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EFFECTS OF LONG-DURATION EARTHQUAKES ON BUILDING SAFETY IN ALASKA-A CASE STUDY

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ABSTRACT

The objective of this study is to perform a preliminary study on the effects of long-duration earthquakes on building safety, using a typical 20-story steel structural building, Atwood Building (AB), in Anchorage, Alaska, as a representative example. The AB has three lateral force resistant systems: moment resistant frames, steel shear walls and steel braces. To monitor AB's seismic behavior, 32 accelerometers were installed in 10 stories and 21 sensors were installed in the downhole array near the AB. A linear elastic model for this building was established using a finite element software Perform 3D and verified by ambient vibration tests and the recorded seismic response time history data from the building. Based on the verified linear elastic model, corresponding inelastic nonlinear and ultimate capacity models were developed, in which beams, columns and penal zones were modeled with stiffness deterioration and strength degradation and the steel shear walls were modeled by "strips" for the ultimate strength. Based on the performance-based design concept, nonlinear time domain analyses were performed to identify critical elements of the structure and assess the structural performance in different levels in terms of Immediate Occupancy, Life Safety and Collapse Prevention due to short vs. long-duration earthquakes. It was found that long duration earthquakes may course much significant damage to structures than short-duration earthquakes with the same pick ground accelerations.

Introduction

Various investigations on ground motion characteristics of strong earthquakes indicate that the duration of a seismic event is critical for quantitative estimation of the structural damage. The knowledge on the cumulative fatigue development of structural components under the long-duration cyclic seismic loading is of significant importance to the earthquake engineering community to develop adequate measure to prevent the costly damage. However, provisions of current seismic design codes were developed considering primarily large surface faulting earthquakes in only 20 to 30 seconds of strong ground motion.

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The duration of strong earthquake ground motion characterizes the total energy excitation of a structure is particularly important in the case of structure behave nonlinearly, as number response cycles is directly related to the duration. Large accelerations may not be always necessary to drive a structure into the nonlinear response. The ground motion of moderate acceleration and long duration can also result many cycles of nonlinear response which may generate cumulative damage or low-cycle fatigue and causes structural instability or eventual collapse.

Modern design methodologies rely on accurate predictions of structural performance and estimate the structural reliability based on code calibration and design. The failure of a steel structure due to strong motion earthquakes may occur either by exceed of a preset displacement level by a single excursion of the response (single excursion mechanism) or by exceed of a preset damage level (Suidan and Eubanks 1973) of structural response due to accumulation of number of the excursion by earthquake excitation (fatigue failure mechanism). Many assessment of cumulative damage on steel structure is based on evaluation models (Park and Ang, 1985; Cosenza et al., 1993; van de Lindt and Goh, 2004a and 2004b). These studies provide valuable idea on the effect of low cycle loading on cyclic deterioration in strength, stiffness and energy dissipation capacity of structural members. However, most of assessment models were based on the assumption of single degree-of-freedom (SDOF) oscillators with idealized elasto-placetic models to obtain statistical measure of dissipated hysteric energy with cumulative fatigue. On the other hand, limited experimental studies (Krawinkler and Zohrei, 1983; Chai and Romstad, 1997; Yamada, 1997; Taucer et al., 1998) have been performed. Nevertheless, most of the experimental studies were limited to typical type of structural members and connections and also very costly. Therefore, the evaluation of available ductility during damaging earthquakes for a specific existing structure requires deterministic detailed research in addition to those investigations mentioned above.

The intent of this study is to provide the structural engineers with demonstration results from a case study for improved understanding of the effects of long duration earthquakes on building structures. This preliminary study focuses on the accumulated damage due to the long-duration earthquakes, In the future study the effects of low-cycle fatigue on material capacity will be included.

Building and Instrumentation Description

The south central region of Alaska is near a plate subduction zone, which is one of the most active seismogenic zones in the world. Earthquakes in subduction zones are characterized by large magnitude and long duration earthquakes. The 1964 Alaska earthquake had a moment magnitude of 9.2 and produced ground surface motions with durations up to four-minutes. In this study, we have therefore selected to predict the long-duration seismic performance of an existing Advanced National Seismic System (ANSS) instrumented moment resistant steel frame structure, Robert B. Atwood Building (AB) in Anchorage, AK. The AB is an idealized choice for such study as it is located on an area where the occurrence of large magnitude, long duration earthquakes are highly probable. The deaggregation of the National seismic hazard maps of U.S. Geological Survey (Wesson et al., 1999) clearly indicates two significant events are highly probable at the building site, a large magnitude (9.2) event from subduction zone (at a distance of 50-60 Km) and a crustal event of magnitude 7.5 (at a distance of 65 km). Both of these events potentially generate the long duration ground motions at the building site.

