondly, the number of specialists in earthquake engineering or GIS is insufficient. There’s no guarantee that the scenario results which were manipulated by users without special training can be trusted.

Now internet technologies are sophisticated enough to access spatial data easily over the world-wide Web. Web-GIS technology shows great promise in sharing and integrating the spatial data among governance. Furthermore, it helps people, who have desire to experience GIS applications, to avoid the costs of software purchase. In view of this factor, we can take advantage of Web-GIS to implement a platform which provides the capability of seismic disaster analysis online. In this paper, we will investigate this state-of-the-art technology, describe the benefits by using it, and finally demonstrate the capabilities in the platform.

GIS Technology Trend: Web-based GIS

The emergence of Web-GIS technology provides a platform for data sharing and integration over the internet. Developments in internet-enabled GIS data integration and analysis allow client-side users to have the opportunity to access the up-to-date data and sometimes let organizations keep themselves off data management and maintenance.

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³ MapInfo Professional is a powerful Microsoft Windows-based GIS application that enables business analysts and professionals to easily visualize the relationships between data and geography.
Sometimes people may think of Web-GIS technology as a simple extension of traditional desktop mapping activities. However, this is not the case. In the strict terms, Web-GIS mapping is not the same as desktop GIS mapping. The spatial analysis functions of GIS include the capabilities of buffering, overlaying, dynamic map object selection, nearest neighbor computation, etc. The full range of analysis capabilities are available in most desktop GIS software, but are not available in contemporary Web-GIS solutions. This is due to the thin-client browser-based concept of Web-GIS technologies (Fitzgerald, 2000).

With rapid technology growth, numerous companies provide various kinds of products. Selecting Web-mapping software to meet task requirements of client and server sides is difficult. For example, user interaction is an important factor and the basic trade-off is time consuming. Two mainstream architectures has been discussed; one is thin-client but fat-server; and the other is fat-client but thin-server. In our case, a thin-client but fat-server framework was selected to build our Web-based Seismic Disaster Simulation System.

AspMap is used to construct the Web-based simulator because of its high-performance in spatial data access, display and analysis capabilities. It is used in ASP.NET, ASP applications and services. In detail, AspMap can create image maps in standard image file format, or even send a bit-stream directly to the browser. The size of image map is generally small and can be downloaded very quickly. Further interaction requires the client to send information to the server, and a new map is generated and resent. Although single transmission time is short, all image-map interactions place large demands on the server and may lead to excessive volumes, which lead to slow response times. In addition, the result of any interaction is unknown until the map is redrawn, and it is unusual for only one interaction to be required before the desired map is displayed. Those disadvantages are the stakes we are going to take (W. Fredrick Limp, 1999).

Description of the Architecture of Web-based Seismic Disaster Simulation System

The architecture of Web-based simulator is shown in Fig. 1. To implement the program, TELES is used to establish seismic scenario databases in the beginning. The modules of TELES contain potential earth science hazard, direct physical damage assessment, induced physical damage assessment, and direct/indirect socio-economic loss estimates. Each of these modules can be decomposed into several sub-modules. All the modules and sub-modules are interdependent as shown in Fig. 2. In this project, we obtain enormous simulation results from running TELES in batch mode. The details of building the seismic scenario database can be found in next section.

![Fig. 1 The architecture of Web-based Seismic Disaster Simulation System](image1)

![Fig. 2 Methodology framework of TELES](image2)

After the database was established, another job for us to do is to design user interface and to write analysis engine to search in the database, which can make clients access the scenario databases efficient and secure. In this project, the major development tools are C# and AspMap Web component. The common languages, such as HTML, CSS, and JavaScript, are also implemented into the operating interface development.

Establishment of Seismic Scenario Database

Probabilistic seismic hazard analysis is often applied in estimating seismic risks in different

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4 C# is a new programming language designed for building a wide range of enterprise applications that run on the Microsoft .NET Framework.
regions or for some specific critical facilities (e.g., nuclear power plants, dams, etc.). The hazard curves obtained from the analysis are often in terms of ground motion intensity parameters (e.g., peak ground acceleration, response spectra, etc). Other quantities, such as soil liquefaction potential, damage-state probabilities of civil infrastructures, number of casualties and amount of losses, are often derived indirectly from the hazard curves of ground motion intensity. Since the influence factors for soil liquefaction and other damage/casualty/loss quantities are very complicated, they cannot be expressed as one-to-one functions of ground motion intensity parameters.

This study intends to study various kinds of seismic risks through scenario simulations. TELES is modified to extend its seismic scenario simulation capability, so that it can run in batch mode to obtain various kinds of analysis results when the study region is subjected to different set of scenario earthquakes. The set of scenario earthquakes may or may not come from probabilistic seismic hazard analysis. If the whole set of scenario earthquakes covers all the interested events and the annual occurrence rate of each scenario earthquake is estimated by probabilistic seismic hazard analysis, the scenario database can be applied in probabilistic seismic risk assessment, which will be explained in the following sections. Otherwise, it can be applied in early seismic loss estimation to save computational time by quick queries soon after occurrences of strong earthquakes.

Seismic-hazard source model

In general, probabilistic seismic hazard analysis involves three steps. The first step is to identify the probable seismic-hazard sources in the neighborhood of a study region. The second step is to select an appropriate ground motion model such as attenuation relationships. The third step is the probabilistic calculation of the effect due to different seismic-hazard sources.

The seismic-hazard source model is simply a description of the spatial and temporary distribution of earthquakes with various magnitudes and occurrence rates. Referring to the fault-rupture model proposed by Der Kiureghian and Ang (1977), the known active faults should be properly taken into consideration as Type 1 sources. For the rest of seismic-hazard sources, they can be modeled as Type 2 or Type 3 sources, depending on the available information of local geological condition, tectonic structure and seismological records.

For simplicity, this study focuses on simulating area sources around Taiwan area. The seismic source zoning scheme and the earthquake catalog used were similar to those used in Loh and Wen (2004). The upper bound magnitude in each zone can be estimated graphically based on the assumption of constant energy accumulation and release (Makropoulos and Burton, 1983). The parameters in the Gutenberg-Richter magnitude recurrence relation can be obtained by least square method or maximum likelihood method (Weichert, 1980).

The seismic sources can be further divided into smaller grids. The annual occurrence rates (per unit area) of various earthquake magnitudes and focal depths in each grid can be assumed to be uniform within individual sub-zones or proportional to the number of historical earthquakes (per unit area) occurred in the grid. Considering the uncertainty in earthquake occurrences and the tendency of occurrence in some specific grids, it is most likely that the true annual occurrence rate of future earthquakes in each grid lies within the range of the previous bounds. The effects of using different rules to assign occurrence rates for scenario earthquakes on the expected annual losses of counties/towns will be studied in the later section.

Seismic source parameters in scenario database

To establish the Taiwan seismic scenario database, the shallow and deep earthquake source zones are divided into grids with 0.2 degree intervals. A representative earthquake magnitude in each 0.2 magnitude interval is selected, starting from lower bound magnitude \( m_L \) to upper bound magnitude \( m_U \) in each grid. Since a fault-rupture model is preferred, the empirical relationship between the fault-rupture length \( L \) and the earthquake magnitude \( m_L \) should be provided and it can be expressed as:

\[
L = \exp[1.06m_L - 3.232]
\]  

It is noted that Eq. (1) is similar to that used in Loh and Wen (2004). However, a minor modification of the first coefficient (changing from 1.006 to 1.06) has been made to match the observation in the Chi-Chi Taiwan earthquake in 1999. The observed fault-rupture length is about 80 to 100 km for an earthquake with estimated Richter magnitude 7.1 to 7.3. Thus, the estimate for fault-rupture length increases a little in this study. To increase the precision of analysis results and to satisfy the assumption of Type 3 sources, the number of fault-rupture directions ranges from one to four depending on the fault-rupture length. In this study, if the fault-rupture length is larger than 15 km, 30 km or 60 km, the number of fault-rupture directions is 2, 3 and 4, respectively.

Although the other seismic source parameters, such as fault-rupture width and dip angle, can be considered in TELES, the fault-rupture width and dip angle are assumed to be zero and 90 degrees, respectively. The focal depths of scenario earthquakes are 10, 20, 30, 50, 70 and 90 kilometers. In summary, there are 99,000 scenario earthquakes.
defined in the database.

Implementation

Through the Web-based Seismic Disaster Simulation System as shown in Fig. 3, users can access the seismic scenario database efficiently without installing any plug-ins or paying any money to buy expensive software. In the web-site, basic GIS functions; for instance, zoom-in, zoom-out, and moving the maps, have provided for the users. More advanced functions, such as identifying the properties of the maps, layers control, spatial search, etc. are under developing now.

Besides manipulating the maps, users can also define seismic source parameters at will, for example, longitude and latitude of an epicenter, focal depth, and magnitude. After the source parameters are sent to and processed by the server engine, analysis results are displayed in the client's screen. The analysis results include the number of buildings in different damage states, the number of injured and death tolls, and the direct economic losses of buildings. Through the Web-GIS interface, people can investigate these results explicitly and quickly by creating thematic maps as shown in Fig. 3, 4 and 5.

Conclusions

Advanced Web-GIS technologies are applied in this project to help developing a seismic disaster simulation system. In this project, we integrate and share both data and research achievements. People in the world can enjoy the benefits of it.

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Database Collection and Historical Earthquake Scenarios (I)

Ching-Lin Wen¹, Ming-Chun Ke¹ and Chin-Hsun Yeh²

文慶霖、柯明淳、葉錦勳

Abstract

The main goals of the project are to collect civil infra-structure database and to simulate historical earthquake events using Taiwan Earthquake Loss Estimation System (TELES). The civil infra-structure database is classified into four categories: general building stocks, essential facilities, transportation systems and utility systems. In 2005, we had collected several database described in details later. In utility systems, we collected database of pipelines and facilities provided by Taipei Gas Company and Taipei Water Company. In transportation systems, we collected and calibrated database of main roads and bridges which belong to national freeway, provincial highway and county-level highway. We also tried to establish the analyzable road network which is composed of bridges, elevated highways and normal road segments. Cooperated with other project to assess seismic vulnerability of elementary and junior-high school buildings, we collected and integrated the school building database in TELES. Based on the various kinds of collected database, it is possible to use TELES to simulate historical earthquake scenarios and to calibrate various kinds of parameters in analysis models.

Keyword: TELES, database of civil infra-structures, historical earthquake scenarios

Foreword

The National Center for Research Earthquake Engineering (NCREE) has developed the Taiwan Earthquake Loss Estimation System (TELES) for years. Because of continuous database collection and software development, the TELES become more and more sophisticate. Many government agencies and cooperative institutions have applied it in disaster mitigation practices. The applications of TELES divide into three categories: early seismic loss estimation, seismic disaster simulation and seismic risk assessment.

In application of early seismic loss estimation, it took about 10-30 minutes to obtain the simulated results two years ago. With the help of establishment of seismic scenario database, it now takes less than 30 seconds to obtain the results. The precision of the estimation results has also improved. For example, the smallest geographic unit to assess damages and casualties is changed from town to village, and the seismic source model is changed from point-source model to line-source model. The automatic outputs of ground motion intensity, building damages and casualties are useful for emergency response personnel in dispatching rescue forces and medical resources.

In application of seismic disaster simulation, the seismic source parameters can be defined through four different approaches, i.e. historical earthquake events, active faults, artificial events and user-supplied ground-motion intensity maps. Depending on the amount and completeness of inventory database, various kinds of estimates can be obtained to assist proposing disaster mitigation plans.

To establish seismic scenario database of Taiwan, the TELES has been modified to run seismic scenario simulations in batch mode. Inputting a series of earthquake source parameters, it can calculate simulation results one by one and store the results in a systematic way to facilitate data query and statistic. Combining the probabilistic seismic hazard analysis module, the seismic scenario database can be apply in seismic risk management.

The key factor in success of TELES depends on the collection of complete and useful database. Nonetheless, database collections are often the most

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time consuming and expensive aspects in performing a comprehensive study. In general, the inventory data are classified by their usage and functionality. For example, the civil infra-structures may be classified into four categories: general building stocks, essential facilities, transportation systems and utility systems. Each category is further divided into several classes according to their specific usage. On the other hand, the inventory data are also classified according to their structural types, seismic resistant capability, etc. to assess damage-state probability of individual object based on ground motion intensity and ground failure extent. The data classification schemes as well as the associated analysis models should depend on the content of inventory database.

The collected database in this project mainly includes utility systems, transportation systems and essential facilities. The utility systems include gas system, potable water system, electric power system, and so on. The only transportation system which is processed in this project is the highway system composed of road segments and bridges. The essential facilities include school buildings, hospitals, fire-fighting stations, police stations, government buildings, and so on. Based on the collected database and the calibrated analysis parameters, TELES is used to analyze several disastrous earthquake events. Investigating and comparing the analysis results, we may improve analysis models and associated parameters.

**Utility System**

In 2005, we processed the database of pipelines and facilities which were provided by Taipei Gas Company and Taipei Water Company. Moreover, the Taipei Gas Company provided the pipeline repair records in 921 Earthquake and 331 Earthquake. Base on these repair records we have studied the relationships between pipeline repair rate and strong ground motion parameters.

The data format of pipelines and facilities from Taipei Gas Company and Taipei Water Company are all following the format specified by the Maintenance Office of Public Works Department of Taipei City Government. The data format is called UIF (utility interchange format). As shown in Fig. 1, we wrote a program to translate the origin UIF data to geographic information system (GIS) and then input to TELES for analysis.

The distribution of gas pipelines (Fig. 2) and the distribution of water pipelines (Fig. 3) were integrated in TELES, which can estimate the number of repairs, repairing cost and restoration time under the scenario earthquake.

The locations of the repairs of gas pipelines were denoted by their address; and therefore we have to find out their coordinates by using digital map (Fig. 4). This project studied the empirical formula for
assessing repair rate of buried pipelines under severe earthquakes

Then we will develop a program to run network analysis and study the consequences of bridge damages after strong earthquakes. Such kinds of study may facilitate analysis of emergency rescue routes and prioritization of bridge retrofit.

**Transportation System**

In transportation systems, we mainly processed the database of main road segments and bridges in national freeway, provincial highway and county-level roads (Fig. 5). Based on the map from the Institute of Transportation, MOTC, we also established the relationship of road segments and bridges and constructed an analyzable road network in 2005.

![Fig. 5 Road distribution map in Taiwan.](image)

In addition, we have been doing some researches with Taiwan Area National Freeway Bureau and Directorate General of Highways and have collected seismic resistant capability parameter of bridges. The location of bridges and their spatial relation with the neighboring road network were checked by using base maps and related maps. We have checked 9,970 bridges in the IOT Traffic Road Network Digital Atlas. In this way, we will transform the original "point" objects into "line" objects to represent bridges (Fig. 6). The project will continue in 2006.

**Essential Facilities**

As far as essential facilities are concerned, we had collected database of elementary and junior-high school buildings in Taiwan. In 2005, the Ministry of Education commissioned NCRE to investigate the seismic resistant capability of elementary and junior-high school buildings. We integrated the database in TELES (Fig. 7). The database of school buildings includes coordinates, floor area, structural type, etc. The applications of school building database are to estimate the possible casualties in day time and to estimate the shelter capacities when strong earthquakes happen.

![Fig. 7 Distribution map of elementary school and junior high school.](image)
Conclusions

Taiwan Earthquake Loss Estimation System (TELES) can be applied in proposing local seismic disaster mitigation plans and act as a decision-making support system soon after occurrence of strong earthquakes. In the near future, TELES will also integrate probabilistic seismic hazard analysis and may have applications in proposing maximum probable earthquakes for each county and in proposing adequate seismic insurance policies. High precision earthquake loss estimation depends on correct database collection. Therefore, constructing a systematic, precise and up-to-date database is the goal of this project.
Seismic Assessment of the Electric Power System in Taiwan and its Applications

Gee-Yu Liu 1, Yi-Jen Wang 2 and Chih-Wen Liu 3

Abstract

Continuing past research work by the authors, more efforts have been made for investigating the seismic performance of the electric power system in Taiwan. In order to provide more informative support to the hazard mitigation and emergency management planning for utilities and local governments, the Taipower’s demand data have been reorganized into smaller administrative districts (i.e. cities/counties, major metropolitan areas, and science-based industrial parks) in Taiwan. The most threatening seismic source grids to the system’s power supply capacity have also been identified. The possible power outages caused by regional earthquakes from source zones of high seismic potential have also been estimated. Finally, an analysis module for the power flow calculation regarding the ‘instant impact’, in addition to the previous module regarding the ‘aftermath influence’, has been newly developed.

Keywords: electric power system, seismic performance, risk assessment

Introduction

Electric power systems serve as one of the most important infrastructures in modern societies. Through complicated interactions between lifeline systems, earthquake-induced power disruption will not only cause severe inconvenience to people and delay the recovery in the affected areas, but also cause huge direct and indirect losses in the industrial and business sectors. It is highly desirable to be capable of understanding and predicting the seismic performance of power systems, and then adequate measures can be taken to enhance their seismic preparedness.

In the previous studies by the authors on the seismic scenario simulation and risk assessment of electric power systems, the following items have been accomplished: (1) Clarify the effects of a damaging earthquake to a system’s performance; classify them into two phases, namely the ‘instant impact’ and ‘aftermath influence’; and identify the issues concerned for analyzing them; (2) Introduce earthquake scenarios of horizontal line-source type for a better prediction of ground motion intensities; (3) Enhance the computational efficiency dramatically while conducting the power flow analysis of a system under a huge amount of scenarios, so that the Monte Carlo simulation become feasible [Liu et al., 2005a and 2005b].

New efforts done in the past year as a part of this series of study could be summarized as: (1) Re-organizing the Taipower’s demand data according to cities/counties, major metropolitan areas or science-based industrial parks, so that the attained results can be refined to meet and needs from local hazard mitigation and emergency management plans; (2) Performing power outage simulation caused by regional earthquakes that might occur (20% probability of exceedance in the coming 20 years) from seven source zones of high seismic potential; (3) Developing a new analysis module for the power flow calculation regarding an ‘instant impact’ circumstance. Major results will be presented in the following sections.
### Analysis Procedure and Assumptions

The seismic risk assessment of the electric power system in Taiwan starts with a set of hazard-consistent scenario earthquakes [Liu et al., 2005a]. Its procedure, shown as a flow chart depicted in Fig.1, could be summarized as: (1) simulate the damage status of equipment (say, transformers) and remove from the inventory if damaged; (2) isolate the disconnected buses and lines of the system due to the removal of damaged equipment; (3) perform power flow analysis and attain the operable solution; (4) check and remove buses with abnormal voltage and lines with abnormal current; (5) remove loads that no longer connect to the system due to the removal of buses and lines in the previous step; (6) repeat Step (3) to Step (5) until all buses have a normal voltage and all lines have a normal current; (7) accumulate the power supply (i.e. remained loads) to the areas of interest; (8) calculate the ‘ratio of reduction in power supply’ (RRPS, between 0.0 and 1.0) of interested areas; (9) combine the attained RRPS value and the annual occurrence rate of each scenario earthquake to plot the corresponding seismic risk curves.

In Step (3), the power flow analysis of a damaged system may refer to wither the ‘instant impact’ or the ‘aftermath influence’. This will depend on whether the power generation is re-dispatched and line capacitance is adjusted (for the ‘aftermath influence’; these mimic the system operation for re-gaining the power supply) or not (for the ‘instant impact’). This is the major difference from the one proposed by Shinozuka et al. [2004]. Step (4) refers to the activation of the system’s relay protection for isolating these buses and lines to prevent the power equipment from any electrical damage. As a result, an iterative process of Step (3) to Step (5) is required for deciding the remained capacity of the damaged system.

In this study, it was assumed that the 345/161 and 161/69kV transformers are, among other substation equipment in the system, the only vulnerable equipment. The two fragility curves developed by UWG for the minimal failure mode (one main porcelain gasket leak) of transformers categorized as TR2 (3-phase 230kV) and TR4 (3-phase 500kV) [Anagons, 1999] were further assumed applicable to the 161/69 and 345/161 kV transformers, respectively.

### The aftermath Influence

Fig.2 depicts the seismic risk curves for the power system in Taiwan regarding the ‘aftermath influence’. The abscissa and ordinate refer to the RRPS value and the annual probability of exceedance. The risk the system is exposed to, in terms of the ‘ratio of reduction in power supply’, is specified by curves of mean RRPS value and ±1.0 and ±2.0 times of standard deviation based on a 20-case Monte Carlo simulation for each earthquake scenario. Therefore, given a RRPS value, one can easily read the chance per year that the RRPS lower than or equal to the specified value will occur. Similarly, the risk regarding any selected area, e.g. each service area (north, middle, south and east of Taiwan; Taipower classification), city/county, metropolitan area, or science-based industrial park, has been assessed in the same way.

In order to identify the most threatening source grids (see [Liu et al., 2005b]) to the power system, an impact factor \( IF_i \) for Grid \( i \) could be defined as

\[
IF_i = \sum_{j=1}^{M_i} RRPS_j \cdot r_j
\]

where \( M_i \) is the number of earthquake scenarios associated with Grid \( i \), and \( r_j \) and \( RRPS_j \) refer to the
annual occurrence rate and the RRPS value associated with the $j$-th earthquake scenario, respectively. How threatening each source grid is then can be decided by the ranking of its impact factor.

Fig. 3 depicts the 10 most threatening source grids to the system’s post-earthquake performance (aftermath influence), with the pink grid having the highest impact factor. The figure is overlapped with the spatial distribution of the system’s major facilities, such as power generating plant, major transmission lines and substations. Usually, the locations of most threatening source grids are close to areas of higher seismicity as well as concentrated power facilities.

Consequence of High Potential Regional Earthquakes

Bases on the method proposed by Wen and co-workers [Wen et al., 2005], Dr. Jean of NCREE has recently estimated major regional earthquakes that might take place in the near future in Taiwan. Fig. 4 depicts the attained 7 source zones. Each is associated with an earthquake with magnitude $M_l$ that may occur with a probability greater than 20% in the coming 20 years. Regarding the source zone ZS07, Fig. 5 depicts the simulated PPRS values (aftermath influence) in each city and county in Taiwan when such an earthquake happens. This result is based on the earthquake scenario with M7.04 and depth 5.25 Km. It is estimated that Yun-lin, Cha-Yi and Tainan will lose about 60, 90 and 30% of power supply, respectively.

The Instant Impact

A new module for the power flow calculation regarding an ‘instant impact’ circumstance of power
systems has been developed and employed to the post-earthquake performance analysis of the system in Taiwan. The corresponding seismic risk curves for the whole system and for each service area are depicted in Fig. 6 and Fig. 7, respectively. For the shake for comparison, the curves for ‘aftermath influence’ are depicted, too. The ‘instant impact’ is theoretically a more critical situation because there is no time then for the system operators to re-dispatch power generation or adjust line capacitance. The remained capacity of a damaged power system regarding the ‘instant impact’ is usually lower than the one for the ‘aftermath influence’. A blackout event is more easily to be attained for the ‘instant impact’, too. This fact naturally leads to the observation from Fig. 6 and Fig. 7 that the risk curves regarding the ‘instant impact’ (blue lines) is higher than the one regarding the ‘aftermath influence’ (red lines).

The power outages caused by devastating regional earthquakes of highest seismic potential have also been estimated. These can provide more informative support to the hazard mitigation and emergency management planning for utilities and local governments. For example, the seismic preparedness of a power system can be improved if the spatial correlation between the threats and the exposures is well considered. Finally, a new analysis module has been developed to deal with the power flow calculation regarding the ‘instant impact’. As a consequence, how and where sudden power disruption will occur immediately after an earthquake event takes place can be simulated, which is very important to critical facilities like communication and medical centers, as well as to power-sensitive industries.

Acknowledgement

The funding from the National Science Council, Taiwan under Grant No. NSC-93-2625-Z-492-003 is gratefully acknowledged. This research is made possible through kind help from the System Operation Department, Taipower for offering the power system data essential to this study.

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An Overview of Recent Studies on the Seismic Performance of Substation Electrical Equipment

Gee-Yu Liu

Abstract

Recent earthquake experiences suggest that one of the most critical failure modes of power systems is the damage in substation electrical equipment. The significance of the IEEE-693-1997 Standard, which has become the basis of experimental study of substation equipment in the U.S., is highlighted in this report. An overview of some of the recent achievements in the seismic performance studies related to substation equipment issues is also provided, which include the behavior of bushings, transformers, disconnect switches, and the characteristics of overturning as well as dynamic interaction between interconnected equipment.

Keywords: substation electrical equipment, seismic performance, bushings, transformers, disconnect switches, overturning, dynamic interaction

Introduction

Ever since the occurrence of 1971 San Fernando, California earthquake, electrical power systems were given serious consideration in their seismic safety issues. The following earthquake experiences indicate that the damage in substation equipment is the most critical cause of post-earthquake power disruption. However, it is until recent years that the understanding of seismic behavior of substation equipment gets significantly improved. This is thanks to the release of the IEEE-693-1997 Standard as well as various research efforts made by the PEER and MCEER investigators. This report aims at presenting an overview of recent achievements related to substation equipment, which include the seismic performance of bushings, transformers, disconnect switches, and the overturning of as well as dynamic interaction between equipment.

IEEE-693-1997 Standard

Currently, the IEEE-693-1997 (IEEE Standard 693-1997 Recommended Practice for Seismic Design of Substations, [IEEE, 1998]) serves in the U.S. as the most important reference for the seismic design, qualification and fragility testing of substation electrical equipment. It provides description and procedure for the seismic criteria, qualification methods and levels, structural capacities, performance requirements for equipment operation, and installation methods.

Three Seismic Performance Levels, i.e. High, Moderate, and Low, are defined by the IEEE-693-1997. Equipment that is shown to perform acceptably in ground shaking consistent with the High Seismic Performance Level (see Fig.1) is said to be seismically qualified to the High Level. The rest may be deduced in the same way. Qualification Level is decided by the site-specific PGA value calculated using a 2% probability of exceedance in 50 years. A site is classified as Low if the PGA value is less than 0.1g, Moderate if between 0.1g and 0.5g, and High if greater than 0.5g.

To compare with its predecessor IEEE-693-1984, the current version makes a great improvement. For example, it becomes equipment specific, meaning that each type of equipment is provided with its own uniquely designed set of requirements. The IEEE-693-1997 identifies 11 experimental methods for the seismic qualification of equipment, including the sophisticated shaking table testing using 3-D earthquake ground motion records. It has become the basis of experimental study of substation equipment in the U.S., from which it is expected that the future version of IEEE-693 Standard will be revised to incorporate some of the latest research findings.

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Research Framework

California Energy Commission has granted a program called PIER (Public Interest Energy Research Program). An annual budget of more than US$ 60 millions has been approved to this Program. It aims at promoting research work that can further enhance the security and reliability of energy services and products in California. Regarding electricity, the issues mainly focused upon its seismic safety.

Under the framework of PIER, research in the U.S. related to the seismic performance of electric power systems and substation equipment has become very vital and highly integrated. Many professors and researchers from the PEER and MCEER Centers have participated in this collaborative research work, which can be summarized as Fig.2 [Chambers, 2005]. Major research topics include seismic hazard assessment, equipment fragility and system performance modeling, repair modeling, direct and indirect economic loss estimation, and so forth.

