1695

COMPARISON OF SEISMIC PERFORMANCE BETWEEN U.S. STEEL PERIMETER FRAME AND JAPANESE SPATIAL MOMENT RESISTING FRAME

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SUMMARY

Major objective of this study is to clarify differences in seismic performance of U.S. steel perimeter frame and of Japanese spatial moment resisting frame. For this purpose, the SAC theme structures (3-story and 9-story) have been redesigned with the Japanese seismic code under similar soil and hazard conditions. By using these frames, a comparison of seismic performance between the U.S. steel perimeter frame and the Japanese spatial moment resisting frame was carried out through the nonlinear pushover analysis and earthquake time-history response analysis. From these numerical analyses, following results were obtained. 1) Push over analysis: The ultimate resistances (base shear / model weight) of the Japanese 3-story frames are about 1.6 times as large as that of the U.S. frame. In the case of 9-story frames, inelastic behaviors of the Japanese frames are almost similar to that of the U.S. frame. 2) Nonlinear time-history response analysis: The maximum inter-story drift angles in each story of the U.S. 3-story frame are larger than that of the Japanese 3-story frames. On the other hand, the damage level of beam in U.S. 3-story frame is a little smaller than that in the Japanese frames. In the case of 9-story frames, both maximum inter-story drifts and damage level in the U.S. frame are almost similar to those in the Japanese frames.

INTRODUCTION

The 1994 Northridge earthquake and the 1995 Hyogoken-nanbu earthquake caused widespread building damage throughout some of the heavily populated communities of Southern California and Kobe. As the typical damage of low-and-medium rise steel building structures, the brittle fractures at the welded joints between the beams and columns were reported in Both U.S.A. and Japan. The typical structural system of steel building in U.S.A. is perimeter frame, while Japanese typical steel structural system is spatial moment resisting frame. The structural system and seismic design procedure between U.S.A. and Japan are different. Major objective of this study is to clarify differences in seismic performance of U.S. steel perimeter frame and of Japanese spatial moment resisting frame. For this purpose, the 3-story and 9-story buildings involved in the SAC project 【Malley, 1996】 have been selected. And these SAC theme structures have been redesigned with the Japanese seismic design code under similar soil and hazard conditions. By using these frames, a comparison of seismic performance between the U.S.

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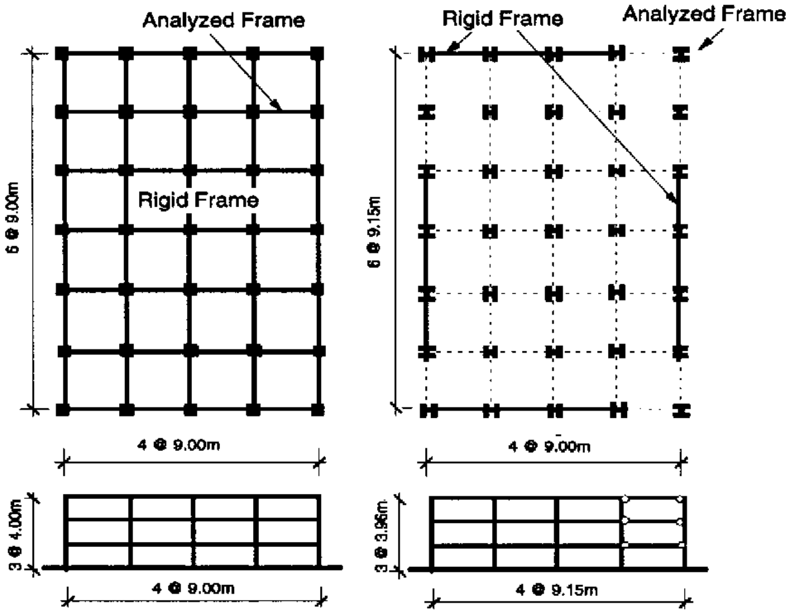
steel perimeter frame and Japanese spatial moment resisting frame was carried out through the nonlinear pushover analysis and earthquake time-history response analysis. This paper presents the results of these numerical analyses.