The AB, designed and constructed in the early 1980's, is located in Anchorage’s downtown area. In 2003, the AB was chosen for seismic instrumentation, sponsored by the Advanced National Seismic System (ANSS) program of United States Geological Survey (USGS), to study the building response and the effect of soil-structure interaction as the surrounding area suffered extensive damage during 1964 Great Alaska earthquake. The building has 20 stories with its basement used as a parking garage. This building is 38.5 m x 38.5 m in plan and 80.5 m in height. The building foundation is reinforced concrete spread footings. The lateral resistant system of the AB consists of a moment-resisting steel frame structure, a 14.63 m x 14.63 m in-plan center steel plate shear wall (SPSW) core, and a bracing system in the N-S direction. The typical elevation and plan views are shown in Figure 1 (Liu, et al., 2005).

The 32-channel seismic sensors (CH 1-CH 32) are located in the basement garage, on the 1st, 2nd, 7th, 8th, 13th, 14th, 19th, 20th and 21st (roof) floors of the building, as shown in Fig. 1 (Liu, et al., 2005). The recorded signal from each recorder is sampled at 200 samples/sec and the recorders are operating in trigger mode with triggering thresholds varying from 1 gal at the basement to 40 gal at the roof level (Liu et. al., 2005).

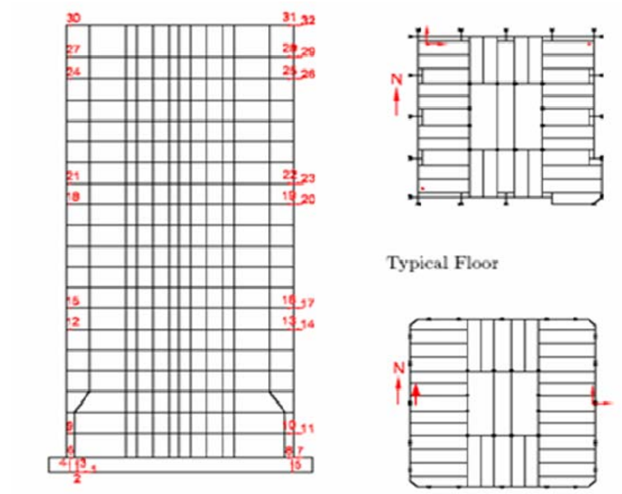


Figure 1. Elevation and plan views of Atwood Building with accelerometer locations indicate.

Modeling Approach

To investigate the effects of long-duration earthquake on building safety based on the concept of performance based design, the finite element (FE) software, PERFORM 3D, was used for the AB structure in the case study. In the modeling approach, three FE models were created to represent different characteristics of the AB structure in different damaging levels, named linear elastic (LE), nonlinear (NL) and ultimate capacity (UC) models, respectively.

Verification of Linear Elastic Finite Element Model

Linear Elastic Finite Element Model

The LE model for this building was established using frame elements for beams, columns and brace members, whereas the shell elements for SPSWs. All steel materials were considered linear elastic with elastic modulus E=200 GPa (29,000 ksi). In the LE model, the composite actions for floor beams were considered and the effects of end zone in frame elements were included.

Verification of Linear Elastic Finite Element Model

The main purpose of the FE LE model was to use as a baseline model; therefore, the LE model was verified by ambient vibration tests and the recorded seismic response time history data from the building instrumentation to make sure the accuracy. A comparison example is shown in Table 1.

Table 1. Comparison of the structural periods from the LE model with the identified from an ambient vibration test.

