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THREE-DIMENSIONAL NONLINEAR TIME-HISTORY ANALYSIS OF TWO-STOREY CONCRETE WALL-TYPE STRUCTURE

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ABSTRACT

A two-storey concrete wall-type structure is analyzed using three-dimensional nonlinear time-history analysis. Ground acceleration records of the magnitude 7.2 Kobe earthquake are used to simulate a scenario earthquake from the 67 km West Valley fault in Metro Manila. Ansys® software is used in the analysis to enable the modeling of concrete inelasticity: due to both concrete cracking and concrete crushing, using Solid65 concrete element. Using this element, smeared reinforcement is also introduced in the model. Results of simulation show that the failure mode is governed by tension cracks in the concrete adjacent to the corners of openings in the walls, especially in the absence of additional diagonal reinforcement thereat. It is also shown that concrete crushing failure is not critical for the structure analyzed, even in the absence of special boundary elements. Comparison of the result of simulation using different ground acceleration records scaled up to the same level shows that both stresses and deflections are higher for the record that has a predominant period of 0.34sec which is relatively nearer that of the natural period of structure of 0.15sec. The effect of frequency content of the input ground motion, which may be attributed to the soil condition, is evident in this study. There is a need for further research on more appropriate ground acceleration inputs for simulations like this study. The computational effort of about 10 hours using 1.83 GHz Intel Core Duo processor and 2.49GB RAM for a 10-sec simulation at 0.01sec interval is too much for most engineering design offices. While the computational effort for a 16-sec simulation at 0.02 sec interval for a small structure as in this paper could be reduced from 8 hours to 2.5 hours using a much powerful HP Proliant ML150 G6 server with 2.26 GHz, 8-core processor and 48 GB RAM, this computer is still not the norm for ordinary engineering design office. Aside from considerable storage disk requirement, software commercially available to engineering design offices are typically less powerful; there is thus still a need for simplified analytical tools, e.g. 2-dimensional nonlinear static analysis by pushover.

Keywords: *Nonlinear Time-History Analysis, Concrete, Wall-Type Structure, Ground Acceleration*

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1. INTRODUCTION

New technologies and structural systems are recently introduced in the Philippine housing industry. One such technology is the reinforced concrete bearing wall type construction system. This construction system without beams and columns is now beginning to be used in multi-storey, multi-dwelling units as well as in one or two-storey single-family dwelling units. Shorter construction period for this construction system makes it an attractive alternative to housing developers. However, while excellent performance of moderate-rise bearing wall type buildings was observed during the 1985 Viña del Mar, Chile earthquake (Wood, 1992), this particular technology, which is characterized by modest reinforcement details by US standards, is still untested under earthquakes in the Philippines. It may be noted that expected earthquakes in our country may exceed the peak ground acceleration of 0.36g experienced in Viña del Mar, Chile located 80kms from the epicenter. Moreover, building code requirements in the country, which are based on US codes, present some technical difficulties for the adoption of this type of construction system locally since it has been observed that the cost of detailing makes this construction system uneconomical following the United States building code (Wallace & Moehle, 1992). The use of bearing wall type system for low-rise single-family dwelling structures also requires particular attention due to the presence of many perforations. It is thus necessary to assess the seismic performance of concrete bearing-wall type structural system considering local earthquake scenario. It is also necessary to develop practical analysis tools for such assessment.

2. OBJECTIVES

Previous unpublished work by the first author on this type of structure using linear elastic analysis implemented using ETABS showed that boundary elements are required on certain areas of the structure following the provisions of the National Structural Code of the Philippines (NSCP, 2010), validating the observation in the United States. The special requirement for boundary elements makes the detailing of wall-type structure rather uneconomical. Moreover, it may be observed that while most commercially available software used by practicing engineers such as ETABS, SAP2000, and STAAD can do nonlinear analysis of frame-type structures, walls elements can only be modeled therein as line elements. Shear-type of failure for walls is not adequately addressed.

This study therefore seeks to simulate the behavior of a two-storey concrete bearing-wall type structure under earthquakes that may be expected to occur in the country using nonlinear time-history analysis. Potential compression mode of failure, which is the reason for special boundary element, as well as critical locations for tension cracking mode of failure are investigated.

Among the challenges are: appropriate modeling of concrete wall-type structure with nonlinear material properties and appropriate modeling of time history of ground motion.

3. NUMERICAL SIMULATION

Nonlinear time-history analysis of the 3-dimensional model, shown in Figure1, is performed using different earthquake record data as alternative input. Transient Analysis facility of ANSYS® (ANSYS, 2010) (ANSYS, 2010) using Ansys Parametric Design Language (APDL) is used in this study. Seismic load is introduced in the form of acceleration applied at the foundation level.

An APDL code is written for transient analysis to enable the software to read the input acceleration applied at different time increment. All points at the ground level are assumed to be synchronously accelerated. The ground acceleration digital record is stored in an array for input in the analysis. Gravity load is introduced in the form of inertia load along the global y-axis.

3.1 Structural Model

For this two-storey wall-type structure, walls are modeled using SOLID65 element. An important aspect of this element is that it can model nonlinear material property. This material type can model both concrete cracking in tension and crushing in compression as well as the reinforcing bars. Eight nodes with three translational degrees of freedom each define the element. Cracking is permitted at each integration point. Reinforcing bars are assumed to be smeared over the element. The slabs, on the other hand, are modeled as concrete SHELL63 elements. The finite element model using mapped meshing for walls and free-meshing for slabs is shown in Figure 1.Wall thickness used is 100mm so as not to exceed the maximum length to thickness ratio of 25 for the unsupported length of 2.6 m per section 414.6.3.15 of the National Structural Code of the Philippines (NSCP, 2010). Floor slabs are 125 mm thick. The computed (fundamental) natural period, using ANSYS, of the structure in Fig. 1 as described above is 0.15 sec.