2. MODEL BUILDINGS

The 3-story and 9-story buildings [Krawinkler and Gupta, 1998] involved in the SAC project have been selected for the analysis. These are typical office buildings, which designed on the basis of 1994 UBC guideline for seismic zone 3. The plan view and analyzed plane frame elevation of the SAC 3-story model building (SAC3-frame) are shown in Figure 1(b). The SAC3-frame has been redesigned with Japanese seismic code as spatial moment resisting frame under similar soil and hazard conditions. The plan view and elevation of the redesigned frame were shown in Figure 1(a) (BRI3-frame). As for the redesigned buildings, two model buildings (BRI3-A, BRI3-B) have been redesigned in this study. The member sections of these buildings and the SAC3-frame (SAC3-LA) were listed in Table 1. The moment resisting frames in the BRI3 and SAC3 buildings are shown in solid and bold lines in each plan view. Only the analyzed frames, whose elevations are shown in Figure1, are utilized in the analysis discussed in this paper.

The plan views and the analyzed frame elevations of 9-story model buildings (SAC9, BRI9-frame) are also shown in Figure 2(a), (b). From figure 1 and 2, it was found that all columns in U.S. perimeter frames bend about the strong axis. On the other hand, in the Japanese spatial moment resisting frames, all columns are box-shaped which are used to resist bi-axial bending. The member sections used in the analyzed 9-story frames are summarized in Table2.



(a) BRI3-Frame (b) SAC3-Frame

Figure 1: Plan views and analyzed frame elevations for 3-story buildings

Table 1: Beam and column sections for 3-story frames

Story/Floor	BRI3-A			BRI3-B			SAC3-LA		
	Column (BCR295)		Girder (SN400B)	Column (BCR295)		Girder (SN400B)	Column (50ksi)		Girder (36ksi)
	Ext.	Int.		Ext.	Int.		Ext.	Int.	
3/4	□-450×16		H-550×200×9×19	□-400×16		H-550×200×9×16	W14×257 (H-419×407×31×49)	W14×311 (H-437×412×36×58)	W24×68 (H-602×228×11×15)
2/3	□-450×19			□-400×19		H-550×200×9×19			W30×116 (H-762×267×14×22)
1/2	□-450×22		H-550×200×12×22	□-400×22		H-550×200×9×22			W33×118 (H-835×292×14×19)

3. ANALYSIS FRAME

The computer program club.f [Ogawa and Tada, 1994] is used in the analysis, which deal with the combined non-linear analysis of plane steel frames and which can take account of the elasto-plastic deformation of joint panel. The nominal yield strength and assumed yield strength in the analyzed frames are summarized in Table3. In the U.S. perimeter frames, masses for the analysis frames have been computed on the basis of area equal to 1/2 floor. While in the Japanese spatial frames, masses for the analysis frames have been computed by considering that 1/6 and 1/5 of the total mass is attributed to each analyzed frame in the 3-story and in the 9-story buildings, respectively. For the six analysis frames, the first mode periods listed in Table 4. 2% viscus damping is set as the stiffness populating damping.

Figure 3 indicates relative member strength of beam, column and joint panel zone for the 3-story frames. Figure 3(a) shows strength ratio of column to beam (R_c) at the beam-to-column joint in each story. Figure 3(b) shows