Performance of Substation Equipment

A database has been developed by Anagons that documents the seismic performance of substation equipment own by California utilities in past 12 earthquakes between 1971 and 1994 [Anagons, 1999]. The data relate to equipment operating at 220/230kV and 550kV, and are organized into 68 data fields including earthquake location, ground motion, site location and conditions, equipment characteristics, performance of equipment, failure mode, and restoration time. In the meanwhile, the classification, failure modes and opinion-based fragility curves for equipment of 220kV and higher developed by UWG (Utilities Working Group, a group of experts from several California utilities, formed in 1993) are also summarized. The merits of this study lies in the success in joining the power utility professionals and earthquake engineers to establish quality database. It can serve as a basis for further developing or improving substation equipment

Fig. 2 Research framework of PEER and MCEER for the studies related to the seismic performance of electric power systems and substation equipment [Chambers, 2005]
vulnerability functions.

Porcelain bushings and insulators are used as parts in substation equipment essential for high-voltage electrical insulation. They are well understood to be very slender, weighty, flexible and, as a result, very fragile seismically. Gilani et al. have performed a series of shaking table testing for studying and qualifying the seismic performance of transformer bushings, which include two 196kV, two 230kV and three 550kV ones [Gilani et al., 1998-99; Whittaker, 2004]. Most bushings survived severe earthquake shaking and were qualified to the High Level, which contradicts the damage experience from past earthquakes. This calls into question the utility of the IEEE-693-1997 procedures used for qualification of substation equipment. The PGA value was found a poor descriptor of damage. New methods and ground motion parameters seem needed for characterizing the fragility of transformer bushings.

196kV bushing [Gilani et al., 1998]

230kV disconnect switch [Gilani et al., 2000]

550kV bushing [Gilani et al., 1999]

525kV one-phase transformer-bushing system [Filiatrault, 2005]

Photos Various shaking table testing of substation equipment

Disconnect switches outnumber else equipment in substations. They are used to open and close circuits, so that, after the protective device has de-energized the circuits, a section of a circuit can be isolated for repair or routine maintenance. Following the testing of transformer bushings, Gilani et al. started working on the disconnect switches [Gilani et al., 2000]. Two horizontal-break and three vertical-break switches were shaking table tested and seismically evaluated. The mathematical model for switch posts was developed, too. In this study, the cast aluminum spacers at the base of the switch posts were found most vulnerable. The IEEE-693-1997 requirement for qualifying disconnect switches was found unrealistic, and, accordingly, an alternate procedure has otherwise been proposed.

Transformers serve as the pivotal equipment to the functionality of a substation. They are no way to bypass, and will be costly and time-consuming to repair if damaged. Filiatrault et al. have performed shaking table testing of a 1-phase 525kV transformer-bushing system [Filiatrault, 2005]. Major findings of the study include: The dynamic characteristics of the bushing are greatly influenced by the flexibility of the top plate of the transformer tank. The tested amplification values (between the input motion at the base of transformer and the motion recorded at the base of the bushing) can easily exceed the one given by IEEE-693-1997. The seismic response analysis of a transformer should take into account its dynamic interaction with the foundation. On the other hand, there has been unexpected loss of transformers attributed to their internal damage. For this reason, Saadeghvaziri et al. have examined the internal packaging of transformers and identified possible dynamic response characteristics and modes of failure (such as sliding of key spacers, loss of close fitting tolerances between limbs and yokes, and flexural and rocking of core-frame system) [Saadeghvaziri et al., 2004].

Overturning and dynamic interaction both comprise a major fraction of causes of damage in substation equipment when an earthquake strikes. For the issue of overturning, Makris et al. have conducted a series of analytical studies for investigating the rocking response of substation equipment items [Makris et al., 1998-2001]. Key factors that control overturning, such as the effect of vertical excitations, anchorage, and the coefficient of restitution during impact have all been addressed. It was reported that while for the most of the frequency range, anchored blocks survive higher accelerations than free-standing blocks, there is a short frequency range where the opposite happens. The commonly used design approach developed in late 70’s for slender structures is found inherently flawed and should be abandoned. The study concludes that the exact rocking spectrum emerges as a distinct, irreplaceable indicator of the shaking potential of ground motions and should be adopted as a valuable analysis and design tool.

While regarding the dynamic interaction between interconnected substation equipment, Der Kiureghian et al. have analytically assessed the effect of interaction in interconnected items and developed design guidelines for reducing the adverse nature of this effect [Der Kiureghian et al., 1999-2000]. In these studies, each equipment item was modeled as a system with distributed mass and stiffness properties and, through the use of a displacement shape function, was characterized by a single degree of freedom. Four types of connection were investigated, namely linear...
rigid buses, nonlinear flexible strap connectors, and extensible cable connectors with and without flexural rigidity. Parametric investigations reveal that the interaction tends to amplify the response of the higher frequency equipment and de-amplify the response of the lower frequency equipment relative to their respective stand-alone responses. A recommendation for the minimum cable length to avoid the adverse effect of interaction is formulated.

Conclusions

An integrated research approach in the U.S. for improved understanding of the seismic performance of substation electrical equipment has been witnessed. The collaboration among government, utilities and research institutes under well coordinated research framework has achieved new findings about the seismic characteristics of substation equipment including bushings, transformers, disconnect switches, and the overturning of as well as dynamic interaction between equipment. New seismic design guidelines will no doubt benefit from the feedback of these findings. Many of the findings are very informative, too, from the viewpoint of performance-based design conception.

There are currently 20 extra-high voltage substations (345/161kV) and 42 primary substations (161/69kV) in Taiwan. The former and the later have a number of 16 and 33 each belonging to the open-air type, respectively. They inevitably share the same seismic safety issues as those in the U.S. (almost all being open-air). In addition, the security issue of transmission towers is very critical and unique in Taiwan. In 1999, the collapse of 345kV transmission towers caused island-wide blackouts twice, including the halt of power supply to around 6.8 million of end-users for several days during the Chi-Chi earthquake event. It is a must to follow up the latest know-how to help monitoring and retrofitting critical power facilities in Taiwan.

References


A Study of Post-tensioned Seismic Structural Systems

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Abstract

Post-tensioned structural systems combine the properties of pretressed strands and anticipated energy dissipating devices. It is a key feature that the high-strength strands enable the structural systems to self-center after earthquakes. Added energy dissipating devices in the beam-to-column joint can produce hysteretic energy dissipating effects in the structures. Therefore, utilizing the properties of self-centering and hysteretic energy dissipation, it could result in reduced transient and residual displacements during and after an earthquake. This structural system has excellent ability of deformation, but it inherently possesses lower hysteretic dissipating energy. This characteristic probably results in more significant vibration than traditional structural system during shake happens. Also, when a structure is subjected to lateral force, openings between beams and columns are be created and elongate the structure’s dimension. The openings not only potentially destroy the compatibility of deformation but also decrease service function for traditional RC slab. Based on those reasons the research project attempts to propose feasible energy dissipating devices and solutions for construction and design problems of RC slab.

This year, which is the second stage, some issues are focused on developing of the energy dissipating devices in the post-tensioned beam-to-column joints based on research results of the first stage. The devices using in post-tensioned RC, steel CFT and RCS moment resisting frames are included. In addition to the experimental investigations, analytical approaches were employed to grasp the earthquake responses of the structural components and systems. Experimental results confirm the post-tensioned beam-to-column joints have probability of re-centing and large space of progressing hysteretic energy dissipating.

Keywords: post-tensioned building systems, self-centering, energy-dissipating device

Introduction

The traditional design philosophy of building structures is that the beam ends or dissipating devices are expected to provide plastic deformation to dissipate energy from earthquake. It is rarely possible to eliminate residual deformations for traditional non-prestressed structures suffering moderate or severe earthquake. The post-tensioned seismic building systems have the advantage of self-centering after the effects of earthquake ground accelerations and thereby eliminate permanent drift and reduce the possibility of being demolished after an earthquake. Besides, all the components can be precasted in advance so that the cost of construction can be reduced. In recent years, many researches on the post-tensioned building systems have been launched in the US and Japan. Fig.1 show the thirty-nine-story post-tensioned RC building was constructed in San Francisco in 2001. It showed that the post-tensioned building systems had good potential in development and practice. These years, many researches on RC beam-to-RC column joints, steel beam-to-CFT column joints, and steel beam-to-RC column joints were conducted in the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. With all these researches, the integrated research on the post-tensioned building systems was launched and was focused on the behavior of different kinds of

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post-tensioned composite joints. Finally, the analytical approaches were employed to simulate the earthquake responses of the structural components and systems.

**Background and Purposes of Research**

This is a 4-year program beginning in 2004. Typically, the beams contribute to the initial stiffness of a post-tensioned beam-to-column joint under minor earthquake loads. When it is under severe earthquakes, there will be a gap between the end of the beam and the column face. The joint stiffness comes mostly from the post-tensioned tendons. The joint can eliminate permanent drift by self-centering effects induced by the prestressing force (PT force). Although there is only little residual deformation in this building system, the input energy due to the earthquake is hard to dissipate. Adopting energy-dissipating devices is one of the most common methods to dissipate the energy input by the earthquakes. In this research, we proposed several kinds of dampers on the joint to increase its damping ratio. Moreover, good hysteresis loops were expected due to the plastic deformation of the dampers.

At the first stage, different kinds of materials, including the post-tensioned steel beam-to-CFT column joints, the post-tensioned RC beam-to-RC column joints, and the post-tensioned steel beam-to-RC column joints were constructed in order to investigate the behavior of the post-tensioned building systems. Besides, as mentioned above, in order to improve the insufficient energy dissipation of the post-tensioned building systems, varieties of energy-dissipating devices were used in the specimens. It was expected that through this integrated project, the related research on this new structural system could be coordinated efficiently.

![Fig. 1 The thirty-nine-story post-tensioned RC building in San Francisco (Englekirk 2002)](image)

**The Behavior of Post-tensioned Joints**

The behavior of the post-tensioned joints is the superposition of the responses of the tendons and the energy-dissipating devices. Fig. 2 is the typical behavior of a post-tensioned steel beam-to-steel column joint. Fig. 2(a) shows the cyclic response of the post-tensioned bars, and (b) shows the cyclic response of the energy-dissipating devices. The behavior of a post-tensioned joint with energy-dissipating devices as shown in (c) is the superposition of (a) and (b). From the superposition, the total moment \( M_{PTED} \) can be obtained and expressed as follows:

\[
M_{PTED} = M_{PT} + M_{ED}
\]  

in which \( M_{PT} \) and \( M_{ED} \) are the moment contributed by the PT force and energy-dissipating devices respectively. The PT force controls the magnitude of the bending moment \( M_J \) beyond which a gap opening occurs between the beam end and the column face. The nonlinear elastic behavior continues until a bending moment \( M_J \) is reached. The moment \( M_J \) is given by:

\[
M_J = M_J + (k_1 + k_2)\theta_B
\]  

where \( k_1 \) = elastic rotational stiffness provided by the PT forces after a gap opening occurs; \( k_2 \) = elastic rotational stiffness provided by the energy-dissipating devices; and \( \theta_B \) = rotation of the joint when the energy-dissipating devices begin to yield.

The joint stiffness between \( \theta_B \) and \( \theta_C \) is the linear superposition of \( k_2 \) and \( k_3 \). When the bending direction is reversed, the joint is still nonlinear elastic, and the rotation of the joint \( \theta_D \) is given by:

\[
\theta_D = \theta_C - 2\theta_B
\]  

To ensure the full self-centering of the post-tensioned joint, the following rule should be met:

\[
M_J \geq (k_2 - k_3)\theta_B
\]  

A post-tensioned building system with appropriate design, should not only have limited residual deformation but dissipate energy efficiently. Further and systematical researches are required before the full implementation of this novel, economical and practical building system in Taiwan.

(a)
**Experimental Setup and Results**

The old and new test frames shown in Fig. 3 and Fig. 4 are conventional experimental setups for beam-to-column connections. The old test frame is that vertical actuators connecting to the end of the beams are displacement controlled. A swivel at the end of each actuator is simulated as a hinge. The lateral actuator connecting to the top of the column holds its position as zero displacement when the test is running. A wide flange section fixture located at the bottom of the specimen column is used to grasp specimen column’s bottom end in the old test frame.

For new test frame, there’s a hinge device to be designed and to be placed at the bottom of the specimen column. Displacement control mode of mastering the vertical actuators are adopted in the two testing frame system in this project. The loading protocol following the requirement in AISC is used in the research and shown in Fig. 5.

Specimen, PTT4 and PTH4, tested in the conventional test frame, was a steel beam-to-CFT column joint with triangular and H shape steel plates to dissipate energy on both the top and bottom of the
beams. When the drift ratio of the joint reached 0.5% radian, there was a gap between the beam end and the column face. When the drift ratio came to 0.75% radian, the steel plates started yielding. When the drift ratio reached 1.0% radian, the steel plates severely yield and the gap between the beam end and the column face reached 10mm. The test was stopped with the drift ratio of 6% radian, and the beam flange and web yielded slightly. The test result is shown in Fig. 6 and Fig. 7.

Specimen S2-3, tested in the new test frame, was a steel beam-to-CFT column joint with steel bars welded to inside surface of top and bottom beam flanges to be as energy dissipating devices. When the drift ratio came to 0.5% radian, there was a gap opening between the beam end and the column face to be occurred. The energy dissipating steel bars yielded as the drift ratio of 1.5% radian reached. When the drift ratio came to 2.0% radian, the east beam flange near the end of stiffened rib plate appeared serious buckling. When the drift ratio reached 4.0% radian, the buckling occurred in the east beam got worse. It caused some residual deformation of specimen not to re-center. In addition, the beam end force resulted in significant decay. The test was stopped. The test result is shown in Fig. 8. The test results show steel bars could provide energy dissipating, but unexpected seriously buckling in the beam occurs. It is obvious that the details of the energy dissipating device should be progressed.

Summary
The test results show that post-tensioned beam-to-column connections have expected self-centering ability. And it could produce better behavior of hysteretic energy as adding appropriate energy dissipating devices.

Based on the experimental results of the two tests, the advantages of the proposed building systems were confirmed: small residual deformation and efficient energy dissipation. There are many unfinished tests in second stage of this project. And besides post-tensioned beam-to-column tests, it will add post-tensioned frame system tests this year. After the whole series of testing, it is expected that the behavior of the post-tensioned joints with energy dissipating devices will be understood more clearly. The primary objective of the project is to implement the post-tensioned seismic structural systems in real practice in Taiwan buildings.

References
The Analysis, Design, and Development of Rolling Type Isolated Bearing
K.C. Chang 1, J.S. Hwang 2, S.Y. Wu 3, S.N. Lee 4

Abstract
This research is concerned with the experimental study on seismic behavior of bridges with rolling type seismic isolation bearings. Unlike base isolation of buildings, bridge columns play an important role on the seismic behavior of the bridges. The columns not only provide the vertical support, but also contribute lateral stiffness and inertia force under earthquake ground motions. The effective period of the isolated bridge becomes longer and effective damping ratio becomes smaller due to the stiffness of the columns. A 1/7.5 scaled bridge model is designed and constructed in this study. A series of experiments are to be carried out at NCREE to verify various types of bearings. This paper summarized the design of the bridge model and some preliminary test results of the shaking table study with Rolling Type Seismic Isolation Bearings [RTB].

Keywords: Isolated bearing, Seismic isolation, Rolling type isolation bearing, Seismic isolation bearing.

Introduction
Extending the nature period of structures is the basic concept of base isolation system. According to the response spectrum of recorded earthquake ground motions, the most ground motion input energy of an earthquake concentrates on short-term and midterm periods of the corresponding spectrum. Increasing the effective period of structures, the maximum spectral acceleration of an earthquake could be significantly decreased. However, the maximum displacement response will be increased with the increasing structural periods as shown in Fig.1. Therefore, additional energy dissipation system could provide extra damping to reduce the displacement response of an isolated system under seismic excitation.

In general, it is difficult to simulate the behavior of seismic isolated bridges under earthquake ground motions by considering only single degree of freedom analysis. The relative spatial distribution of stiffness and mass of the bridges are quite different from those of buildings even though the longer vibration period is the main dynamic characteristic for both isolated bridges and buildings.

Comparing to buildings, most of the inertia mass and main lateral stiffness of a bridge is provided by the upper deck and the bridge columns, respectively. Furthermore, the effective damping of the isolated bridges will be degraded due to the stiffness of bridge columns [Hwang et al. 1997]. In this study, a 1/7.5 scaled bridge model is designed and constructed with the consideration the bridge column. A series of experimental study will be carried out to verify various types of bearings. This paper summarized the design of the model bridge and the preliminary shaking table results study of the model bridge with rolling type bearing.
Basic Dynamic Characteristic of Rolling Type Bearings

Fundamental mechanism of a rolling type bearing may be demonstrated by the figure 1. In the figure, a roller with radius ‘r’ is located on a slope surface with an inclined angle of $\theta$. The roller is sustained to the gravity load from the superstructure on it and may roll along the slope surface. Since rolling resistance is always smaller than sliding resistance on a same surface, the horizontal force transmitted to the substructure of a rolling type bearing would be smaller than that of a sliding one. Restoring force of the rolling type bearing is provided by the parallel component of the gravity load to the inclined surface.

Fig. 1 Fundamental mechanism of rolling type bearing

External forces acting on the roller are shown in figure if the roller is rolling down the slope surface. In the figure, N is the normal force acting on the surface and the rolling friction f is equal to $\mu*N$, where $\mu$ is the coefficient of rolling friction. ‘x’ indicates the horizontal displacement of roller. From the equilibrium of horizontal forces, the equation of motion for the rolling bearing is written as

$$m x = -mg \sin \theta \cos \theta (1 - \mu \cot \theta)$$

(1)

Also, it can be proved that the rolling bearing would have an effective natural frequency as [Ref. 1]

$$f_x = \frac{1}{8} \sqrt{\frac{2g \sin \theta \cos \theta (1 - \mu \cot \theta)}{x}}$$

(2)

From the above formula, it is seen that the effective natural frequency of the rolling bearing is primarily dependent on the inclined angle and its displacement response for a constant coefficient of rolling friction. Also, resonant frequency of the rolling bearing does not exist due to the displacement dependency. The effective natural frequency of the bearing would increase with its displacement response. This means that the rolling bearing has larger effective stiffness under frequent small excitations to restrict the bearing displacement. As the displacement response increases, the effective stiffness of the bearing would decrease to activate its seismic isolation mechanism.

Design Concept of Bridge Model

The geometrical and physical properties of a scaled model must resemble the ones of prototype. From a structural engineering perspective, the geometrical and physical quantities such as the moment of inertia, acceleration, frequency etc. of the model can be well determined as the length scale between the prototype and model is decided based on the dimensional analysis. The design flow chart used in this study is shown in figure 2. One can see that additional masses are necessary because of the unchangeable material density.

The prototype of this scaled bridge model is primarily based on the standard drawings published by Directorate General of Highways (DGH) in Taiwan. It is a multi-span PCI bridge with simple support. The compressive strength of concrete is 210 kgf/cm$^2$ and the steel material tensile yielding strength is 4200 kgf/cm$^2$. The width of deck is nine meters and the span of prototype is thirty meters. The height of columns including cap beam is eleven meters. Based on the scaling factor of 7.5, the width of deck of model is 1.75 m and the clear span of prototype is 4.4 m. The height of columns including cap beam is 1.47 m. The configuration of the bridge model is shown in figure 3.

In order to avoid cracking in the RC columns, the RC columns are replaced with CFT columns based on the theory of effective cross section. Therefore, the strain on the bottom of CFT columns must be monitored to keep them from yielding. In this study, five real earthquake time history records are used, including EL Centro, KOBE, the TCU068 and TCU129 of Chi-Chi earthquake, and the Northridge earthquake. Every earthquake time history record was also scaled down to satisfy the relationship between prototype and model.

![Fig. 2 Design flow chart of the shaking table test](image)
Preliminary Test Results

Since the dominant frequency of the isolation system with rolling type bearings depends on the relative displacement of bearings, the resonance frequency of this isolation system doesn’t exist. The larger the relative displacement of bearings, the higher the system period would be. As can be seen in Figure 4 and 10, the acceleration response of the deck is limited within 100 gal because the transmissible force is controlled by the slope of Rolling Type Bearings. It should be noted that the acceleration on the top of cap beams was amplified due to the contribution of columns stiffness as shown in Figure 5 and 11. And in this study, the relationship between the relative displacement of Rolling Type Bearings and the input ground motion PGA value was approximating linear. This linear relationship also existed between the relative displacement of Rolling Type Bearings and the dominant frequency of system.
Conclusions

The acceleration response of deck under seismic excitation may be well controlled and limited within 100 gal by the RTB. The acceleration response of cap beam is amplified due to the contribution of column stiffness and mass. Therefore, it might be important to take columns contribution into consideration for seismic isolation. The relationship between the relative displacement of RTB and the input ground motion PGA value is approximating linear. This linear relationship also exists between the relative displacement of Rolling Type Bearings and the dominant frequency of system.

References


Earthquake-resistant Behavior of Pre-stressed Steel Beams and CFT Column Connections with X-shaped Dampers

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ABSTRACT

The structural system of post-tensioned pre-stressed steel beams with concrete filled tubular (CFT) columns has remarkable performance in strength, ductility, load-bearing capacity and construction duration. After the structure is experienced an earthquake, the beams and columns can return to their original location. However, behavior of beam-to-column connection is so complicated that it is not widely applied. For the beam-to-column connections of this structural system, a detailed design method was proposed in this research. This research established a mechanics model of the beam-to-column connection and performed a series of structural earthquake-resistant experiments. The research results proved that the beam-to-column connection has remarkable earthquake-resistant performance in strength, ductility, energy dissipation and self-centering ability. The structural system remained standing even when story drift reached 6% and plastic angular displacement of the beam-to-column connection reached 5%. It demonstrates that the earthquake-resistant performance of this beam-to-column connection exceeds the seismic code requirements for earthquake-resistant connections in Taiwan and U.S.

Keywords: pre-stressed structural system, self-centering ability, concrete filled tubular column

Introduction

The ductility of moment-resistant structure usually depends on the strength of beam-to-column connections. Traditional beam-to-column connection is fully penetrating welded using a backing bar on jobsite. The voids between backing bar and column flange may cause stress concentration when beam end is loaded. The crack can even grow into column and tear the welding zone apart. It is the most common steel structure failure mode in North Ridge earthquake, 1994. Because of the weak welding zone, the beam-to-column connections had failed before the strength and ductility of beams and columns were fully developed. The traditional beam-to-column connection must be improved.

In recent years, the beam-to-column connection design is improved to avoid failure mentioned ahead. Some connection types have good stiffness, strength and ductility, however the structure still have residual displacements after an earthquake.

To make structural system capable for self-centering, post-tensioned pre-stressed structural system was developed in recent years. The concept of post-tensioned pre-stressed structural system was initially adopted for pre-cast concrete structure (Priestley 1993). Its mechanics behavior was proved in the U.S. research program “Pre-cast earthquake-resistant structural system” (PRESSS, Precast Seismic Systems). The program, lasted for 7 years from 1993 to 1999, was an integrated research program including numerical analysis,
beam-to-column connection sub structure experiments, and 5-story pre-cast concrete structure simulating dynamic experiments, etc. The results showed that the post-tensioned pre-stressed structural system had the ability to reduce residual displacement.

In 2001, Ricles introduced post-tensioned pre-stressed structural system concept into steel structure (Ricles 2002). He attempted taking angle steel as an energy dissipation element and performed a series of experiments for beam-to-column connection. The results showed that post-tensioned pre-stressed structural system enhanced self-centering ability of the structural system better when applied to steel structure rather than concrete structure. In 2002, steel rods are used as energy dissipation elements and performed a series of cyclic loading experiments (Christopoulos 2002). The results also showed that the structural system had good self-centering ability.

This study proposes a detailed design method of post-tensioned pre-stressed steel beams and CFT column connection with an X-shaped damper as an energy dissipation element. A theoretical analysis method was developed and the feasibility of this earthquake-resistant connection was proved by the resulted strength, ductility, energy dissipation and self-centering ability from a series of cyclic loading experiments.

THEORETICAL ANALYSIS

The loading direction of X-shaped damper was perpendicular to steel plate surface. The height of steel plate was much greater than its thickness. Since the cross-section moment is proportional to the perpendicular distance to the X shape center, the stress \((\sigma = M/S)\) of each cross-section caused by moment is equal. When loading increased to a certain value, i.e. became plastic stage from elastic stage, the X-shaped damper would yield. Therefore, X-shaped damper had extreme efficient deformation and energy dissipation ability.

When X-shaped damper is loading, the center is the inflection point and can be taken as a hinge. The relationship of lateral loading \((V)\) and the lateral displacement \((\Delta)\) can be obtained as follow:

\[
\Delta = \frac{1}{2} \int M(y) \times m(y) \, dy = \frac{1}{E_s} \int \frac{V y^2}{12} \, dy = \frac{5hV}{6E_s h^2} \tag{1}
\]

where \(E_s\) is Young's Modulus, \(h\), \(b\) and \(t\) are the height, width and thickness of X-shaped damper. \(M(y)\) is the moment induced by lateral loading \(V\), \(m(y)\) is the moment induced by unit lateral loading, \(l(y)\) is the section moment of initial.

Thus, the lateral stiffness of X-shaped damper \((k_{dh})\) is:

\[
k_{dh} = \frac{2E_s bt^3}{3h^3} \tag{2}
\]

The behavior of beam-to-column connections is divided into 3 stages:

The first stage is before beam-to-column gap opening: The perpendicular displacement of beam midpoint is contributed by beam, column, and panel zone. Thus, the stiffness in perpendicular direction \((k_1)\) of beam-to-column connection at beam midpoint is:

\[
k_1 = \frac{k_c k_k k_{pc}}{k_c k_k + k_c k_h + k_h k_{pc}} \tag{3}
\]

where \(k_c = 3EI/L^3\) is the perpendicular direction stiffness contributed by beam, \(k_c\) is the perpendicular direction stiffness contributed by column. The moment of concrete at column edge reaches its maximum strain 0.003 is treated as its yielding moment. From the surface stress and moment of the column, the EI of column can be solved. \(k_{pc}\) is the perpendicular direction stiffness contributed by panel zone. When the beam-to-column gap is open, the displacement at beam midpoint is \(F_1 = k_1 \Delta_1\).

The second stage is from beam-to-column gap opening to X-shaped damper yielding: The X-shaped damper and tendon are affected by the same opening angular displacement that causes perpendicular direction displacement at beam midpoint. Therefore, the stiffness of X-shaped damper and tendon are connected in parallel. For the perpendicular direction stiffness \((k_2)\) of beam-to-column connection at beam midpoint, it is first connecting in parallel the stiffness of X-shaped damper before yielding with the stiffness of tendon. Then, it is connected in series to the stiffness at beam, column and panel zone. It is solved in the following:

\[
k_2 = \frac{k_k k_k (h_1 + k_{pc})}{k_k k_k (h_1 + k_{pc}) + k_h k_h (h_1 + k_{pc}) + k_h k_{pc} (h_1 + k_{pc}) + k_h k_{pc}} \tag{4}
\]

where \(k_1\) is the stiffness in perpendicular direction contributed by tendon, \(k_2 = (d^2/I_s^2) \times k_{dh}\) is the stiffness in perpendicular direction contributed by X-shaped damper. The displacement at beam midpoint is \(\Delta_2\) when X-shaped damper yields. The shear force at beam midpoint is \(F_2 = k_1 \Delta_1 + k_2 (\Delta_2 - \Delta_1)\).

The third stage is after X-shaped damper yields: Because X-shaped damper has no stiffness after yielding, \(k_{dy} = 0\) at this time. From eq. (4) the
beam-to-column connection perpendicular direction stiffness ($k_3$) at beam midpoint in this stage is:

$$k_3 = \frac{k_f k_m k_p k_s}{k_f k_m k_s + k_f k_s k_p + k_s k_p k_x + k_s k_x k_f}$$  \hspace{1cm} (5)

From the information mentioned above, the shear force-displacement relationship of beam-to-column connection at the beam midpoint can be obtained as Fig. 1.