	E-W (second)			N-S (second)			Torsion (second)		
	1st mode	2nd mode	3rd mode	1st mode	2nd mode	3rd mode	1st mode	2nd mode	3rd mode
6/4/2001 Ambient	2.19	0.66	0.34	1.82	0.57	0.3	1.59		
LE model	2.187	0.6866	0.3615	1.828	0.5951	0.3209	1.585	0.5745	0.3404
NL model	2.19	0.6823	0.36	1.822	0.5943	0.321	1.592	0.5759	0.3411
UC model	2.192	0.6598	0.3335	1.819	0.5859	0.3134	1.538	0.5497	0.3193

Nonlinear Finite Element Model

Nonlinear Finite Element Model

Based on the baseline LE model, a corresponding nonlinear inelastic FE model was developed. The purpose of the FE NL model was to predict the structural nonlinear performance of the AB structure under the design level strong earthquake shaking, including the possible larger deformation, P-Δ and P-δ effects, material yielding, plastic hinge development, and tension-only action for brace systems. Since the PERFORM 3D can't take the buckling into account for the SPSWs, the design concept of steel plate girders was used in the nonlinear shear wall modeling. The determination of nominal shear strength of a SPSW was based on the elastic and inelastic buckling behavior.

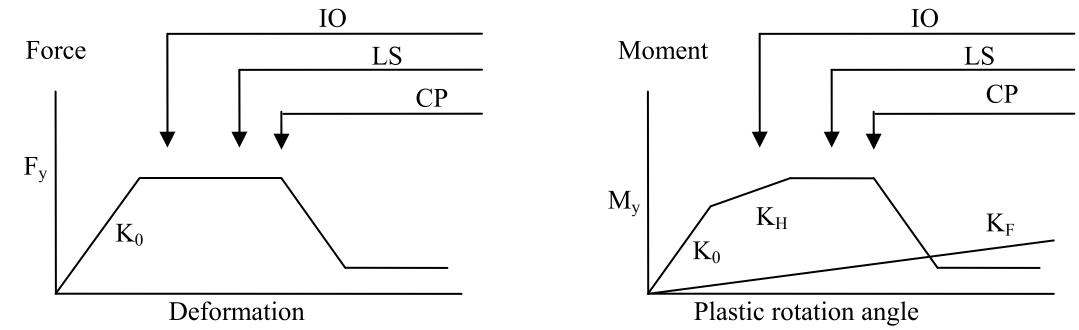
In the NL and UC models, beams, columns, penal zones, braces and SPSWs were defined based on different performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Table 2 shows examples of the component deformations used in this study base on suggestions from FEMA 356 for steel structures.

In Table 2, θ_y is the plastic rotation angle at the yield level. The acceptance criteria for SPSWs were $0.5\theta_y$, $10\theta_y$ and $13\theta_y$ for IO, LS and CP, respectively, in FEMA 365 for shear walls with stiffeners to prevent shear buckling. When applying these criteria to SPSWs without stiffeners in this study, adjustments were considered; therefore the values of $0.5\theta_y$, $10\theta_y$ and $13\theta_y$ were modified into $<0.5\theta_y$, $<10\theta_y$ and $<13\theta_y$, respectively. For SPSW's with different dimensions and boundary conditions, these adjustments were not the same.

Table 2. Examples of the component deformations for different performance levels

	Component /Action	Acceptance Criteria		
		Plastic Rotation Angle, Radians		
			Primary	
		IO	LS	CP
1	Beams	θ_y	$6\theta_y$	$8\theta_y$
2	Columns	θ_y	$6\theta_y$	$8\theta_y$
3	Panel zones	0.0075	0.0228	0.0300
4	SPSWs	$<0.5\theta_y$	$<10\theta_y$	$>13\theta_y$

Both stiffness deterioration and strength degradation were considered in the modeling of beams, columns and penal zones for different acceptance of performance levels for IO, LS and CP. Figure 3 (a) represents the general force-deformation relationships for beams and columns using the elastic-perfect-plastic model, and Figure 3 (b) for penal zone using the tri-linear model with strain hardening.



3 (a) Deformation and acceptance criteria of a typical column.

3 (b) Deformation and acceptance criteria of a typical penal zone connection.

Figure 3. Examples of the force-deformation relations of columns and penal zone connections (with strain hardening) for depicting modeling and acceptance criteria.

Ultimate Strength for Steel Shear Walls

Many SPSWs in the AB structure are mainly steel plates without strong columns as boundary elements; therefore, they may not have significant post-buckling capacity and were modeled by the buckling model categories even for the ultimate capacity. For the walls with two-side strong columns were modified in the FE UC model by strip models to allow the development of the post-buckling capacity in the structure, because with strong boundary beams and columns, the steel plate can develop the post-buckling strength significantly even after the shearing tear-up in the diagonal direction.