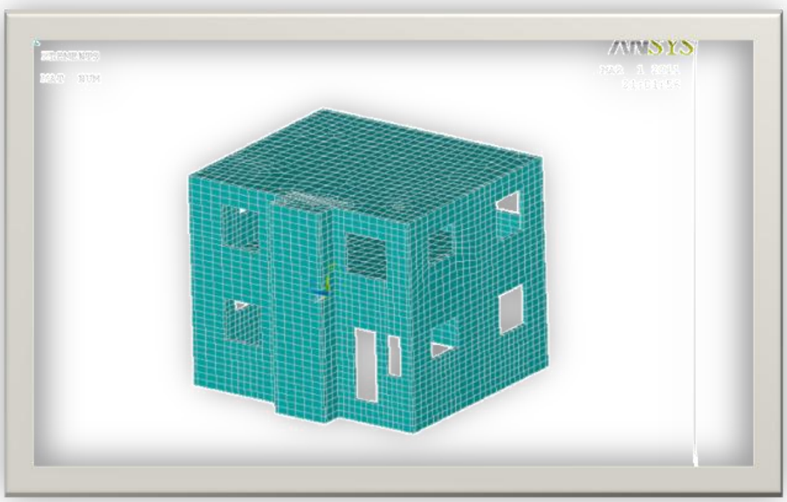

Figure 1. Structure 3-D Model

Input data includes volumetric ratio and direction of reinforcing bars in addition to concrete properties.

The following material properties are used for SOLID65 element in the analysis:

Concrete Properties:

Concrete Strength = 13.3 MPa; Compressive (Crushing) Strength = 8.666 MPa; Cracking Strength = 1.13 MPa; E = 14.131 GPa; Poisson's Ratio = 0.25

Concrete softening is introduced in the model by using a concrete softening factor. Concrete softening is the reduction of compressive strength due to tension in the transverse direction. This phenomenon was first understood when it was observed by J. Peter in his 1964 dissertation at Technische Hochscule Stuttgart (Mo & Rothert, 1997) that concrete panels subjected to compression were softened by tension in the transverse direction. A softening parameter was first quantified by Vecchio and Collins in 1981 in their Compression Field Theory based on test results of 17 reinforced concrete panels (Mo & Rothert, 1997). By combining equilibrium, compatibility, and a softened stress-strain relation of concrete, Vecchio and Collins also developed the Modified Compression Field Theory in 1986 (Palermo & Vecchio, 2004). Hsu (1991) also proposed a softened truss model theory based on the work of Vecchio and Collins. Hwang, Fang, Lee, & Yu (2001) used a softened strut and tie model to predict the strength of squat walls. The Modified Compression Field Theory was also implemented by Palermo and Vecchio (2004) for finite element formulation. Salem & Maekawa (2006) also applied concrete softening using the Okamura-Maekawa concrete softening model in finite element analysis. An advantage of the Okamura-Maekawa concrete softening model is the consideration of the strain-rate factor in the model.

Ansys default failure criteria for concrete is based on the work of Willam and Warnke [Ansys Theory Reference, (Ansys, Inc., 2009)]. Failure criteria is expressed in the form:

$$\frac{F}{f_c} - S \geq 0 \qquad \text{[Eq. 1]}$$

Where F is a function of state of principal stress
S is a failure surface in terms of principal stresses and concrete parameters
f_c = uniaxial crushing strength

The functions F and S are defined for four different domains depending on the nature of stresses at the point. These domains are:

$0 \geq \sigma_1 \geq \sigma_2 \geq \sigma_3$ (compression-compression-compression),
$\sigma_1 \geq 0 \geq \sigma_2 \geq \sigma_3$ (tension-compression-compression),
$\sigma_1 \geq \sigma_2 \geq 0 \geq \sigma_3$ (tension-tension-compression), and
$\sigma_1 \geq \sigma_2 \geq \sigma_3 \geq 0$ (tension-tension-tension)

Graph of the failure surface is shown in the following:

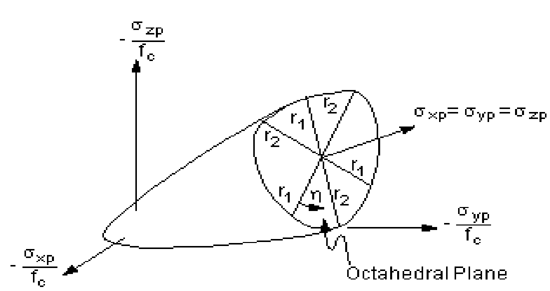

Figure 2. 3-D Failure Surface in Principal Stress Space (Ansys, Inc., 2009)

Other details of the concrete failure criteria can be found in Ansys Theory Reference (Ansys, Inc., 2009).

In Ansys concrete modeling, shear transfer coefficient is used to represent the condition of the crack face. A shear transfer coefficient of zero means complete loss of shear transfer while a shear transfer coefficient of 1.0 means no loss of shear transfer (for rough crack). Different values of shear transfer coefficient were used by different authors. Baetu and Ciongradi (2011) used shear transfer coefficient of 0.4 and 0.8 respectively for open (β_t) and closed (β_c) cracks. Greeshna, Jaya, and Annilet (2011) used shear transfer coefficients of 0.2 and 0.9 respectively for open and closed cracks. Qi Zhang (2004) used shear transfer coefficient of 0.125 to 1.0 for open cracks and observed that "Although there is no significant difference of the ultimate loading with the different β_t , accompanying with the increase of β_t , the ultimate loading is slightly increased after the displacement is beyond 3mm". For this study, shear transfer coefficient is taken to be zero for open crack while closed crack shear transfer coefficient is conservatively taken to be 0.6, considering the small amount of reinforcement used. Effect of using higher values of β_t and β_c is currently under study. Discussion of concrete material property can be found in Ansys Theory Reference – Concrete (ANSYS, 2010).

It may be noted that a study conducted by Cheng, Sun, and Fan (2002) used the Okamura-Maekawa sheared concrete softening model for 2-D analysis in Ansys to account for concrete softening.

In this study, Ansys concrete material property was used with concrete softening coefficient of 0.65 in compression based on the result of a previous study (Germar, 2011) for lattice wall model. A need for further research on concrete softening for finite element application is acknowledged. Viscous damping is used in Ansys in the form $C = \alpha M + \beta K$. In this study, α was taken to be 0 while β was taken to be 0.05: hence damping is assumed to be proportional to the stiffness.

It may also be observed that the concrete strength used in this study is smaller than what may be commonly used in actual construction practice. Initial analysis using 20 MPa concrete showed that compression mode of failure is not critical. Hence, concrete strength is reduced to 13.3 MPa to see potential compression mode failure at reduced concrete strength. Moreover, code requirement for 16mm bars around the openings is not applied, coupled with reduced concrete strength, to expose critical locations in tension cracking during the simulation.