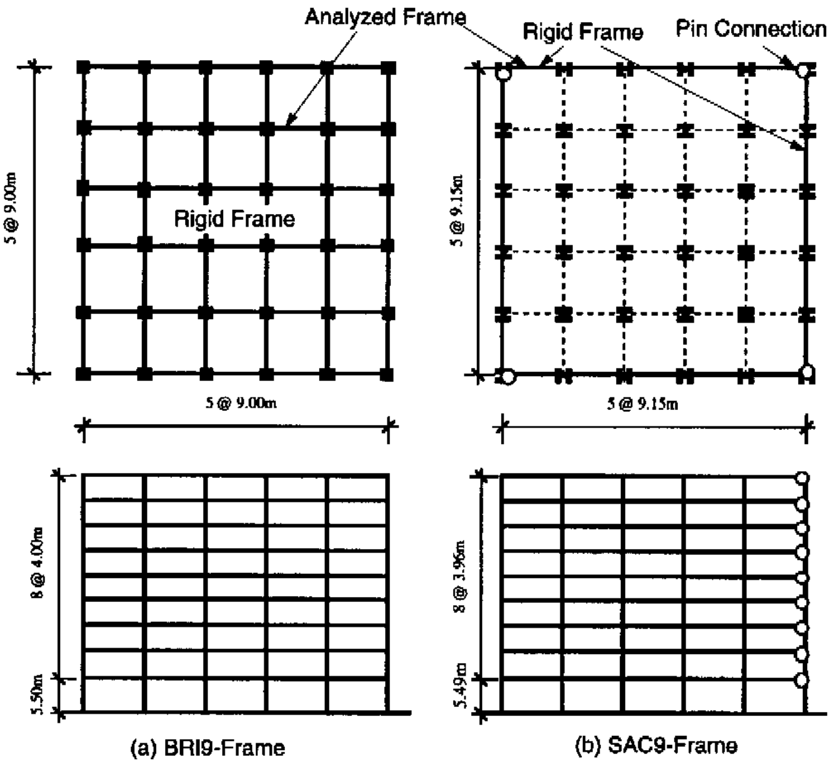


Figure 2: Plan views and analyzed frame elevations for 9-story buildings

Table 2: Beam and column sections for 9-story frames

<table>
<tr><th rowspan="3">Story/Floor</th><th colspan="3">BRI9-A</th><th colspan="4">BRI9-B</th><th colspan="3">SAC9-LA</th></tr>
<tr><th colspan="2">Column (BCP325)</th><th rowspan="2">Girder (SN490B)</th><th colspan="2">Column (BCP325)</th><th colspan="2">Girder (SN490B)</th><th colspan="2">Column (50ksi)</th><th rowspan="2">Girder (36ksi)</th></tr>
<tr><th>Ext.</th><th>Int.</th><th>Ext.</th><th>Int.</th><th>Ext.</th><th>Int.</th><th>Ext.</th><th>Int.</th></tr>
<tr><td>9/10</td><td colspan="2" rowspan="3">☐-450×19</td><td rowspan="3">H-450×250×9×16</td><td colspan="2" rowspan="2">☐-450×16</td><td colspan="2" rowspan="2">H-500×250×9×16</td><td rowspan="2">W14×233 (H-409×404×28×44)</td><td rowspan="2">W14×257 (H-419×407×31×49)</td><td>W24×68 (H-602×228×11×15)</td></tr>
<tr><td>8/9</td><td>W27×84 (H-678×253×12×16)</td></tr>
<tr><td>7/8</td><td colspan="2" rowspan="3">☐-450×19</td><td>H-500×250×9×19</td><td>H-500×250×9×16</td><td rowspan="2">W14×257 (H-419×407×31×49)</td><td rowspan="2">W14×283 (H-427×410×33×53)</td><td>W30×99 (H-753×266×13×17)</td></tr>
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<tr><td>5/6</td><td rowspan="2">W14×283 (H-427×410×33×53)</td><td rowspan="2">W14×370 (H-456×418×42×68)</td></tr>
<tr><td>4/5</td><td colspan="2" rowspan="3">☐-450×22</td><td rowspan="3">H-600×250×12×22</td><td rowspan="3">H-600×250×12×19</td></tr>
<tr><td>3/4</td><td colspan="2" rowspan="3">☐-500×22</td><td rowspan="2">H-550×250×12×22</td><td rowspan="3">W14×370 (H-456×418×42×68)</td><td rowspan="2">W14×455 (H-484×427×51×82)</td></tr>
<tr><td>2/3</td><td rowspan="2">W36×160 (H-914×305×17×26)</td></tr>
<tr><td>1/2</td><td>H-650×250×12×25</td><td colspan="2">☐-500×25</td><td>BH-700×250×14×22</td><td>BH-700×250×14×19</td><td>W14×500 (H-499×432×56×89)</td></tr>
</table>

strength ratio of joint panel zone (R_p), where the R_p means the ratio of the joint panel strength to the lesser strength of column and beam at a beam-to-column joint. From the Figure 3(a), it was found that strength of column members is about 2 times as large as that of beam members in second and third floor. From the Figure 3(b), it was found that the R_p values of the SAC3 frame are less than 1.0 in each story, which means weak joint panel structure. Table5 shows total weight of steel members in each model, which is the sum of column, girder and beam.