![Fig. 1. Relationship between force and displacement at beam midpoint](image)

**EXPERIMENTAL SETUP**

Schematic diagram of the connection is showed as Fig. 2 and the testing frame is shown in Fig. 3. The column is 400 × 400 mm square cross section steel tube having thickness of 10 mm. The rectangular steel tube is made of A572 (Grade 50) steel and filled with concrete. The beams of specimen are made of H-shaped A36 steel with cross section of H500 x 200 x 10 x 16 mm. The end of each beam is welded to an end plate, which is extended with two X-shaped dampers above the top beam flange and below the bottom beam flange, respectively. Four holes are drilled in the end plate and four compatible pipe sleeves are also imbedded in the concrete-fill steel tubes so that 16 steel strands with post-tension can pass through the holes and pipe sleeves and mount at the midpoints of the pair of the beams. There are 4 sets of testing specimens and the design parameters are listed in Table 1.

![Fig. 2. Schematic diagram of connection](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Dimension of X-shaped damper</th>
<th>Initial post tension force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>thickness (mm)</td>
<td>width (mm)</td>
</tr>
<tr>
<td>PBCCb30h20</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>PBCCb30h15</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>PBCCb20h15</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>PBCCb20h20</td>
<td>25</td>
<td>20</td>
</tr>
</tbody>
</table>

**EXPERIMENTAL RESULTS AND DISCUSSION**

There were 4 sets of specimens in this research but only specimen PBCCb30h20 is taken for examples in this paper. The relationship between its loading and angular displacement for each specimen are shown in Fig. 4.

![Fig. 3. Schematic diagram of testing frame](image)

![Fig. 4. Relationship between loading and angular displacement for specimens](image)

The figure shows that the specimen maintained its strength even when story drift reached 6%. Plastic angular displacement is about 5% when story drift reached 6%, which satisfied the requirements, Federal Emergency Management Agency (FEMA) and Taiwan Steel Structure Design Specification, 1994. Therefore, it is proved to be an excellent earthquake-resistant beam-to-column connection. The experimental results showed that there was 3mm
residual displacement at beam midpoint when the structural system was loaded and story drift reached 3%. Even when the story drift reached 6%, there was only 35mm residual displacement at beam midpoint. It proved that the structural system is capable for self-centering.

Fig. 5 shows the energy dissipation proportion of beam, column, panel zone and X-shaped damper. The Figure shows that the X-shaped damper contributed the most energy dissipation. When maximum story drift reached 6%, the percentage of energy dissipation by X-shaped damper, beam, column and panel zone are 90%, 10% and almost 0%, respectively.

![Fig. 5. Proportion and distribution of energy dissipation for connections](image)

**Table 2. Displacement of beam, column, panel zone and damper with maximum story drift**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$k_1$ (kN/mm)</th>
<th>$k_2$ (kN/mm)</th>
<th>$k_3$ (kN/mm)</th>
<th>$F_1$ (kN)</th>
<th>$\Delta_1$ (mm)</th>
<th>$F_2$ (kN)</th>
<th>$\Delta_2$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PBCCb30h20</td>
<td>theoretical</td>
<td>5.931</td>
<td>3.401</td>
<td>0.354</td>
<td>75.221</td>
<td>12.992</td>
<td>95.620</td>
</tr>
<tr>
<td></td>
<td>experimental</td>
<td>5.213</td>
<td>2.837</td>
<td>0.332</td>
<td>74.304</td>
<td>14.341</td>
<td>95.090</td>
</tr>
<tr>
<td>PBCCb30h15</td>
<td>theoretical</td>
<td>5.931</td>
<td>3.179</td>
<td>2.876</td>
<td>98.148</td>
<td>17.585</td>
<td>119.10</td>
</tr>
<tr>
<td></td>
<td>experimental</td>
<td>5.881</td>
<td>0.355</td>
<td>0.402</td>
<td>84.610</td>
<td>17.522</td>
<td>115.051</td>
</tr>
<tr>
<td>PBCCb20h15</td>
<td>theoretical</td>
<td>5.931</td>
<td>1.872</td>
<td>0.354</td>
<td>72.338</td>
<td>13.119</td>
<td>88.555</td>
</tr>
<tr>
<td></td>
<td>experimental</td>
<td>5.833</td>
<td>2.354</td>
<td>0.302</td>
<td>82.850</td>
<td>14.780</td>
<td>93.600</td>
</tr>
<tr>
<td>PBCCb20h20</td>
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<td>5.931</td>
<td>1.771</td>
<td>0.364</td>
<td>70.135</td>
<td>12.767</td>
<td>89.548</td>
</tr>
<tr>
<td></td>
<td>experimental</td>
<td>4.787</td>
<td>1.687</td>
<td>0.475</td>
<td>63.026</td>
<td>14.689</td>
<td>88.263</td>
</tr>
</tbody>
</table>

The theoretical equations of the beam-to-column connection mechanics model developed in this study were proved to be close to experimental results. As a result, the theoretical equations of the mechanics model and the experimental results are worth referring.

**CONCLUSIONS**

The design method of connection for pre-stressed steel beams and CFT column was proved that the major structure elements remain undamaged after an earthquake while the X-shaped damper absorbed most energy. The replacement of the steel plates to repair structure after an earthquake is simple and fast. Besides, this study proved that the structural system is capable of self-centering and well earthquake-resistant with remarkable strength, ductility and energy dissipation performance. The structural system remained its integrity even when the story drift reached 6%. Also, the plastic angular displacement of the beam-to-column connection can reach 5%. It demonstrated that the earthquake-resistant performance of this beam-to-column connection exceeds the requirements of earthquake-resistant connection specification in Taiwan and US.

The theoretical equations of the beam-to-column connection mechanics model developed in this study were proved to be close to experimental results. As a result, the theoretical equations of the mechanics model and the experimental results are worth referring.

**REFERENCE**


Pseudo Dynamic Test and Analysis of A Full Scale Two-Story Steel Plate Shear Wall Substructure

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林志翰 ¹ 、蔡克銓 ²

Abstract

The purposes of this research combined experimental and analytical investigation on the seismic responses of SPSW structures include: (1) investigate the seismic performance of a full scale two-story steel panel shear wall structure, (2) investigate the capacity design criteria of the boundary beam and column elements surrounding the steel panels, (3) investigate the effectiveness and seismic design of the lateral restrainers for the SPSW, (4) validate the TensionOnly material implemented for the PISA3D in simulating the responses of the SPSW structure, (5) incorporating the PISA3D into the networked substructure pseudo-dynamic tests of the 2-story SPSW structure, (6) investigate the seismic performance of the RBS connections in the SPSW specimen.

Keywords: SPSW, tension field, strip model, TensionOnly material

Introduction

Steel Plate Shear Wall (SPSW) frame system is the lateral force resisting frame of structure with infilled steel plate (Fig. 1). Because of the extra stiffness and strength of SPSW frame system, the thin steel plate often be used when this system applying. Although the thin plate wall without any stiffening is easy to buckle in shear, after the infill plate to buckle in shear can develop diagonal tension field action (Fig. 2). And then dissipate energy through the yield of tension field.

Strip material

The strip model is often used when taking structure analysis of SPSW frame. Strip model (Fig. 3) is developed by Kulak et al. (1983). In this model, a series of inclined, pin-ended, tension member are taking to represent the steel plate. The single strip’s behavior will have a influence in simulation result of SPSW. Because of the thin steel plate will tend to yield in tension and buckle in compression. In this research, developing a new material (TensionOnly material) in PISA3D computer program (Lin and Tsai, 2003) to simulate single strip behavior. The
TensionOnly material behavior is just like Fig. 4. Then compare with the past test result (Lin and Tsai, 2004) to validate the performance of new strip material. The specimen and analysis model is showed in Fig. 5. The analysis result (Fig. 6) reveals that taking this new material (TensionOnly) to simulate strip behavior can get good result in SPSW frame simulation.

**Experimental Program**

This specimen in this research is a full scale two-story SPSW frame, which simulate a lateral force resisting frame in a virtual structure (Fig. 7). This specimen is measuring nine meter tall and four meter wide. Considering the convenient of material preparation in future apply, SS400 grade steel plate is used. The thickness of steel plate for first story wall is 3mm and for second story wall is 2mm. The yield strength for steel plate of each story are 335MPa (1F) and 338MPa (2F). All the boundary beam and column elements are A572 GR. 50 steel. Detail specimen size is showed in Fig. 8.

On each side of steel plate, this specimen is constructed with three horizontal restrainers. The main purpose of restrainer is to raising the serviceability (improve loud buckling sounds, and large out-of-plane displacement). The section design of restrainer is base on the past research (Lin and Tsai, 2004), It recommend to take 3% of SPSW maximum shear as the restrainer force demand. The design result is rectangular tubes (Tube 125x75x4 mm for 1F, Tube 125x75x2.3 mm for 2F).

In order to avoid the fracture happen in column face, all beam-to-column connection detail is to use reduced beam sections (RBS) at each end. When using RBS connection detail, we must consider the deep column effect. After RBS section buckle, in this section may occur large lateral displacement. If the column section is more deeply, it may cause additional wrapping stress in column. Possibly it may cause column occur unexpected failure. So when using the RBS detail in beam-to-column connection, we should take the deep column effect into consideration. In order to avoid this situation, we increase the lateral support beam to control lateral
displacement of RBS section. The two-story specimen is showed in Fig. 9 to Fig.10.

Two-story SPSW frame analytic model

In order to predict the seismic response of test SPSW frame, using PISA3D computer program to construct a 3D model. In this analytic model, taking two series of strips to represent steel plate wall, and the inclined angle of strips is ±41°. Each series of strips contain twelve strips. About strip, we take truss element and using the new developed material (TensionOnly) to simulate strip behavior. Boundary frame are using BeamColumn element and Bilinear material. Both in TensionOnly and Bilinear material, the decision of yield strength and past yield stiffness is base on the coupon test of material. The PISA3D model is showed in Fig. 11.

Because of the material property (Only take tensile force) used to simulate strip, we must construct two direction inclined strips to represent SPSW behavior. But it may bring some problem, especially in calculating of structure period. In my two-story SPSW model, using two direction strip may cause the double calculation of steel plate wall’s stiffness. So when I get a structure period, it may be wrong. It needs some modify in model, when calculating the structure period. In order to solve this problem has several solutions. One solution is that it only construct one series of inclined strips when calculating structure period. Then construct another direction of inclined strips when analysis the seismic response.
Conclusions

In this research, we have some conclusions and recommend as follow:

1. Using PISA3D computer program to construct unstiffened steel plate shear wall model. When we use strip model and take TensionOnly material to simulate the strip behavior, it is valid comparing with test result. According to above result, we can say that TensionOnly material is a reliable material property to represent strip when simulating SPSW frame.

2. When beam-to-column connection detail is to use reduced beam sections (RBS). In order to avoid unexpected failure in column, the lateral support beam must be add to control RBS section lateral displacement. So that we can avoid deep column effect to damage the boundary column.

3. The new developed strip material (TensionOnly material), we use bilinear behavior to simulate the tension behavior. It still has some difference with real material. If we can use simulate hardening behavior in the tensile section of TensionOnly material, it may have better performance in SPSW frame’s simulation.

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Experimental Study of Genetic Algorithm on Structural Control

Tzu-Kang Lin1, Hua-Hsuan Kao2, Kuo-Chun Chang 3

林子剛 1 高華旋 2 張國鎮 3

Abstract

Recently, how to protect and control the civil structures during earthquakes has been an important issue over the last few decades. Commonly, the structure control can be classified into three categories: passive control and active control and hybrid control. Though theoretical analysis has shown that active control can offer better control results than the passive control, the complexity and the practical reliability problem are still unsolved. For this reason, this research will focus on proposing a more reliable way for the prevalence of active control in the future. The new method is then verified by a down-scale shaking table experiment and the result is carried out.

Keywords: genetic algorithm, smart structure, structure control

Introduction

As commonly known, earthquakes are the most unpredictable accident among all natural disasters. The damage and losses including properties and lives could be overwhelmed during a major earthquake. Nowadays, how to protect the structure from destruction and collapse has become more and more important. Accompanying with the improvement of the material ductility and strength by the new-developed technology, the earthquake-resistant ability of structures has been largely improved. The geotechnical and faults investigations have offered great contribution in hazard analysis. The use of energy-absorbing elements could also prevent any unexpected damage during the earthquake. Among these elements, the dampers seem to be the most well-known and practical ones in these days. Meanwhile, research on active control has been prosperous these years and the results from theoretical analysis have also demonstrated their advantages in reducing the structural response during external excitations. However, the active control method is still bothered by some factors that cannot be overcome. For example, the reference frame is needed for measuring the relative structural response in most active control theories and it is not practical for high-rise buildings. For these reasons, the active control system with a tunable mass installed on the roof floor is the only method that has been implemented in the practice. The first aim of this research is to improve the feasibility and reduce the cost without sacrificing any control efficiency in active control.

Over the last few decades, human has been exploring lots of fields to solve some complex problems. As shown in nature life, the intuition of creatures is to follow the most adaptive and valuable way in life. Following the concept, artificial intelligence was proposed and developed. Up to date, the most famous branch of artificial intelligence includes fuzzy theory, artificial neural networks, and genetic algorithm. By setting proper parameters in these theories, artificial intelligence can be easily applied to the engineering field and offer an alternative way to solve the complex nonlinear problems.

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This research will focus on integrating the traditional active control method and other new proposed technologies. The goal is to develop diverse ways for a more robust active control system. The improved genetic-algorithm-based method will be evaluated with the linear quadratic gain (LQG) control to prove its superiority in control efficiency.

Theoretical Analysis and Simulation

In order to improve the feasibility of LQG, the commonly used feedback signals such as the relative displacements and relative velocities of the structure is replaced by strain gauges in this paper. By using these source, the modified active control law can calculate the control force by an optimized gain and it can be expressed as equation (1).

\[ u_{ci}[k] = u_{ci}[k - 1] + u_{ci}[k - 2] \cdots + u_{ci}[k - 3] + u_{ci}[k - 2] \cdots + u_{ci}[k - 1] \]  \( (1) \)

To find out the optimal feedback gain, which is the most suitable for the specified structure, genetic algorithm is applied into the modified active control to find the optimal gain. The structural response would be effectively alleviated while it is stroked by arbitrary external force.

The structure used for the theoretical analysis is a scale-down eight-floor structure, which is referred from a real scale model. The first-mode natural frequency of the specimen is designed to be 5.79 rad/sec with 2% first-mode damping ratio. The damping matrix is supposed to be proportional to the mass and the stiffness. The stiffness and mass matrix of the structure can be obtained for the numerical simulation. The final natural frequency and the damping could be calculated by numerical analysis after determining the dimension of the structure. The size and characteristics of the structure are shown as the Table 1.

To obtain the optimal gain for the control force, some parameters used for the genetic algorithm has to be defined before the numerical simulation. As mentioned before, the cost function would play the most important role in the optimization process. Since the goal of this research is to reduce the response of the structure simultaneously during external excitation, the ratio between the response of the uncontrolled structure with that of the proposed method is taken as an evaluation criterion for controlling the structure. The cost function used in this research is defined as Equation (2).

\[ C_i = \sum_{i=1}^{8} (1 + \frac{|x_i|_{\text{max}}}{|x_i|_{\text{max}}^{\text{max}}})^{a_i} \]
\[ C_s = \sum_{i=1}^{8} (1 + \frac{|x_s|_{\text{max}}}{|x_s|_{\text{max}}^{\text{max}}})^{b_i} \]
\[ C_1 = \sum_{i=1}^{5} (1 + \frac{R_{dA_i}}{R_{dA_i}}) \beta_i \]
\[ C_4 = \sum_{i=1}^{5} (1 + \frac{R_{dA_i}}{R_{dA_i}}) \beta_i \]
\[ C_5 = \sum_{i=1}^{5} C_i \]  \( (2) \)

Where the suffix i represents the response of the structure with control, suffix u represents the behavior of benchmark. \( |x|_{\text{max}} \) represents the absolute maximum of relative acceleration of each degree of freedom of the structure and \( |x|_{\text{max}}^{\text{max}} \) represents the absolute maximum of relative displacement of each degree of freedom of the structure. \( RA \) represents the root-mean-square value of relative acceleration of each degree of freedom of the structure and \( RD \) represents the root-mean-square value of relative displacement.

Comparison of Numerical Analysis and Experimental Results

By the above-mentioned process, the optimal feedback gain is obtained. EL Centro earthquake time history with sampling frequency of 50Hz is used to search for the optimal feedback gain. The gain is then applied on the structure under excitations of EL Centro and Kobe time history. Numerical comparison including relative displacements and absolute accelerations of each degree of freedom is shown in Table 1. In order to have a comparing benchmark for the proposed control method, structure response with the modified active control system is also compared with those under LQG method. The structure responses from the fifth floor to the eighth floor are listed in Table 1.

Table 1 Theoretical reduction percentage under EL Centro earthquake

<table>
<thead>
<tr>
<th>EL Centro 150gal</th>
<th>LQG</th>
<th>GA_strain</th>
<th>GA_accel</th>
</tr>
</thead>
<tbody>
<tr>
<td>MaxA%</td>
<td>48</td>
<td>47</td>
<td>55</td>
</tr>
<tr>
<td>MaxD%</td>
<td>45</td>
<td>43</td>
<td>58</td>
</tr>
<tr>
<td>RmsA%</td>
<td>62</td>
<td>61</td>
<td>68</td>
</tr>
<tr>
<td>RmsD%</td>
<td>68</td>
<td>67</td>
<td>70</td>
</tr>
</tbody>
</table>
The theoretical analysis has shown that the structure response can be effectively reduced by the GA-based method. Basing on the satisfying result, the control method is then applied to a practical structure on a shaking table in National Center for Research on Earthquake Engineering (NCREE). The active control system is composed of multi components including sensors, actuators, and control force command center. Each of the components should be carefully examined before experiment to make the verification process successful. For example, any failure of the sensors may cause serious deviation on the control force and the response of the structure might diverge during the experiment process. For that reason, each step of the experiment is carefully scheduled and prepared. The specimen was manufactured according to the theoretical model, which is an eight-floor down-scale steel structure.

The experimental process can be classified into two main groups. The first one is the active control experiment with the GA-based algorithm and the second one is the benchmark experiment with the LQG method. The first part of the experiment can be divided into two sub-groups. In the first sub-system, strains of specific locations on the structures and the control force time history is used as the feedback data for the control system. Due to the installation problem, the strain of the structure in the first method is changed to the acceleration of each floor in the second sub-system to improve the feasibility of the GA-based method. Experimental results of the structural response under EL Centro earthquake is shown in Table 2. Same as shown in Table 1, the maximum relative displacement, maximum absolute acceleration, root mean square of relative displacement and root mean square absolute acceleration during the whole time history is listed in the table.

<table>
<thead>
<tr>
<th>EL Centro 150gal</th>
<th>LQG</th>
<th>GA_strain</th>
<th>GA_accel.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6F</td>
<td>7F</td>
<td>8F</td>
</tr>
<tr>
<td>MaxA%</td>
<td>38</td>
<td>41</td>
<td>39</td>
</tr>
<tr>
<td>MaxD%</td>
<td>51</td>
<td>47</td>
<td>43</td>
</tr>
<tr>
<td>RmsA%</td>
<td>53</td>
<td>55</td>
<td>52</td>
</tr>
<tr>
<td>RmsD%</td>
<td>63</td>
<td>59</td>
<td>58</td>
</tr>
</tbody>
</table>

The results in Table 2 have shown that all the three methods can effectively reduce the structural response. The displacement of structure can be alleviated much more than the acceleration. According to the experimental results, the performance of the GA-based subsystem 1, which uses the strain and control force as the feedback sources, is similar to those of the LQG control method. The performance of the GA-based subsystem 2, which uses the acceleration and control force as the feedback sources is a little downgraded. The results are also compared with the data obtained from theoretical analysis. It is shown that the control efficiency is about 90% of the theoretical value. Comparison of the analytical control force and experimental achieved control force has also shown large difference between them. Namely, the system does not achieve its best performance during the experimental process. To demonstrate the correctness of theoretical analysis, the control force from experiment is used as the feedback force in numerical simulation. The data has shown great compatibility with the experimental results and the accuracy of the model is verified. However, connection between each component of the experimental control system should be improved to enhance its practical performance.

### Table 3 Comparison of control force under EL Centro earthquake

<table>
<thead>
<tr>
<th>Time history</th>
<th>Control Method</th>
<th>Theoretical Analysis</th>
<th>Experimental Achievement</th>
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</thead>
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<tr>
<td>EL Centro</td>
<td>LQG 4874</td>
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<td>GA_strain 4919</td>
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<td>GA_accel. 4658</td>
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Conclusions

A new GA-based method focusing on improving the traditional active control system is proposed in this paper. To get better performance than the conventional control method, the proposed method tries to use a new approach to the optimal control force.

By using the genetic algorithm, the optimal feedback gain is obtained with searching the global domain in GA-based active control. Moreover, instead of using relative displacement and relative velocity, strains, accelerations of the structure, and control force are taken as the feedback data to calculate the control force. By these obvious improvements, reference frame is no longer needed and the feasibility of active control would be largely improved.

The proposed method is implemented in a down-scale eight-story steel structure. Theoretical analysis and experimental result have shown that under different earthquake excitations, the response of the structure, both the maximum relative displacement and absolute acceleration can be effective reduced. The root mean square value of the time history has also demonstrated
that the structure response can be controlled in a smooth way.

To check the reliability of the numerical model, the experimental command force was input as the control force in the simulation process. Compatible result was obtained and the robustness of the model is approved again. However, the efficiency downgraded approximately 10 % during the experiment for the insufficient achievement of the control force. Comparison of the control force has shown serious diminution of the control force during experiment. It is concluded that the difference among theoretical analysis and experimental data was caused by integration problems between control components. The pragmatic setup will be improved in the future work.

Outlooks

The research has successfully applied genetic algorithm in direct search of optimal feedback gain in an active control system. However, the control algorithm is obtained by a special fitness function with several parameters defined by the researcher. By using diverse feedback sources and different parameter settings, the choosing of the parameters could also be optimized and it is believed that the performance of the control system would be enhanced. The annoying local minimum problem, frequently met during the optimization process, could also be avoided. Moreover, to strengthen the reliability of the smart control system, the GA-base active control system will be integrated with other artificial intelligence technologies. The feedback source will also be evaluated by using other instrumentation devices to make the proposed control system more practical in the application field.

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A Study of Precast/Prestressed Beam-to-Column Connections

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葉勇凱\textsuperscript{1}, 黃世建\textsuperscript{2}, 黃建勳\textsuperscript{3}

Abstract

Precast/prestressed concrete structures have the advantages of high quality and fast constructing. And due to the prestress the residual displacement after earthquake will be restrained. It can keep the functions of buildings. But at high seismic region the design of prestressed structures must be careful. The prestressed tendons can not yield to keep the prestress at any conditions. So the requirement of energy dissipation is not easily achieved. In this study, we intend to use energy dissipation devices in beam-to-column connections and study the provided dissipation energy. It is hoped that those devices can provide the seismic performance of the precast/prestressed concrete structures. Five reinforced concrete specimens with or without energy dissipating devices were tested to study the energy dissipating capacity and the damage patterns of concrete. Three exterior joint specimens LPTC, HPTC and HPTC-M used steel plates as the energy dissipating device. Interior joint specimen PTC had different lateral steel configurations in the left and right connected beams to study the damage patterns. Another interior joint specimen Hybrid PTC used partially unbonded mild steel bars as the energy dissipating device.

Keywords: precast/prestressed concrete structures, plastic hinges, steel plates, energy dissipating devices

Introduction

Traditional reinforced concrete structures have some disadvantages such as too heavy self-weight, small span, cracks, slow constructing speed and hard field casting work. On the contrary, precast structures have some advantages such as light self-weight, small sections, long span and fast constructing speed and good quality control. Precast members can be fast produced in shop and easily constructed in field. Although the cost of precast members is higher than reinforced concrete members, the less construction time and easy field working let precast/prestressed concrete structure system be a competent choice.

The major difference between precast/prestressed concrete structures and reinforced concrete structures is the behavior of energy dissipation. The traditional reinforced concrete structures use the yielding of reinforced bars to dissipate the energy. But, the prestressed tendons in the precast/prestressed concrete structures can not yield. So it needs extra devices to accomplish the requirement of energy dissipation.

In this study, we tested three different energy dissipation devices in beam-to-column connections and study their influence to the plastic hinge of beams and the connection zone. First is an interior joint case. Its right beam used spiral lateral reinforcement and left beam used rectangular hoop reinforcement to study their different damage patterns. Second is an interior joint case too. We let steel bars pass through the beam-to-column connection and partially bounded with no-shrinkage mortar to observe its energy dissipation effect. The third is an exterior joint case. It used steel plates as the energy dissipating device. We
changed the length of steel plates and the prestress of tendons to observe the energy dissipation effect.

**Specimens and Set-up**

In this study there are five specimens. All specimens used posttensioned tendons to connect the beam and the column. Due to the initial prestress is acting on the beam, the applied moment needs large enough to open the gap between the beam and the column. When the gap is opened, the energy dissipation device starts to function. The nonlinear deformation is concentrated at the device to avoid the damage of beams and columns. The prestress forces the gap closed after the seismic loading. So the system can reduce the residual displacement and its energy devices can be easily replaced. Three specimens, LPTC, HPTC and HPTC-M, are exterior beam-to-column connections. Two specimens, PTC and Hybrid PTC, are interior beam-to-column connections. For all specimens, the column dimensions are 65×65×370 cm and the beam dimensions are 50×60×305 cm. As shown in Fig. 1, the exterior joint specimens used two L shape energy dissipation steel plates fastened on the both sides of the beam. Specimen LPTC has energy dissipation plate length 180 mm and posttension force 1500 kN. Specimens HPTC and HPTC-M have energy dissipation plate length 130 mm and posttension force 2000 kN. The difference of specimen HPTC and HPTC-M is the welding type of cover plates. HPTC used 10 mm field welding and HPTC-M used 10 mm penetration welding.

![Fig. 1 The exterior joint specimen](image1)

The interior joint specimen PTC used rectangular hoop reinforcement in the east beam and spiral lateral reinforcement in the west beam to observe the damage patterns of beam ends. As shown in Fig. 2, another interior specimen Hybrid PTC used four 6.85 m steel bars as the energy dissipation device. After the pre stressing procedure, let the steel bars pass through the beam-to-column connection and grouted with non-shrinkage cement. The steel bars were wrapped with plastic sheet at the beam end to form a 30 cm energy dissipation region.

![Fig. 2 The interior joint specimen Hybrid PTC](image2)

In the test, the bottom end of the column was fixed at an A frame. The upper end of the column was hold by an actuator to maintain its position. At the each end of the beams had an actuator controlled under a prescribed displacement time sequence. The time sequence had a triangular wave form. There have six cycles at each of drift ratios 0.375%, 0.5% and 0.75%. The following drift ratio 1.0% had four cycles. For each of the following drift ratios 1.5%, 2.0%, 3.0%, 4.0%, 5.0% and 6.0%, all had two cycles.