Steel Properties:

The reinforcement provided in the model is the minimum requirement for the horizontal reinforcement. The vertical reinforcement of 0.0025 is adopted similar to the horizontal reinforcement for simplicity though higher than the minimum requirement of 0.0015.

3.2 Earthquake Ground Motion Data

Scenario earthquake from the West Valley fault system based on the Metro Manila Earthquake Impact Reduction Study (MMEIRS) (JICA, 2004) is considered in this study. This 67km fault is a strike-slip fault capable of generating a magnitude 7.2 earthquake. The fault system is similar in mechanism to the San Andreas Fault, Kobe Fault, and the North Anatolian Fault in Turkey which are also mainly strike-slip faults. The 1995 magnitude 7.2 Kobe earthquake thus appears to be a reasonable approximation of the West Valley Fault scenario earthquake.

Accelerograms of Kobe earthquake recorded at JMA station and Port Island station were used in the MMEIRS. JMA and Port Island stations have epicentral distances of 18.27km and 19.25km respectively. For this study, the JMA record and the Nishi-Akashi record are used with or without scaling of the amplitudes. Based on PEER database (Pacific Earthquake Engineering Research Center), the JMA record was recorded on diluvium while the Nishi-Akashi record, at epicentral distance of 8.5km, was recorded on soft soil (Manfredi, Polese, & Cosenza, 2003). Ground motions in two perpendicular directions are used as input data in this study. Considering the source faulting mechanism and maximum capable magnitude, the earthquake records are selected because of their proximity to the epicenter. It should be noted that due to limited earthquake record available, the present structural code (NSCP, 2010) requires earthquake records to be used to be scaled only to a certain level regardless of the soil condition under which the earthquakes were recorded. Based on the Code, "Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake)."

Accelerograms and acceleration spectra at Nishi-Akashi station are shown in Figures. A.1, A.2, and A.5. Earthquake record NIS000 has a maximum ground acceleration of 0.509*g* and predominant period of 0.46sec. NIS090 has a maximum ground acceleration of 0.503*g* and predominant period of 0.44sec.

As shown in Figures A.3, A.4, and A.6,for JMA station, maximum recorded ground acceleration is 0.821*g* with predominant period of 0.34sec for JMA000 and 0.599*g* with predominant period of 0.38sec for JMA090. Note that for this record, spectral acceleration at 0.15 sec is almost 1*g*.

Time-history, acceleration spectra, and other ground motion parameters are obtained using SeismoSignal and SeismoSpect by Seismosoft® (Seismosoft, 2010).

Selection of earthquake ground motion based on National Structural Code (NSCP, 2010) provisions for time-history analysis requires an average value of SRSS spectra of at least 1.4 times the 5% damped spectrum of the design basis earthquake for period from $0.2T$ second to $1.5T$ seconds. For the computed period of the structure of 0.15 second, this corresponds to the range of 0.03 to 0.225 seconds.

For Nishi-Akashi station records, the response spectrum at 0.15 second is approximately 0.80*g* in both directions. Due to limited available earthquake records of similar magnitude, fault distance, and source mechanism, the Nishi-Akashi record is used in this study with a scale factor of 1/0.80 or 1.25 to approximate the target demand spectrum of 1*g* for 5% damped spectrum of the design basis earthquake. A scale factor of 1.75 is also used for the Nishi-Akashi record to approximate the code requirement that the response spectrum be at least 1.4 times that of design basis earthquake within the period $0.2T$ to $1.5T$.

As previously noted spectral acceleration at 0.15 sec is almost 1*g* for the JMA record. For JMA record, a scale factor of 1.27 is also used in this study to raise it to the same level of response as that of 1.75 x Nishi-Akashi record which is 1.4 times that of the design basis earthquake. It is acknowledged though that simply scaling the acceleration amplitudes without considering the frequency content may not be entirely realistic. Hence it is imperative to continue the research on realistic ground acceleration time-histories that may be used in nonlinear simulation like this study. For now, the Kobe records with amplitude scaling as described above are considered appropriate.

The selection of appropriate ground motion for performance evaluation still presents a problem in many studies. As noted by Krawinkler (1997), in the seismic demand evaluation of steel frame structures in SAC steel project, the first storey drift for a 2.4 sec structure had a coefficient of variation of 0.68 and a ratio of maximum to mean drift of 3.9 using a 10% probability of exceedance for 50 year earthquakes for Los Angeles, California.

4. DISCUSSION OF RESULTS

4.1 Result of 3-D Time History Analysis

Detailed results of the above simulation are presented in Germar (2011), where it is also noted that at the time of this study it takes around 10 hours to compute a 10-second response time history at 0.01 sec time step using 1.83 GHz Intel Core Duo processor and 2.49GB RAM. A 16-second simulation using a time step of 0.02 sec took 8 hours to complete using the same machine.

Example results of nonlinear time-history analysis using the Nishi-Akashi record are shown in Figures 2; 4 - 6. Relative displacement response for the first 10 seconds of a sample point at roof level of longitudinal wall 1 for 1.75 x Nishi-Akashi record is shown in Figure 2. The maximum relative displacement of 0.93mm occurs at time 9.01 sec.

Maximum and minimum principal stresses at time 9.01 sec are shown in Figures4 and 5. Maximum principal stress shows the location of elements in tension. Minimum principal stress, on the other hand, shows the location of elements under compression. The diagonal pattern of both tensile and compressive stresses is quite evident in both figures. (The slabs are not shown in the figures, to show the stresses in the walls.)

Largest compressive stress amounts to around 3.3 MPa (color yellow) in the vicinity of the corners of windows and door openings and the base corner. Largest tensile stress amounts to around 1.13 MPa, (color yellow-green), which is the cracking strength of concrete, also in the vicinity of the corners of windows and door openings and forming diagonal lines into the base corner.

Figures 3; 7-9 show the corresponding results for the JMA record scaled by 1.27. Maximum roof displacement for the 1.27x JMA record for the first 14 seconds amounts to 1.37mm at time = 8.58 sec.

Maximum compressive stress amounts to around 5.8MPa (color green) in the vicinity of the corners of windows and door openings and the base corner. Maximum tensile stress amounts to around 1.13 MPa, (color yellow-green), which is the cracking strength of concrete, also in the vicinity of the corners of windows and door openings and forming diagonal lines into the base corner.

For both the 1.75 xNishi-Akashi and 1.27 x JMA records, the compressive stresses are within the compressive strength limit while the tensile strength limits are reached in the vicinity of the openings (in the absence of additional diagonal reinforcing bars as intended in the simulation).