In most of research results, it was recognized that the flexural stiffness of boundary columns were affects the slope of the strips. Consisting with the research result, the design codes are also suggests the slope of SPSWs should take the value as

$$\tan\alpha = \sqrt[4]{\frac{1+\dfrac{tL}{2A_c}}{1+th\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)}} \qquad (1)$$

in which, α is the angle between column and strip, *t* is the wall thickness, *L* and *h* are the width and height of the wall , A_c and A_b are the cross sectional area of column and beam. I_c is the moment inertia of column cross section. All units are millimeter.

Since it is impossible to set the strip angle exactly as the equation produced (because it will cause the strips from upper and lower floors not in the same locations), the closest angle which were created between the divided segment of column and beam were adopted, which was the same situation and treatment seen in many experimental work. Table 3 shows the angles and areas of strip elements in the UC model.

Table 3. Angles and areas of strip elements in the UC model

	t_w (cm)	Angle α of UC Model	Area Assigned (cm^2)
EW Wall Strip	7.94 (0.3125 in.)	45.4°	39.1 (6.06 in^2)
	9.53 (0.375 in.)	43.2°	44.7 (6.93 in^2)
	12.7 (0.5 in.)	43.2°	59.6 (9.24 in^2)
	17.15 (0.675 in.)	42.8°	79.89 (12.38 in^2)
NS Wall Strip	6.35 (0.25 in.)	36.17°	33.55 (5.25 in^2)
	9.53 (0.375 in.)	35.13°	50.12 (7.77 in^2)
	12.7 (0.5 in.)	35.13°	66.45 (10.34 in^2)

Earthquake Ground Motions-Long vs. Short

Selection of Long Duration Earthquake Motions

Three earthquakes were selected for this study: 1964 Great Alaska Earthquake (Alaska EQ), 1995 Mexico City Earthquake (Mexico EQ) and 1985 Llolleo Chile Earthquake (Chile EQ). The Alaska EQ, which had a Richter magnitude of 9.2 and lasted about four minutes, was no recorded the ground acceleration data for Anchorage area available. A synchronized acceleration time history generated by Mavroeidis, et al. (2008) was developed based on the geological structure of south-central Alaska and evaluated for the outcrop motion underneath the area. To get the corresponding ground motion time history in the Anchorage area, a linear wave propagation analysis from the outcrop to the ground surface was done by using the computer program Pro-Shake (EduPro, 2007) and the shear wave velocity profile at the AB site.

Generate the Corresponding Short Duration Acceleration Time Histories

To keep the same amplitude and the frequency contents in the short and long duration earthquakes, the short duration acceleration time histories have to be generated from the corresponding long duration earthquakes. For each above selected long duration earthquake, the Hamming window filtering is applied to include the maximum intensity of the ground motion in the time frame from 20-30 seconds, compatible with the procedure in creating the design spectra.

Scale the Ground Acceleration Input to Different Levels

Based on the concept of perform based design, the seismic hazards must category into deference levels according to probability analysis. In this project, the seismic hazards are approximately categorized into (1) weak motion: no specified probability, (2) design level earthquake: probability of 10% exceedance in 50 years, and (3) Maximum considered earthquake: probability of 2% exceedance in 50 years.

To simplify the evaluation, for the weak motion, several local earthquakes are considered as representative. The design level earthquakes were scaled from the selected three groups of short and long duration earthquakes. The scale factors were determined by comparing the design spectrum at the AB location (longitude = -149.8925 and latitude = 61.2156) and the response spectrum of the selected earthquakes at the structural fundamental period 1st mode T_1=2.2s. in the E-W direction and T_1=1.8s. in the N-S directions, respectively.

Structural Response to Earthquake Ground Motions-Long vs. Short

Time history analyses were performed on LE, NL and UC models using the weak ground motions, design level severe damaging earthquakes and maximum considered extreme rare earthquakes as input, respectively. Performing the linear elastic time history analysis was mainly for the purpose to compare the results with recorded data from the AB instrumentation to verify the LE modes. The analyses for the design level and the maximum considered earthquakes were for the purpose to evaluate the long-duration effects. Only some of the results were presented here.

