Refining Historical earthquake Data Through Modeling and Scale Model Tests

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ABSTRACT

This study was performed for the reevaluation of historical earthquake records which occurred in Korea through tests and numerical analyses. For the scale model tests, static and cyclic lateral load tests on wooden frames that constitute a Korean ancient commoner's house were conducted. Full-scale models of two types of frames were used for testing. Two 1:4 scale models were tested for rock and soil foundation conditions. Scaled real earthquake time histories were inputted for the tests. The peak ground acceleration (PGA) at the collapse of the house at the soil site was 0.25g, whereas PGA for moderate damage at the rock site was 0.6g. The intensity of major historical earthquake records related with house collapses was reevaluated based on the results of these scale model tests. The magnitudes of historical earthquake records related with house collapses were estimated considering the magnitude, epicentral distance, soil condition and aging of the house. Eighteen artificial time histories for magnitudes 6-8, epicentral distances 5 km - 350 km and hard and soft soil condition were generated. The aging effects of the house was modeled as the lateral loading capacity of wooden frames represented by hysteretic stiffness decreased linearly with time.

INTRODUCTION

The seismic hazard for countries that are located in a stable continent is usually based on historical earthquakes. In Korea, one major source of uncertainty in seismic hazard analysis was identified as the over-estimation of intensity of historical earthquake records that lead to the relatively high occurrence rate of large earthquakes.

A typical damage record of a historical earthquake is the "*ground was shaken and commoners' houses collapsed and people died*". The number of deaths in the largest event was approximately 100. The estimated Modified Mercalli(MM) Intensity of these events in Kyungju City area varied from VIII to X depending on experts [KAERI, 1982].

The MM Intensity or Japanese Meteorological Agency (JMA) intensity scale may not be directly applicable to the evaluation of Korean earthquakes, because the structure of a Korean ancient commoner's wooden house is different from that of one found in Japan or Europe.

For the reduction of uncertainties in seismic hazard analysis, the intensity and magnitude of large historical earthquakes were reevaluated through the experimental and numerical studies. In this study, the typical Korean ancient house was defined as a three-bay-straw-roof house

HISTORICAL EARTHQUAKE RECORDS AND LOCAL SITE CONDITION

Typical Earthquake Records

The number of historical earthquakes in Korea for the periods between 2 AD to 1905 is about 1,900. Among the historical earthquake records, approximately fifteen events include descriptions of house collapses followed by the loss of lives. Among those, at least six events occurred in the Kyungju City area where the Yangsan Fault systems passes through. The estimated MM intensity of these events in Kyungju City area varied from VIII to X depending on experts [KAERI, 1982]. It is worth noting that during the 1995 Kobe earthquake the damages of wooden houses at soft soil, and houses degraded due to termite or fungi, were found to be much more severe than those founded on rock, and those relatively un-degraded houses [Doi, et al., 1996].

Several large historical earthquake records in Kyungju City area, located 35 km from the northwest of the Wolsong NPP site in the southeastern part of Korea, have raised same important issues, such as the seismic design level of NPPs and the activity of the nearby fault systems since 1982. Also, the results of the probabilistic seismic hazard analysis (PSHA) for several NPP sites have shown relatively high hazard levels and large uncertainties.

<u>Soil Characteristics of Kyungju Site</u>

Among historical earthquake records, most of the large events related with house collapses and loss of lives occurred in Kyungju, Kaesung, and Seoul area. Kyungju City area is the typical alluvium soil site in Korea [KAERI, 1999]. The bedrock of the Kyungju City area consists of several kinds of sedimentary rocks formed in the Cretaceous period of the Mesozoic Era, and is intruded by granite. Sedimentary rock layers are composed of complex layers of shale and sandstone. The upper layer of the bedrock is weathered rock overlain by residual soil or alluvium layers. Alluvium is widely distributed around the Hyeongsan River, Namchun River, and Pukchun River, which run across Kyungju City. The thickness of the alluvium layer is increased as the distance from the Mt. Kumi, located in the west of Kyungju City, is increased. The thickness of the alluvium layer varies from 1 m to 20 m depending on location.