Based on the results of simulation, it can be observed that while 1.75 x Nishi-Akashi record and the 1.27 x JMA station record have the same level of maximum acceleration and response spectrum, the 1.27 x JMA records result in higher roof displacement and state of stress. A possible explanation could be the effect of frequency content of JMA record nearer that of the structure: the Nishi-Akashi record has a predominant period of 0.46 sec while the JMA record has a predominant period of 0.34sec that is nearer that of the fundamental natural period of structure which is 0.15sec. It is even possible, and potentially more adverse to the structure, to have ground accelerations with predominant periods even closer to 0.15sec: e.g nearer the fault and/or on stiffer ground.

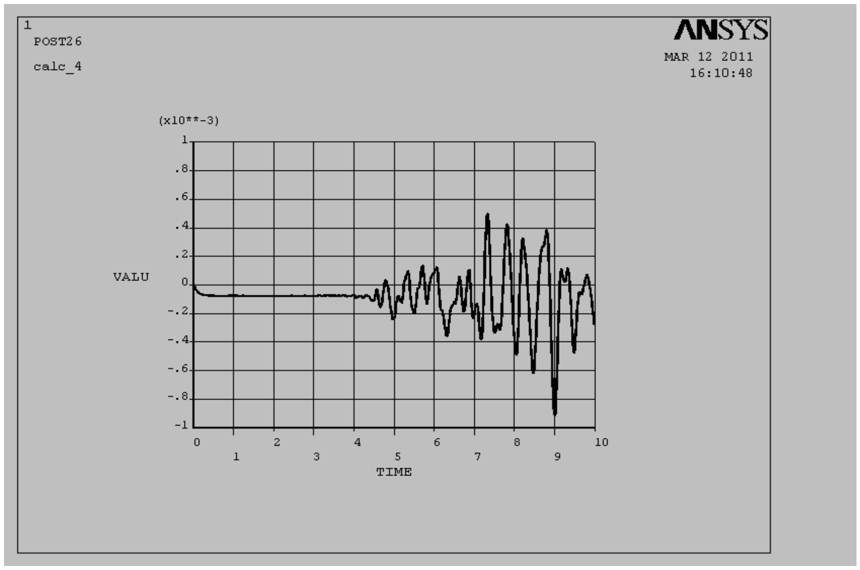

Figure 3. Longitudinal wall 1 relative roof displacement (1.75 x Nishi-Akashi)

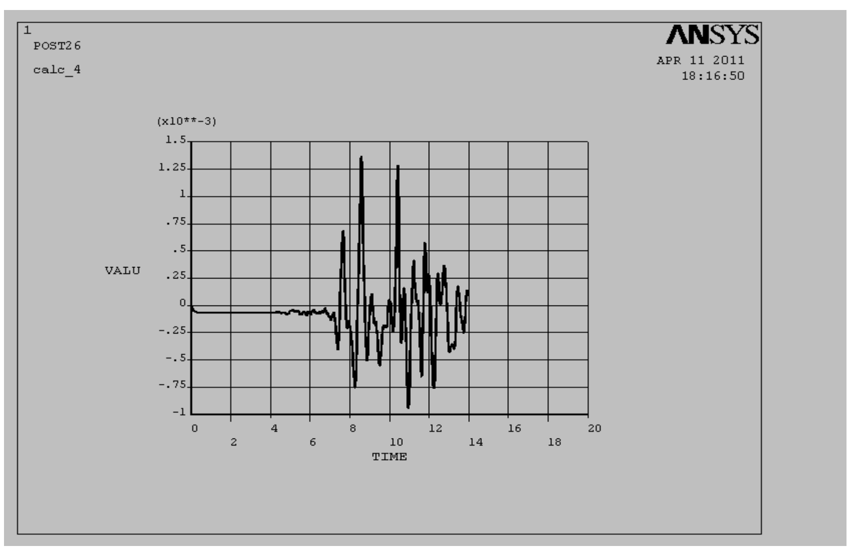

Figure 4. Longitudinal wall 1 relative roof displacement (1.27 x JMA)

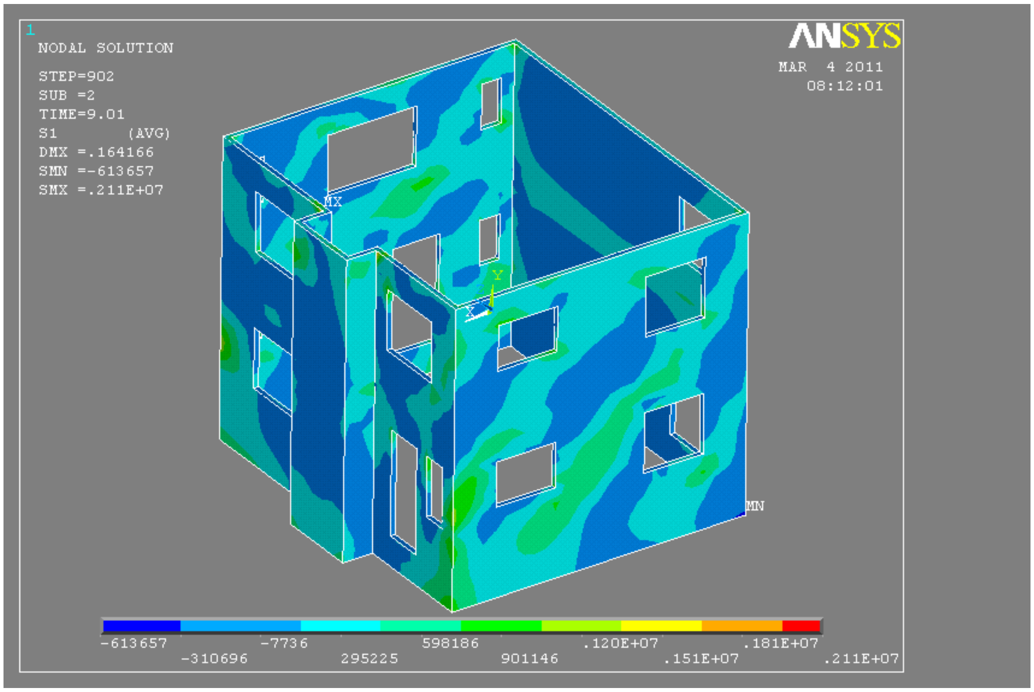

Figure 5. Maximum principal stress (1.75 x Nishi-Akashi)