![Fig. 3 The energy dissipation behavior of a posttensioned connection](image3)

**Energy Dissipation Behavior**

As shown in Fig. 3, the mechanism of energy dissipation of a posttensioned connection can be divided into five stages. At the first stage, the gap between the beam and column is closed and energy
dissipation device is not stretched. This stage is in an elastic state and the stiffness of the system is same as the original beam-to-column system. At the second stage, the gap between the beam and column is opened and energy dissipation device is stretched but not yield. Because the gap provides additional displacement, the stiffness of the system is smaller than the stage 1. This stage is also in an elastic state. At the third stage, the energy dissipation device is yield and even hardening, so the stiffness is much smaller. This stage is in a nonlinear state. At the fourth stage, the loading is reverse and the energy dissipation device elastically unloads. The system stiffness is same as the stage two. Because the device already has plastic deformation, the system starts to dissipate the energy. At the fifth stage, the unloading device is yield and stiffness is reduced to the value of the stage three. Further, the gap is closed again and system is back to elastic state. These five stages form the complete energy dissipation hysteretic loop and have a self-centering mechanism.

### Test Results

The differences of the specimens are shown in Table 1. In this paper, the positive of loading-displacement curve is defined as the push-up direction of the actuator. The following will discuss test results of each specimen.

#### Table 1 Differences of the specimens

<table>
<thead>
<tr>
<th></th>
<th>LPTC</th>
<th>HPTC</th>
<th>HPTC-M</th>
<th>PTC</th>
<th>Hybrid PTC</th>
</tr>
</thead>
<tbody>
<tr>
<td>type</td>
<td>exterior</td>
<td>exterior</td>
<td>exterior</td>
<td>interior</td>
<td>interior</td>
</tr>
<tr>
<td>steel plate</td>
<td>steel plate</td>
<td>steel plate</td>
<td>no device</td>
<td>steel bars</td>
<td></td>
</tr>
<tr>
<td>ED length</td>
<td>180cm</td>
<td>180cm</td>
<td>130cm</td>
<td>300cm</td>
<td></td>
</tr>
<tr>
<td>PT force</td>
<td>1520kN</td>
<td>1800kN</td>
<td>2040kN</td>
<td>1650kN</td>
<td>1080kN</td>
</tr>
</tbody>
</table>

#### LPTC

Fig. 4 is showing the loading-displacement curve of specimen LPTC and the numerical analysis results. At the stage of larger drift ratio, the hysteretic loop shows strength degeneration and the residual displacement is left. Due to the slip of the energy dissipation steel plate, the plate’s yielding is delayed. We expected the plate should yield at drift ratio 0.5% but actually it is occurred at drift ratio 3%. This is the reason of strength degeneration and residual displacement. At the drift ratio 5%, the loading reached its maximum value 480.80kN and the cover plate was failure due to the buckling of energy dissipation plate. The testing was stopped at the second cycle of drift ratio 5%.

#### HPTC

Fig. 5 is showing the loading-displacement curve of specimen HPTC and the numerical analysis results. Because the posttension force was larger than LPTC, the gap was opened later. The welding of PT plate was strengthened and the slip did not occur during the testing. Before the drift ratio 3%, the energy dissipation effect was better than LPTC and the hysteretic curve was close to the predicted value. After the drift ratio 3%, the ED plate was buckling and the cover plate was failure and the energy dissipation effect was reduced. At the drift ratio 5%, the loading reached its maximum value 536.80kN. The testing was stopped at the second cycle of drift ratio 5%.

#### HPTC-M

Fig. 6 is showing the loading-displacement curve of specimen HPTC-M and the numerical analysis results. At the stage of larger drift ratio, the hysteretic loop shows strength degeneration and the residual displacement is left. Due to the slip of the energy dissipation steel plate, the plate’s yielding is delayed. We expected the plate should yield at drift ratio 0.5% but actually it is occurred at drift ratio 3%. This is the reason of strength degeneration and residual displacement. At the drift ratio 5%, the loading reached its maximum value 480.80kN and the cover plate was failure due to the buckling of energy dissipation plate. The testing was stopped at the second cycle of drift ratio 5%.
of specimen HPTC-M and the numerical analysis results. The posttension force was same as HPTC but the welding of cover plate was strengthened. After the drift ratio 3%, the ED plate was buckling and the cover plate was failure. The energy dissipation effect was worse than the predicted value but better than HPTC. At the drift ratio 5%, the loading reached its maximum value 540.20kN. The testing was stopped at the second cycle of drift ratio 5%.

- **Hybrid PTC**

Fig. 8 is showing the loading-displacement curve of specimen Hybrid PTC and the numerical analysis results. This specimen used four 6.85 m steel bars as the energy dissipation device. The energy dissipation effect was better than PTC but the damage of beams was similar with PTC. The testing was stopped at the second cycle of drift ratio 6%.

![Fig. 6 The hysteretic loop of specimen HPTC-M](image)

![Fig. 7 The hysteretic loop of specimen PTC](image)

![Fig. 8 The hysteretic loop of specimen Hybrid PTC](image)

**Conclusions**

This study has proved that the posttension precast/prestressed beam-to-column connections can achieve the requirement of energy dissipation through the using of energy dissipation devices. The system can dissipate energy and have no residual displacement. The contact faces of the beam and column using steel plates to protect will be no damage. All damages will occur at energy dissipation device. After the earthquake, we only need to replace the energy devices and not to repair the beams or columns.

**References**

Seismic Behavior of Low-Rise Shear Walls with SMA Bars

Y.L. Mo¹ and W.I. Liao²

Abstract
In recent years, researchers have focused their study on many possible approaches to enhancing the seismic performance of structures. One promising solution which is receiving attention today is the application of Shape Memory Alloys (SMA). In this study, high seismic performance shear walls have been proposed to have SMA bars acting as a kind of structural bracing system at both sides of the shear walls to increase the ductility and the energy dissipation capacity of the low-rise shear walls. This paper presents the results of the reversed cyclic tests on low-rise shear walls with SMA bars. The height, width, and thickness of the designed shear walls were 1.0 m, 2.0 m and 0.12 m, respectively. SMA bars were provided in the directions of 25 degrees to the horizontal (from the top corner to the bottom corner of the wall). The steel ratio in both perpendicular directions of the shear walls was 0.24%. The main parameter used in the study is the type of SMA bars, namely Superelastic and Martensite SMA bars. The force-displacement hysteretic loops of the low-rise shear walls under reversed cyclic loading are presented. Test results show that the maximum shear strengths of the tested walls are affected by SMA bars. It was found that the shear wall with Martensite SMA bars has greater residual displacement. In contrast, the shear wall with Superelastic SMA bars has less residual displacement. At the ultimate state, one of the four Superelastic SMA bars buckled, resulting in less energy dissipation capacity than the expected value. How to prevent the buckling of SMA bars needs to be investigated in the near future.

Keywords: shear wall, Shape Memory Alloy, Martensite, Superelastic

Introduction
Shape Memory Alloys (SMAs) are the type of alloys which have the ability to dissipate energy through repeated cycling without significant degradation or permanent deformation. Superelastic SMA possesses the stable hysteretic behavior over a certain range of temperature, where its shape is recoverable upon removal of load. On the other hand, Martensite SMA also possesses the ability to recover its shape after undergoing large deformations through heating. Both types of SMA show a particular promise in civil infrastructural applications, especially in seismic resistant design and retrofit of structures.

In recent years, the unique properties of SMA have attracted many researchers’ attention to study its possible application to civil infrastructural members such as shear walls to increase the ductility and the energy dissipation capacity and to reduce the permanent deformation. This paper presents the results of evaluating the effect of SMA application to low-rise shear walls. Reversed cyclic loading tests were performed on three low-rise shear walls, two of which were using SMA bars as external bracings. It was found that the shear wall with Martensite SMA bars had greater residual displacement, while the shear wall with Superelastic SMA bars had less residual displacement. The buckling of Superelastic SMA bars were experienced during the tests resulting in the need of more investigations to prevent buckling.

Test Program
Three low-rise shear walls were tested under
reversed cyclic horizontal loading. The height to width ratio of walls is 0.5. The height, length and width of the designed shear walls were 1.0 m, 2.0 m and 0.12 m, respectively. Boundary elements are provided on the two sides of the shear walls. Steel bars were provided in horizontal and vertical directions as shown in Fig. 1 and Fig. 2.

Materials

The average concrete compressive strength of the shear walls and foundations were 206.4 kg/cm² (20.23 MPa) and 262.4 kg/cm² (25.72 MPa), respectively.

As shown in Fig. 1 and Fig. 2, No.3 bars were used as the stirrups of boundary elements, as the steel grids of the two walls, and the stirrups of bottom foundations of the three walls. No.4 bars were used as the stirrups of the top beam. No.5 bars were used as the longitudinal bars of the boundary elements. No.6 bars were used as the longitudinal bars of the top beam and the foundation. The yield and ultimate stresses of the used rebars are listed in Table 1.

<table>
<thead>
<tr>
<th>Table 1 Yield and ultimate stress of rebars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield stress</td>
</tr>
<tr>
<td>#3 Rebar</td>
</tr>
<tr>
<td>#4 Rebar</td>
</tr>
<tr>
<td>#5 Rebar</td>
</tr>
<tr>
<td>#6 Rebar</td>
</tr>
</tbody>
</table>

Table 2 Mechanical Properties of SMA

<table>
<thead>
<tr>
<th>Property</th>
<th>SMA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Superelastic</td>
</tr>
<tr>
<td>Maximum strain recovery</td>
<td>8%</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>30 - 83 GPa</td>
</tr>
<tr>
<td>Yield strength</td>
<td>195-690 MPa</td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>1068 MPa</td>
</tr>
<tr>
<td>Elongation at failure</td>
<td>17.50%</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>0.33</td>
</tr>
</tbody>
</table>

Notes: Data provided by Special Metals Corporation

There were two types of SMA bars used in this research, namely Superelastic and Martensite SMA bars. Both types have the density of 6.5 g/cm³. SMA has an excellent performance against corrosion. Mechanical properties of SMA are shown in Table 2. Fig. 3 shows the position of SMA bars on a typical specimen.

Structures

The height, length, and width of the shear walls were 1.0 m, 2.0 m and 0.12 m, respectively. It should be noted that the end regions of the shear walls were provided with columns as boundary elements. Each wall specimen was built with heavily reinforced top beam and bottom foundation to provide high axial stiffness relative to the adjacent wall. The top beam was used to apply the horizontal load along the top of the wall uniformly. The bottom foundation was designed to provide full fixity on the strong floor. The cross section of the columns was 180 mm x 240 mm and provided with longitudinal bars and stirrups. The steel ratio in both horizontal and vertical directions for SMAS (reinforced with Superelastic SMA bars) and SMAM (reinforced with Martensite SMA bars) was 0.24% as shown in Fig. 2, the other 0.24%, when compared to SMAC, was replaced with SMA bars.
applied on the top of the shear wall. The test procedure is controlled by the horizontal displacement at the top of the wall. The scheme of the displacement control is shown in Fig. 5.

Fig. 3 Position of SMA bars and LVDTs on the SMA bars

Fig. 4 Test set-up

Table 3 Steel Ratios

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Steel Ratio (%)</th>
<th>SMA Bars (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal</td>
<td>Vertical</td>
</tr>
<tr>
<td>SMAC</td>
<td>0.48</td>
<td>0.48</td>
</tr>
<tr>
<td>SMAS</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SMAM</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Test Results

Figs. 6, 7 and 8 show the cracking patterns and failure mode of specimens SMAC, SMAS and SMAM, respectively. The cracking of the concrete was drawn on the white painted faces of the specimens during the tests. During the tests, uniformly distributed cracks were observed in all specimens. No failure at the boundary elements of the walls was found in specimens SMAS and SMAM during the tests. For specimen SMAC, failure occurred at the bottom of the boundary elements causing the ultimate strength of the shear wall to be lower than expected value, as shown in Fig. 6. Concrete crushing and rebars buckling were observed at the middle region of specimen SMAS and SMAM. As shown in Fig. 7, one of the Superelastic SMA bars on specimen SMAS buckled. When the drift of specimen SMAM reached 0.625%, the width of cracks on the wall was about 1 mm, and the actuator was released. Then the Martensite SMA bars were heated to 150°C to promote the shape memory effect. This heating of SMA bars reduced the displacement at the top of the wall by 0.05 mm.

Fig. 6 Specimen SMAC at failure stage

Fig. 7 Specimen SMAS at failure stage

The force-displacement relationship of specimens SMAC, SMAS and SMAM are shown in Fig. 9, 10 and 11, respectively. Pinching effect was found from the hysteretic loops of the entire specimens. It is the typical phenomenon for low-rise conventional shear walls when rebars are provided in horizontal and vertical directions.
The ductility and maximum shear force of specimens SMAC, SMAS and SMAM are shown in Table 4. It is noted that the specimen SMAC failed due to the weak boundary elements and interface between the boundary element and the foundation, the strength and ductility of SMAC does not reflect the maximum capacity of the wall. The strength and ductility would be increased if failure did not happen at boundary elements.

Conclusion

Test results show that the maximum shear strengths of the tested walls are affected by SMA bars. It was found that the shear wall with Martensite SMA bars has greater residual displacement. In contrast, the shear wall with Superelastic SMA bars has less residual displacement. At the ultimate state, one of the four Superelastic SMA bars buckled, resulting in less energy dissipation capacity than the expected value. How to prevent the buckling of SMA bars needs to be investigated in the near future.

Table 4 Ductility and maximum shear force of LN and LB

<table>
<thead>
<tr>
<th></th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\mu$</th>
<th>Maximum shear force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMAC</td>
<td>4.97</td>
<td>13.93</td>
<td>2.80</td>
<td>89.67</td>
</tr>
<tr>
<td>SMAS</td>
<td>3.63</td>
<td>11.41</td>
<td>3.14</td>
<td>114.66</td>
</tr>
<tr>
<td>SMAM</td>
<td>3.03</td>
<td>10.91</td>
<td>3.60</td>
<td>105.41</td>
</tr>
</tbody>
</table>

Note: $\Delta_y$ = yield displacement
$\Delta_u$ = displacement corresponding to 80% of the maximum shear force in the descending branch

$\mu = \text{ductility (} \Delta_u / \Delta_y \text{)}$
Bridge Local Scour Monitoring System Using MEMS Sensor Zigbee Network

Yung-Bin Lin\textsuperscript{1} and Kuo-Chun Chang\textsuperscript{2}

林詠彬\textsuperscript{1}、張國鎮\textsuperscript{2}

Abstract

Wireless MEMS sensors network has been widely used in many fields. In this paper the MEMS pressure sensor is integrated on a sensor board with Zigbee sensor network for real-time local bridge scour depth monitoring during a flood. The wireless MEMS pressure sensor scour monitoring system has been developed and tested in the laboratory. Local scour is one of the major factors for bridge failure. Scour failures tend to occur suddenly and without prior warning or sign of distress to the structure. Bridges subject to periods of flood/high flow require monitoring during those times in order to protect the traveling public. This wireless MEMS sensors scour-monitoring system can measure both the processes of scouring/deposition and the variations of water level. Several experimental runs have been conducted in the flume to demonstrate the applicability of the MEMS sensors network system. The experimental results indicate that the real-time monitoring system has the potential for further applications in the field.

Keywords: Zigbee, MEMS, wireless sensor network, bridge, scour

Introduction

It is well known that scour is one of the major causes for bridge failure\textsuperscript{1,2}. When scouring occurs, the bed materials around the pier footing can be eroded, leaving the infrastructure such as bridge piers and abutments in an unsafe condition and in danger of collapse with the distinct possibility for loss of life. More than 1000 bridges have collapsed over the past 30 years in the U.S.A., with 60\% of the failures due to scour\textsuperscript{1}. This serious problem also happens in many East-Asian countries such as Taiwan, Japan, Korea…etc., owing to the fact that these areas are subject to several typhoon and flood events each year during the summer and fall seasons. Scour failure tends to occur suddenly and without prior warning or sign of distress to the structures. The nature of the failure is usually defined as the complete collapse of an entire section of a bridge. There were 68 bridges damaged due to scour damage in Taiwan, based on the survey from 1996 to 2001\textsuperscript{1}. Scouring at a bridge pier in the river can be caused by general scour, contraction scour or local scour. Among them, local scour is the most critical and generally caused by the interference of the structures with river flow, and it is characterized by the formation of the scour hole at bridge piers or abutments. A great deal of time, money and efforts have been dedicated to the development and evaluation of scour detection and instrumentation in order to obtain more accurate measurements. However, it is not easy to measure or monitor the depth variations of scouring at piers, especially in a flood.

Therefore, the local scour depth monitoring system faces the challenge of developing a real-time, reliable and robust system, which can be installed in a river bed near the bridge pier or abutment. Moreover, it is well known that the established scour formula for estimating the maximum scour depth relates to the characteristics, including the flow depth, velocity and sediment size. In practice, the limitations of these scour formula should be addressed before one can apply them adequately. The recognition of any possible aggradation and degradation of the river-bed level in response to a channel disturbance is important for the prediction of channel bed variations. Besides, the scour process around the pier or abutment is essentially complex due to the three-dimensional flow patterns.

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\textsuperscript{2} Director, Department of Civil Engineering National Taiwan University, ciekuo@ntu.edu.tw
interacting with sediments. However, most of the data obtained to develop the scour formula are collected from the laboratory instead of from the field. Thus, it is necessary to develop a real-time system for monitoring and measuring the scour depth in the field.

There is growing interest in wireless sensor networks (WSN) in civil engineering and industrial applications. However, many challenges still lie in the way of improving the capabilities of wireless sensors. WSN monitoring has emerged in recent years as a promising technology that will greatly impact the field of structural health monitoring. Smart, wirelessly networked sensors can collect and process a vast amount of data, from monitoring and control of structural damage, air quality, traffic conditions, to weather conditions and tidal flows. In the present study, wireless MEMS sensors Zigbee network local scour monitoring system is developed and utilized to the real-time measurement in the process of the local scour. Applying the wireless MEMS sensors Zigbee network systems, the laboratory data of the water level, scour depth and deposition height are collected and analyzed herein.

**MEMS pressure sensor**

Over the past two decades, the micro electro mechanical systems (MEMS) sensor industry has continuously made progress. MEMS devices have already been used in a number of commercial applications, including projection displays, and the measurement of pressure and acceleration. New applications are emerging as the existing technology which is applied to the miniaturization and integration of conventional devices.

Today, MEMS sensors offer very high accuracy at very low cost and provide an interface between the mechanical world and the electrical system. In this paper, MEMS pressure sensor was fabricated using a 4 inch double side polished P(100) wafer. The sensor die consists of a thin Si diaphragm fabricated by bulk micromachining. Prior to the micromachining, piezoresistors are patterned across the edges of the diaphragm region using standard IC processing techniques. After etching of the substrate to create the diaphragm, the sensor die is bonded to a Pyrex glass substrate to realize a sealed vacuum cavity underneath the diaphragm.

Conventionally, the die is mounted on a package such that the top side of the diaphragm is exposed to the environment through a port. Herein, the house of the pressure sensor die was filled with silicon oil. To preventing corrosion or conduction, an indirect stainless foil bonding approach with different thickness is applied to weld onto the bottom of the sensor house for adapting in the fluid field. This pressure sensor as shown in Fig. 1 is then ready to integrate into the Zigbee sensor board for test.

**Zigbee wireless sensor network**

The wireless monitoring system with micro-electro-mechanical system (MEMS) sensors could reduce system cost and time consume dramatically. A wireless monitoring system should provide relevant data from the observed structure without the requirement to inspect. So the data has to be transmitted in a sufficient way to the users. An on-site central unit for data collection and storage in a database and further to analyze the data from sensor node is needed. The central unit also should allow a calibration and a wireless reprogramming of the sensor nodes to keep the whole system flexible.

Moreover, in a situation when the network goes down, for example due to power failure, the application should be designed to take care of the recovery of the network to its previous state and continue with data transmission without any errors. Choosing the right network topology that best suits the application is an important decision. When data reliability is crucial, mesh architectures provide the best shield against signal degradation and loss of data.

Zigbee is a wireless standard based on 802.15.4 that was developed by the Zigbee Alliance (an organization of semiconductor manufacturers, technology providers, and OEM's). Zigbee incorporates the network, security and application layers that reside on top of the IEEE 802.15.4 WPAN standard.

Zigbee supports star, mesh, and cluster-tree network topologies as shown in Fig 2. It is important to ensure applications using robust routing algorithm to obtain the best possible route for data transmission from one node to another and remember them.

**Test and results**

Fluid pressure can be measured directly by the pressure sensor. Absolute pressure was measured using MEMS pressure sensors under atmospheric pressure. The MEMS pressure sensor Zigbee network local scour monitoring system is developed in the present study. As shown in Fig. 3, these two series aligned pressure sensors, namely s1 and s2, are individually mounted in 10 cm interval on the pier placed in the flume. The s1 is installed at the 10 cm height near the lower end of the pier while s2 is installed at the upper site. These sensors are connected directly to the Zigbee sensor board to wireless send all the real-time measured data to the coordinator (notebook) for further analysis.

The experiments were conducted in a 12 m-long, 1 m-wide and 1.2 m-deep flume with glass sidewalls at the Hydrotech Research Institute of National Taiwan University, Taipei, Taiwan. The pier was fastened in the middle of the flume. The prescribed discharge and its corresponding depth for each experimental case were controlled by adjusting the
inlet valve and tailgate. To simulate scouring/deposition of the river bed, a dune-like bed formation is assumed in the experimental setup.

Items measured include the calibration of the sensors under static pressure, the response of the sensors under current, upraised water flow measurement, and scour progress. When the rising water surface reaches the sensors, pressure will be detected directly.

As presented in Fig. 4 the s1 sensor was continuously impacted under a 2 m/sec spouted flow using a water pipe to test the performance of the sensor. Due to the unsteadiness of the jet flow in the water pipe, oscillation pressures in terms of volts were observed. These oscillation volts/signals under a 2m/sec flow velocity demonstrated the performance of the designed MEMS pressure sensor can be used in a swift/storm flow under a flood. As shown in Fig. 5, the static pressure is measured under steady state conditions as the flow tapping into the sensors. The calibration factors between the water depths and the pressures are obtained.

The rising flood with strong current not only attacked bridge but also caused disaster when it overflowed the riverbank during a torrential flood. Real-time measurement of the rising water level is also important. To measure the water level during the upraised flow, results of the flow pressures against time are shown in Fig. 6. The s1 sensor was impacted by the elevated flow firstly at about 30 second while the s2 was at 65 second. Due to the static pressure in the flume, the pressures of the s1 were steeper than those of s2. About 0.14 psi static pressure difference between these two sensors was 10 cm in vertical. As mentioned, a dune-like bed formation is assumed in the experiment to simulate scouring progress of the river bed. Scouring progress is shown in the Fig. 7 from the responded sensors. Both sensors are submerged in the dune-like bed with 50 cm deep water. As the water flowing toward the pier, scouring results from the flow shear near the bottom will be detected by the pressure sensors and that revealed the scour depth as these sensors emerged from the riverbed.

The experimental results indicate that the real-time monitoring system has the potential for further applications in the field. It is evident that results from the MEMS pressure sensor using Zigbee sensor network has been developed and tested in this study. Several experimental runs of measuring water level and scour depth using this system have been conducted and demonstrated in the laboratory. The experimental results indicate that the real-time monitoring system has the potential for further applications in the field.

It is evident that results from the MEMS pressure sensor using Zigbee sensor network is much convenient and innovative for real-time scour monitoring. Moreover, this monitoring system not only useful for scouring safety of a bridge, but also it benefited to integrate the accelerometers onto the single sensor board for bridge health diagnosis during earthquake attacked. The resistant performance characters under torrential flood or earthquake attacked of a pier and abutment is then monitored at real-time and at anytime as these events happened. Definitely, wireless sensor network using Zigbee protocol which provides real-time information will help engineers and bridge governor for bridge maintenance and operation under natural disasters.

Summary

Scour is one of the major causes for bridge failure. Scour failures tend to occur suddenly and without prior warning or sign of distress to the structure. Bridges subject to periods of flood/high flow require monitoring during those times in order to protect the traveling public.

MEMS pressure sensor scour monitoring system using Zigbee sensor network has been developed and tested in this study. Several experimental runs of measuring water level and scour depth using this system have been conducted and demonstrated in the laboratory. The experimental results indicate that the real-time monitoring system has the potential for further applications in the field.

References

Fig. 1 MEMS Zigbee network sensor board

Fig. 2 Zigbee network

Fig. 3 MEMS pressure sensor Zigbee network scour monitoring system setup in the laboratory

Fig. 4 Calibration of pressure sensors under static pressure

Fig. 5 Calibrations of the MEMS pressure sensors under still water condition

Fig. 6 Progress of pressure sensor during upraised flow

Fig. 7 Progresses of pressure sensor during scour
Development and Shake Table Test of the Semi-active Control Base-isolation System for Building Structure

Pei-Yang Lin

Abstract

In this study, the shaking table test of semi-active controlled base-isolation system is presented. The full-scaled one-story building structure is used as the upper structure in the shaking table test. The semi-active controlled base-isolation system combined the rolling pendulum system and one Magneto-rheological damper. The rolling pendulum system provide the suitable restoring forces, while the semi-active controlled MR damper provide the online adjusted damper force. The Fuzzy-Logic control is used to calculate the optimal command voltage to the MR damper. Eight different earthquake excitations with several PGA levels are tested. The shaking table test result shows that the semi-active controlled base-isolation system can isolated the ground acceleration input with limited stroke of the isolation system. Also, it is stable and adaptive to different intensity of excitation.

Keywords: semi-active control system, Magneto-Rheological Damper, Base-isolation system

Introduction

Low power consumption, high reliability and fail-safe operation make the semi-active control technique one of the more promising approaches for mitigation of seismic responses in civil engineering structures. Currently, magneto-rheological (MR) dampers are being widely studied for their potential use as semi-active control devices [Dyke][Spencer][Chang]. An MR damper resembles an ordinary linear viscous damper but the cylinder of the damper is filled with special fluid that contains tiny polarizable particles. The fluid state can be changed drastically from liquid to solid and vice versa by adjusting the magnitude of an applied magnetic field produced by a coil that is wrapped around the piston head of the damper. When no current is supplied to the coil, the damper behaves as an ordinary viscous damper. On the other hand, when current is sent through the coil, the fluid inside an MR damper becomes a semi-solid, the yield strength of which depends on the level of current applied. Since control commands simply adjust the parameters of a MR damper that is placed in a real structure, control instability never occurs and only a small amount of energy is required. Therefore, MR dampers are reliable and fail-safe.

This study combined the rolling pendulum system and the MR damper to the semi-active controlled base-isolation system. The rolling pendulum system provides the suitable restoring forces, while the semi-active controlled MR damper provides the optimal damper force. The command voltage to the MR damper is calculated by Fuzzy-Logic control. Different earthquake excitations with several PGA levels are tested. All the test result shows that the semi-active controlled base-isolation system is stable, reliable, fail-safe and effective. Also, it is adaptive to different intensity of excitation as compared to the traditional passive base-isolation system.

Design of Semi-active Control

The semi-active control device used in this study is the Magnetorheological damper. Although its resistance to motion can be changed on command, it can not be treated like an active actuator for purposes of numerical simulation. Moreover, traditional active control algorithms cannot be directly applied to this hybrid control system. Therefore, semi-active controllers are developed in the context of the nonlinear base-isolated structure with rolling pendulum system and MR damper subcomponents.

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Fuzzy logic is used to map an input space to an output space by means of if-then rules, see Fuzzy Logic Toolbox Users Guide. Control components of the input signal are transformed into linguistic values through a fuzzification interface at each time step. Use of a fuzzy controller is advantageous in that performance is not overly sensitive to changes in the input signal. For output the mapped linguistic values are transformed into numerical values through a defuzzification interface.