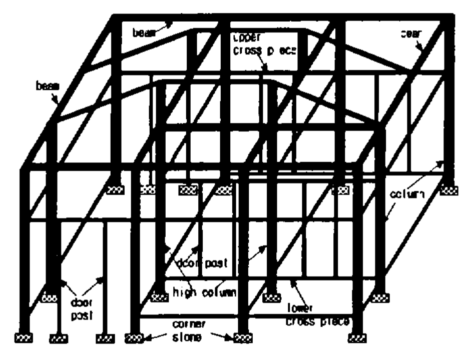
Recently, geotechnical seismic sensitivity analysis for Kyungju City was performed by several researchers [Sun, et al., 2000]. For the sensitivity analysis, they performed in situ tests including boring investigation, cross-hole, down-hole, and SASW (Spectral Analysis of Surface Waves) for the typical 13 areas in Kyungju City. It was found that most of the areas have deep alluvial layers up to 40 m thick. The alluvial layer can be classified as S_C and S_D based on the results of in situ test including seismic techniques. The acceleration amplification characteristics of the alluvial layer was analyzed using SHAKE91. From these analysis results, it was noted that the acceleration at surface is larger than that at bedrock. The max. amplification factor was 2.3. The frequency contents of the surface wave showed that the band of resonant frequency moved to the lower frequency range.

The amplification of acceleration at ground surface, due to the alluvial layer, can cause more significant structural damages. In the case of the ancient wooden house, the structural damage can be serious because the natural frequency of a wooden house is very low [Seo, et al., 1999b]. From this point of view, it is possible to assume that house collapses written in the historical earthquake records, were caused by the large amplification of a surface wave induced by a small or intermediate earthquake. Also the possibility that the liquefaction or differential settlement causing the house to collapse cannot be ignored.

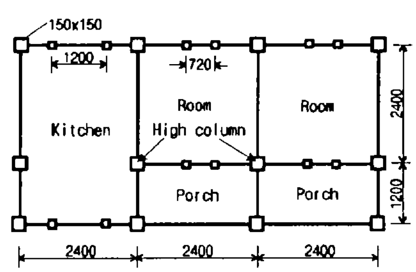
STATIC AND DYNAMIC TEST OF WOODEN HOUSE

<u>Tests of Wooden Frames</u>

The most general shape of a commoner's house in the ancient period up to the 19th century was judged as the three-bay-straw-roof wooden house that consisted of one kitchen, two rooms and a porch [Seo et al. 1999]. Fig. 1 shows the typical wooden frame of the prototype house less the roof frame. The dimensions of the prototype house are 7.2 m (l) x 3.6 m (w) x 2.4 m (h). Two types of wooden frames, one connecting the perimeter columns and the other connecting center columns and perimeter columns in the transverse direction are used. The beam-column joint typically used is a tenon. No nails or bracing are used in the frame. Natural stones two or three times bigger than the column size and having a relatively flat surface are used as cornerstones. The structure is standing freely on the corner stones. The mass of the roof is about 8,840 kg and is distributed by 12 columns. Pine lumber is used for the structural members.



(a) Perspective view



(b) Plan view

Fig. 1. Structure of the prototype wooden house (unit: mm)

<u>Model Fabrication</u> Two typical types of full-scale wooden frames were fabricated with fresh pine lumber. The type 1 frame connects the perimeter columns which the type 2 frame links an inner high column and perimeter columns in the transverse direction. The beams used for the type 1 frame are smaller than the one in the type 2 frame. Two cross members

are placed below the beam between columns of the frame. The beam-column joint typically used at the top of the perimeter columns is a tenon, whereas the joint between the high column and beams is a dovetail but wooden nails were also used. The joint between the cross member and column is a dovetail

Test Setup Each frame was installed in a vertical position. The bottom of each column was connected at the 60 mm hole to the steel frame fixed to the floor by hinge devices. The top of the frame was linked horizontally with an actuator attached to the reaction wall. Two LVDTs (Linear Variational Displacement Transducers) were installed to measure the horizontal deflection.