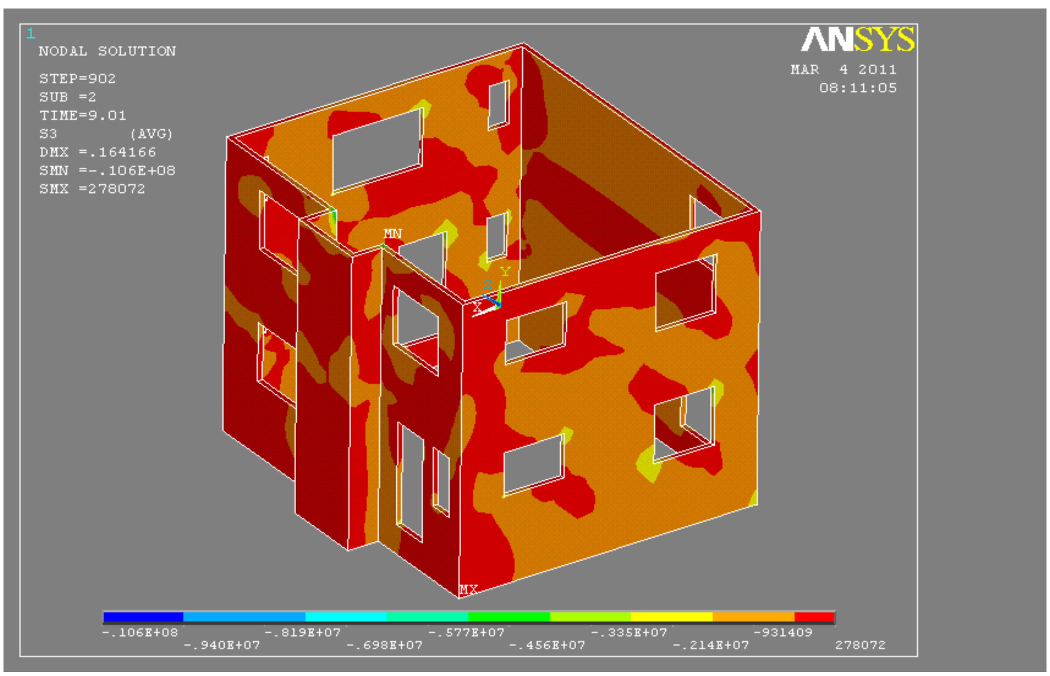

Figure 6. Minimum principal stress (1.75 x Nishi-Akashi)

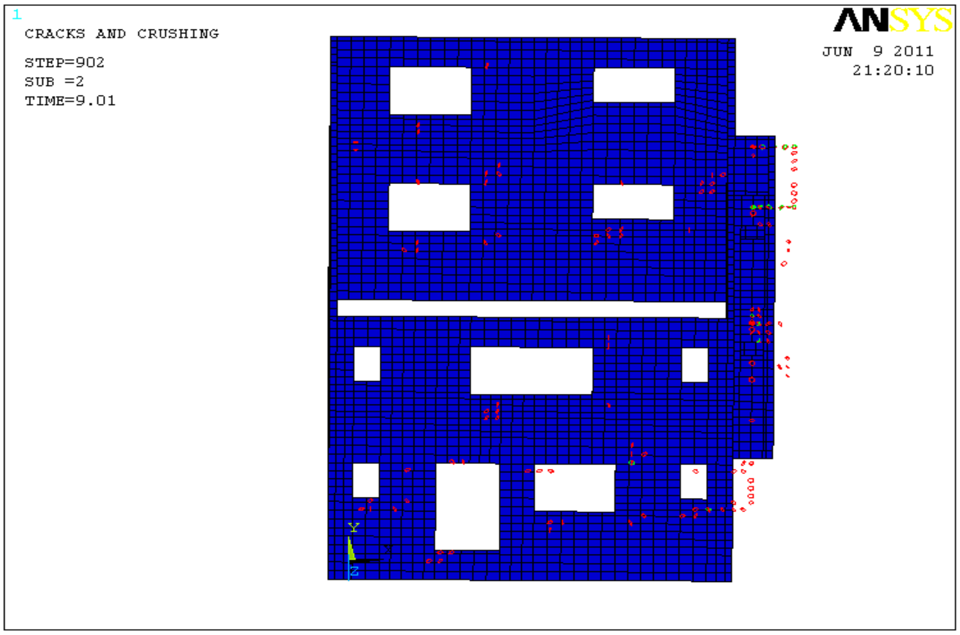

Figure 7. Crack locations (1.75 x Nishi-Akashi)

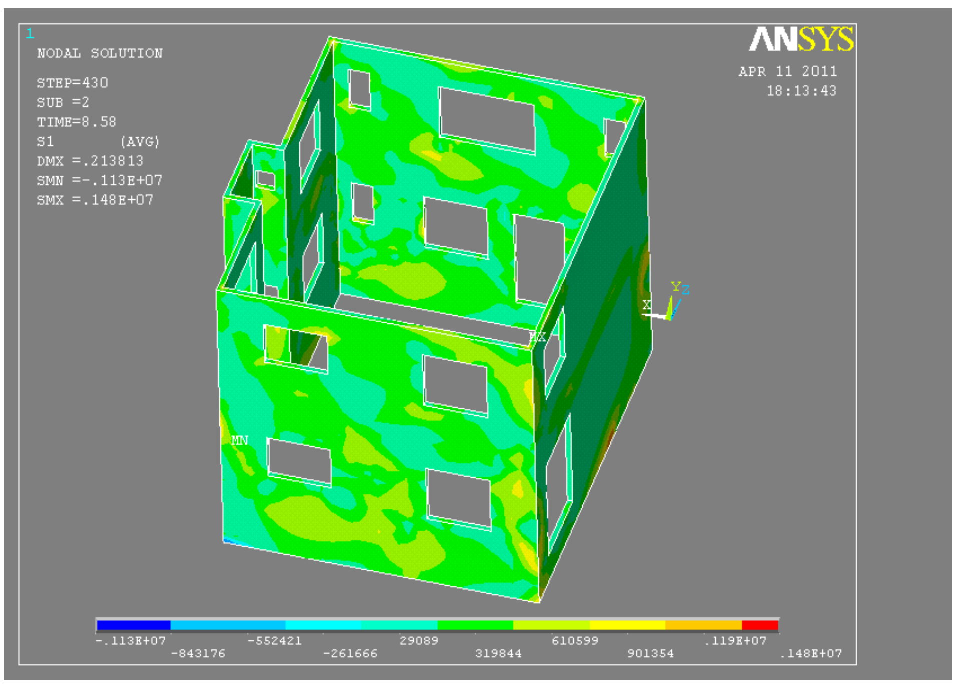

Figure 8. Maximum principal stress (1.27 x JMA)

