APPLICATION OF OPTIMUM DESIGN METHODS TO ACTUAL HIGH-RISE BUILDING WITH HYSTERETIC DAMPERS

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SUMMARY

The purpose of this paper is to show the practical applicability of a new optimum design method by the authors to an actual high-rise building structure with hysteretic dampers. The building as a hotel has no basement floors and 25 stories above ground. The overall height is 96.60 (m) and the total floor area is 23,978 (m^2). The main structure is composed of steel rigid frames above ground and the foundation is composed of a rigid reinforced concrete mass. Several hysteretic dampers are installed in all the stories. To design the structure with such dampers, the optimum design method is applied. The results show that the dampers can reduce the dynamic responses against earthquakes, which maximum ground velocity is 50 (cm/s). It is revealed that the new design method is very efficient in saving amount of structural steel and the time required for structural design.

INTRODUCTION

In designing a high-rise building, much analysis is required to calculate the static and dynamic responses of the building frame under design loads. Likewise to redesign the member sizes based on the analysis results requires much recalculation. It depends on the designer's skill and intuition. It is therefore strongly desired to improve the process of structural design.

The purpose of this paper is to show a practical applicability of an optimum design method due to the present authors [Tsuji et al. 1999] to actual high-rise building structures with hysteretic dampers. The building, which is a hotel, has no basement floors and 25 stories above ground. The overall height is 96.60 (m) and the total floor area is 23,978 (m^2). The main structure is composed of steel rigid frames above ground and the foundation is composed of a rigid reinforced concrete mass. Some hysteretic dampers are installed in all the stories. To design the structure with such dampers, the optimum design method is applied. It is shown that the dampers can reduce the dynamic responses against an earthquake of a maximum ground velocity is 50 (cm/s), and that the new design method is very efficient in saving the amount of structural steel and time required for its design.

BUILDING DESCRIPTION

2.1 Location:

This building is located to the west of Osaka-city, 7 km from JR Osaka station.

2.2 Outline:

The building has no basement floors and 25 stories above the ground. The maximum height is 96.60 m and the total floor area is 23,978 m^2. (see Fig.1)

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2.3 Structural Planning:

Plan of the building in the upper stories is square in plan 30.5m by 30.5m (see Fig.2 & Fig.3). The main structure consists of steel columns, columns of concrete-filled-steel-tube (CFT) and steel beams. Hysteretic dampers are installed in the walls of the core.

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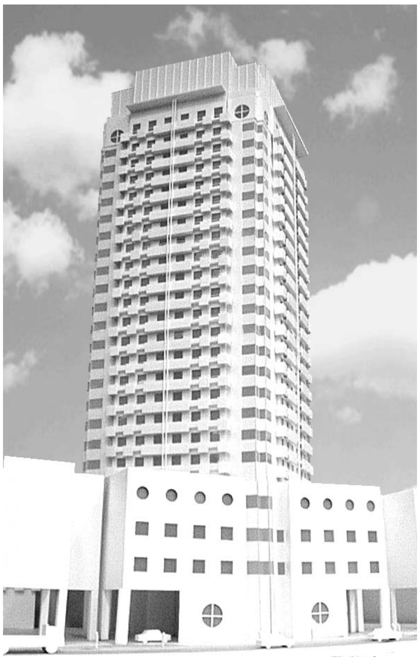
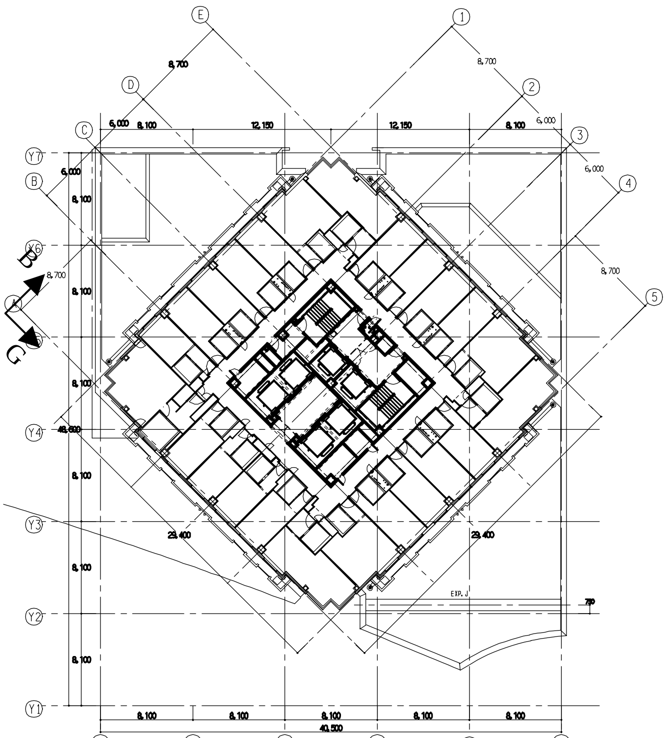