Comparison of Linear Elastic Time History Results with Recorded Data from the AB Instrumentation

A relative weak ground shaking from one of the local earthquakes (Feb. 16, 2005, M_L=4.6) was chosen for conducting time history analyses to obtain model responses. The accelerations recorded from sensors channel CH 7 (in the E-W) and CH 8 (in the N-S) (see Figure 1) at the ground level were used as the input. The absolute acceleration and displacement time histories in the N-S and E-W directions at a node in the roof level in the LE model were computed and compared with the recorded time histories from the CH 31 (in the N-S) and CH 32 (in the E-W), respectively. Figure 4 shows the comparison between the recorded and computed time histories in the E-W direction.

Fairly good agreement can be observed between the simulated and recorded responses.

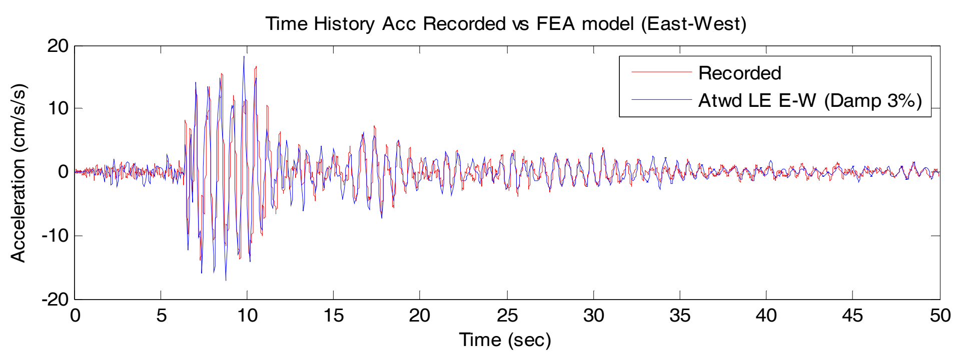
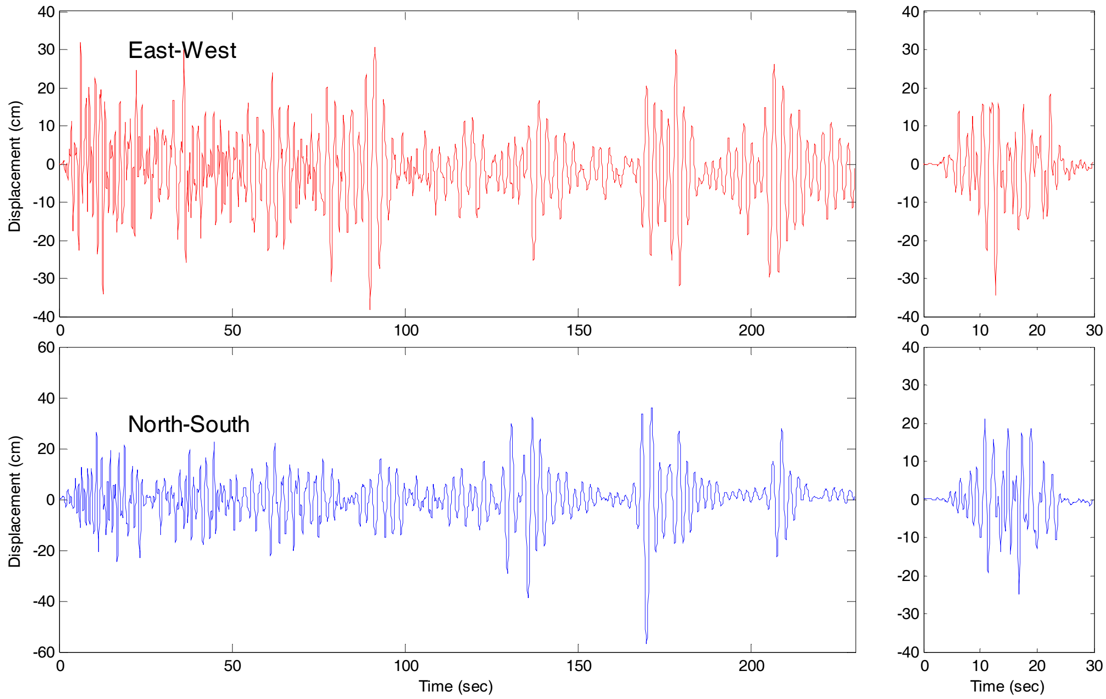


Figure 4. Comparison of time history of the LE model and its results with the recorded data from the AB instrumentation (3% damping, in the E-W)