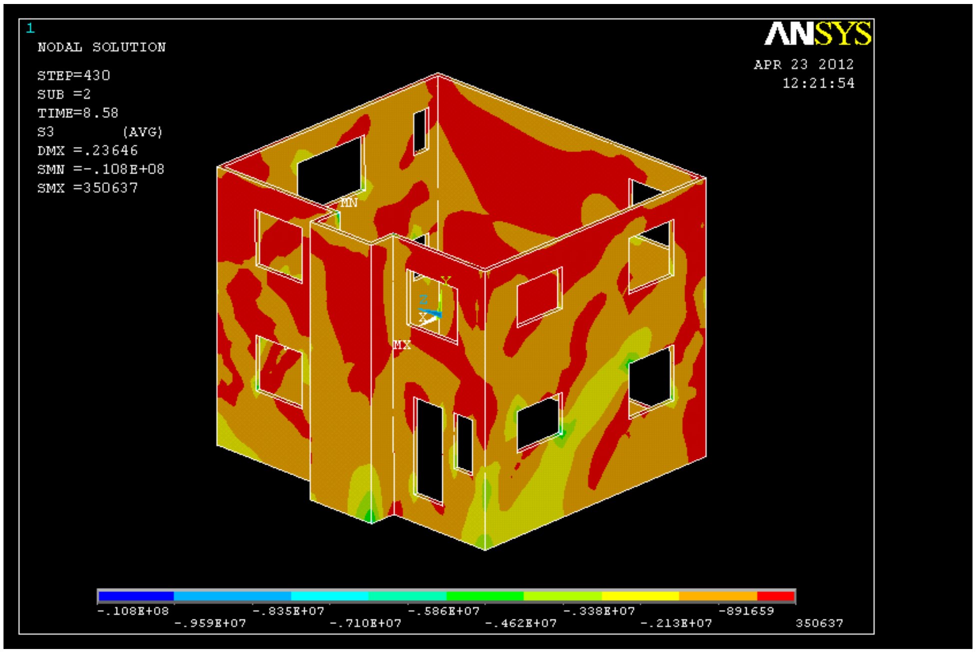

Figure 9
Minimum Principal Stress (1.27 x JMA)

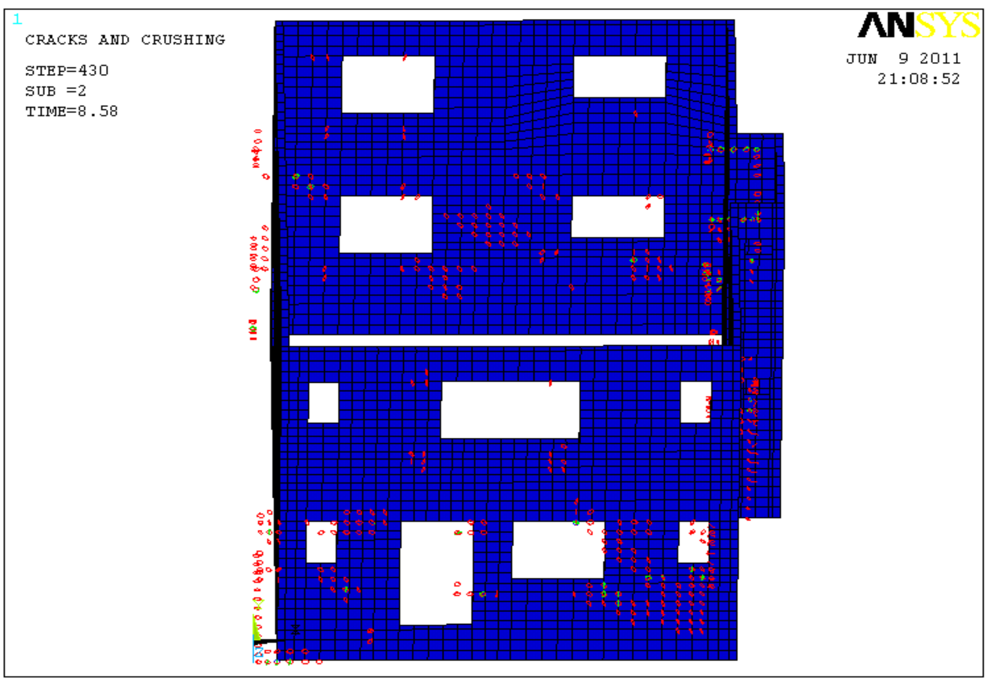

Figure 10 Crack locations (1.27 x JMA)

4.2 Comparison of 2-D and 3-D Analysis

While 3-D time-history analysis could show the state of stress at any instant of time, time-history analysis, coupled with solid modeling of the 3-dimensional structure requires a large amount of Random Access Memory (RAM) and long computation period. The hard disk storage requirement for output data recorded at each time step is also considerable. A 10 sec simulation at 0.01 sec time step interval takes about 10 hours to run on a 1.83 GHz Dual Core processor with 2.49 GB RAM. A 16 sec simulation at 0.02 sec time step interval takes around 8 hours to run. The output storage requirement for the 16 sec simulation at 0.02 sec time step amounts to around 70 gigabytes of memory. While an 8 hour run could be reduced to 2.5 hours using a much powerful HP Proliant ML 150 G6 server with 2.26 GHz, 8-core processor and 48 GB RAM, this high end equipment is still not the standard equipment for most engineering design offices at the present (2012). Analysis of medium rise, multi-bay wall type structure would also take much longer compared to a 2-storey, single-bay wall-type structure analyzed in this study. Output data storage would also be considerable.

Also, although 3-D modeling may be considered more straight-forward compared to 2-D modeling where effects of connecting transverse walls and slab have to be indirectly accounted for, 3-D modeling of walls requires expensive, complicated software such as ANSYS® which ordinary engineering office might not have access to. A 2-D solid model could reduce the RAM and hard disk requirement but would still need sophisticated software to model inelastic behavior. A time-history analysis of 2-D solid model would still need a large amount of hard disk storage for the output data.