Test Procedure A displacement control method was used in static and cyclic tests. The rate of displacement of the actuator was 0.2 mm/sec in the static test. Static tests were performed up to the displacement limit of 200 mm except for one type 1 frame that was initially tested up to the drift of 400 mm for the purpose of identifying its ultimate strength.

In the cyclic load tests, displacements were increased from 3 mm to 96 mm at six levels. Four cycles with equal displacement were loaded in each step. The duration of each cycle of loading was 1 min/cycle.

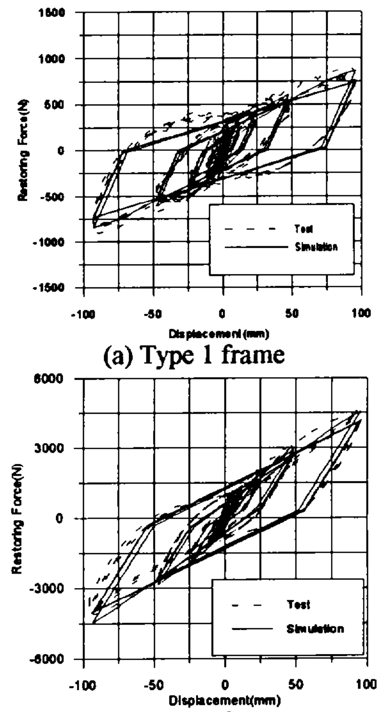
Test Results and Discussion The static force-displacement curves from the static tests show that the lateral loading capacity of the frame is very small compared with other types of frames, such as braced frames [Sugiyama, 1996]. Also, very large nonlinearity and no distinct yield points were identified. Local fluctuations of strength, like a saw tooth, were observed as the displacement increases. It is judged that the slip among the members that constitute the joint and/or the change of contact pressure occurs at the joint.

The hysteretic curves of each Type 2 frame obtained from the cyclic tests are shown in Fig. 2 with the simulation results using the Double Target model [Ohtori and Ishida, 1995]. It is noted that the shapes are quite different from those of the nailed frame or braced frame [Ewing et al., 1980; Kamiya, 1988], but similar to that of a rubber bearing [Ohtori and Ishida, 1995]. The characteristic stiffness reduction of wooden frames, due to the loading history and the magnitude of the maximum experienced displacement, can be well represented by the modified Double Target model.

Failure Mode The failure mode of each frame was investigated at the end of the static and cyclic tests. Failure modes observed in the static and cyclic tests were almost the same. The typical failure mode of the type 1 frame was shear failure of the mortise branches of the tenon. The maximum displacement of the type 1 frame at failure was about 400 mm (1/6 radian). The failure mode of the type 2 frame was bending failure of the mortise branches of the tenon. The maximum displacement of the type 2 frame at failure was 250 mm (1/9.6 radian).

Shaking Table Test of Wooden House Model

Model Fabrication Two 1:4 scale models nominally identical to the prototype house were fabricated with fresh pine lumber. Two different carpenters, each with comparable skill, made each model. The beam-column joints of the frames were also scaled. The artificial mass method was applied in the model test.



(a) Type 1 frame

(c) Type 2 frame

Fig. 2. Comparison of hysteretic curves; test vs. simulation

The mass of the roof, one half of the wall masses, and other added masses were lumped together at the roof level. A total mass of 930 kg made of 150 mm (wide) x 600 mm (long) x 25 mm (thick) steel plates was uniformly distributed at the roof, as shown in Fig. 3.

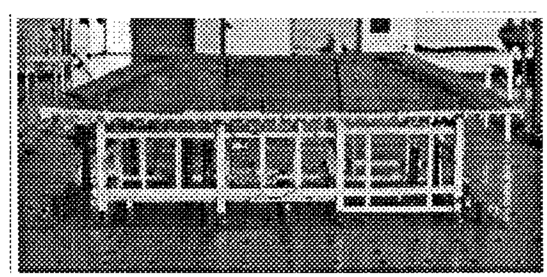


Fig. 3. 1/4 Scale model of ancient wooden house

Tests Setup Nine accelerometers and six linear variable displacement transformers (LVDT) with 25 mm capacity were installed to measure the responses at the top of the two corner columns and one center column

The boundary condition between the cornerstone and the bottom of the column was assumed to be a hinge and was modeled by mechanical devices specially made of ball bearings. The assumption of the hinge boundary is based on the facts that the contact surface is irregular in plane and the mass center is located at the top of the flexible frame. So rotation rather than sliding at the contact surface is more likely to happen during an earthquake.