4. PUSHOVER ANALYSIS

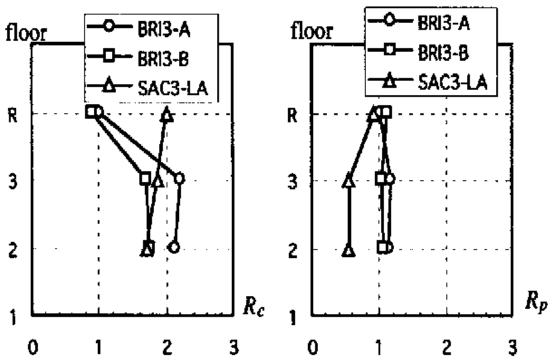
The nonlinear pushover analysis has been carried out on a single frame, which elevation is shown in Figure1 and 2. Figure 4 shows pushover results, in terms of base shear versus 1st story drift angle (1st story displacement over story height) diagrams for all model structures. The results of the analysis are shown for the 3-story frame and the 9-story frame respectively. In each Figure, the behavior of the SAC-frame was compared to that of the BRI-frames. From the Figure 4, in case of 3-story frame, it was found that the ultimate resistance and the stiffness of the BRI3 frames are larger than that of the SAC3-LA frames. The ultimate resistances (base shear / model weight) of the BRI 3 frames are about 1.6 times as large as that of the SAC frame. On the other hand, in case of 9-story frame, inelastic behaviors of the BRI frames are almost similar to that of the SAC frame.

Table 3: Yield strength of members

	BRI3 Frame		BRI9 Frame		SAC Frame	
	Column (BCR295)	Girder (SN400B)	Column (BCP325)	Girder (SN490B)	Column (50ksi)	Girder (36ksi)
Nominal Yield Strength	3.0 t/cm^2	2.4 t/cm^2	3.3t/cm^2	3.3t/cm^2	3.51 t/cm^2	2.53 t/cm^2
Assumed Yield Strength	3.3 t/cm^2 (3.0×1.1)	2.64 t/cm^2 (2.4×1.1)	3.63t/cm^2 (3.3×1.1)	3.63t/cm^2 (3.3×1.1)	4.04 t/cm^2 (3.51×1.15)	3.47 t/cm^2 (2.53×1.37)

Table 4: First mode periods

	BRI-A	BRI-B	SAC-LA
3-Story Frame	0.65sec.	0.70sec.	0.96sec.
9-Story Frame	1.91sec.	1.87sec.	2.08sec.



(a) R_c values (b) R_p values

Figure 3: Relative member strength

Table 5: Total weight of steel members

	BRI-A	BRI-B	SAC-LA
3-Story Frame	400.4ton	391.4ton	326.9ton
9-Story Frame	1312.5ton	1302.9ton	1534.4ton