Thus, there is still a need for simplified 2-D inelastic analysis of wall structures.

Germar (2011); Germar and Pacheco, (2012a), and Germar and Pacheco, (2012b) show that 2-dimensional nonlinear static analysis by pushover, using lattice model of wall elements, could adequately capture the tension cracking and compression crushing type of failure. An advantage of a 2-D nonlinear static analysis, using lattice model, is the relatively short time that the analysis could be done. A 2-D analysis of a wall model takes only minutes to finish compared with 8 to 10 hours using the same machine. Challenges of the 2-D nonlinear static analysis, on the other hand, include modeling the properties of the equivalent lattice model, modeling plastic axial hinges, and indirect modeling of transverse wall and slab elements.

5. CONCLUSIONS AND RECOMMENDATIONS

Three-dimensional nonlinear time-history analysis of an example two-storey wall-type structure, relatively a new structural type, subject to a representation of a scenario earthquake that may emanate from the West Valley Fault in Metro Manila, Philippines, can be reasonably performed through commercial state-of-the art finite element software, with judicious modeling of the nonlinear material properties of the structure and the ground acceleration history. The computational effort is very high, however: as much as 10 hours to compute a 10-second response time history using 1.83 GHz Intel Core Duo processor and 2.49GB RAM. An 8-hour run could be reduced to 2.5 hours using a much powerful HP Proliant ML 150 G6 server with 2.26 GHz, 8-core processor and 48 GB RAM but this high-end equipment is still not the standard equipment for most engineering design offices. The hard disk storage requirement for output data is also considerable even for just a small structure as in this paper. The software commercially available to engineering design offices are also typically less powerful. Hence there is still a continuing need to develop simplified analytical tools. Germar(2011); Germar and Pacheco, (2012a), and Germar and Pacheco, (2012b) show that 2-dimensional nonlinear static analysis by pushover, using lattice model of wall elements, could adequately capture the tension cracking and compression crushing type of failure.

For the time history in this study, the failure mode is governed by diagonal tension in the wall piers. Compression mode failure is also shown to be not critical for the structure analyzed despite the slenderness ratio of the wall reaching the Code-prescribed limits. Even with reduced concrete strength, compression mode failure is still not critical. A practical significance of this finding is that the Code requirement for buildings to have special boundary element details may need to be re-assessed for low-rise bearing wall-type residential structures. Arguably, it may be simpler and more effective to increase slightly the uniform steel reinforcement in all the walls of the structure or at the vicinity of the openings than to make special details at designated boundary elements.

The simulation results also show the significance of frequency content of the earthquake record. The effect of soil condition on the frequency is also acknowledged. It shall be noted that to date (2012), selection of earthquake records for purposes of nonlinear time-history simulation is not required to consider yet the effect of soil condition. Additional research on proper selection of input earthquake ground motion is strongly recommended for future studies.