Input Motion The Nahanni earthquake, which occurred in eastern Canada in December 1985 and was recorded at a rock site, was inputted to model 1 as a typical rock motion. The Imperial Valley earthquake of October 1979 in the western United States, recorded at El Centro Array No. 5, was used in model 2 to simulate the soft soil condition. The topographic and local soil characteristics of both Kyungju City and El Centro areas are similar. So the Imperial Valley earthquake was chosen as the input motion for the soft soil condition.

Test Procedure Three components of earthquakes were inputted simultaneously. The vertical component was scaled to 2/3 of the horizontal component. The PGA was increased in increments of 0.1g from 0.1g to 0.6g for model 1, and for the rock conditions, whereas the PGA was increased from 0.05g to failure in increments of 0.05g for model 2, for the soft soil conditions. Random white noise was inputted separately for measuring the natural frequency and damping ratio at the elastic level.

Test Results and Discussion The fundamental natural frequencies of model 1 measured in the elastic range for the white noise input with peak acceleration of 0.025g were 3.32 Hz and 3.52 Hz in longitudinal and transverse direction, respectively. Those of model 2 for the same input were 3.32 Hz and 4.29 Hz, respectively. The difference of natural frequency in transverse direction was caused by the differences in model fabrication and the carpenter's skill. Since the length scale is 1/4, the frequency of the model is twice that of the prototype.

The modal damping ratio of the wooden house in the elastic range calculated from the random test results was about 7% in both directions. But the equivalent viscous damping ratios of the frame system measured from the cyclic load tests for large displacements [Seo et al. 1999a] were 27% and 13% in the longitudinal and transversal directions, respectively.

The peak responses of model 1, at the top of the column for the rock site condition, are summarized in Table 1. A permanent displacement of about 0.5 mm in the longitudinal direction of model 1 was observed at a PGA of 0.6g after the test. The max. responses of model 2 are at the top of the columns C1, C2, and C3 for the soft soil condition, which are presented in Table 2. The trends in acceleration and displacement response were almost the same as for model 1, but model 2 collapsed in the longitudinal direction at a much lower PGA level of 0.25g..

Nonlinear Dynamic Analysis Nonlinear dynamic analyses using the modified Double-Target model were also performed. Model parameters such as stiffness and damping ratio, which were obtained from full-scale frame tests, were used. Test models were idealized as single-degree-of-freedom lumped mass systems. As shown in Fig. 4, the Modified Double-Target model is useful for the simulation of nonlinear behavior of the wooden house.

Table 1. Max. response of model 1 at the top of the column for Nahanni earthquake, rock site

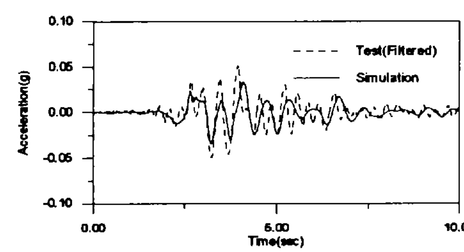
Input PGA level		0.1g	0.2g	0.3g	0.4g	0.5g	0.6g
Displ. Response (mm)	X	1.19	2.85	3.91	6.24	7.83	9.84
	Y	1.08	3.28*2	5.89*2	4.69	6.21	7.40
	Z	-	-	-	-	-	-
Acc. Response*1 (g)	X	0.067	0.086	0.111	0.056*3	0.168	0.172
	Y	0.052	0.071*2	0.082*2	0.063*3	0.087	0.153
	Z	0.093	0.173	0.251	0.024*3	0.379	0.398