**Fig. 1 Input and output membership functions of semi-active control surface “S3”**.

Shake Table Test

In this study, a base-isolated structure with a rolling pendulum system and a 20 kN magnetorheological damper is tested on a shake table. Unlike traditional isolators, such as high damping rubber bearings (HDRB) (see for example [Lin]) or friction pendulum bearings (see for example [Wang]), the rolling pendulum system (RPS) is used in this study. The rolling pendulum system can provide the suitable restoring force with lowest resistance. The semi-active magnetorheological damper can provide suitable hysteresis behavior according to the command voltage. As result, the natural period and the hysteresis characteristics of the semi-active controlled base-isolation system (combined the RPS system and MR damper) can be selected by the control designer individually. And it maximize the control range, and it results in the semi-active controlled base-isolation system is much adaptive to different intensity of excitation. Figure 2 shows a schematic drawing and test photo of the experimental setup. The shaking table controlled by the MTS system delivers the desired earthquake excitation. Responses of the system are measured by numerous accelerometers, load cells, thermal couples and LVDTs, and are sent to the data acquisition system. Some transducer readings are also feed back to the semi-active control system (PC/Simulink). Through a semi-active control surface (pre-loaded in PC/Simulink) and the feedback signals, the command voltage can be calculated online. The command voltage is converted to command current by a voltage control current source (VCCS) and fed to the magnetorheological damper in order to optimally control the hybrid base-isolation system.

The floor plane of the upper structure is 2m by 3m, the floor height is 3m. All the structure members are made by steel. The section sizes of the columns, beams and girders are all “H150x150x7x10”. The beams and girders are rigidly connected to 25mm’s steel plate (floor plate). As result, the floor can be seen as rigid floor. The mass of each floor are 12 tons (including the structural weight and the mass blocks). Figure 3 (a) shows the configuration of the experimental sensors. All the displacement, velocity, acceleration, command voltage, damper force, shear and axial force are measured and recorded by suitable sensors and “Pacific” data acquisition system in NCREE.

The natural period of the rolling pendulum system (As shown in Figure 3(c)) is designed as 2.77 sec. Ends of the Magnetorheological damper (as shown in Figure 3(b)) are securely attached to the top surface of the shake table and the bottom of the isolated structure. The command voltage is sent from the “dSPACE” control system with one BNC line to the “VCCS”. The VCCS converts the command voltage to the command current and sent to the MR damper. The power supply provide the 24volts power to the VCCS.
Eight excitation cases, including El Centro NF (1970 array#6), El Centro FF (1940), three Chi-Chi earthquake records (1999) from three different station TCU075, TCU076 and TCU082, another earthquake record from station TCU075 (Far-field case), Kobe and random excitations, are tested in this shaking table test. Four passive control cases (with four different command levels), three semi-active control cases (with different feedbacks and control design) and fixed-base case are tested in this study. Table 1 lists the descriptions of these test cases. The passive cases are used to verify the capacity and also the failure scheme of the semi-active controlled base-isolation system. The fixed-base test is used to compare the structural response w/o isolation system.

Semi-active control case “S3” used the A1a and Dmr as feedback signals. The control object is to minimize the acceleration of the isolated structure (A1a) with the constraint of the stroke of the isolation system (Dmr). That means this control case wish to isolate the acceleration responses without exceeding the stroke capacity.

Figure 4 shows the comparison of the maximum controlled responses of two passive control cases (P-off and P-max) and the semi-active control case “S3” under two different kinds of excitations (TCU075-FF and TCU075-NF). TCU075-FF is a far-field earthquake records, while TCU075-NF is a near-field record. The near-field earthquake has more low-frequency content and it will induce a great displacement of the isolation system. As we can see from this figure, the passive control case “P-off” has the best acceleration reduction, but the displacement of the isolation system (Dmr) is also the biggest. The displacement increases with the PGA levels. Fortunately, the displacement did not exceed the system capacity (0.15m) during the tests of far-field earthquake “TCU075-FF”. In the near-field earthquake test “TCU075-NF”, the displacement reaches the capacity while the PGA level is about 260 gals. That means the passive controlled base-isolation system is effective and reliable when the excitation is not to intensive (ex: far-field eq., or small near-field eq. (less than 250 gals)). When the structure is designed to survive and function able in some intensive earthquakes, the power of the must be on. Comparing the maximum responses of the passive (P-max: maximum command voltage all the time) and semi-active controlled system in Figure 4, we found that the acceleration reduction pf the semi-active control system is much better, especially in the low PGA cases. The traditional passive isolation system designs the system according the design earthquake of spectrum. As results, the control effect of the isolation system is perfect when facing the design PGA level. But when the excitation is not so strong, the control effect (isolated acceleration responses) decreased. That means the passive control system is not adaptive to different intensity of excitations. Accoding to Figure 4, it is found that the semi-active control system is adaptive to different intensive of excitation. The acceleration is isolated more under small
excitation, and the control effect of the extreme excitation is compactable to the max-power case “P-max”.

Figure 5 shows the command voltage, MR damper force time histories, hysteresis loops of the MR damper and rolling pendulum system of semi-active control case “S3” under El Centro earthquake excitation (normalized to 700gals). According to this figure, the command voltage to the MR damper is changing according to the feedbacks. The hysteresis loops of the MR damper varied with the command voltage and have butterfly shapes. It just represent the design control object “reduce the acceleration with the constraint of the stroke”

Conclusions

The shake table test of semi-active controlled base-isolation system that includes rolling pendulum system and a Magnetorheological damper shows the great benefit of the smart damper. The most different between the base-isolation system with passive damper and semi-active control Magnetorheological damper is that the semi-active control system is adaptable to various kinds and intensity of excitations. While the passive control system can only focus on some cases (P-max: intensive earthquakes; P-off: small earthquakes). Also, this study provides evidence of full-scale, real-time control of augmenting a common base-isolation system with a smart Magneto-Rheological damper that is modulated with a fuzzy controller

References


Development and Shake Table Test of the Hybrid Controlled Bridge-isolation System

Pei-Yang Lin¹

Abstract

In this study, the development and shake table test of hybrid controlled bridge-isolation system are presented. The hybrid controlled bridge-isolation system is composed of the rolling pendulum system and the smart MR damper. The rolling pendulum system provides the suitable restoring force, and the smart MR damper provides the online adjusted hysteretic force. The scaled-down bridge model is used in the shake table test. Both passive and semi-active control cases are compared in this study. According to the shake table test results, the hybrid controlled bridge isolation system can reduce the deck acceleration effectively with the limited stroke. Also, it is stable and adaptive to different intensity of excitation.

Keywords: semi-active control system, Magneto-Rheological Damper, Bridge-isolation system

Introduction

Low power consumption, high reliability and fail-safe operation make the semi-active control technique one of the more promising approaches for mitigation of seismic responses in civil engineering structures. Currently, magneto-rheological (MR) dampers are being widely studied for their potential use as semi-active control devices [Spencer][Chang]. An MR damper resembles an ordinary linear viscous damper but the cylinder of the damper is filled with special fluid that contains tiny polarizable particles. The fluid state can be changed drastically from liquid to solid and vise versa by adjusting the magnitude of an applied magnetic field produced by a coil that is wrapped around the piston head of the damper. When no current is supplied to the coil, the damper behaves as an ordinary viscous damper. On the other hand, when current is sent through the coil, the fluid inside an MR damper becomes a semi-solid, the yield strength of which depends on the level of current applied. Since control commands simply adjust the parameters of a MR damper that is placed in a real structure, control instability never occurs and only a small amount of energy is required. Therefore, MR dampers are reliable and fail-safe.

This study combined the rolling pendulum system and the MR damper to the hybrid controlled bridge isolation system. The rolling pendulum system provides the suitable restoring forces with tiny friction, while the semi-active controlled MR damper provides the optimal and online adjusted damper force. As result, almost all the damping force in the bridge isolation system is controllable. The Fuzzy-Logic control algorithm and tool box in Matlab-Simulink software are used to design the semi-active controller. The absolute acceleration of the deck and the relative displacement of the isolation system are used as the feedback signals. The command voltage of the smart MR damper is calculated from the feedback signals, and then sent to the MR damper to generate the suitable damping force. The scaled-down bridge model is used in the numerical study and shake table test. Different excitations with several PGA levels are tested. Both passive and semi-active control cases are tested and compared.

Design of Semi-active Control

The semi-active control device used in this study is the Magneto-Rheological damper. Although its resistance to motion can be changed on command, it can not be treated like an active actuator for purposes
of numerical simulation. Moreover, traditional active control algorithms cannot be directly applied to this hybrid control system. Therefore, semi-active controllers are developed in the context of the nonlinear base-isolated structure with rolling pendulum system and MR damper subcomponents. Fuzzy logic is used to map an input space to an output space by means of if-then rules, see Fuzzy Logic Toolbox Users Guide. Control components of the input signal are transformed into linguistic values through a fuzzification interface at each time step. Use of a fuzzy controller is advantageous in that performance is not overly sensitive to changes in the input signal. For output the mapped linguistic values are transformed into numerical values through a defuzzification interface.

Design of a fuzzy logic controller is separated into three parts: (1) use a fuzzy inference system (FIS) editor to define the number of input and output variables and choose the type of inference to be used; (2) define membership functions for the input and output variables; and (3) define if-then rules. In this study, a trial and error process results in the use of two inputs (displacement and acceleration) and one output (voltage) variable. Next, triangular membership functions and the range of their variables are defined for each input and output variable. Finally, the if-then rules are edited so as to connect each input and output.

For the semi-active controller, S3, the absolute acceleration of the deck and relative displacement of the isolation system are selected as inputs, and the output is the command voltage. The number of membership functions used for the inputs are five and six, while seven membership function are used for the output. Figure 1 (a), (b) and (c) show the result of applying the inputs and output membership functions over the whole range of the input variables for semi-active control case “S3”. Figure 1 (d) shows the control surface of semi-active control case “S3”. The design approach for semi-active control case “S3” is to control both the absolute acceleration and the relative displacement.

**Numerical Simulation**

To get a preliminary understanding, the numerical simulation with the of the hybrid controlled bridge isolation system and the scaled-down bridge model is made. Figure 2 shows the setup view of the hybrid controlled bridge isolation system in the scaled-down bridge model.

![Fig. 2 Front and 3D view of the scaled-down bridge model with the hybrid controlled bridge isolation system](image)

The whole structure is simplified to the upper structure (the deck) and lower structure (the pier). The hybrid controlled bridge isolation system (purple parts) connects the upper and lower structure. Then, the equation of motion of the whole system can be written as follows:

\[
\{M\}\ddot{X} + \{C\}\dot{X} + \{K\}X = -\{M\}\ddot{X}_R + \{B\}F_{MR} \tag{2}
\]

The equation of motion contains two degree of freedom (the deck and the pier). The stiffness of the rolling pendulum system is written in the \([K]\) matrix. The nonlinear MR damper is written independently.

In the numerical simulation, the 1940 EL Centro earthquake excitation (NS) is used. As the object of the numerical study is to get the preliminary
understanding of this kind of hybrid control bridge isolation system, only the passive control cases are simulated in this section. Different PGA levels and command voltage to the MR damper are simulated and compared. Figure 3 shows the time history responses of the deck acceleration (A2), isolation displacement (D2) and pier displacement (D1) with (W) and without (W/O) MR damper. According to the figure, the deck acceleration is greatly reduced (PGA: 700gals, W/O:185gals, with MR: 116gals). The stroke of the isolation system is also reduced (W/O:0.39m, with MR: 0.14m). The displacement of the pier also decreased slightly.

![Fig. 3 Time history responses of the deck acceleration (A2), displacement of the isolation system (D2) and pier displacement (D1) with (W) and without (W/O) MR damper.](image)

**Shake Table Test**

To validate the real control effect, the hybrid controlled bridge isolation system is equipped onto the scaled-down bridge model and tested on the shake table. Figure 4 shows the photo of the shake table test in NCREE in 2005. Four rolling pendulum pads and one MR damper is used in the hybrid controlled bridge isolation system. The natural period of the rolling pendulum system is 2.77sec. The stroke of the MR damper is 300mm. Maximum force capacity is 20kN. The scaled-down bridge model belongs to a jointed NSF program. The scale factor of the bridge model is 7.5. The span is 4.5m, clear width of the deck is 1.75m, height of the pier is 1.47m and the weight of the upper structure is 21.72ton.

Figure 5 show the sensor setup diagram in the shake table test. Relative displacement and acceleration of the deck, top of the pier, bottom of the pier are measured. The command voltage, displacement and force of the MR damper are also measured. Six passive control cases with different constant command voltage levels to the MR damper (from the minimum to the maximum) and three different semi-active control cases are tested in the shake table test. The semi-active control procedure is illustrated as follow: The shaking table controlled by the MTS system delivers the desired earthquake excitation. Responses of the bridge model are measured by numerous accelerometers, load cells and LVDTs, and are sent to the data acquisition system. Some transducer readings are also feed back to the semi-active control system (PC/Simulink). Through a semi-active control surface (pre-loaded in PC/Simulink) and the feedback signals, the command voltage can be calculated online. The command voltage is converted to command current by a voltage control current source (VCCS) and fed to the Magneto-Rheological damper in order to optimally control the hybrid controlled bridge-isolation system.

![Fig. 4 Shake table test photo of the scaled-down bridge model with hybrid controlled bridge isolation system.](image)

![Fig. 5 Sensor setup diagram of the shake table test.](image)

Figure 6 show the comparison of input ground acceleration and deck acceleration time history responses (upper plot), relative displacement time histories of the bridge isolation system (middle plot) and pier displacement time histories (lower plot) of the semi-active control case under El Centro (NS dir.)
earthquake excitation (normalized to 500gals). According to the figure, the deck acceleration of the semi-active control case is greatly reduced. While the maximum stroke of the bridge isolation system is controlled to be smaller than 0.1m (Stroke capacity: 0.15m) under 500gals excitation. The semi-active controlled MR damper can online adjust the damper force, and it results the better deck acceleration reduction.

![Figure 6](image1.png)

Fig. 6 Comparison of input ground acceleration and deck acceleration time history responses (upper plot), relative displacement time histories of the bridge isolation system (middle plot) and pier displacement time histories (lower plot) of the semi-active control case under El Centro (NS dir.) earthquake excitation (normalized to 500gals).

Figure 7 shows the deck acceleration (upper plot), relative displacement of the bridge isolation system (upper middle plot), and command voltage to the MR damper (lower middle plot) and the MR damper force (lower plot) time history responses of the semi-active control case under El Centro NS earthquake excitation (normalized to 500gals). The deck acceleration and (A_Dl) and relative displacement of the bridge isolation system (Dmr) are used as feedbacks in this semi-active control case. The command voltage is varied with feedbacks and the damper force (LMR) is changed according to the command.

![Figure 7](image2.png)

Fig. 7 Deck acceleration (upper plot), relative displacement of the bridge isolation system (upper middle plot), command voltage to the MR damper (lower middle plot) and the MR damper force (lower plot) time history responses of the semi-active control case under El Centro (NS dir.) earthquake excitation (normalized to 500gals).

**Conclusions**

The hybrid controlled bridge isolation system with rolling pendulum system and smart MR damper is developed, simulated and tested in this study. The rolling pendulum provides the suitable restoring force with almost no friction, while the smart MR damper provides the online adjusted hysteretic force. As results, all the hysteretic force is controllable during the excitation. It maximized the controllable range of the hybrid controlled bridge isolation system. Both the numerical and shake table test results show that, this hybrid controlled bridge isolation system can effectively reduce the deck acceleration responses with the consideration of limited isolation stroke. Also, it is adaptive to different kinds and intensities of excitations.

**References**


Earthquake Engineering Experimental Data Model and Virtual Laboratory


Abstract

This paper introduces the preliminary study on the developments of experimental data model and virtual laboratory system for the National Center for Research on Earthquake Engineering (NCREE). On the development of experimental data model, a draft data model was proposed and a web-based user interface has been prototyped. The draft data model defines the relationship among experimental data elements. A web-based testing user interface allows researchers to input the experimental data into the database and users to browse the data. On the development of virtual laboratory, 3D geometric models of the NCREE building major experimental facilities including reaction walls, strong floors, the shake table, cranes and lifters, are established. In addition, a laboratory touring system and the recurrence of a shake table experiment based on virtual reality technology are implemented.

Keywords: Experimental Data Model, Virtual Laboratory, 3D Computer Graphics, Laboratory Touring System, Recurrence of Experiments

Introduction

It commonly takes a lot of time, money, and manpower to complete a structural experiment in an earthquake engineering laboratory such as National Center for Research on Earthquake Engineering (NCREE). Therefore, there should be no doubt that the experimental data resulted from experiments are valuable assets of the earthquake engineering laboratory and the procedure and methodology of structural experiment are valuable as well. However, it is not easy to manage very complicated and heterogeneous experimental data to re-use and record experiment process to re-establish in the future.

The aim of this research is to integrate information technology between data management and virtual reality to manage typical experimental data and re-establish experiment processes at NCREE. The research results can be divided into three categories: (1) experimental data model and test user-interface, (2) laboratory 3D model and tour system, and (3) experimental result and process visualization.

Experimental Data Model

To address the issues about the archive, reuse, and sharing of experimental data, an effort is currently in progress at NCREE toward the development of an experimental data management system. The present focus of the effort is on the development of a good data model that can capture sufficient information for future reuse of experimental data and is effective enough to support data management for the experimental data management system.

A survey on international effort related to experimental data management is conducted at first. It is found that the reference NEESgrid data model

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proposed by Peng and Law (2004) has provided a comprehensive data model for supporting the major activities in the NEES community involved in earthquake engineering experiments and simulations. Furthermore, after some reviews and discussions on the reference data model by a group of NCREE researchers with expertise in either experiments or information technologies, a preliminary data model suitable for modeling typical experiment at NCREE prototyped in this work. Both the objected-oriented (Rumbaugh & Blaha, 2004) and the entity-relationship approaches (Chen, 1976) are employed in the design of the proposed data mode.

The NCREE preliminary data model divides experimental data into five fundamental classes, including Activity, Apparatus, ApparatusSetup, DataElement and ComplexType. Figure 1 shows the five fundamental classes of data model and their sub-classes. Figure 2 shows a sketch of class diagram of the data model. Both of Figs. 1 and 2 are shown in UML (Unified Modeling Language) representation standard. Figure 3 shows the entity-relationship (E-R) diagram of the data model and will be implemented into a relational database of a central management server in the near future.

To assure the sufficiency and effectiveness of the preliminary NCREE experimental data model, a testing web-based user interface has been implemented (see Fig 4). In the interface, a relational database is employed to realize the data model and web-based user interfaces are designed to reflect the information representation of the data model.

Virtual Laboratory Touring System

Earthquake engineering education is one of the the duties of the NCREE. In order to introduce earthquake engineering and NCREE to public, a laboratory touring system based on virtual reality technology is being developed in this work. Users
can visit the NCREE laboratory using in the virtual environment or follow a virtual touring agent, which works as a guide to introduce the experimental environment and the facilities step by step. Taking advantage of interactive virtual reality technology, this system allows users to operate some of the virtual facilities, such as controlling the virtual cranes interactively.

On the 3D modeling side, this research employs 3D Studio MAX 6.0 (Autodesk, 2005), which is a kind of 3D modeling software to create the 3D models, including, such as office block the NCREE building, laboratory, the ground, shake table, reaction walls, strong floor and cranes and so forth. In order to improve the visual reality, surfaces of these 3D models are textured with proper colors, patterns, or photos taken by digital cameras. In addition, this work divides 3D models into some geometrical blocks hierarchically for maintenance and extensibility.

On the establishment of virtual environment side, this research employs Quest3D 3.0d (Act 3D, 2005), which is a kind of virtual reality software, to build the virtual environment. Form Figures 5 to 8 show the 3D models of the NCREE building, shake table, reaction walls and beam a crane, respectively. The above facilities can be introduced by the touring agent with translucent dynamic text boards. Besides, the virtual environment simulates an interactive interface for users to operate the beam crane. The virtual environment with touring system offers more convenient and intuitive way for users to browse the NCREE laboratory and understand more on earthquake engineering researches.

**Figure 5 NCREE building 3D model**

**Figure 6 Shake table 3D model**

**Figure 7 Touring system (in Chinese) of reaction wall and strong floor**

**Figure 8 Crane 3D model with user operating interface**

**Experiment Recurrence System**

An experiment recurrence system was prototyped in this work to represent an experimental or numerical analysis result using 3D computer graphics. This system translates
numerical data from experimental measured data or numerical simulation results to a series of scripts for 3D Studio MAX, making the geometric models of the structure deformed dynamically. This system allows researchers, engineers or students to observe the dynamic responses in different view angles of their interests.

A numerical simulation of a tuned-mass-damper (TMD) controlled 3D frame structure is used as an example to demonstrate the recurrence system. In this example, the system shows four view areas (A, B, C and D in Fig. 9). They are introduced as follows:

Areas A and B: These two views represent the 3D frame structure and its dynamic response with and without the controlling components (TMDs in this case), respectively. Researchers or engineers can compare the effectiveness of the controlling components to the structure subjecting to a specific ground motion. Users can interactively change the view angles using the keyboard and the mouse.

Area C: This area plots a selected dynamic response, which can be the response of a sensor in an experiment or a monitoring point in a numerical simulation.

Area D: This area plots the input ground motion of the experiment or the numerical simulation.

The above four viewing areas are synchronized when showing dynamic animations and responses.

Summary

This work preliminary prototyped (1) a data model for earthquake engineering experimental data management and web-based interfaces to browse the experimental data; (2) a virtual touring system offering a friendly environment using virtual reality technology for visitors or students to browse the laboratory and to learn more on earthquake engineering researches; (3) an experiment recurrence system representing an experimental or numerical simulation result using 3D computer graphics.

At present, these three systems operate and are still being developed independently. Future work includes not only to further enhance the functionalities of these systems, but also to integrate them into an earthquake engineering experimental data management and virtual reality system. The vision of future work is to offer a user-friendly environment to manage the earthquake engineering experimental data which have been completed and to represent these data through a virtual reality approach, allowing researchers, engineers and students to learn state-of-the-art knowledge and technologies through this system.

Figure 9 Prototyped recurrence system

Figure 10 Flowchart of using the recurrence system

References


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Extension and Application of Nonlinear Structural Analysis Platform

Bo-Zhou Lin¹, Fang-Wei Hsu¹, Ming-Chieh Chuang¹, Ker-Chyuan Tsai², and Yuan-Sen Yang³

林柏州、許芳瑋、莊明介、蔡克銓、楊元森

Abstract

The emphases of this research are (1) continuing to develop and apply the existing environment of nonlinear structural analysis and extending PISA3D as an analysis engine for sub-structural pseudo dynamic testing, (2) improving the fundamental framework of PISA3D to construct a more flexible object-oriented framework, and developing the function of nonlinear pushover analysis, (3) adopting the latest computer graphics techniques to improve the 3D visualization efficiency of the pre/post-processor.

Keywords: Nonlinear structural analysis, pre-processing, post-processing, PISA3D, VISA3D, GISA3D, pseudo-dynamic testing, pushover analysis, 3D computer graphics, object-oriented technique

1 Introduction

Structural analysis using numerical simulation technique plays an important role in modern seismic design. Either in engineering or academic circles needs a feasible platform for nonlinear numerical simulation to update analysis models of advanced structural systems or seismic components, to support proper analysis and design, and to enhance academic researches. According to these demands, National Center for Research on Earthquake Engineering (NCREE) cooperates with National Taiwan University to develop a nonlinear structural analysis program PISA3D [1], a visualization post-processor VISA3D [2], and a graphical pre-processor GISA3D [3]. The emphases of this research are continuing to improve the software framework of these programs, to extend the libraries to assist users in efficiently nonlinear structural analysis, and to combine the latest computer graphing techniques and software module with the structural analysis pre/post-processor.

Extending PISA3D as an analysis engine for pseudo-dynamic testing

Pseudo-dynamic test is a popular testing method in structural laboratories. It only uses the equipments of static tests to simulate dynamic structural response. One of the achievements of this research in 2005 is to integrate PISA3D and ISEE testing platform (Internet-based Simulation for Earthquake Engineering) [4], so that it can be the analysis engine for sub-structural pseudo-dynamic testings. Figure 1 shows the architecture of CGM (Command Generation Module) and FEM engine (PISA3D) in PNSE (Platform for Networked Structural Experiments) of ISEE. The CGM-shell handles all the networked communication with the PNSE server. PISA3D is responsible for the computation of structural analysis. After getting the restoring force of the sub-structural specimen in a laboratory and combining it with analytical response of other structural part, PISA3D can compute all DOFs’ displacements of next step, and completes the step-by-step time integration for transient dynamic analysis. Being the analysis engine, PISA3D extends interface for setting specimen’s restoring force, getting specimen’s displacement, and starting or terminating analysis. At the same time, this research inherits a SpecimenBC class from the super class Element to represent the specimen in the laboratory. This class gets the measured restoring force from PNSE instead of calculating its internal force.

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Improving PISA3D framework and extending libraries

This work modifies the nonlinear element framework of PISA3D, including the Element class, Section class and Material class. This defines more realistic responsibilities and mutual relationships among these classes, and improves the numerical convergence in nonlinear analyses. After the upgrade, the programming flexibility of element nonlinear analysis codes is strengthened. Researchers can extend their research achievements, nonlinear elements, sections and materials into PISA3D’s libraries with high efficiency.

Besides the improvement in framework, a section class BCSection02 is developed. As shown in Fig. 2, the feature of this section is that one can specify different materials to different DOFs in a single section. The element using this section can perform different hinge properties so that user can define more detailed behavior and strength for different hinges.

Development of GUI engine library for finite element structural analysis

This research develops a graphical user interface (GUI) engine library and a GUI for a new generation finite element structural analysis program. For future usability and extensibility, the program emphasizes to construct a more flexible object-oriented framework by ameliorating the fundamental framework. Furthermore, to improve the efficiency and extensibility of 3D rendering, NCREE and a software company, AnCAD Inc., collaboratively developed post-processing module including “Visualization Layer” and “Graphing Technique Layer.” According to the demand of visualization for structural analysis, Visualization Layer processes the analyzed data from structural analysis programs. Graphing Technique Layer provides the functions of resizing, rotating, zooming, moving, animation recording, value picking, object picking, translucency setting, color mapping, and so forth. In the design of the framework of extensibility, Graphing Technique Layer provides a “Basic Graphing Layer” drawing basic figure units by using MATFOR [6] (a product of ANCAD Inc.), a “Model Display Layer” and an “Element Display Layer” processing respective C++ class module, and an “Application Program Layer” using Borland C++ Builder to develop user interfaces.

The aforementioned post-processing module can be supported after the fracturing point C. Figure 5 is a simple steel frame analyzed by PISA3D. The story height is 3.6m, cross sections are W4×13, and the fracture material is adopted to specify the hinge properties. After pushover analysis in PISA3D, the hinge distribution while roof displacement reaches 1m is shown in Fig. 5(a), and the frame response of the entire pushover is shown in Fig. 5(b).
statically and dynamically display un-deformed geometric model of a structure, deformed shape of a structure, modal shapes of a structure, distribution of plastic hinges, and deformed shape-plastic hinges (Fig. 6). In addition, the program is also capable to provide the visualization of load distribution (Fig. 7), section of elements (Fig. 8), and color distribution of stress/strain/force (Fig. 9). Users also can look up the analytic results by database system (Fig. 10).