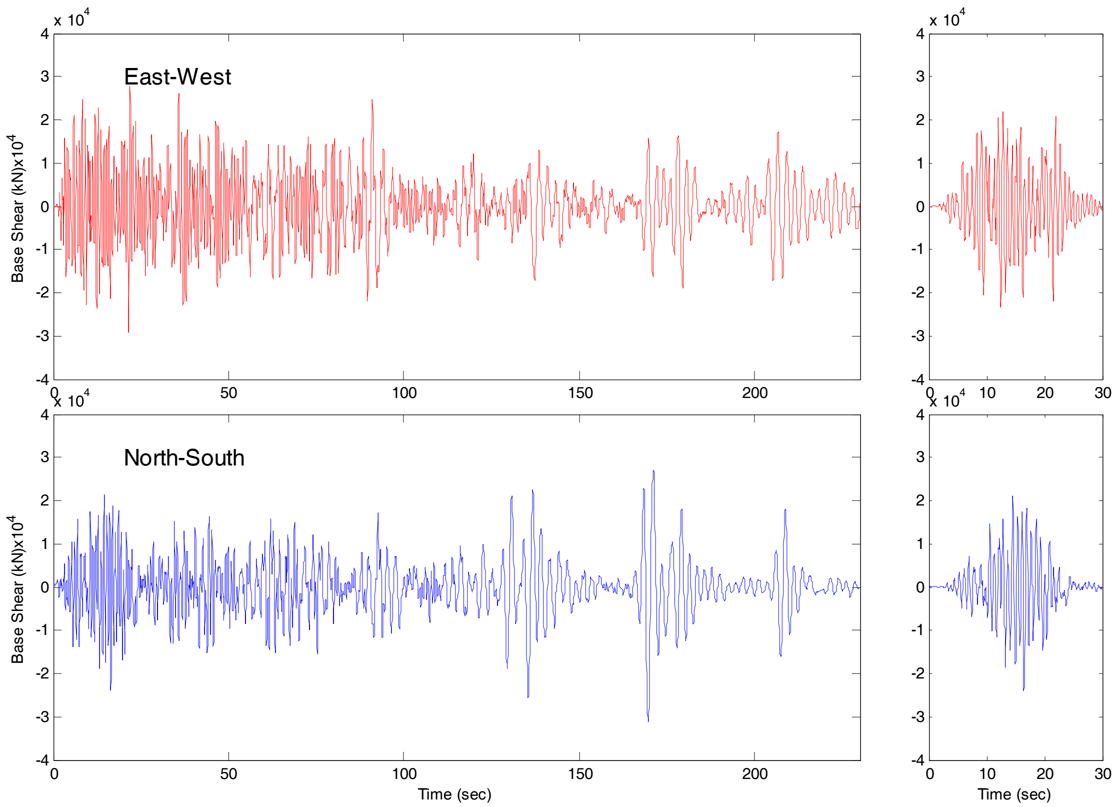
Nonlinear Time History Results From Design Level Damaging Earthquake

Three pairs of design level short-duration and long-duration earthquakes from the Alaska EQ, Mexico EQ and Chile EQ were used as input to the NL model to study the long-duration effects on structural responses in the design level. Figure 5 (a) and (b) show the displacement time history from long and short versions of design level Alaska EQ. Much larger displacement can be seen in the N-S direction in the long duration response. Figure 6 (a) and (b) show the base shear and Figure 7 (a) and (b) show the overturning moment time histories from long and short versions of design level Alaska EQ. Much larger base shear and overturning moment can be seen in both N-S and W-E directions in the long duration response.