*1: acceleration responses were low-pass filtered at 50 Hz
*2: accelerations were input 150-170% greater than specified level in all frequency bands.
*3: data measured are not reliable

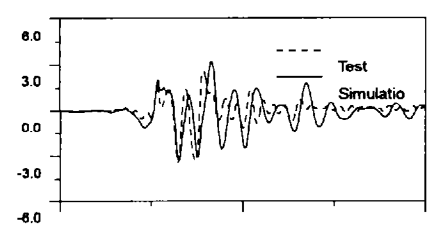
Table 2. Max. response of model 2 at the top of three different columns for Imperial Valley earthquake, soil site

Input PGA level	Model Response at Top of the Column						
	Direction	Max. Acceleration (g)*1			Max. displacement(mm)		
		C1	C2	C3	C1	C2	C3
0.05g	X	0.038	0.043	0.052	4.28	4.20	5.13
	Y	0.036	0.029	0.035	2.33	1.59	0.80
	Z	0.041	0.045	0.040	-	-	-
0.08g	X	0.056	0.058	0.064	9.74	9.40	10.71
	Y	0.045	0.040	0.041	4.13	3.14	2.44
	Z	0.072	0.078	0.072	-	-	-
0.10g	X	0.065	0.062	0.067	14.29	13.29	14.96
	Y	0.047	0.053	0.049	4.97	4.29	2.46
	Z	0.090	0.083	0.082	-	-	-
0.15g	X	0.099	-	-	-	-	-
0.20g	X	0.146	-	-	-	-	-
0.25g	Collapse						

*1: responses were low-pass filtered at 50 Hz



(a) Acceleration response



(b) Displacement response

Fig. 4. Comparison of the displacement response results of test and simulation, imperial Valley earthquake, input PGA

= 0.08g, y-direction

Intensity Estimation As mentioned above, the prototype wooden house was damaged severely at 0.6g at a rock site, but it collapsed at PGA=0.25 g at a soil site. These results are for new houses built with sound materials and can be thought of as upper bound or close to upper bound values. This is because, the collapsed houses in historical earthquake records may be comprised of very old houses, i.e. 50 or 100 years old. This hypothesis was clarified after the Kobe earthquake in 1995.

In order to convert the test results to intensity, we utilized empirical formula between peak ground acceleration and MM intensity. Several equations such as Trifunac and Brady, (1975), Ambraseys (1974), Hershberger (1956), Gutenberg and Richter (1956), Murphy and O'brien (1977) were developed from the earthquake data [ATC, 1985]. However, wide variations can be observed. As a conclusion, it is estimated that both site conditions had an MM intensity VIII.

NUMERICAL STUDY FOR MAGNITUDE ESTIMATION

The magnitudes of historical earthquake records related with house collapses are estimated considering the magnitude, epicentral distance, soil condition and aging of a house. Eighteen artificial time histories for magnitudes 6-8, epicentral distances 5 km - 350 km and hard and soft soil condition were generated. The aging effects of the house was modeled as the lateral loading capacity of wooden frames represented by hysteretic stiffness decreased linearly with time.

Input Motion Generation

Frequency contents of ground motions change according to the characteristics of wave propagation and local site conditions. In this study, design response spectrum developed by Ohsaki [Hisada, et al., 1978] was used for the generation of various input motions. Table 3 shows the spectral velocities at control points.

Table 3. Design response spectrum [Hisada, et al., 1978]

Field	Magnitude, M	Epicentral Distance, Δ(km)	Control Point									
			A		B		C		D		E	
			T_A	S_V	T_B	S_V	T_C	S_V	T_D	S_V	T_E	S_V
Near	8	25		0.6	0.10	10	0.30	30	0.50	30		12
	7	10		0.7	0.10	11	0.23	24	0.45	24		7
	6	5		1.2	0.10	17	0.13	21	0.35	21		3
Inter-mediate	8	120		0.5	0.20	18	0.35	32	1.00	32		26
	7	45	0.02	0.5	0.13	11	0.33	28	0.80	28	2.0	19
	6	15		0.6	0.10	10	0.25	24	0.60	24		12
Far	8	350		0.5	0.22	26	0.37	44	1.20	44		42
	7	150		0.5	0.14	15	0.35	38	0.90	38		32
	6	60		0.5	0.10	10	0.33	33	0.70	33		20