![Fig. 6 Visualization of deformed shape and plastic hinges.](image)

**Fig. 6 Visualization of deformed shape and plastic hinges.**

![Fig. 7 Visualization of load distribution.](image)

**Fig. 7 Visualization of load distribution.**

![Fig. 7 Section of elements.](image)

**Fig. 7 Section of elements.**

![Fig. 9 Color distribution of stress.](image)

**Fig. 9 Color distribution of stress.**

**Development of GUI for the finite element structural analysis program**

The project also focuses on the enhancement of VISA3D, which is a GUI for PISA3D to assist PISA3D’s user to verify model and examine analysis results. Obviously we can shorten the time spent in operating PISA3D via VISA3D. Following VISA3D, the authors developed a new GUI named GISA3D [3] for PISA3D. GISA3D (Fig. 11) not only offers parts of service of post-processing adopted from VISA3D but supports a visual pre-processing function assisting users in constructing structural models. Based on Borland C++ Builder and OpenGL graphical technology, GISA3D provides a basic interface function, allowing users to define grids for building a model. It also supports easy “snap-to” grid for laying out elements. GISA3D facilitates the model maintainability and reduces the effort of model generation.

The Smalltalk-80 Model-Viewer-Control (M-V-C) framework [8] is an important concept employed in the development of GISA3D. The M-V-C framework provides a classical and well-known example for the architecture of GUI system. Based on Borland C++ Builder environment, we completed the windows application by Borland’s VCL (Visual Component Library) and enhanced GISA3D graphic performance via OpenGL. Once we adopted the VCL framework, the framework restricted us the idea of design about the relationship between Viewer and Controller. The Model is the only part in M-V-C framework that we can modify. The Model means the data information of display. Changes of requirement about Model are considered in the exploratory study and authors propose object-oriented analysis and design by which GUI’s framework becomes true object oriented. It is an important issue about maintainability in the future besides runtime efficiency.

At the same time as development of GISA3D, NCREE executes an experiment of 2-story BRB frame [4]. The specimen is the substructure of the 2 story BRB frame and is subjected to bi-directional earthquake loads. The experiment is more complicated than other common ones, so we convey and modify
original GISA3D in order to provide the function of simultaneous 3D display in the NCREE’s PNSE (Fig. 12). Simultaneous display supplies more intuitive and easy way to represent the current experimental state. Undoubtedly, it is the best evidence to proof GISA3D is “reusable”.

As we all know, GISA3D originally adopts OpenGL to expedite the graphical efficiency and ability but now MATFOR [6] provides us an alternative solution. and its functions in the libraries enhance our program with dynamic visualization capability and speed up the development process.)Through the collaborative project [7] with ANCAD, MATFOR is specially enhanced to facilitate the visualization for the application of structural engineering. It is a more easy way to implement these functions (i.e. mouse dragging, pointing and selecting in 3D space) in GISA3D via MOTFOR, so we will replace OpenGL with MATFOR in the next GISA3D version and provide more friendly functions for PISA3D’s users in the future.

Conclusions and future developments

This research continues to develop a nonlinear structural analysis platform which is feasible for both engineering and academic circles. This research also employs the object-oriented framework to integrate the software with analysis demands in the lab, and provides distinct interface for libraries extension to perform re-use of existing software components. The completed improvements in the framework and function help future developments on nonlinear libraries and numerical methods. By cooperating with a software company, pre/post-processor adopts advanced 3D graphic techniques capable of increasing graphics performance and efficiency. In the future, this research will keep developing more integrated and efficient graphic user interfaces.

References

Construction and Application of Earthquake Engineering Knowledge Base (II)

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Abstract

The long-term goal of this study is to develop a knowledge management system for National Center for Research on Earthquake Engineering (NCREE) in order to promote the sharing and effective use of the knowledge generated and accumulated from the NCREE researches. This study first takes advantage of Internet technology, which can distribute information in a quick and timely fashion, to establish the knowledge portal and knowledge application systems for NCREE. Two topics are selected as this year’s focus on development of NCREE knowledge base and its related knowledge application systems. The first topic is on seismic design for buildings. A seismic design knowledge website and a web-based forum for seismic design knowledge community have been constructed. The second topic is on management of the monitored structural strong motion data collected from a set of sensor-equipped structures in Taiwan. A web-based strong motion information management system has been developed.

Keywords: knowledge management, knowledge portal, topical knowledge website, knowledge community, seismic design, structural strong motion monitoring

Introduction

Since established in 1990, National Center for Research on Earthquake Engineering (NCREE) has generated an amount of valuable research results. These research results are generated from large-scale structural experiments; innovating experimental technologies; academic research on earthquake engineering; design, evaluation, and retrofit of structural earthquake resistance; and simulation and evaluation of earthquake disaster. If these research outcomes can be managed, presented, and disseminated effectively, the NCREE knowledge can be extensively applied; then the newer and more valuable knowledge of earthquake engineering can be derived to contribute to the society. Therefore, how to effectively manage and disseminate NCREE knowledge is the goal here we strive for.

For quickly and timely disseminating knowledge, the internet is no doubt the best helper for knowledge management. So developing an internet or network platform for NCREE knowledge management is one of the most important objectives this project hopes to achieve. To interact effectively with users of NCREE knowledge, the NCREE knowledge management platform (see Fig. 1) provides NCREE knowledge portals, topical knowledge application and community websites. We expect that these functions can offer users explicit knowledge and help them share their implicit knowledge or experience. And at the same time, the NCREE knowledge management platform could serve as a helpful tool for people who are interested in acquiring the knowledge of earthquake engineering, and hopefully, boosts that knowledge’s evolution.

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There are many aspects in the field of earthquake engineering. In order to match NCREE’s objectives, this study focuses on NCREE’s current research results related to seismic design knowledge and accumulated knowledge on management of strong motion data.

In the year of 2005, this study has established a NCREE knowledge Internet portal, a topical website and knowledge community forum for seismic design, and constructed a database for monitored strong motions in a selected set of Taiwan’s structures.

Prototype of NCREE Knowledge Internet Portal

A portal provides the single interface for user to find information and application systems he or she needs. In order to be effective, a portal should be designed according to two principles: minimization of complexity for accessing information; and simplification on the usage of application systems.

In accordance to the two principles, a portal in general offers the following functions: (1) searching, publishing, subscribing, and categorizing web-site information; (2) integrating information and the application systems; (3) providing personalized information; and (4) effective navigation of the web-site information [1]. As shown in Fig. 2, this study has established a prototype for NCREE knowledge Internet portal. This prototype currently has supported some of the functions mentioned above, namely, information publishing and subscribing.

As to portal information publishing, knowledge user can acquire through this portal the information of all NCREE publications and related papers (See Fig. 3). On the other hand, through this portal, knowledge users can subscribe the NCREE newsletter; and then receive the most updated news of NCREE that inform the related activities. In such a situation, the NCREE newsletters will be automatically e-mailed to users’ mailboxes. Evidently, this kind of subscribing mechanism will guarantee a more effective dissemination of information than just posting news on any related website.

Seismic Design Knowledge Application Website and Knowledge Community Forum

This study cooperates with the project sponsored by the Construction and Planning Agency of Ministry of the Interior (Establishment of Knowledge Map and Knowledge Sharing Standard for Construction Industry) to establish knowledge websites for application of topical knowledge maps. In 2005, two topics are focused for establishing the topical knowledge maps and websites: seismic design for buildings and education of disaster precaution. In order to reinforce the content of the knowledge base, this study continues to collect more knowledge information for the topic on seismic design of buildings.
Nevertheless, since collecting every possible piece of seismic design knowledge, including seismic design specifications, are very valuable for earthquake engineering community, this study developed an XML schema for the seismic design specifications in 2004. In 2005, according to this XML schema, this study further establishes XML documents for the seismic design specifications and commentary of buildings [2].

Besides, this study uses the technology of eXtensible Style Language Transform (XSLT) [3] to support different scenarios of applying the same knowledge content. As shown in Fig. 4, and Fig. 5, users can search and apply the same specifications according to either the design procedure or the consequence of the keyword search.

In addition, this study establishes a knowledge community forum for researchers and engineers to discuss all matters related to the seismic design of buildings (see Fig. 6). In the future, we hope to support functions for users to discuss their problems with references to the related items in the on-line seismic design specifications. And such discussion could be accumulated and extracted as new knowledge for future revision on the seismic design specifications.

Management of Monitored Structural Strong Motion Data

Earthquake Strong Motions is an important research topic in the domain of earthquake engineering. Many researchers are interested in analyzing the behavior of structures that are monitored during the occurrence of an earthquake strong motion.

In Taiwan, many important buildings and bridges are monitored by the Central Weather Bureau (CWB). This study cooperates with CWB to address the needs of CWB in managing a huge amount of response records of its monitored structures, as well as NCREE researchers’ needs in applying those records in their research analyses. This study establishes a web-based system for managing the monitored data, in which nearly all information researchers often need are integrated. The information includes the monitored data, free-field strong-motion records, event information, basic information of the monitoring system, and structural design information of the monitored structures.

Figure 7 shows the portal of the management system. Through this portal, users can acquire strong-motion data by either describing the earthquake event (see Fig. 8) or choosing a particular station (see Fig. 9). The strong motion data includes the original acceleration history (see Fig. 10), the histories of accelerations, velocities, and displacements with and without baseline correction; and the Fourier analysis results of the strong-motion data. Besides, by choosing the monitoring station, users can acquire the details of the monitoring station, including the pictures of the station, the setting of the monitoring console, and the allocation of the monitoring sensors (see Fig. 11).

Conclusions

The NCREE knowledge Internet portal and the Taiwan structural strong motion data management system have been completed in this study. For the
seismic design, this study continues to reinforce the content of the knowledge base. In addition, this study develops an XML version of the seismic design specifications and commentary of buildings and prototypes the application website for the specifications.

Acknowledgement
The authors would like to thank the research team of the Yen Tjing Ling Industrial Research Institute of the National Taiwan University and the officials of the CWB earthquake center for their support and collaboration on the related cooperative research topics for this work.

References
The Engineering Geological Database for Strong Motion Stations in Taiwan

Kuo-Liang Wen¹ and Hung-Hao Hsieh²

Abstract

More than 650 seismic stations all over Taiwan have been installed by the Central Weather Bureau (CWB) to record the ground motion data. In order to obtain the geological conditions and soil profiles of these strong motion stations, a site investigation project was established by the National Center for Research on Earthquake Engineering (NCREE) and CWB in 2000. The site investigation mainly consists of three parts: the basic description of a site, the on-site boring, and the Suspension P-S Logger technique which is used to determine the P and S wave velocities of the stratum at various depths. The suspension P-S Logger technique, using a single down-hole probe with one source and two receivers, allows continuous measurements of wave velocities with high resolution.

There are 26 seismic stations, which had been investigated in 2005. With reference to Kyoshin Net in Japan and ROSRINE in USA, a preliminary engineering geological database for 260 seismic stations investigated during 2000~2004 has been constructed on NCREE’s website for convenient accession.

Keywords: Geological, Database, Wave velocity, P-S Logger

Introduction

Taiwan is located on the Circum-Pacific seismic belt which is the most active seismic region in the world. Preventing severe losses of lives and properties from large earthquake is a major concern for the people in this region. The Taiwan Strong Motion Instrumentation Program (TSMIP) was initiated by CWB in 1991 to monitor the ground motions at over 650 free-field stations around Taiwan. Once a major earthquake happens, all the records of ground motions from TSMIP provide useful information for the operation of hazard mitigation. The ground responses monitored by seismographs reveal the characteristics of ground motions in different geological conditions which can be used to improve the design spectrum and the building codes of current use.

More than 1,000 seismic stations have been installed in Japan to monitor the ground response during earthquake. Users can download the data of ground response on a web site called “Kyoshin Net”. The basic information of a station site, the physical properties of soils, and the wave velocity of the stratum measured by the down-hole velocity logging technique are also available on the Kyoshin Net. After 1994 Northridge earthquake, a project called “Resolution of Site Response Issues from the Northridge Earthquake”, ROSRINE, has been activated to study the site response in the USA. Users also can access to a web site to download the geological information and the wave velocity profile of a station site.

The distribution of seismic stations in Taiwan is the densest in the world, although the amount of seismic stations installed by CWB in Taiwan is less than that in Japan and USA. However, the application of earthquake data would be restricted without a complete geological database. Therefore, NCREE and CWB collaborated to perform the site investigation to obtain the basic soil properties and the wave velocity of the stratum in 2000. There are 26 seismic stations which had been investigated in 2005 is shown in Figure 1. These station names are shown in Table 1. There are 286 seismic stations, which had

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² Assistant Research Fellow, National Center for Research on Earthquake Engineering.
been investigated during 2000–2005, and are shown in Figure 2. With reference to Kyoshin Net in Japan and ROSRINE in USA, a preliminary engineering geological database for the 260 seismic stations investigated during 2000–2004 has been constructed on NCREE’s website for convenient accession. The engineering geological database established on a GIS web site provides convenient access for researchers in earthquake engineering.

The local site conditions play an important role in the ground response during earthquake. Different site conditions could induce amplification or deamplification at different period ranges in the response spectra. It is called the site effect. Besides, in the seismic hazard analysis, the motion at bed rock of a site is predicted by the attenuation law from the earthquake source. According to the 2000 Uniform Building Code (UBC), 1997 National Earthquake Hazards Reduction Program (NEHRP) provisions in the USA, and the revising earthquake-resistant codes in Taiwan, the ground motion at free field is evaluated by the response at bed rock times the coefficient of site effect. The coefficient of site effect is related to the magnitude of earthquake and the local site conditions. Thus, a complete geological database is essential to the evaluation of site effect for earthquake engineering.

Table 1. The seismic stations investigated in 2005.

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Suspension P-S Logging Technique

The wave velocity profile at a site can be measured by various conventional methods including the up-hole, down-hole, and cross-hole logging techniques. The source and the receiver in those logging techniques are separated in different units. For the up-hole and the down-hole logging techniques, the distance between the source and the receiver is varied at different measuring points. The Suspension P-S Logging Technique, developed by the OYO Corporation in Japan, is used in this project to measure the primary wave velocity \( V_p \) and the shear wave velocity \( V_s \) of the stratum. The source and the receiver of this measuring system are integrated into a single probe within a short distance. Therefore, the wave velocities of the stratum can be measured continuously and precisely.

The borehole was first drilled at an interest site and filled with water. If the borehole is not well-standing, the borehole may be lined with a plastic tube. The probe was then put into the borehole at a specified depth. A primary wave or a shear wave could be generated by the source in the probe. The primary wave would be propagated through the surrounding soil in the direction perpendicular to the borehole axis.
The shear wave would be propagated through the soil along the vertical direction. Each receiver consists of a hydrophone for receiving the primary wave and a geophone for receiving the shear wave. A normal pulse and a reverse pulse are triggered by the source in order to check the signals received by two receivers. The time histories of those received signals should be in the same shape but with 180 degrees of the phase difference, since the two shear waves were propagated through the same soil media.

Typical measured signals of the primary waves and the shear waves from the logging computer are shown in Figure 3, where $H_1$ and $H_2$ represent the signals received by the upper receiver in normal and reverse directions, $H_2$ and $H_2$ represent the signals received by the lower receiver in normal and reverse directions, $V_1$ and $V_2$ represent the signals received by the upper and lower receivers, respectively. From the time histories of $H_1$ and $H_2$, the first arrival time for the upper receiver and the lower receiver could be picked as $t_{s1}$ and $t_{s2}$. Since the distance between the two receivers is 1 m, the shear wave velocity could be determined as:

$$v_s \text{ (m/sec)} = \frac{1}{t_{s1} - t_{s2}} \quad (1)$$

Similarly, the primary wave velocity is:

$$v_p \text{ (m/sec)} = \frac{1}{t_{p1} - t_{p2}} \quad (2)$$

![Figure 3. Typical measurements from the Suspension P-S Logging System](image)

**Engineering Geological Database**

There are three major items in the Engineering Geological Database in Taiwan. The first item is the general information of the station site, including latitude and longitude of the station site, ground water level, geographical/topographical conditions, and surrounding structures. The second item is the physical properties of soils. The SPT-N value, water content, unit weight, soil classification, and grain size distribution are obtained by on-site boring, sampling, and laboratory testing. After the borehole was drilled, the Suspension P-S Logging Technique was used to measure the wave velocity of the stratum in depth for every 0.5 m. The wave velocity of the stratum is an important index for site classification, so it is selected as the third item in the database. If the geological condition of the station site is classified to the rock outcrop, only the general environmental investigation was performed to collect the basic information of the station site.

This project has been conducted for six years. Till now, the site investigations at 286 station sites were completed, including 52 stations in 2000, 65 stations in 2001, 49 stations in 2002, 54 stations in 2003, 40 stations in 2004, and 26 stations in 2005. The stations are located on the alluvial deposit, gravel or even rock sites. All the results are summarized on NCREE’s website. As shown in Figure 4, the general information for station TTN023 (the photo of the seismograph, the plan section and the cross section of the surrounding environment), the soil profile, the SPT-N value, the shear wave velocity, and the primary wave velocity of the stratum are all available on NCREE’s website.

![Figure 4. The information for station TTN023 in the database shown on NCREE’s website. (a) The soil profile, SPT-N value, and wave velocity profile. (b) The photo of the seismic station in the field. (c) The description of the plan section and the cross section in the field.](image)
In the 1997 UBC and 1997 NEHRP provisions in the USA, the average of the shear wave velocity for the top 30m of soils is used as an index for the site classification. In the site classification of Taiwan free-field strong-motion stations, the site conditions are classified as class B (rock), class C (soft rock or very dense soil), class D (stiff soil), and class E (soft soil) according to the geological age, rock type, and the average of SPT-N values for the upper 30m of the stratum. With detailed subsurface soil profile and quantitative soil properties (SPT-N values and wave velocities) on a station site, the site effect of ground motions could be thoughtfully analyzed for a certain class of site conditions. Engineers may evaluate appropriate peak ground acceleration for the earthquake-resistant design of structures. According to the average of the shear wave velocity for the top 30m classified code (Table 2), the 26 seismic stations, which were investigated in 2005, should be classified. The classification is shown in Table 3.

Table 2. The shear wave velocity for the top 30m classified code (1997 UBC and NEHRP provision).

<table>
<thead>
<tr>
<th>Classification</th>
<th>The average of the shear wave velocity for the top 30m (V₃₀)</th>
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<tr>
<td>A</td>
<td>V₃₀ ≥ 1500 m/sec</td>
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<td>B</td>
<td>760 m/sec ≤ V₃₀ &lt; 1500 m/sec</td>
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<td>C</td>
<td>360 m/sec ≤ V₃₀ &lt; 760 m/sec</td>
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<tr>
<td>D</td>
<td>180 m/sec ≤ V₃₀ &lt; 360 m/sec</td>
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Table 3. The classification of 26 seismic stations which investigated in 2005.

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</table>

Conclusions

The site investigation at 286 TSMIP stations was completed by NCRREE cooperating with CWB in Taiwan. By sampling soils in the borehole and using the Suspension P-S Logger Technique, specific geological and geotechnical data are obtained including the soil profile, the physical properties of soils, and the wave velocities of the stratum. All the results of investigation are systematically organized in the database available on a preliminary web site. This project will be continuously performed in the following years. Combining with the GIS technique, the engineering geological database for strong motion stations in Taiwan will be more convenient for web querying. If an engineering project site is close to the strong-motion station, engineers may retrieve the geological and geotechnical properties of soils from the database for evaluating the ground response at the site. This database is helpful to the site effect analysis and the earthquake-resistant design.

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Ground-Motion Time Histories for Seismic Design
Wen-Yu Chien¹, Yu-Wen Chang² and Juin-Fu Chai¹

Abstract
This study collects a proper earthquake database and proposes a procedure to select ground motions for seismic design and analysis. Based on the reliability theory, the time-predictable characteristic earthquake model is used to calculate the hazard contribution for the Type I active faults. The analyses of aggregation and de-aggregation of the seismic hazard are performed to generate maps of the magnitude and distance for the controlling earthquakes. More than 200 ground motion records are selected and grouped to properly reflect the magnitude and distance of the controlling earthquakes for each site. The criteria to select ground motion records from the earthquake database are based on the corner frequency of the spectrum shape. The study of generating spectral compatible time history is also discussed.

Keywords: hazard de-aggregation, controlling earthquake, spectral compatible time history,

Introduction
The new version of Taiwan Building Code provisions was issued in July 2005. According to these seismic design provisions, the engineers must use ground motion time histories to process the dynamic response analyses for specific structures such as large scale structures, high-rise buildings, and structures using isolation system and/or damping devices. The ground motion time histories for design must reflect the seismic hazard potentials. All the earthquake sources, faults, and source effects must be taken into account for selecting time histories refer to the design earthquake. These provisions imply that the ground motion used for structural design must be hazard- consistent and site-dependent. A scaling criterion is also provided by the provisions to properly scale the ground motions. The spectral compatible time histories can also be used as the input for dynamic response analysis.

The TSMIP strong motion array collected a lot of earthquake data that can provide a large amount of earthquake database for the seismic design and analysis. However, the criteria for selecting strong ground motion are still problems for engineering practices. Collecting ground-motion records of large PGA from the large magnitude earthquakes do not guarantee the proper accelerograms for seismic design. The objective of this study is to collect a small amount earthquake database that could reflect the seismic hazard potentials of different locations for seismic design. The indices related to the response spectrum of the ground motion are provided so that engineers can select most proper ground-motion for seismic design and analysis.

Hazard Model for the Type I Active Faults
The study work of paleo-earthquakes investigation performed by Chen (Chen, et al., 2005) showed that the rupture of the Type I active faults in Taiwan followed the occurrence-time-predictable earthquake model. Based on these results of the paleo-earthquakes investigation and the damaging earthquake data related to these active faults, the occurrence-time-predictable characteristic earthquake model is used to model the Type I active faults for the hazard analysis and de-aggregation analysis. Let the \( f(t) \) denotes the probability density function of the recurrence interval of the failure of fault(rupture and cause earthquake), the probability of the fault surviving up to time \( t \), counted from the last occurrence of rupture, can be express as,

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\[ R(t) = 1 - Q(t) = 1 - \int_0^t f(\xi) \cdot d\xi = \int_0^t f(\xi) \cdot d\xi \]  

where, \(Q(t)\) is the failure distribution function of the fault. The hazard function is defined as

\[ \gamma(t) = \frac{f(t)}{R(t)} \]  

If the elapse time of a time-predictable active fault is given as \(T_e\), the hazard analysis is trying to estimate the occurrence probability of the fault during the next \(T_p\) years. For most structures the design earthquake is estimated based on the structural life-time. In this case, the \(T_p\) is set as the structural life-time and the seismic hazard due to the active fault can be estimated by the engineering reliability theory.

Giving a condition that the fault has survived during \((0, T_e)\), the probability of its failing during \((T_e, T_e + T_p)\) is called the posteriori failure probability,

\[ Q(T_p) = \frac{\int_{T_e}^{T_e+T_p} f(\xi) \cdot d\xi}{\int_{T_e}^{\infty} f(\xi) \cdot d\xi} = 1 - \frac{R(T_e + T_p)}{R(T_e)} \]  

\[ = 1 - \exp\left[ - \int_{T_e}^{T_e+T_p} \gamma(\xi) \cdot d\xi \right] \]  

The Log-normal distribution is assumed to represent the probability density function of the earthquake recurrence interval of the active fault as shown in Figure 1. The axis of time is normalized by the expect value \(T_e\) of the earthquake recurrence interval. With regard to the report of the paleo-earthquakes study performed by Chen (Chen, et al., 2005), the cumulative probability data of the CheLonPu fault which cause the 1999 Chi-Chi earthquake are also shown in Figure 1 for comparison. These data show good match with the assumed probability distribution function.

### Determination of Controlling Earthquakes

The assumption of Poisson process is used to calculate the probability of exceedance of structural life-time for the area source of earthquake. The life-time hazard due to the active is also calculated by the Eq.(3). The procedure of de-aggregation of the seismic hazard described in the appendix C of NRC Regulatory Guide 1.165 was modified to determine the controlling earthquakes for Taiwan area. The de-aggregation analysis is a seismic hazard based procedure which is able to summarize the contribution of individual magnitude and distance ranges to the seismic hazard. The magnitude and distance values of the controlling earthquakes of the design level can be found for each study site.

Based on the geologic, tectonic structure, subduction model and seismological information, this study selected four most representative seismogenic zoning schemes. A grid size of \(0.25^\circ \times 0.25^\circ\) is used to discretize the earthquake sources for each zoning scheme. The hazard contributions of individual magnitude ranges \((m_i, m_i + \Delta m)\) are calculated for each earthquake source. The expected magnitude of the magnitude range \((m_i, m_i + \Delta m)\) is expressed as

\[ m_i = m_k + \frac{1}{\beta} \log \left( 1 - \exp(-\beta \Delta m) \right) \]  

Where, \(\beta\) is the model parameter of G-R magnitude and recurrence relationship.

Referring to the return period of 475 years of the design earthquake, the analyses of de-aggregation of the seismic hazard are performed to generate the contour maps of the magnitude and distance of the controlling earthquake as shown in Figure 2. The main hazards are contributed from the large magnitude and short distance earthquakes. Based on these maps, The Taiwan area is divided into four sub-areas, and the ground motion database are collected for each sub-areas.

![Fig. 1: Comparison of probability density function(pdf) and probability distribution function(PDF) of the recurrence-time of active faults. The probability distribution data of the CheLonPu fault are denoted as “+” for comparison.](image)
Ground Motion Database

The TSMIP strong ground motion array collects many large magnitude earthquake data. The magnitude of controlling earthquake (Fig. 2) is greater than 6.7. But, it would be difficult to collect enough accelerograms for the basic ground-motion database. This study finally selects 12 earthquakes of magnitude larger than ML6.0 as shown in Table 1 to generate the ground-motion database. There are 200 ground-motion records with 3-components in the database. For each record, the indices such as the spectral values of the short period and long period, and the corner values of spectral shape are provided.

Spectral Compatible Time History

There are many procedures for generating the spectral compatible time history. Both the time- and frequency- domain procedures are studied to generate the artificial accelerogram as shown in Figure 3. The response spectrum can fit the target response spectrum very well. The artificial accelerogram can also be generated from the phase spectrum generated from the database (frequency-domain procedure). Both of the time- and frequency- domain procedures could generate artificial accelerogram well. But, the velocity of velocity time history is too large compared with the real one. Therefore, the PGV of the artificial velocity time history can not be used for design. The study on the relationships between PGV and PGA are performed for hard and soft site.

The case studies for earthquake data collected from the Taipei basin show a special relationship between the PGV and PGA as shown in Figure 4. For earthquakes of magnitude greater than ML6.8, large PGV values could be experienced due to the basin effects. In general, a linear relationship similar to the one of the soft site can be applied in Taipei for the case of large PGA (PGA>0.2g). For the case of small PGA (PGA<0.15g) the ratio of PGV/PGA is much larger. The contour maps of the ratio of PGV to PGA are generated for different design earthquake levels for Taipei basin. The relationships between the PGV and PGA should be used to check the artificial velocity time history. Further work is still necessary for generating artificial ground-motion time history which can also fit the PGV values.