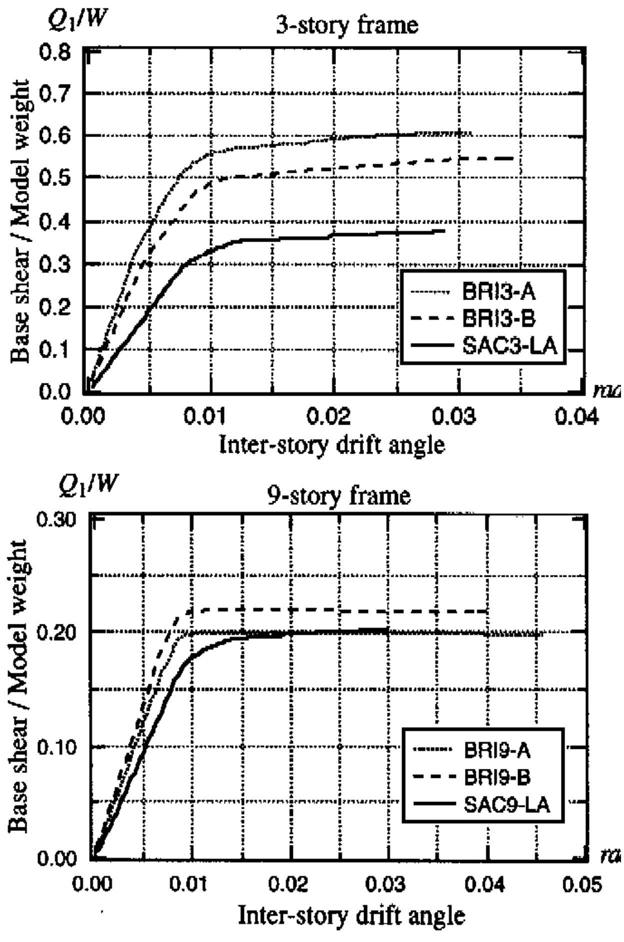


Figure 4: Push over analysis

5. NONLINEAR TIME-HISTORY RESPONSE ANALYSIS

5.1 Input earthquake ground motions

Table 6 shows input earthquake ground motions used for the nonlinear time-history response analysis. Three recorded earthquake ground motions, EL CENTRO NS (1940 El centro), TAFT EW(1952 Taft), NTT NS(1995 Kobe) and one artificial earthquake ground motion, YOKOHAMA were selected for this analysis. The maximum velocities of these input earthquake ground motions were set to be about 50cm/sec.. By using these 4 input earthquake ground motions, nonlinear time-history response analysis of 3-story (BRI3-A, BRI3-B, SAC3-LA) and 9-story (BRI9-A, BRI9-B, SAC9-LA) frames were performed.

5.2 Analysis results of 3-story frames

5.2.1 Total input energy

Figure 5 demonstrates energy spectrum (V_T) of the EL CENTRO NS used in the analysis. The marks plotted in this figure indicate total input energy represented by velocity (V_T) into the 3-story frames (BRI3-A, BRI3-B, SAC3-LA). The V_T value of SAC3-LA frame is smaller than that of the BRI3 frames, because of the difference in first mode periods between SAC3 frame and BRI3 frames.

Table 6: Input earthquake ground motions

Input Earthquake Ground Motions	Maximum Velocity	Maximum Acceleration	Duration
EL CENTRO NS	50cm/sec	511gal	20sec
TAFT EW	50cm/sec	498gal	20sec
YOKOHAMA	52cm/sec	312gal	40sec
NTT NS	50cm/sec	186gal	40sec