The peak ground acceleration values were calculated using the following equation developed by Shin [Shin, et al., 1998].

$$\ln(a) = 0.49 + 1.2M - 0.84\ln\Delta - 0.0061\Delta \qquad (1)$$

where a is the peak ground acceleration (gal), M is the earthquake magnitude, and Δ is the epicentral distance (km).

When the level of ground motion becomes large at a soil site, the soil surface shows non-linear characteristics so that the ratio of soil surface to bedrock motion is changed. The ground spectrum, defined for the bedrock, was converted to that of one for the soil surface. The acceleration response spectra at the soil surface can be obtained from equation (2) [Sugito, et al., 1986].

$$S_s = \beta_s \cdot S_r(M,\Delta,T,h) \qquad (2)$$

where S_s and S_r are the spectral acceleration at ground surface and bed rock, respectively. β_s is the conversion factor. T and h are period and damping ratio, respectively. The conversion factor, β_s, is determined from the N-value of standard penetration tests. In this study, the soil depth at Kyungju city is assumed 20 m. The average N-values for hard and soft soil site are assumed 30 and 6 respectively. A typical converted acceleration response spectrum for the near field earthquake is shown in Fig. 5. Eighteen artificial ground motions that can envelop the converted acceleration response spectrum were generated as input motions.

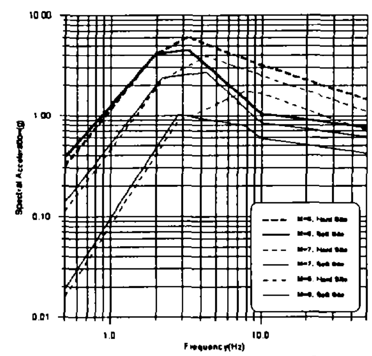


Fig. 5. Acceleration response spectrum for near field earthquakes

Aging Characteristics of Wooden House

Since no data for the lateral load capacity reduction of a traditional wooden house is available in Korea, the reduction rate developed in Japan was used in this study. In Japan, the degradation rate of a wooden house is represented by the reduction rate of lateral load capacity considering the service years and the maximum service life of 150 years. The remaining capacity of a wooden house is assumed to be decreased linearly as in the following equation [Suzuki, 1995].

$$Remaining\ Capacity\ \text{Ratio} = 1 - \frac{Service\ Years}{150} \qquad (3)$$

Damage Assessment Criteria

In Japan, the damage level of a wooden house is divided into three levels, minor damage, severe damage, and collapse. Collapse and severe damage of a wooden house are defined as the story drift exceeding 100 mm (1/30 rad) and 50 mm (1/60 rad), respectively [Kitahara and Fujiwara, 1988]. Converting to a wooden Korean house, the collapse and severe damage can be defined as the max. story drift of 80 mm and 40 mm, respectively.

Magnitude Estimation of Historical Earthquake

Max. displacement responses of wooden houses for far, intermediate, and near earthquakes are shown in Fig. 6 ~ 8. For the far field earthquakes, the wooden houses suffer only minor damage, regardless of earthquake magnitude, epicentral distance, site soil condition, and the degradation level of the wooden house. Wooden houses at the soft soil site suffered more damage than that at the hard soil site. As shown in Fig. 7, the wooden houses at the soft soil site suffered severe damage from the earthquakes greater than magnitude 7. The 100 year old wooden house at the soft soil site collapsed under the magnitude 7. All of the wooden houses subjected to a magnitude 8 earthquake suffered severe damage.