Figure 1: Perspective of building



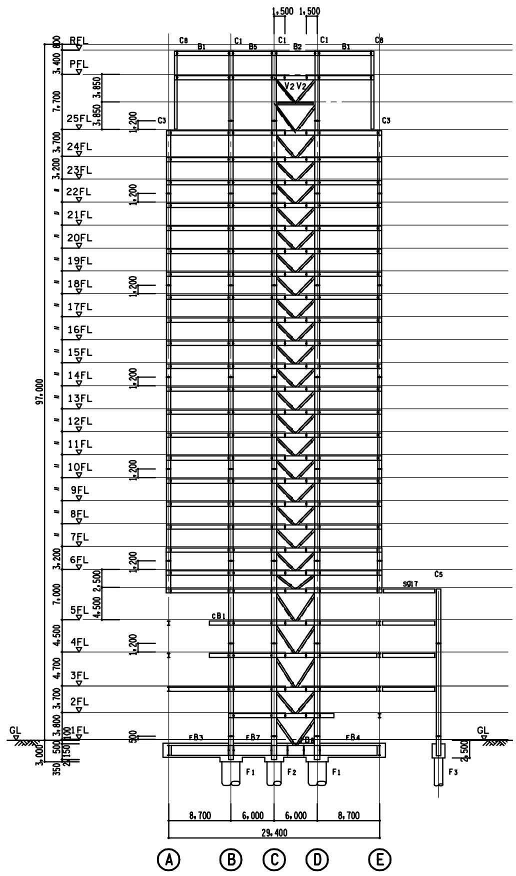


Figure 2: Floor plan **Figure 3: Framing elevation**

CONCEPT OF THE STRUCTURAL DESIGN

3.1 Seismic Design:

In the seismic resistant design, design story shear forces are decided through the preliminary seismic response analysis using design earthquake ground motions shown in Table 1. The initial member sizes are determined by using the optimum design method [Tsuji et al. 1999]. Then the dynamic seismic response analyses are carried out to check seismic response performances. (see Fig.4)

3.2 Design Earthquakes:

The design earthquakes are as follows:

EL CENTRO 1940 NS •BCJ•

TAFT 1952 EW •BCJ•

HACHINOHE 1968 NS •BCJ•

UEMACHI OSAKA L1 (artificial)

UEMACHI OSAKA L2 (artificial)

3.3 Target of Seismic Response Performances:

Target of seismic response performances are shown in Table 1 (dual targets for two-level design earthquakes).

Table 1•Target of seismic response performances

Input level	Maximum amplitude of seismic velocity •cm/sec•	Target value	
		Maximum story deformation angle $\bullet\times10^{-3}$rad•	Maximum ductility factor
Level 1	25	Less than 6.0	Less than 1.0
Level 2	50	Less than 12.0	Less than 2.0

The L1 and L2 ground motions (maximum velocities are 43.6cm/sec and 66.9cm/sec) are used to determine whether the responses satisfy design criteria for level 2 earthquakes. Story ductility factor is defined as the ratio of the maximum story deformation to its yield deformation.

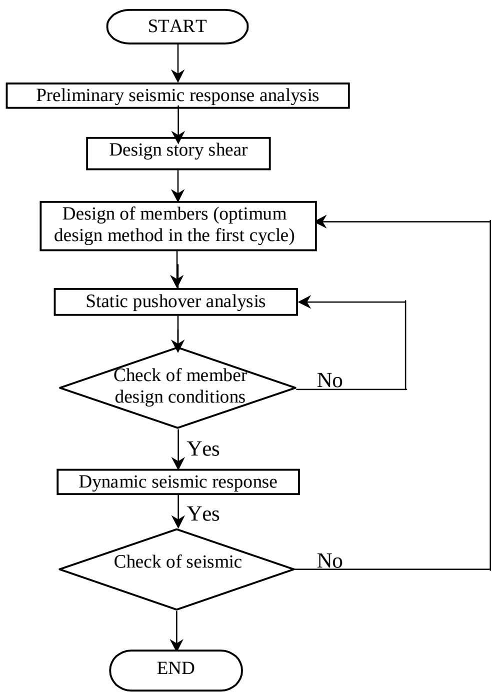


Figure 4: Flowchart of seismic design

OPTIMUM DESIGN

4.1 Structural Design Conditions:

The optimal design method [Tsuji et al. 1999] is applied under the following conditions. Vertical loads through columns are applied on the beam-to-column joints, and floor weights are considered as the distributed loads. The member weights may change during a design sensitivity analysis cycle, but this change of self-weight is ignored in the analysis. The member cross-sectional areas are assumed to be the following:

For beams $187.2 \leq Aj \leq 288.3$ (6F-25F) (1)

$231.5 \leq Aj \leq 300.3$ (1F-5F)

For steel columns $385.8 \leq Aj \leq 671.6$ (2)

For CFT columns $821.3 \leq Aj \leq 1247.8$ (3)

All members are SM490 (tensile strength is 490MPa). The relations between the cross-sectional area, second moment of area and section modulus are assumed to be the following:

For beams $I=4.0A^{2.0}$ $Z=1.5A^{1.5}$ (4)

For steel columns $I=1.2A^{2.0}$ $Z=0.8A^{1.5}$ (5)

For CFT columns $I=0.27A^{2.07}$ $Z=0.41A^{1.46}$ (6)

The cross-sectional area of a CFT column is evaluated so that its strength is equivalent to that of the corresponding steel column. CFT members are selected based on the constructional conditions that the concrete can fill the box-type column through more than 37.2(m). Composite effect of reinforced slab to steel beams is considered in determining the second moment of area. Response strain condition is expressed as

$$-0.00157 \leq \varepsilon j \leq 0.00157 \quad (7)$$

Design story shear forces are determined according to the preliminary seismic response analysis.

4.2 Structural Modelling:

Fig.2 shows the square plan of the building. Only three planar frames along the B loading direction are considered. Design variable grouping is shown in Fig. 5. The same number indicates that those members have a common member section, and the member sections without numbers are not design variables. The bottom of the first-story column is assumed to behave as pin joints supported by stiff foundation beams. The size of the foundation beams is not a design variable. Each slab is considered to have sufficient stiffness to transfer horizontal loads (rigid floor-diaphragm assumption). Hysteretic dampers are installed in the 2nd-line frame. Under the design story shear forces, all dampers experience plastic behaviors and can be assumed to have constant yield forces regardless of those response displacements. At the joint in the 5th floor where the frame line A and the frame line 2 meet, and the frame line B and the frame line 2 meet, there is a beam which crosses over the section. The effect of this beam is considered by elastic spring with equivalent strength. The member sections are shown in Table 2.

4.3 Results:

Fig.6 shows the variation of total member steel weight with respect to the design step number. This total member steel weight does not contain sub-structures such as small beams. It can be observed that total member steel weight almost reaches a constant value after 50 steps. Fig.7 shows the result at the 50th step. The line widths in Fig.7 indicate the cross-sectional areas of beams and columns calculated from the present optimum design method. It takes about 7 seconds in each step using Apple iMac(CPU: PowerPC G3 233MHz, memory 256MB). The available members are shown in Table 2. Table 3 shows the comparison of the members selected by the optimum method and those at the final design. The difference between member sections at optimum