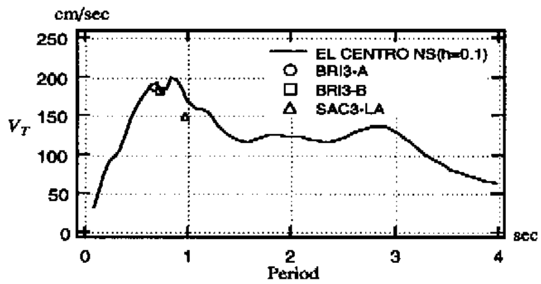


Figure 5: Energy spectrum

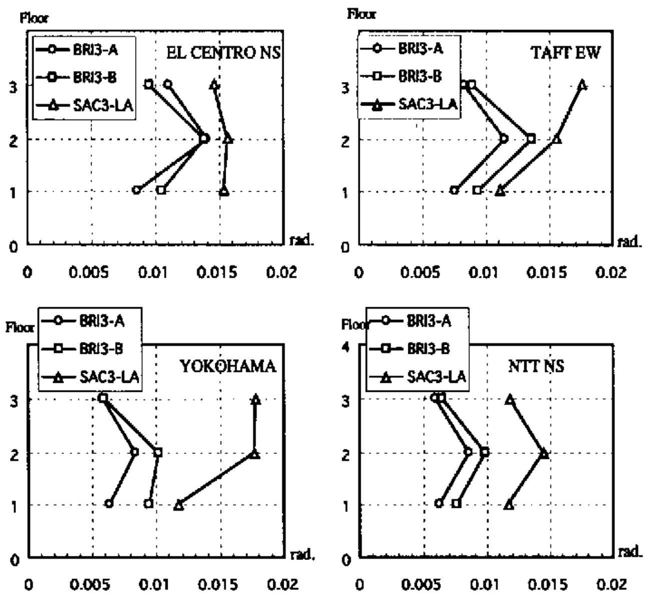


Figure 6: Maximum inter-story drift angles

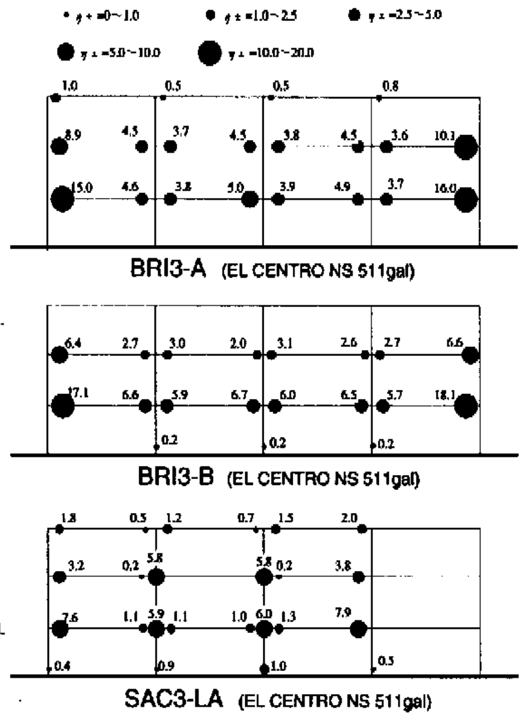


Figure 7: Damage distribution into frames

5.2.2 Maximum story drift and damage

Figure 6 shows maximum inter-story drift angles along the height of the frames against each earthquake ground motion. The maximum inter-story drift angles of the SAC3 frame in each story are larger than that of BRI3 frames against the selected four earthquake ground motions. Maximum inter-story drift angles of these three frames against the 50kine-simulated earthquake ground motions are within 0.02rad. Figure 7 shows damage distribution into each frame against EL CENTRO NS. The damage is represented by maximum cumulative ductility ($\eta_{\pm}$) which is defined by the following equation.

$$\eta_{\pm}=\frac{W_{\pm}}{M_p \cdot \theta_p} \tag{1}$$

Where, $W_{\pm}$: larger cumulative absorption energy of positive and negative lording for a hinge in member, M_p: full plastic moment of member(panel), θ_p: elastic limit rotation angle.
As for the beam damage, the damage level of beam in the SAC3 frame is smaller than that in the BRI3 frames. The damage to panel zones is occurred in the SAC3 frame. The relative member strength of panel zone (R_p) is smaller than 1.0 in the SAC3 frame, which was shown to previous Figure 3. In case of the SAC3 frame, the beam damage may decrease by the energy absorption into the panel zones.

5.2.3 Analysis under same total input energy

From figure 5, it was found that total input energy (V_T) into the BRI3 frames are larger than that into the SAC3 frame. So, the maximum accelerations of EL CENTRO NS for the BRI3-A and B frames were changed to 410gal and 400gal respectively, in order to make same energy input (V_T =150kine) condition into each analysis frame. Figure 8 shows the maximum inter-story drift angles of the three analysis frames against EL CENTRO NS under the same total energy input condition. Figure 9 shows damage distribution into the BRI3 frames obtained from this analysis. Comparing the beam damage of the frames in Fig.9 to that of the SAC3 frame obtained from Fig.7, it is found that the damage level of beam in the SAC3 frame is a little smaller than that in the BRI3 frames. It is considered that the energy absorption into the panel zones and large elastic limit rotation angles of the beams in the SAC3 frame make the beam damage ($\eta_{\pm}$) decrease.

5.3 Analysis results of 9-story frames

5.3.1 Total input energy

Figure 10 demonstrates energy spectrum (V_T) of the YOKOHAMA earthquake ground motion used in the analysis. The marks plotted in this figure indicate total input energy represented by velocity (V_T) into the 9-story frames (BRI9-A, BRI9-B, SAC9-LA).

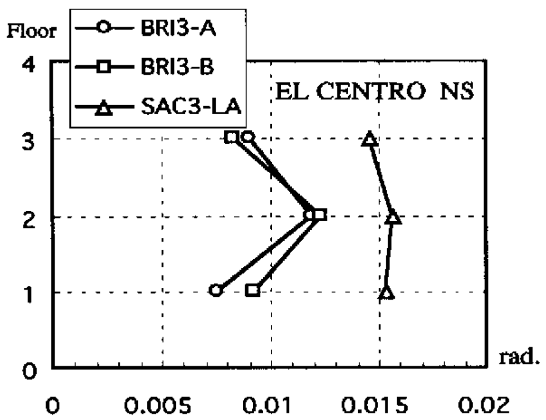


Figure 8: Maximum inter-story drift angles (same input energy)