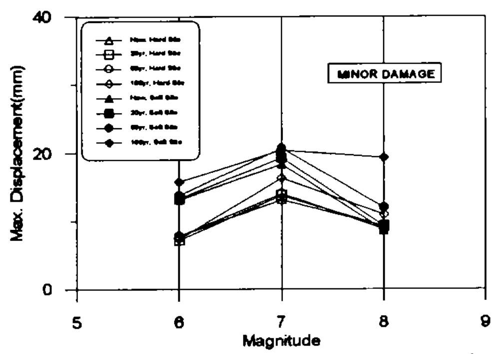


Fig. 6. Max. displacement response for far field earthquakes

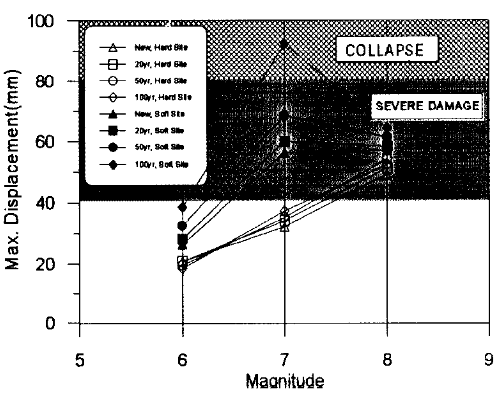


Fig. 7. Max. displacement response for intermediate field earthquakes

As shown in Fig. 8, all of the wooden houses subjected to a near field earthquake of magnitude 8 collapsed. For magnitude 7, they suffered severe damage, regardless of earthquake magnitude, epicentral distance, site soil condition, and degradation level of the wooden house. The degradation levels of the wooden house were considered as a remaining lateral load capacity of 0, 20, 50, and 100 years old wooden house calculated by the equation (3).

The damage description of the largest historical earthquake is "*commoners' houses collapsed and about 100 people died*". Assuming 5 residents per house, the max. number of houses collapsed could be about 20. Also, assuming that only the damages of capital and nearby areas were recorded up to 14th century, the actual damages would have not been great. Because, if a large earthquake really had happened in a capital city with large population, the damage description would have been a fatal one. It is also natural to presume that houses with small capacity due to degradation collapsed or were damaged.

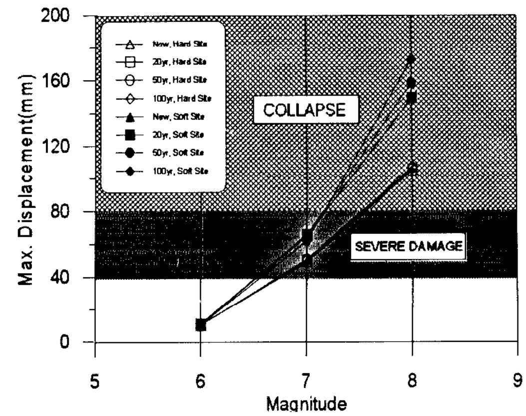


Fig. 8. Max. displacement response for near field earthquakes

It was identified from the figures that, for near field earthquakes, severe damage occurred at soil and rock sites regardless of aging at magnitude 6.5 and 6.7, respectively. For intermediate field earthquakes at soil site, severe damage occurred at magnitude 6.2 for the 50-year old house. Therefore, the magnitude of historical earthquakes occurred in Kyungju City, which is a typical soil site, is estimated about 6.2 with epicentral distance of average 15 km. Among historical earthquake records of house collapses, it is hard to distinguish any specific event related with rock site condition. However, the magnitude of earthquakes at the rock site is estimated to be the same as for a soil site.

CONCLUSIONS & RECOMMENDATIONS

The shake table tests on two 1:4 scaled models of an ancient wooden house were performed to estimate the PGA level and earthquake intensity. The PGA of the collapse of the house at soil site was 0.25g, whereas the PGA of the severe damage of the house at rock site was 0.6g. The intensity of major historical earthquake records related with house collapses was quantitatively estimated to be MM VIII.

From the results of seismic analysis considering input motion characteristics and structural degradation level, the magnitude of historical earthquakes occurring in Kyungju was estimated to be about 6.2.

This is an on-going project with the next refinement expected from the evaluation of mathematical models of structures and distributed soil-structure stiffness, calibrated with scale model tests.

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