Table 1 : Earthquakes collected for the study

<table>
<thead>
<tr>
<th>Origin Time (UT)</th>
<th>Epicenter</th>
<th>Dep. (km)</th>
<th>ML</th>
</tr>
</thead>
<tbody>
<tr>
<td>1999/09/20</td>
<td>23°51.12’</td>
<td>120°48.96’</td>
<td>8.0</td>
</tr>
<tr>
<td>2000/06/10</td>
<td>23°54.06’</td>
<td>121°06.6’</td>
<td>16.2</td>
</tr>
<tr>
<td>2000/07/28</td>
<td>23°24.66’</td>
<td>120°56’</td>
<td>7.3</td>
</tr>
<tr>
<td>2001/06/13</td>
<td>24°22.87’</td>
<td>122°36.4’</td>
<td>64.4</td>
</tr>
<tr>
<td>2001/06/14</td>
<td>24°25.13’</td>
<td>121°55.7’</td>
<td>17.3</td>
</tr>
<tr>
<td>2001/12/18</td>
<td>23°52.02’</td>
<td>122°39.1’</td>
<td>12.0</td>
</tr>
<tr>
<td>2002/03/31</td>
<td>24°08.39’</td>
<td>122°11.5’</td>
<td>13.8</td>
</tr>
<tr>
<td>2002/05/15</td>
<td>24°39.06’</td>
<td>121°52.3’</td>
<td>8.5</td>
</tr>
<tr>
<td>2002/05/28</td>
<td>23°54.78’</td>
<td>122°23.8’</td>
<td>15.2</td>
</tr>
<tr>
<td>2002/08/28</td>
<td>22°15.65’</td>
<td>121°22.3’</td>
<td>12.0</td>
</tr>
<tr>
<td>2002/09/16</td>
<td>25°06.09’</td>
<td>122°23.3’</td>
<td>175.7</td>
</tr>
<tr>
<td>2003/06/10</td>
<td>23°30.22’</td>
<td>121°42.0’</td>
<td>32.3</td>
</tr>
</tbody>
</table>
Conclusions

Based on the de-aggregation of seismic hazard analysis, this study collects a ground-motion database of 200 records, and suggests a procedure to select ground motions for engineering practices and studies. Adopting the time domain analysis procedure, these ground-motion records are modified to generate the spectral compatible time histories. The shape of accelerogram is good and acceptable. However, some values of PGV of the modified velocity time history are about two times of the original one. A further study on the artificial velocity time history must be performed in details.

References


Fig. 3: Comparison of accelerograms and spectra for the original and spectral compatible time-histories.

Fig. 4: The relationship between the PGV and PGA for Taipei basin.
Simulation of Near-fault Ground Motions at Layered Rock Sites

Juin-Fu Chai¹, Tsung-Jen Teng² and Wen-Shin Shyu³

Abstract

The objective of this study is to simulate the near-fault ground motions at layered rock sites. Based on the 3D quasi-dynamic model, in which the rupture and healing processes are taken into account, the space-time slip function of rupture points on the fault plane can be determined. In this study, the slip dislocation at each rupture point on the fault plane is taken for a double couple point source, and then, the induced wave field within an infinite space can be determined in the frequency domain through the solutions of scalar potentials which are expressed by the double spectral integration form. After that, in this study, a 3D layered half-space is considered to model the site conditions where a significant soft rock layer is over the semi-infinite hard bedrock. For this case, the aforementioned wave field within the infinite space can be recognized as the incident wave within the source layer (hard bedrock), and further, the associated wave field within the upper soft rock layer can be determined on the basis of the wave scattering theory for a layered half-space. Therefore, based on the slip function at each rupture point and the Green’s function due to a double couple point source, the total ground displacement can be obtained by the integration over the finite fault plane. Finally, the time history of the near-fault ground motion can be determined by the inverse Fourier transformation from the frequency domain to the time domain. Then, the associated structural responses spectra can be carried out to evaluate the impact of the site-specific near-fault ground motions.

Keywords: near-fault ground motions, site effect, layered half-space

Introduction

Near-fault ground motions, which have resulted in severe damages in recent disastrous earthquakes, are characterized by a short-duration impulsive motion that will transmit large energy into the structures at the beginning of the earthquake. For a near-fault site with soft rock or soil layers over the semi-infinite hard bedrock, due to the site effect, the duration of the near-fault velocity pulse will be elongated and its intensity will be amplified, and hence it will result in much more damages.

In this study, a 3D quasi-dynamic rupture model is adopted to generate the pulse-like near-fault ground motions (Chai, et al., 2004). The source time function can be defined directly without any time-consuming calculation by complicated numerical methods such as HBEM, and hence, this model is much easier than the spontaneous rupture model. However, both the rupture and healing processes which are essential to cause the slip pulses are considered in the proposed model, and hence the properties of a real dynamic solution can be still captured by the proposed quasi-dynamic rupture model.

A 3D layered half-space is considered in this study to model the site conditions where a significant soft rock layer is over the semi-infinite hard bedrock. The fault is defined within the bedrock, and the dislocation

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at each rupture point on the fault plane is taken for a double couple point source. The induced wave field within a 3D infinite space can be determined in the frequency domain by the solutions of scalar potentials which are expressed by the double spectral integration form, and it can be recognized as the wave field within the source layer (hard bedrock). On the other hand, the induced wave field within the upper soft rock layer can be determined on the basis of the wave scattering theory for a layered half-space. Thus, based on the slip function of each source point and the Green’s function due to a double couple point source, the total ground displacement can be determined by the inverse Fourier transformation from the frequency domain to the time domain, and further, the structural response spectra can be carried out to evaluate the impact of site-specific near-fault ground motions under the consideration of site effect.

**Site-specific Near-fault Ground Motions**

**Scalar Potentials of Incident Wave Field**

As shown in Fig. 1, a fault plane $\Sigma$ with a dipping angle $\delta$ is located in the half-space (hard bedrock) of a layered half-space. The global Cartesian coordinates $(x-y-z)$ system is defined such that the $y$-axis is parallel to the intersection of the fault plane and the ground surface. The free surface and the interface between the soft rock layer and the semi-infinite hard bedrock are defined by $z=-h$ and $z=0$, respectively, and $h$ is the thickness of the upper soft rock layer. Moreover, the local Cartesian coordinates $(\xi-\eta)$ system is defined on the fault plane with the origin coinciding with the hypocenter. The $\xi$-axis is along the slip direction, and the slip angle between $\xi$-axis and $y'$-axis (parallel to the $y$-axis) on the fault plane is defined by $\alpha$. Thus, based on the dipping angle $\delta$ and the slip angle $\alpha$, the rupture point $x_0=(x_0, y_0, z_0)$ on the fault plane can be expressed by the 2D local coordinates as

$$\begin{align*}
x_0 &= x_0 + \xi \sin \alpha \cos \delta - \eta \cos \alpha \cos \delta \\
y_0 &= y_0 + \xi \cos \alpha + \eta \sin \alpha \\
z_0 &= z_0 - \xi \sin \alpha \sin \delta + \eta \cos \alpha \sin \delta
\end{align*}$$

where $x_0=(x_0, y_0, z_0)$ denotes the hypocenter.

The slip dislocation of each rupture point on the fault plane can be recognized as a double couple point source with vanished net moment, and the individual moment is $M=\mu D_0$, where $\mu$ is the shear modulus and $D_0$ is the slip displacement. Based on the solutions of scalar potentials caused by a unit point source in a 3D infinite space, the incident scalar potentials $\phi_0, \chi_0$ and $\psi_0$ at $x=(x,y,z)$ caused by a double couple point source with unit spectral slip at $x_0=(x_0,y_0,z_0)$ on the fault plane can be solved in the frequency domain and expressed by the double spectral integration form as

$$\begin{align*}
\phi_0 &= \frac{1}{4\pi^2} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} A_x e^{i(x-x_0)\frac{\eta}{\nu} - i(y-y_0)\frac{\nu}{\delta} - ik_z z} dk_d k_y \\
\chi_0 &= \frac{1}{4\pi^2} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} B_x e^{i(x-x_0)\frac{\eta}{\nu} - i(y-y_0)\frac{\nu}{\delta} - ik_z z} dk_d k_y \\
\psi_0 &= \frac{1}{4\pi^2} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} C_x e^{i(x-x_0)\frac{\eta}{\nu} - i(y-y_0)\frac{\nu}{\delta} - ik_z z} dk_d k_y
\end{align*}$$

where $A_x, B_x$ and $C_x$ are the source coefficients, and can be expressed under the case of $z=z_0$ as

$$\begin{align*}
A_x &= -\frac{1}{2k_z^2} \left[ \sin \alpha \left( \sin 2\delta \frac{k_z^2 + \nu^2}{\nu} + \cos 2\delta \cdot 2k_z \right) \\
&\quad + \cos \alpha \left( \sin \delta \frac{2k_z}{\nu} + \cos \delta \cdot 2k_z \right) \right]
B_x &= \frac{1}{2k_z^2} \left[ \sin \alpha \left( \sin 2\delta \left( \frac{k_z^2 + k_\nu^2}{k_z^2} \right) + \cos 2\delta \cdot \frac{ik_z A}{k_z \nu} \right) \\
&\quad + \cos \alpha \left( \sin \delta \frac{2k_z}{k_z^2} + \cos \delta \cdot \frac{ik_z A}{k_z \nu^2} \right) \right]
C_x &= \frac{1}{2k_z^2} \left[ \sin \alpha \left( \sin 2\delta \left( -\frac{k_z}{k_z^2 + \nu^2} \right) + \cos 2\delta \left( -ik_z \right) \right) \\
&\quad + \cos \alpha \left( \sin \delta \frac{k_z^2 - k_\nu^2}{\nu^2} + \cos \delta \cdot ik_z \right) \right]
\end{align*}$$

Parameters $\nu, \nu'$ and $A$ are defined by

$$\begin{align*}
\nu &= \sqrt{k_z^2 - k_p^2} \quad \nu' = \sqrt{k_z^2 - k_s^2} \quad A = 2k_z^2 - k_p^2
\end{align*}$$

with $k_z^2 = k_p^2 + k_\nu^2$, and $k_p = \omega C_p$ and $k_s = \omega C_s$ are the longitudinal and shear wavenumbers, respectively, for a specified circular frequency $\omega$.

**Ground Displacement for a Layered Half-space**

In this study, a 3D layered half-space is considered to model the site conditions where a significant soft rock layer is over the semi-infinite hard bedrock. For this case, the aforementioned wave field in the infinite space can be recognized as the upward propagating incident wave in the semi-infinite source layer (hard bedrock). In addition, the wave field in the upper layer
can be also separated into the upward and downward propagating waves, and the general solutions of the associated scalar potentials can be express as

\[
\begin{align*}
\phi^i_L &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} A^i_L e^{i \pi x x_0} d\xi d\eta, \\
\phi^d_L &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} A^d_L e^{i \pi x x_0} d\xi d\eta, \\
\psi^i_L &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} C^i_L e^{i \pi x x_0} d\xi d\eta, \\
\psi^d_L &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} C^d_L e^{i \pi x x_0} d\xi d\eta.
\end{align*}
\] (5)

The superscript ‘L’ represents the wave in upper soft rock layer, and the subscripts ‘U’ and ‘D’ denote the upward and downward propagating waves, and the general solutions of the associated scalar potentials can be express as

\[
\begin{align*}
\phi^i_U &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} B^i_U e^{i \pi x x_0} d\xi d\eta, \\
\phi^d_U &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} B^d_U e^{i \pi x x_0} d\xi d\eta, \\
\psi^i_U &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} C^i_U e^{i \pi x x_0} d\xi d\eta, \\
\psi^d_U &= \frac{1}{4\pi} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} C^d_U e^{i \pi x x_0} d\xi d\eta.
\end{align*}
\]

Then, due to the condition of vanished normal stress on the free surface \((z = -h)\), we can obtain

\[
A^d_U = R^d_U A^i_U
\] (7)

Where \(R^d_U\) is defined as the reflection matrix at the free surface for the upward propagating wave in the upper soft rock layer. On the other hand, due to the continuity conditions of stress and displacement at the interface between the upper soft rock layer and the semi-infinite hard bedrock, we have

\[
A^d_U = T_d A_U + R^d_B A^d_U
\] (8)

Where \(T_d\) is the transmission matrix at the interface for the upward propagating incident wave from the hard bedrock, and \(R^d_B\) is the reflection matrix at the interface for the downward propagating wave in the upper soft rock layer. As a result, the upward and downward coefficient vectors of the scattered waves in the upper soft rock layer can be solved from Eqs. (7) and (8), and can be expressed in terms of the source coefficient vector as

\[
\begin{align*}
A^i_U &= \left[ I - R^d_B R^d_U \right]^{-1} T_d A_U, \\
A^d_U &= R^d_U \left[ I - R^d_U R^d_B \right]^{-1} T_d A_U.
\end{align*}
\] (9)

Then, the scalar potentials of scattered waves in the upper soft rock layer can be determined by Eq. (5). Subsequently, the associated displacement vector of the scattered wave field in the upper soft rock layer can be determined. By setting the observe point onto the free surface, the ground displacement vector \(u_{x0}\) at \(x=(x,y,-h)\) caused by a double couple point source with unit spectral slip at \(x=0=(x_0,y_0,z_0)\) can be solved.

Then, based on the slip function of each source point and the Green’s function due to a double couple point source, the total ground displacement can be obtained by the integration over the finite fault plane, and it can be expressed in the frequency domain as

\[
u_{x0}(x) = \int \overline{D}(x_0) \cdot \overline{u}_{x0}(x; x_0) d\xi d\eta
\]

and \(\overline{D}(x_0)\) denotes the spectral slip function at each rupture point, and can be determined by the Fourier transformation of the slip function \(D(\xi; t)\) that can be defined by the quasi-dynamic rupture model (Chai, et al., 2004), and the rupture point \(x_0=(x_0,y_0,z_0)\) can be expressed in terms of the local coordinates by Eq. (1). Finally, the time history of the near-fault ground motion can be simulated numerically by the inverse Fourier transformation from the frequency domain to the time domain. It is noted that the explicit form of all of the reflection and transmission matrices at the free surface and the interface, as well as the explicit form of the ground displacement components can be found in the published report (Chai, et al., 2005).

### Numerical example

A reserve slip fault \((\alpha=\pi/2)\) buried in the semi-infinite hard bedrock is considered in this example, the dipping angle and the focal depth of hypocenter are defined by \(\delta=40^\circ\) and 13.0 km. The 3D global coordinate system is constructed such that the \(z\)-axis passes through the hypocenter. The thickness of the upper soft rock layer is defined by \(h=1.0\) km, and hence the position vector of hypocenter can be defined by \(x_0=(0,0,12)\) km. In addition, the longitudinal and shear wave velocities of the hard bedrock are specified by 5.6 km/sec and 3.2 km/sec, and the values for the upper soft rock layer are 5.0 km/sec and 3.0 km/sec, respectively. The ground observation points are along the \(x\)-axis with the epicentral distances of 12, 14, 16, 18 and 20 km. It is noted that, due to the focal depth and dipping angle, the location with \(x=15.5\) km on the ground surface is the boundary between the hanging wall and the foot wall.

The red lines in Fig. 2 show the comparison of the near-fault ground motions under the consideration of a soft rock layer with thickness of 1.0 km. It shows that, according to the increasing travel distance, the arrival time and duration of the pulse will be delayed and elongated, and its intensity will be decayed. On the other hand, the pure half-space case is also considered for comparison, in which the upper soft rock layer is replaced by the same medium as the hard bedrock. The resulted ground motions are shown by the blue lines in Fig. 2. It can be found that the arrival time and
duration of the pulse on soft rock site will be delayed and elongated due to the smaller wave velocities in the upper soft rock layer, and further, the intensity will be amplified in accordance with the soft site effect.

Fig. 2: The near-fault ground motions at the observation points along the x-axis

The near-fault response spectra for the considered ground observation points are compared in Figure 3. Consistent with the pulse duration, the plateau range in the structural acceleration spectra will be extended, and the associated corner period will be increased to be larger than 1.0 second. Based on the ratio of the spectral response on the soft rock site respected to that on the hard rock site, the site amplification factor of the near-fault spectral responses can be determined, and Figure 4 shows the resulted amplification factor for the spectral response accelerations and velocities. It can be observed that the larger amplification factor can be resulted for the period range close to the pulse duration. In addition, caused by the directivity effect, the spatial variation of the amplification factor is in accordance with the growth and decline of the ground velocity pulse.

Conclusions

In this study, a 3D layered half-space is considered to model the site conditions where a soft rock layer is over the semi-infinite hard bedrock. Based on the numerical method developed in this study, the site-specific near-fault ground motions can be predicted. Therefore, for designing structures, the site-specific structural response spectra can be developed from the scenario earthquakes instead of the scarcely observed near-fault ground motions. In addition, the simulated site-specific near-fault ground motions can be applied as the input excitations for a time-history analysis to evaluate the structural earthquake performance under the consideration of not only the near-fault effect but also the site conditions.

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Investigation of Velocity Structures of Seismic Stations

Ding-Shing Cheng¹, Kuo-Liang Wen², Yu-Min Kang³ and Tao-Ming Chang⁴

Abstract

The objective of this project is to investigate the velocity structures of shallow and deep soil layers of 30 seismic stations in Chiayi, Tainan, Hsinchu and Miaoli region. Two kinds of methods were considered in this study. One used the stress wave propagation methods combining the SASW method and the impulse response method to obtain the S-wave velocity structure of shallow soil layers by applying a mechanical source. The stress wave propagation theory and the forward iteration process were used to determine the wave velocities and thickness of shallow layers. The other used the micro-tremor array methods, which natural surface vibrations are recorded by 10 recorders arranged in an array. The frequency-wave number analysis is used to identify the dispersion curve of soil layers, and the genetic algorithm search method is used to obtain velocity structures of deep layers by an inversion process. The results from the field tests showed that the wave velocity structure of shallow layers within 10 to 20 meters could be obtained by the stress wave propagation method, and the velocity structures of deep layers below 10 to 20 meters could be obtained by the micro-tremor array tests.

Keywords: Stress wave propagation, micro-tremor array, SASW, Impulse Response method

Introduction

In Taiwan the Central Weather Bureau has installed a lot of seismic stations, and in order to improve the usefulness of seismic data the Bureau has commissioned the National Center for Research on Earthquake Engineering to buildup a geology database for all seismic stations.¹ Accompanied with borehole works, the suspension P-S logging method [1] was used to investigate the velocity structures of soil layers beneath seismic stations. Due to the limitations of the suspension P-S logger, the velocity structure of soil layers within 5 m depth could not be measured correctly, and the properties below borehole depth could not be measured also.

In order to obtain full velocity structure of soil layers, a stress wave propagation method[2] combining the SASW method [3-5] and the impulse response method[6,7] and a micro-tremor array method[8-12] were used to improve the results of the suspension P-S logger. The stress wave propagation method is mainly for depth less than 10 to 20 m. It uses a mechanical manmade source producing high frequency stress waves. The amplitude of manmade ground vibration is greater than that of natural ground vibration. The SASW method was used to obtain the dispersion curves of surface waves, and a forward iteration was used to obtain the S-wave velocity structure of soil layers. The impulse response method was used to obtain the mobility of body waves, and a forward iteration was used to obtain P-wave velocity structures of soil layers. For depth greater than 10 to 20 m, the micro-tremor method was used. By recording natural ground vibration from 10 recorders arranged in an array with three concentric circles for a long time greater than one hour, a dispersion curve of soil layers could be obtained by a
frequency-wave number analytical method. The S-wave velocity structure of soil layers could be obtained by a genetic algorithm search method.

Velocity structure of soil layers of 30 seismic stations in Chiayi, Tainan, Hsinchu and Miaoli region had been obtained although 13 stations have no results of micro-tremor tests due to the limitation of space in test sites.

The Stress Wave Propagation Method

In order to provide P-wave and S-wave velocity structure of soil layers, the stress wave propagation method combining the SASW method and the impulse response method is used. The SASW method can provide the S-wave velocity structure, and the impulse response method can provide P-wave velocity structure. Two methods use a same mechanical manmade source shown in Figure 1. The SASW method use two receivers with a distance of 4, 8, 16 or 32m. The dispersion curve of a SASW test at the CHY012 station was shown in Figure 2. For the impulse response method, a vertical and a horizontal receivers jointed together were put 0.5 m away from the source. The mobility plot of the vertical receiver was shown in the Figure 3, and the mobility plot of the horizontal receiver was shown in Figure 4. After a forward iteration process, the velocity structure of the CHY012 station was shown in Figure 7.

The Micro-Tremor Array Method

Figure 5 schematically shows the field test setup of the micro-tremor array method. 10 recorders are arranged in an array with three concentric circles of maximum radius of 32 or 64 m. In each circle, three recorders were arranged in an angle of 120 degree. The three concentric circles has radius in an order of 2. The total time for measurement at one station should be at least one hour. After a frequency-wave number analysis of data, a dispersion curve for the CHY012 station was shown in Figure 6. By using an inversion process of the genetic algorithm search method, the velocity structure of the CHY012 station was shown in Figure 7

Conclusions

In this project 30 seismic stations shown in Table 1 were investigated by the stress wave propagation method and the micro-tremor array method except for 13 stations. For depths smaller than 10-20 m, the velocity structure provided by the stress wave propagation method is close to result of P-S logging. The result from the micro-tremor array method is close to the result of P-S logging at depth greater than 10 to 20 m. Therefore, a combination of the stress wave propagation method and the micro-tremor array method can provide a full dispersion curve and velocity structure of soil layers. If the location of test sites is big enough, the micro-tremor array method can be used for depth larger than 200 to 300 m.

References


Table 1. Tests conducted in seismic stations in Changhua region.

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Stress Wave</th>
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<tbody>
<tr>
<td>CHY007</td>
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<tr>
<td>CHY008</td>
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</tr>
<tr>
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<tr>
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</table>

Figure 1. test setup of Stress wave propagation

Figure 2. Dispersion curve of a SASW test

Figure 3. Mobility plot of vertical receiver from an impulse Response test
Figure 4. Mobility plot of horizontal receiver from an impulse response test

Figure 5. Test setup of micro-tremor array method

Figure 6. Dispersion curve from a micro-tremor array test.

Figure 7. S-wave velocity structure from a stress wave propagation test and a micro-tremor test
Earthquake Source Parameters and Micro-Tremor Site Characteristics (Micro Earthquake Monitoring)

Kuo-Liang Wen¹, Tao-Ming Chang², Yu-Wen Chang³, Hung-Hao Hsieh⁴ and Wen-Yu Chien⁵

Abstract

The research project is one of series studies which guiding by NCREE regarding to the high technology Science parks of Taiwan. The research goal is to obtain the vibration characteristic of site, the activities of surrounding faults, and provide as input parameters for the earthquake hazard mitigation programs. Also, the results can be integrated with other projects to form a comprehensive research report for government’s decision making. Three major methods used. First of all, a very dense microtremor survey, 259 measurements, has been performed for obtaining the dominate resonance frequencies throughout the Hsinchu science parks. Second of all, the seismic activities around the Hsinchu science park will be monitored by setting up a broadband seismic network consisting of 10 broadband seismometers. And hopefully get enough data in the following years to understand current status of the nearby Hsincheng fault. Third of all, a statistical method was used to produce a Shakemap for the Hsinchu science parks for a certain time frame. All results will be provided for analyze and improve the methodology for a better earthquake hazard mitigation purpose.

Keywords: Hsinchu science park, site effect, earthquake monitoring, hazard potential

Introduction

The Science parks of Taiwan have become the centers for many different types of industries. For a foreseeable future, Taiwanese corporations will choose their bases surround these parks, make their own global strategies, and perform global businesses energetically. But for these locations, earthquakes caused by active faults will be a potential threaten due to Taiwan’s tectonic activities. To possible reduce the losses during the shaking of strong earthquakes; it is necessary to a have a very good hazard-mitigation plan. So far, such a plan is not completely convinced because of lacking some critical factors. In this project, two factors will be produced for that purpose. First is to estimate the status of active faults near the science parks. Second is to measure the site effect of the science parks.

In 2005, the main study area is the Hsinchu Science Park. Using dense microtremor measurements, the sediment thicknesses and site dominate resonance frequencies can be estimated. In the future, these results will be included with the strong motion seismic station drilling results in a site-characteristic database and contributed in constructing large scale micro zonation maps. On the other hand, to monitor the activities of the Hsinchu and Hsinchen faults, a seismic network consisting of 10 broadband seismometers was set up in the study area. The seismicity monitoring will keep on going for at least three years; the collected data will be used to analyze the moving capabilities of the active faults and related source parameters. These source parameters and the status of faults are the most critical input factor for the earthquake hazard estimation programs (TELES) which is the major project by NCREE.
Dense Microtremor Measurements at The Hsinchu Science Park

Inside the Hsinchu Science Park, more than 80% are semiconductor production factories which are very sensitive to the ground vibrations. Therefore the ambient background noise of site, i.e. microtremor, was specially planned to perform throughout the Hsinchu Science Park to understand the site resonance frequencies. During 2005, a total 259 microtremor measurements were collected in the Hsinchu Science Park and its surrounding area. The measurement is very dense spatially; the average distance between every two measurements is 100 meters inside the Science Park, and 200 meters for the area surrounding the Science Park. The measurements are usually located on the road, the distribution map is shown as figure 1. The topography relief of study area is shown as figure 2. The measured microtremor data has been processed using the spectrum H/V ratio method (Nakamura, 1989). And then systematically identified the dominate site resonance frequency and pre-dominate site resonance frequency for every microtremor measurement. Thus the pre-dominate frequency map and the dominate frequency map of the Hsinchu Science Park can be plotted as figure 3 and figure 4. From figure 3, it is concluded that the geological layers beneath the study area are deformed as a serial folds with the axis parallel to north-south direction. The distance between each fold peak is roughly about 2 kilometers. This phenomenon is similar to the ground surface topography. From figure 4, the dominated frequency, it implies that the geology condition of the northeast part of Science Park (the south bank of Touchien River) is fairly simple. There are no strong reflectors between the basement and ground surface. But at the southwestern part of the Science park, there exist a strong reflector which produce a resonance frequency around 4.5Hz. The terminology “reflector” used here represents a strong contrast in velocity profile which will lead the incident energy from top easily reflect back to ground surface and produce a resonance peak in the H/V spectrum.

From the 1/50000 geological map of Hsinchu, published by Central Geological Survey in 1988, the northeastern side of the science park is the Pleistocene Tientzuhu formation (Laetrile, gravel and sand, intercalated sand and silt lentils); the southwestern side of the science park is the Pleistocene Toukoshan formation (sandstone, mudstone and shale). Therefore the microtremor measurements results are mostly reflect the geological transition between these two geological layers. So far, the dense microtremor survey with the velocity well-logging profile can be used to mapping the sediment thicknesses very well.

In the future, the microtremor measurement results can be shown as 1/3 Octave band for the frequency 1~100Hz which is an industrial standard used by most semiconductor equipment manufactures. These figures will be used as the basic information of the Hsinchu Science Park for evaluating vibration effects in the future.
Micro Earthquake Monitoring Network

Based on the observation experiences, there are several kind of precursors before the earthquake happen. These precursors may not be observable for every place, but should be investigated to ensure their statistical significance. In this project, both geophysical and geochemical methods will be used to monitor the same study regions to ensure their usability. We hope to clarify the advantages and drawbacks for each method, and establish a reliable methodology for future earthquake predictions.

Although CWB installed more than 30 strong motion stations in our study area, but there is only two velocity type seismometers. This is not enough to capture the small micro earthquakes. In our study area, there is no big earthquake since 1935 Hsinchu-Taichung earthquake, but only small earthquakes. We believe that if the micro earthquakes have been studied carefully we can get some important insight about the seismic zone. Nine high resolution broadband seismometers (Guralp 6TD) were purchased together with another one identical seismometer bought last year, to form a micro earthquake monitoring seismic network which was set up in the vicinity of Hsincheng fault to monitor earthquake activities and to understand their rupture mechanism. The location of each seismometer can be seen in figure 5.

Fig. 5 The micro earthquake monitoring seismic network is shown as black dots.

Seismic Hazard Potential of The Hsinchu Science Park

The seismic potential analysis is used to compute the possibilities for a target fault with particular parameters to produce a characteristic earthquake during a time frame.

The active faults near the Hsinchu Science Park include the Hsin-Cheng fault, the Shih-Tan fault. From the investigation reports published by the Central Geological Survey, the parameters of these active faults are listed in the following table.