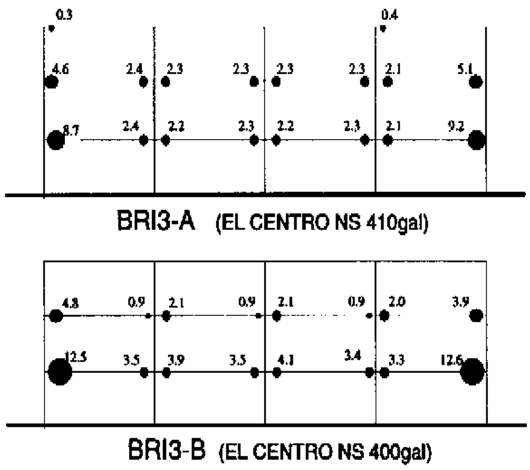


Figure 9: Damage distribution into frames (same input energy)

5.3.2 Maximum story drift and damage

Figure 11 shows maximum inter-story drift angles along the height of the 9-story frames against each earthquake ground motion. From this figure, following results were obtained. The inter-story drift angles of these frames against each earthquake ground motion are almost similar in over all tendencies. Maximum inter-story drift angles of these three analysis frames against the 50kine-simulated earthquake ground motions are within 0.02rad.. The

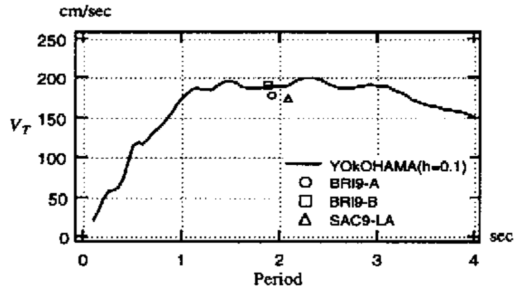


Figure 10: Energy spectrum

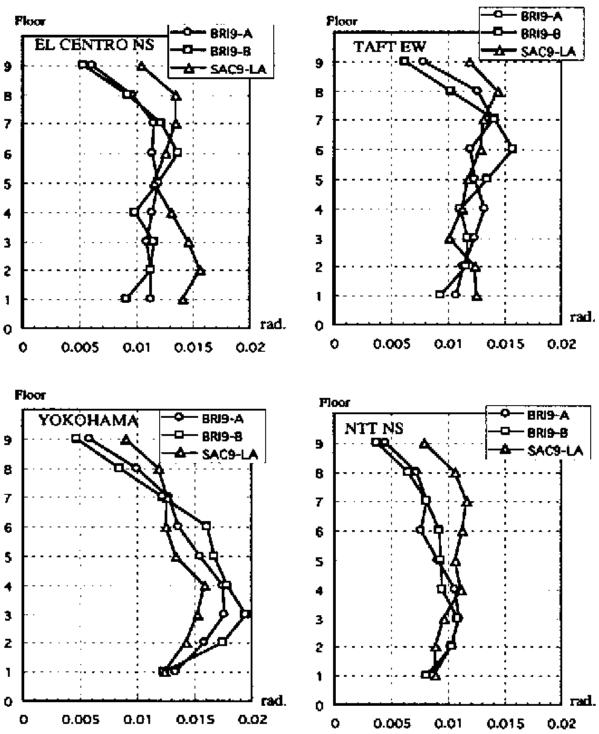


Figure 11: Maximum inter-story drift angles

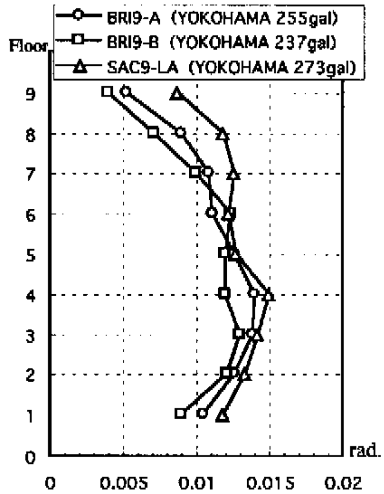
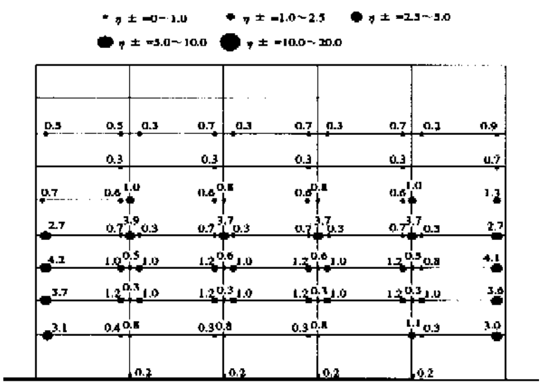
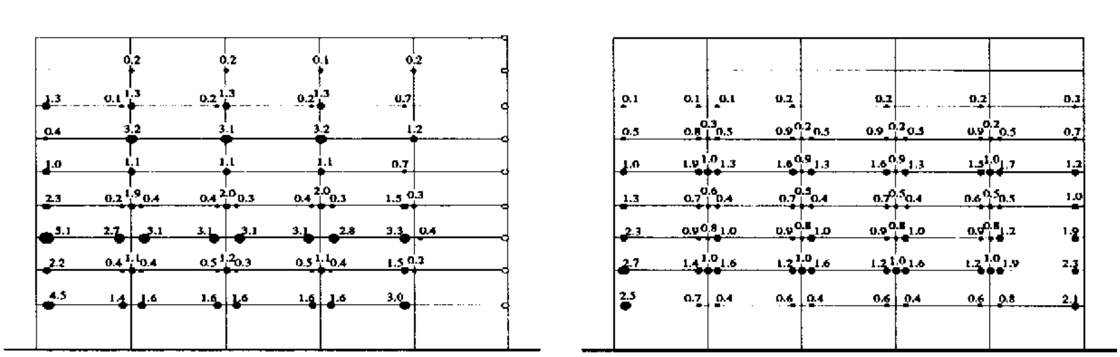


Figure 12: Maximum inter-story drift angles (same input energy)



BRI9-A (YOKOHAMA 255gal)



SAC9-LA (YOKOHAMA 273gal) BRI9-B (YOKOHAMA 237gal)

Figure 13: Damage distribution into frames (same input energy)

YOKOHAMA earthquake ground motions caused larger inter-story drifts than the other earthquakes.

5.3.3 Analysis under same total input energy

The maximum accelerations of YOKOHAMA for BRI9-A and B frames were changed to 255gal and 237gal respectively, which makes the same energy input (V_T =150kine) into the frames. Figure 12 shows the maximum inter-story drift angles of three frames against the YOKOHAMA earthquake ground motions under the same total energy input condition. Figure 13 shows damage distribution to the 9-story frames under same input energy condition. From the 9-story frame analysis, following results were obtained. 