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APPENDIX

Nishi-Akashi Record

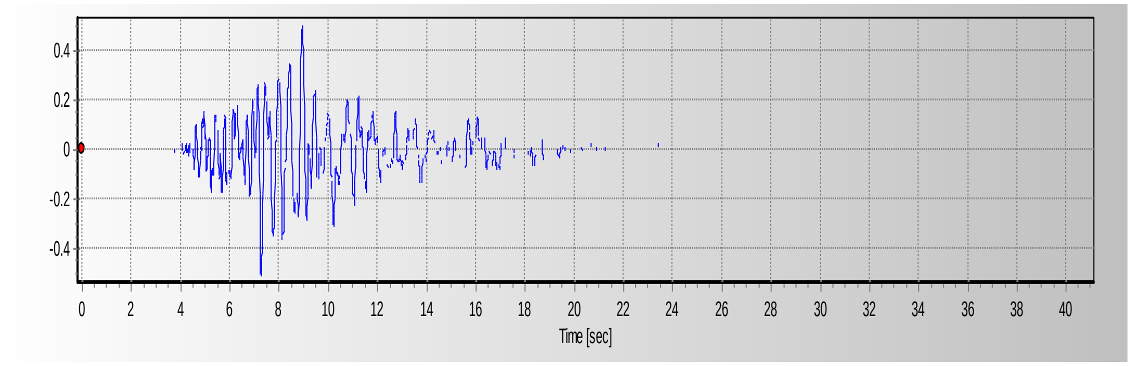

Figure A.1 NIS000 acceleration record

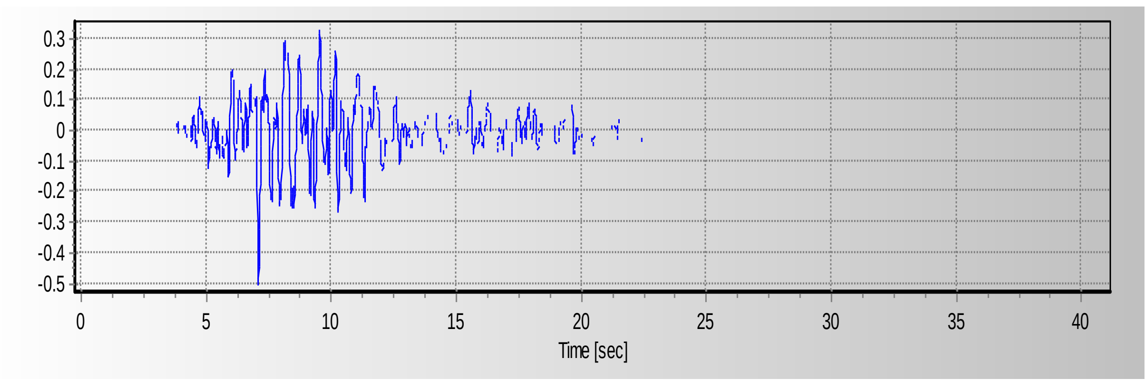

Figure A.2 NIS090 acceleration record

JMA Station Record

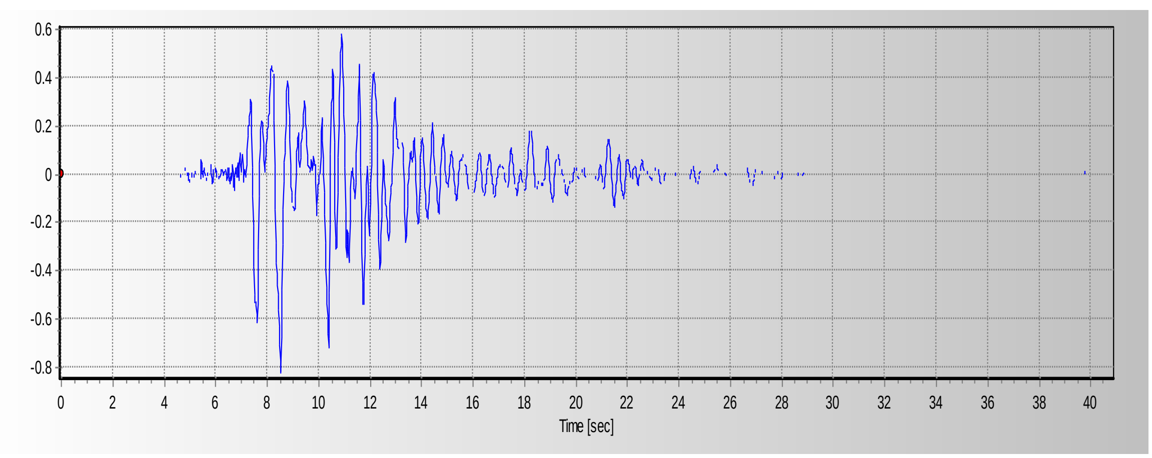

Figure A.3 JMA000 Acceleration record

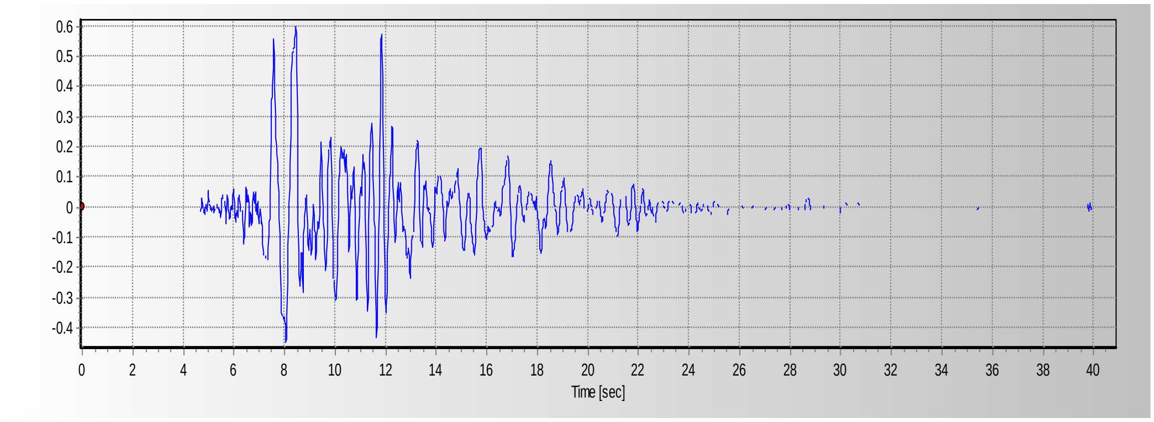

Figure A.4 JMA090 Acceleration record

Response Spectra

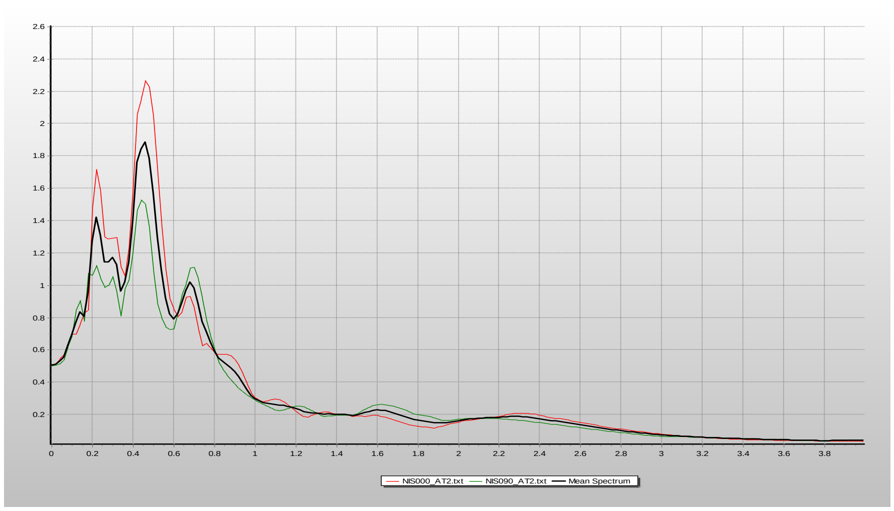

Figure A.5 Response spectra for Nishi-Akashi record

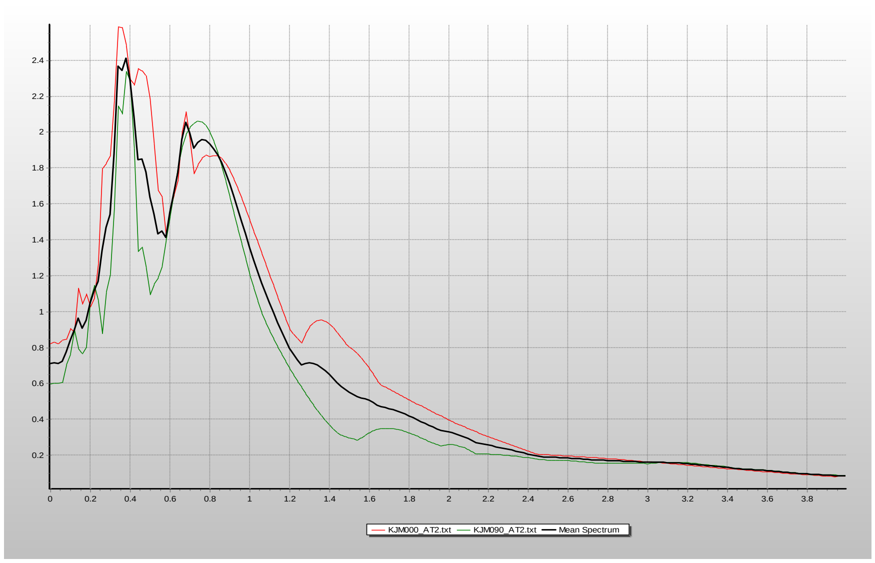

Figure A.6 Response spectra for JMA station record