<table>
<thead>
<tr>
<th>fault</th>
<th>Tr (yr)</th>
<th>Te(year)</th>
<th>ML</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hsin-Cheng</td>
<td>1000</td>
<td>300</td>
<td>6.3-6.8</td>
</tr>
<tr>
<td>Shih-Tan</td>
<td>300</td>
<td>71</td>
<td>6.4-6.9</td>
</tr>
</tbody>
</table>

The parameter $M_L$ shown on above table is the local magnitude of a characteristic earthquake which is the result from fault trenching investigation. Here assume the characteristic earthquake is repeatable which means its magnitude and recurrence time are predictable. In this research, the fault length, recurrence period are used to estimate the possibility for happening characteristic earthquake on active fault. The elapse time $T_e$ is estimated according to historical earthquakes. A detailed description for every used parameters can be found at (NCREE-05-032).

The possibility for happening hazard earthquake means the difficulties of happening a hazard earthquake in a particular region on a certain time frame. Assume the average characteristic earthquake ($E_m$) recurrences possibility of active faults in study region is a log normal distribution. Here the $P(T_e | E_m)$ represents the possibility density function which has the characteristic earthquake magnitude greater or equal to $E_m$ and has its recurrence period equals to $T_e$. Therefore, the possibility for happening a earthquake greater than magnitude $E_m$ during future $T_p$ years is $P(T_e + T_p | E_m)$; the possibility for happening magnitude greater than $E_m$ is

$$P(T_p | E_m, T_e) = 1 - \frac{1 - P(T_e + T_p | E_m)}{1 - P(T_e | E_m)} \quad (1)$$

Assume $\text{COV}(\sigma / \mu)$ has a value 0.5, the computed possibility distribution model of a particular fault is shown as figure 6. When $T_e=300$yr, from the surface area under the curve of possibility distribution model, the possibility for happen a big earthquake is 1.33% for the future 50 years ($T_p=50$yr). 71 years ago, in 1935, Shih-Tan fault ruptured and produce huge hazards. Therefore the possibility of happening a hazard earthquake on this fault for the future 50 years is 4.26%. The elapse time, which is the surface area under the possibility curve, will strongly affect the possibility of happening earthquake for the future $T_p$ years. When the elapse time $T_e$ approach the average recurrence period which represents the possibility of happening earthquake for any future time point become larger and larger. In this project, the potential for happening hazard earthquake for the future 10, 30, 50 years for two particular active faults near the Hsinchu Science Park were listed in the following table.

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After computation of characteristic earthquakes, the shakemap is shown as figure 6. The computed magnitude for Hsin-Cheng fault and Shih-Tan fault is $M_c6.5$ and $M_c6.8$ respectively.

Conclusions

From microtremor measurements, the site resonance frequencies were obtained for the Hsinchu Science Park which is very important for factory seismic-resistant design and hazard mitigation preparations.

Micro earthquake monitoring network can be used to get the important source parameters and even play an important role in earthquake warning system.

From the statistical study, the shakemaps for the Hsinchu Science Park were obtained. This is a good reference for emergency earthquake hazard mitigations.

References


Earthquake Source Parameters and Micro-tremor Site Characteristics Study - Geochemical Monitoring


瓦里亞 1 、林世榮 2 、洪瑋立 3 、楊燦堯 4 、溫國樑 5

Abstract

The present study is proposed to investigate geochemical variations of soil-gas composition in the vicinity of geologic fault zones of Hsincheng fault and the Hsinhua fault within the Hsinchu and Tainan areas, respectively, and to determine the influence of such formations on enhanced concentrations of different gases in soil. Active faults favour gas leaks because they increase permeability of soil which help the gas to migrate easily. Spatial variability of gases like radon, helium, carbon-di-oxide, methane etc. in soil-gas can be used for finding fault system. Soil-gas surveys have been conducted across the Hsincheng fault and Hsinhua fault, to find out the regional activity of these fault systems. During the surveys soil-gas samples were collected along the traverses crossing the observed structures. The collected soil-gas sample bags are analyzed for He, Rn, CO₂, CH₄, Ar, O₂ and N₂. The data analysis clearly reveals anomalous values along the fault. To find the fault system, where the migration of gases is governed by advection, it is essential to identify the anomalies in radon, helium and carrier gases like CO₂, CH₄ and N₂ together. The consistency of this pattern confirms that soil-gas can act as a powerful tool for the detection and mapping of active fault zones. A continues monitoring station has been established inside the Hsinchu National Science Industrial Park (HNISP) at the end of September, 2005. Preliminary results of the monitoring station shows that the site is good the earthquake monitoring and soil-gas variations have shown good correlation with impending earthquakes.

Keywords: Soil-gas, Faults, Earthquake, Radon, Helium

Introduction

Soil-gas geochemistry is currently recognised as a reliable tool to seismotectonic studies, including fault tracing and seismic surveillance as a precursor. Several gases with different origins and contrasting behaviours in soil have been documented for characterising a fracture network allowing degassing (Fu et al., 2005; Toutain et al., 1999; Ciotoli et al., 1998; Walia et al., 2005a). The composition and distribution of gases in the soil atmosphere is affected by surface features such as pedological, biogenic and meteorological factors. However, these are thought to have a subordinated effect on gas leakage from deep fault-related features. Radon and helium are recognized as potential tracers of fault system (Al-Taminmi and Abumurad, 2001; Banwell and Parizek, 1988; Walia et al., 2005a) and are commonly used as precursors for earthquake prediction studies (e.g., Virk et al., 2001; Walia et al., 2005b; Yang et al., 2005).

Spatial variability of gases like radon, helium, CO₂, CH₄ etc. in soil-gas can be used for finding fault system and for seismic surveillance. This method to investigate active tectonic structures, using soil gas composition at faults, provides relevant information about regional stress conditions which can be obtained rapidly and at relatively low cost. Both radon and helium (³He) are products of uranium decay series. Radon due to its short half-life displays poor intrinsic mobility and therefore in diffusive system it obviously

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comes from a short distance below the measuring instrument. Deep origin signals can be observed only if convection/advection occurs; radon being carried upward to subsurface by a rising gas/water column (Etiope and Martinelli, 2002; Yang et al., 2003b). Helium is characterized by its high mobility and low solubility in water. Due to these reasons helium shows highly diffusive character with diffusion coefficient about ten times higher than those of N2, O2 and CO2. Helium has a low and constant concentration of 5.220 ± 0.004 ppm in air. Due to its characteristics and deep origin with respect to radon, helium appears as a powerful pathfinder for crustal discontinuities, faults and fractures (Ciotoli et al., 1998).

Carrier gases like CO2, CH4, nitrogen etc. play an important role in the migration of rare gases (Etiope and Martinelli, 2002; Yang et al., 2003b). Many parameters have been identified which effect emanation of noble gases like radon and helium i.e. the uranium content in rocks, soil porosity/permeability or degree of fracturing, moisture contents and meteorological parameters (Reimer, 1990; Sharma et al., 2000).

The soil-gas method has been used extensively worldwide. This method has been tested along The present work is aimed to study on the Hsincheng fault in Hsinchu area and the Hsinhua fault in Tainan area and later establish geochemical observatory on the above said faults for earthquake monitoring in different phases.

Methodology

Principle

Faults can be described as least-strength zones composed of highly fractured materials and can generate spatial contrasts for many geophysical parameters. Active faults favour gas leaks because they increase permeability of soil which help the gas to migrate easily. The anomalies, however, display complex characters, both in space and time (Sugisaki et al. 1980; King et al., 1993). In general, soil-gases concentration exhibit entirely composition in air and deep-crust derived components. The later usually exhibit higher concentration of He, Rn, CO2 and CH4 which will diffuse upward to the surface and balance with the air. That’s why soil gas compositions are rather different from air. Deep faults or fractures underlie the surface may provide gases the conduits to migrate upward from deep crust or mantle which produces anomalously high deep source signatures in the soil above (e.g.,Ciotoli et al., 1998; Guerra and Lombardi, 2000; Yang et al.,2003b; Walia et al., 2005). Therefore, anomalous soil gas compositions could be able to represent the presence of faults and/or fractures below the ground. Based on the principle, the soil gas technique is commonly applied to detect buried faults or trail of faults disappeared by human activity and/or natural events.

Sampling Procedure

To carry out the present investigation soil-gases were collected along number of traverses on the Hsincheng and Hsinhua faults using the hollow steel probe. A long hollow steel probe of 3 cm diameter and 130 cm long with a disposable sharp awl, which can make steel probe favorable for drilling into the soil and prevent soil to block the probe, was inserted into the soil at the depth of about 100 cm. A thin solid billet is used to displace the awl and allows the lower end of the probe to be in contact with soil-surface. A hand-pump through a specially designed rubber tube (with two filters: one is for dust and another one is for mist) connected with the hollow steel probe is used to collect gas in 1 liter and 3 liter vacuum created sample bags (Fig. 1).

Helium and other gases like carbon dioxide, methane, argon, oxygen and nitrogen from the collected 1 liter sample bags were analyzed within few hours after sampling by using helium leak detector (ASM100HDS, Alcatel) and micro gas chromatography (CP4900, Varian), respectively. Collected 3 liter sample bags are used for radon analysis using Radon detector RTM 2100 (SARAD).

Results and Discussion

Hsincheng Fault

During 1st phase of our investigation, in order to find the fault trace of Hsincheng fault in Hsinchu area numbers of soil-gas surveys using steel probe technique have been conducted specially in National Science Industrial Park. Soil gas survey was performed across 12 profiles (Fig.2) and more than 250 samples were collected for 222Rn, 4He, CO2, CH4, Ar, O2 etc. analyses covering an area of 30 km2 across the fault system. From the spatial distribution found to be the major carrier gas for 4He and 222Rn in this area. of 4He, 222Rn, CO2 and 222Rn anomalies (Fig. 3), the trace of Hsincheng fault has been recognized. N2 is found to be the major carrier gas for 4He and 222Rn in this area.
A profile, crossing fault inside the Hsinchu National Science Industrial Park (HNISP), the biggest Industrial science park in Taiwan, was selected for continuous sampling to understand the relationship between temporal variations of the soil concentration and seismic/crustal activity in this area. The location is close to the trench site where paleoseismic evidence confirmed that it is the surface trace of Hsincheng fault. Weekly monitoring indicates that soil gas variations at fault zone are closely related to the local crustal stress and hence, are suitable for further monitoring. High values of radon and helium were recorded on 8th March, 2005 which can further be correlated with twin earthquakes of Ml5.9 on 6th March, 2005 which rocked the Ilan city and having local intensity of 3 at Hsinchu.

Further few very close points where identified to for probable site for establishing continues motoring station. The site of these probable points has been reconstructed to reduce the meteorological effects and tested for some weeks before selecting a point for final setup of monitoring station. A continues monitoring station has been established at the end of September, 2005 (Fig. 4). Initially only radon detectors RTM 2100 along with carbon-di-oxide detector and Strong Motion Seismograph were installed but later a Quadrupole Mass Spectrometer (QMS) is added to the monitoring site to monitor the variations in the other gases like Helium, Methane, Argon etc. Preliminary results of the monitoring station shows that the site is good the earthquake monitoring and soil-gas variations have shown good correlation with impending earthquakes. Earthquake of Ml5.5 having local intensity of 2 on 30th November, 2005 with epicentral distance of 113 kms from the monitoring station is good example for the efficiency earthquake. Although variation in radon concentration don’t show big increase, but thoron concentration shows big increase which continue for almost two days i.e till 1st December before coming to normal values (Fig.5). From this it can be concluded that the selected site is sensitive to stress variations.

Fig. 2. Spatial distribution of different profiles along Hsincheng in Hsinchu area.

Fig. 3. Distribution of soil-gas data points and anomalies (with the increase in size and change in colour): (A) Helium (Red Triangle represents the anomalous values) (B) Radon (C) Carbon-di-oxide and (D) Nitrogen.


**References**


Seismic Response and Liquefaction of a Large Sand Specimen on Shaking Table

Tzou-Shin Ueng¹ and Chia-Han Chen²

翁作新¹、陳家漢²

Abstract

A physical model test using a large biaxial laminar shear box on the shaking table at the National Center for Research on Earthquake Engineering (NCREE), Taiwan was conducted to study the seismic responses of the saturated sand under multidirectional earthquake shakings. Pore pressures and accelerations within the soil, and the displacements and accelerations of the frames at various depths were measured during tests under both one- and multi-directional shakings of various amplitudes. It is found that the excess pore water pressures generated during a two-dimensional shaking are substantially higher than those generated under the one-dimensional shaking of the same magnitude of acceleration. The settlement of the sand inside the shear box after each shaking test was also measured and evaluated. The test results showed that significant settlements occurred only when there is liquefaction of the soil and the volumetric strain of the liquefied sand decreases with the relative density of sand.

Keywords: biaxial laminar box, pore water pressure, volumetric strain, shaking table, liquefaction

Introduction

At present, most studies of seismic soil behavior such as liquefaction are tested and modeled as a one-dimensional problem. The understanding of the soil behavior under two- and three-dimensional seismic loading conditions is very limited. (e.g., Pyke et al., 1975; Ishihara & Yamazaki, 1980; Ishihara & Nagase, 1988; Endo & Komanobe, 1995; Kammerer et al., 2002). The existing liquefaction laboratory experimental equipments, such as cyclic triaxial tests, torsional shear tests etc. are limited in practical applications because of the small specimen size, sample disturbance, and representation of the state of stresses. Therefore, large soil specimens have been placed on shaking tables that can reproduce the actual seismic ground shaking according to the earthquake recording under either 1 g or centrifugal conditions.

A large scale laminar biaxial shear box with a specimen size of 1880 mm × 1880 mm × 1520 mm on the shaking table at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan has thus been developed to test a large soil specimen under two-dimensional (multidirectional) earthquake shakings for the study of liquefaction and soil-structure interaction in a level ground. For the two-dimensional shaking, the loading and the soil movements can be in any direction on the horizontal plane of two (X- and Y-) axes, and they also change with time. Accordingly, the soil is under a multidirectional shaking.

Sand Specimen Preparation

A commercially available clean silica fine sand from Vietnam was used in this study. The grain size distribution of the sand is shown in Fig. 1. The basic properties are given in Table 1.

A special pluviator was designed for the preparation of the sand specimen inside the shear box by raining method. The wet sedimentation method was adopted for the sample preparation in this study. The sand was rained down into the shear box filled...
with water to a pre-calculated depth. The uniformity and density of the sand specimen were evaluated by undisturbed sampling from the shear box after pluviation of the sand. The saturation of the specimen was checked by the P-wave velocity measurements across the specimen horizontally. Details of sample preparation for the large shear box were discussed in Ueng et al. 2003 and 2006.

Fig. 1 Grain size distribution of Vietnam sand

Table 1: Properties of Vietnam sand

<table>
<thead>
<tr>
<th>Shape</th>
<th>D₅₀ (mm)</th>
<th>Cₛ</th>
<th>e_max</th>
<th>e_min</th>
<th>ρ_max (kg/cm³)</th>
<th>ρ_min (kg/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subangular</td>
<td>2.65</td>
<td>0.32</td>
<td>1.52</td>
<td>0.912</td>
<td>1.644</td>
<td>1.386</td>
</tr>
</tbody>
</table>

Test Results

Water Pressure Changes

Figure 3 is the water pressure changes measured by the water pressure transducer at the depth of 441 mm below the sand surface under one- and two-dimensional sinusoidal shakings with an input amplitude (A_max) of 0.035 g. It can be seen that the excess pore water pressure generated under a two-dimensional shaking are higher than those generated under one-dimensional shaking of the same magnitude. The excess pore water pressures generated under 1-D and 2-D shakings prior to liquefaction are compared and the ratios of excess pore water pressure induced by 2-D shaking to that by 1-D shaking were calculated. Generally, the ratio ranges approximately from 2.5 to 3.5 according to the shaking table tests performed at NCREE.

![Fig. 2 Shaking table tests for the sand specimen](image2)

Series of shaking tests have been conducted on the sand specimen in the biaxial laminar shear box on the shaking table at NCREE on two separate dates in 2005. Various one- and multi-directional input motions were imposed by the shaking table. The input motions included sinusoidal (1 Hz, 2 Hz, 4 Hz and 8Hz) accelerations, with amplitudes (A_max) ranging from 0.03 g to 0.15 g in X and/or Y directions. In the two-dimensional (multi-directional) shaking, there is a 90° phase difference between the input acceleration in X and Y directions, i.e., a circular or ellipse motion was applied. The acceleration, full and reduced amplitudes, recorded at seisnograh stations in Chi-Chi Earthquake, Kobe Earthquake and Loma Prieta Earthquake were also imposed in X and Y directions. In addition, in the latest shaking test a surcharge of 2860 kg was also placed on top of the specimen to simulate an about 3 m overlying soil layer.
The comparison of the pore water pressure changes at the depth of 401 mm below the sand surface under one-dimensional sinusoidal shaking of $A_{\text{max}} = 0.075$ g with frequencies from 1 Hz to 4 Hz is given in Fig. 4. It is found that sand specimen under 1-D shaking with higher frequency caused the higher water pressure increases.

Figure 5 shows the excess pore water pressure distribution along the depth of the specimen at various time during 1-D and 2-D shaking tests. It can be seen that one-directional shaking induced less excess pore pressure and probably caused only a very shallow liquefied layer, while the multi-directional shaking caused a deeper liquefaction zone down to about 760 mm below the sand surface. A multi-directional shaking induced a higher excess pore water pressure and it took a longer time to dissipate in a multi-directional shaking test than under a one-directional shaking. According to the measurements of water pressure changes at different depths in the sand specimen, it is found that the sand at a shallower depth is more susceptible to liquefaction than that at a greater depth.

The settlements after the shaking tests without liquefaction are very small (less than $\approx 2$ mm) and insignificant compared with those when there is liquefaction of the soil, as shown in Fig. 6. The settlements resulted from the multi-directional shaking were larger than those under one-directional shaking in both cases of liquefaction and non-liquefaction of the soil. Soil samples were taken using short thin-walled tubes at different locations and depths after the completion of the shaking table tests.

The volumetric strain of the sand after liquefaction caused by shaking was calculated considering the depth of liquefaction depth. The depth of the liquefied sand is determined based on the measurements of mini-piezometers and accelerometers on the inner frames. The volumetric
strains of the sand specimen under shaking without liquefaction obtained using the settlement measurements divided by the thickness of the specimen. With consideration of depth of the liquefied sand, the test results show that the volumetric strain after liquefaction, under sinusoidal shakings decreases with the relative density of the sand regardless of the amplitude, frequency and directions of shaking. Figure 7 shows volumetric strains after liquefaction under sinusoidal shakings with durations of 5, 10, 20 and 30 seconds in this study. It can be seen that the volumetric strain after liquefaction increases with the shaking duration. These relations can be used to estimate in-situ settlements after liquefaction if the shaking duration can be related with the characteristics of an earthquake.

Figure 7 shows volumetric strains after liquefaction under sinusoidal shakings with durations of 5, 10, 20 and 30 seconds in this study. It can be seen that the volumetric strain after liquefaction increases with the shaking duration.

Conclusions

A large laminar shear box with a specimen size of 1880 mm × 1880 mm × 1520 mm was developed and manufactured at NCREE. A series of one- and multi-directional shaking table tests were performed on saturated Vietnam sand in the shear box to study the responses of the sand specimen. The test results showed that a two-dimensional shaking induced higher pore water pressure generation and deeper liquefaction depth than those under the one-dimensional shaking of the same acceleration magnitude. Sand specimen under one dimensional shakings with higher frequency caused the higher water pressure increases. The multi-directional shaking also caused larger settlements of sand than those under one-directional shaking in both cases of liquefaction and non-liquefaction of the soil. The volumetric strain of the sand after liquefaction decreases with the relative density of the sand while it increases with shaking duration.

Acknowledgments

This study is supported by National Center for Research on Earthquake Engineering, Taiwan, R.O.C. The authors wish to thank the assistances of Messrs. C. W. Wu, F. J. Shi, C. F. Zou and engineers at NCREE in conducting tests on the shaking table.

References

Application of Wireless Sensors for Health Monitoring And Control Structure

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Abstract

Wireless sensors have been proposed for use in structural health monitoring systems because they offer low-installation costs and automated data processing functionality. To validate the performance of the proposed WiMMS (wireless modular monitoring system) on the vibration measurement of large-scale civil structures, a three-story half-scale steel structure is instrumented with a wireless monitoring system assembled from a network of six wireless sensors and tested it on a shaking table to ensure the reliability of the data communication. Field application of WiMMS to ambient vibration survey of Gi-Lu cable-stayed bridge is investigated. Finally, the preliminary study on structural control using MR-damper through WiMMS is also presented.

Keywords: wireless sensor, health monitoring, structural control

Introduction

The practice of using extensive cabling and high cost labor as is typical of the traditional monitoring systems will be changed to a system of inexpensive wireless embedded systems, maintained and operated with ease. Strong interest in applying wireless sensing technologies within structural health monitoring systems has grown in recent years. The advantages of wireless sensors are: they emerging as a viable monitoring system tool and provide rich amounts of mobile computing power. The use of wireless communication for SHM data acquisition was illustrated by Straser and Kiremidjian [1]. Recently, Lynch et al. extended the work by embedding damage identification algorithm into wireless sensing unit [2, 3]. With the rapid advancement of sensing, microprocessor, wireless technologies, it is possible to assess the benefits from the application of such technologies in the structural engineering field. The purpose of this paper is to use the developed wireless modular monitoring system (WiMMS)[4] for civil infrastructural health monitor. Both shaking table test and field experiment are conducted to enhance the reliability and applicability of the system. Seismic response control of building using WiMMS is also conducted in this study for the first time.

WiMMS Hardware Profile

The wireless sensing unit includes three subsystems: the sensing interface, the computation core, and the wireless communication system. The sensing interface is responsible for converting the analog sensor signals into digital forms. The digital data is then transferred to the computational core by the Serial Peripheral Interface. External memory is associated with the computational core for local data storage or analysis. The hardware profile of wireless modular monitoring system is shown in Fig.1. Picture of the wireless sensing unit is also shown in this figure. The Maxstream 9XCite wireless modem is used for the wireless communication subsystem. Its outdoor communication range is up to 300m, which is reduced to about 100m when it is used indoors. The hardware design was focused on improving the unit’s performance of high-precision real-time data acquisition. The functional diagram of the unit is shown in Fig.2.

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Data acquisition subsystem
- 4-channels @ 16-bit A/D conversion

Computational core
- 8-bit ATmega128 microcontroller
- 128 kB SRAM (data storage)
- 128 kB flash ROM (program storage)

Wireless communication channel
- Maxstream 9XCite radio
- Highly reliable spread spectrum radio

Completed Unit
- Form factor: 10 cm x 6 cm x 2 cm
- Power consumption: 80 mA at 7.5V

Field experiment using the sensing unit was also conducted. Ambient vibration survey of a cable-stayed bridge using wireless sensing unit was also conducted. Since the sensing unit was designed for any analog signal between 0 and 5 Volt so as to be accepted by the A/D converter, than for any sensor output signal (accelerometer or velocity sensor) must meet this input voltage constraint of WiMMS. The output voltage of velocity sensors for ambient vibration survey is ± 10 Volt which can not meet the input requirement of wireless sensing unit (0~5V). A signal converter must be designed. Fig.4 shows the circuit print of the designed converter and the voltage converter in operation. Fig. 5 shows the comparison of the recorded velocity signal from both wired and wireless sensor of the ambient vibration of Gi-Lu cable stayed bridge (velocity response of stayed cable).

Application to Structural Vibration Monitoring

To validate performance of the entire wireless sensing unit, validation tests using shaking table on a 3-story half-scale laboratory structure are devises (floor area: 3m x 2m and story height: 3m). A total weight of the test structure is 19 tons. The Crossbow CXL02 MEMS Accelerometers (± 1.0g) were installed on each floor. Both El Centro and Chi-Chi earthquake records were used as input motion to the structure. Both traditional cable-based and the wireless sensing unit systems were used to collect the response of the test structure. Fig. 3 shows the comparison on the recorded acceleration of 3rd floor from both wired and wireless sensing systems. Good agreement was observed.

Fig. 1 Hardware profile of wireless modular monitoring system (WiMMS)

Fig. 2 Hardware functional diagram of the wireless sensing unit.

Fig. 3a A 3-story steel frame instrumented with wireless sensors for shaking table test,

Fig. 3b Comparison on the recorded 3rd floor acceleration from wired and wireless system.
Fig. 4 A signal converter was designed for converting the voltage difference between velocity meter and the sensing unit.

Fig. 5 Comparison on the recorded velocity from wireless sensing unit and traditional data acquisition system (“Red”: wireless, “Blue”: traditional).

Application of WiMMS to Structural Control

A three story steel frame with the installation of a MR-damper in the first story was used to study the application of wireless sensing unit for active structural control. Wireless sensing units was placed on each floor and connected with velocity sensor (or acceleration sensor) to transmit the structural response wirelessly to the receiver at the basement floor. The building was tested on shaking table using El Centro earthquake ground motion data as input motion. Fig. 6 shows the schematic diagram of the arrangement of control devices on the structure. Three important devices were needed to control the structure: (1) VCCS: convert the voltage signal (0. volt~1.0 volt) to current, (2) DAC: digital to analog converter (from action board to VCCS), (3) Action Board: convert the received commend voltage (16 bit digital signal) to analog signal with 0.~1.0 volt for VCCS. The sensing unit at the bottom floor was also embedded with control algorithm so as to calculate the control voltage to action board, as shown in Fig. 7. The embedded computation algorithm in the sensing unit (at the basement) will cover two major computations: one is to calculate the control force (by multiplying the collected signals with the embedded gain matrix), the other is to convert the commend force to voltage (match with the MR-damper). An action board must be placed between the receiver unit and the VCCS. This action board will conduct the digital to analog converter (0.0~1.0 volts). Fig. 8 shows a picture of the designed action board.

Two different control algorithms were used to calculate the control force (or voltage) for MR-damper: one is the velocity feedback and the other is the acceleration feedback. For velocity feedback control velocity signals at all floor levels were collected wirelessly and multiple by the control gain vector (already embedded) to estimate the required control force.

Fig. 6 Overall instrumentation arrangement for structural control test using WiMMS and MR-Damper

Fig. 7 Configuration between wireless sensing unit and the action board at the 1st floor.
Fig. 8 Action board that convert the 16bit digital voltage to analog signal with 0.~1.0 Volt.

Fig. 9 Comparison on the control force and commend voltage from simulation and wireless sensing unit.

Fig. 10 Comparison on the 3rd floor displacement for case of un-control, simulation, and control using wireless sensing unit.

Conclusions

This paper presents the preliminary verification and applicability of wireless modulus monitoring system to monitoring the seismic response of building structure, ambient vibration survey of large civil infrastructure and structural control. The results show that the WiMMS can provide broad applications to monitoring and control of civil infrastructures. With the designed converter different sensor signals can be used as input to the wireless sensing unit for monitoring purpose. For structural control purpose, one can embedded the control gain as well as the control algorithm in the sensing unit. Through this research the structural control can be conducted using wireless sensing unit.

The shake table test of semi-active controlled base-isolation system that includes rolling pendulum system and a Magnetorheological damper shows the great benefit of the smart damper. The most different between the base-isolation system with passive damper and semi-active control Magnetorheological damper is that the semi-active control system is adaptable to various kinds and intensity of excitations. While the passive control system can only focus on some cases (P-max: intensive earthquakes; P-off: small earthquakes). Also, this study provides evidence of full-scale, real-time control of augmenting a common base-isolation system with a smart Magneto-Rheological damper that is modulated with a fuzzy controller.

Acknowledgements

The authors wish to express their appreciations for the funding support of this research from Central Weather Bureau and the technical support from NCREE, particularly from Director Dr. K.C. Tsai and Deputy Director Mr. C. C. Hsu.

References