1) The damage level of beam in the BRI frames is almost similar to that in the SAC frame. 2) The damage into panel zones occurred to these three frames. 3) Over all behaviors (inter-story drift, damage distribution) of BRI9 frames are almost similar to that of SAC9 frame. Because the ultimate resistance and stiffness of the BRI9 frames are also similar to that of the SAC9 frame (see Fig.4).

6. CONCLUSIONS

Major objective of this study is to clarify differences in seismic performance of U.S. steel perimeter frame and of Japanese spatial moment resisting frame. For this purpose, the SAC theme structures (3-story and 9-story) have been redesigned with the Japanese seismic code under similar soil and hazard conditions. This paper presented the results of the comparison of seismic performance between the U.S. frame and the Japanese frame through the push over analysis and nonlinear time-history response analysis. From the results of analyses, following conclusions were obtained.

1) Push over analysis: The ultimate resistances (base shear / model weight) of the Japanese 3-story frames are about 1.6 times as large as that of the U.S. frame. On the other hand, in the case of 9-story frames, inelastic behaviors of the Japanese frames are almost similar to that of the U.S. frame.

2) Nonlinear time-history response analysis: The maximum inter-story drift angles in each story of the U.S. 3-story frame are larger than that of the Japanese 3-story frames. On the other hand, the damage level of beam in U.S. 3-story frame is a little smaller than that in the Japanese frames. In the case of 9-story frames, both maximum inter-story drifts and damage level in the U.S. frame are almost similar to those in the Japanese frames.

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