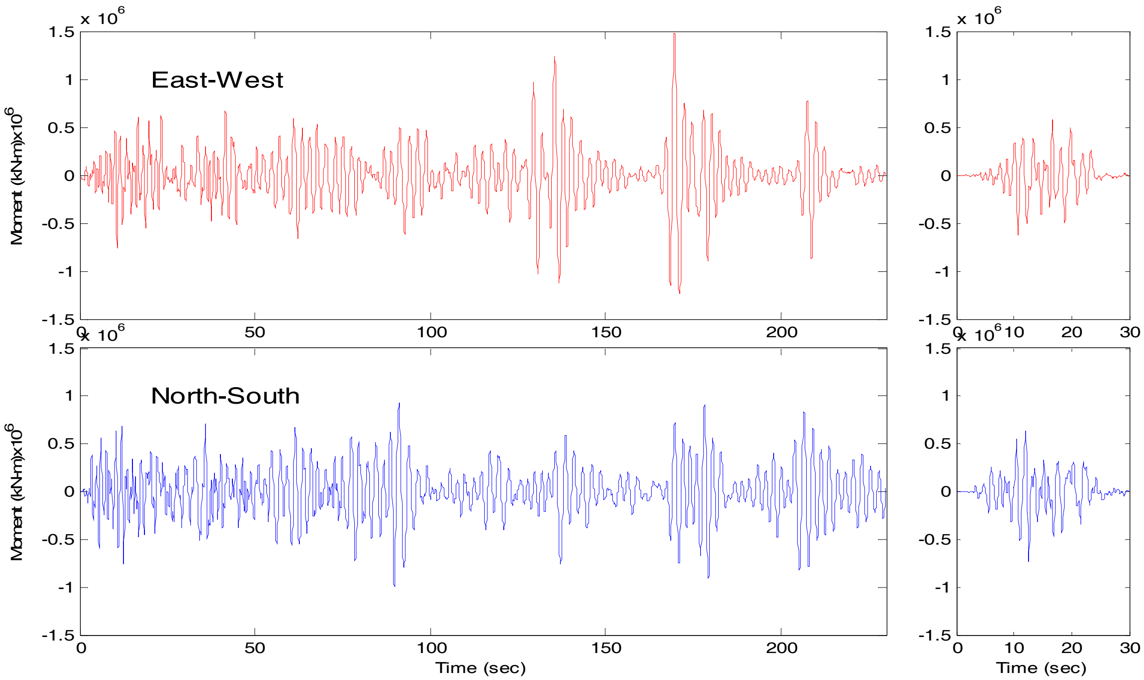
(a) Long duration (b) Short duration

Figure 5. Displacement time histories at the roof level, Alaska EQ



(a) Long duration (b) Short duration

Figure 6. Base shear time histories, Alaska EQ



(a) Long duration (b) Short duration

Figure 7. Base overturning moment time history, Alaska EQ

It is seen from above figures that in most of cases the maximum responses from the long duration earthquakes are greater than that from short versions. However, in a few cases, the results were reversed. Because the non-linear analysis is path dependent, when the Hamming windows filter is used to generate a short version earthquake, the ground acceleration input is modified although peak acceleration values are kept the same. This modification will cause the minor changes in the resulting hysteretic loops and the maximum response. Another way to compare the long duration effect was used and is listed in Table 4, in which the values in the "Short" rows were taken from the same long-duration computing runs but count the" maximum" only from shorter period of time cover the peak ground acceleration.

Table 4. Comparison of the maximum responses from the peak ground acceleration and the corresponding long duration earthquakes (NL model)

EQs	Dir.	Type	Max Roof Displacement (cm)	Max Inter-story Drift (cm)	Max Base Shear ($x10^4$ kN)	Max Turning Moment ($x10^5$ kN·m)
AK EQ	EW	Long	38.2	12.8	2.92	14.9
		Short	34.5	12.6	2.33	6.21
	NS	Long	57.1	11.1	3.12	9.96
		Short	24.8	11.1	2.38	7.37
Mexico EQ	EW	Long	43.5	10.0	2.11	15.9
		Short	52.4	11.2	2.25	15.6
	NS	Long	69.2	11.0	3.22	11.4
		Short	65.7	11.5	3.04	12.0
Chile EQ	EW	Long	25.6	7.2	2.32	11.8
		Short	25.1	6.8	2.31	9.2
	NS	Long	36.7	12.4	3.73	6.7
		Short	29.6	9.2	3.51	6.6

The similar comparison results were obtained from the long and short durations of maximum considered earthquakes. To keep the paper short, the nonlinear time history results from three maximum considered earthquakes will not be shown here.

Conclusions

Comparison of the time history outcomes of the short and long duration design level earthquakes exposes some interesting results:

(1) The roof maximum displacement, maximum inter-story drift, maximum base shear and maximum base over-turning moment were all increased from long duration earthquakes.
(2) Most significant increase in these values is the increase of the maximum over-turning moment. In one particular case, this quantity increased as much as 70%. This issue needs additional detailed study to confirm the level and the reason for this increase.
(3) In many cases studied, the permanent deformation at the roof level and inter-story drift were observed clearly in the long duration events.
(4) Residual based shear was also observed in the long duration events.

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