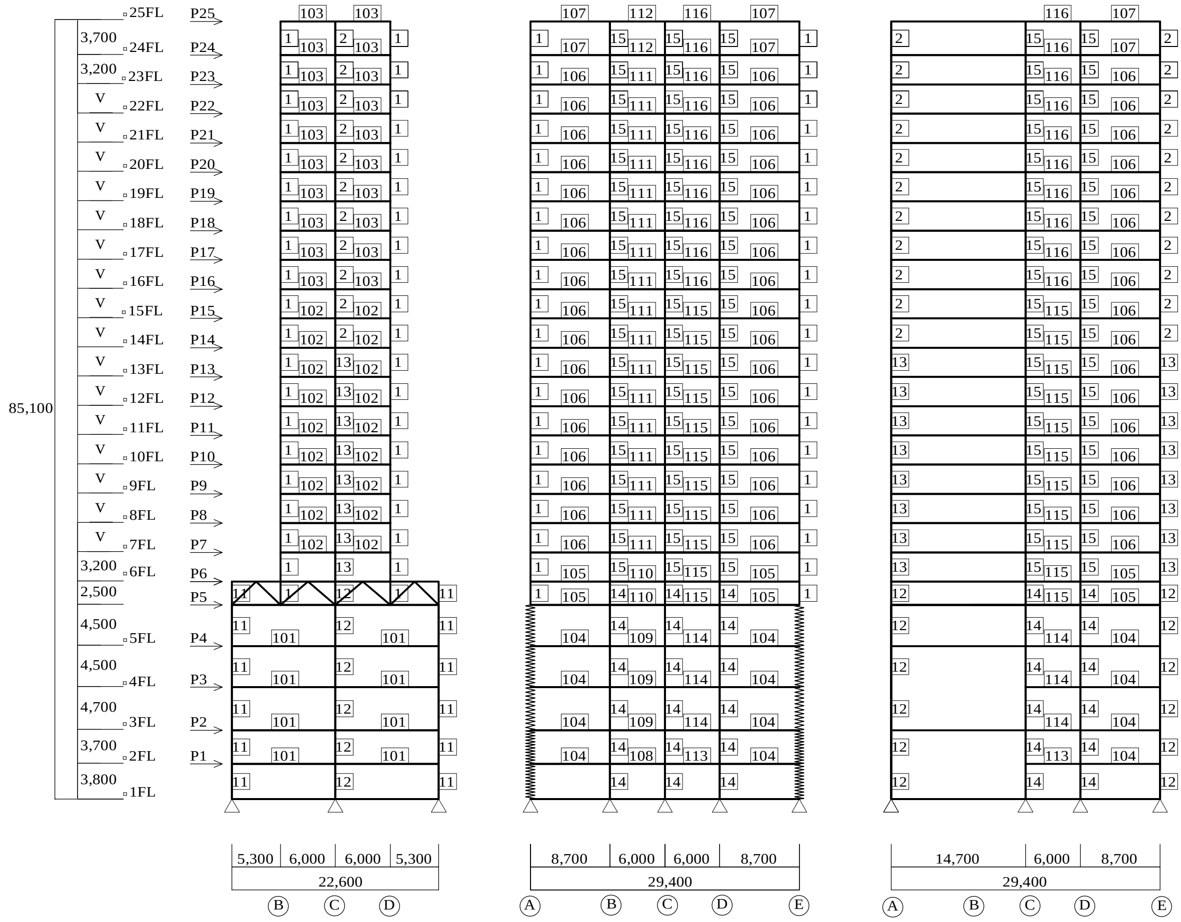


Figure 5: Member grouping in optimum design

design and those at final design arises from various design conditions that are not take into consideration in the optimum design method.

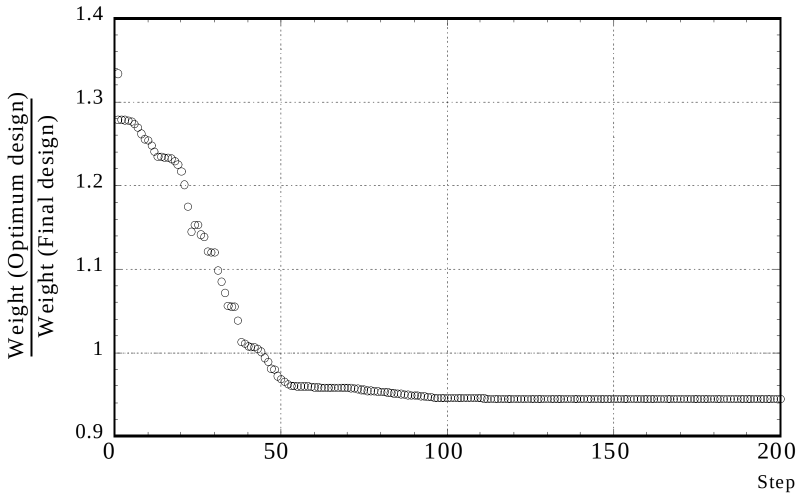


Figure 6: Variation of total steel weight

Table 2: Available member sections (beam)

	Member	Area (cm²)		Member	Area (cm²)
1-5 floor	H-700×300×13×24	231.5	6-25floor	H-588×300×12×20	187.2
	BH-700×300×12×28	245.3		SH-600×300×12×22	200.2
	BH-700×300×12×32	268.3		SH-600×300×12×25	217.4
	BH-700×350×12×32	300.3		SH-600×300×12×28	234.7
				BH-600×300×12×32	256.3
				BH-600×350×12×32	288.3

(Column)

Steel column	Area (cm²)	CFT column	Area (cm²)
□－550×19	385.8	□－550×25	862.7
□－550×22	432.2	□－550×28	906.8
□－550×25	487.4	□－550×32	963.5
□－550×28	537.0	□－600×22	942.7
□－550×32	607.6	□－600×25	989.2
□－600×19	423.8	□－600×28	1038.5
□－600×22	476.2	□－600×32	1102.0
□－600×25	537.4	□－650×25	1123.1
□－600×28	593.0	□－650×28	1177.4
□－600×32	671.6	□－650×32	1247.8
□－650×22	520.2		
□－650×25	587.4		
□－650×28	649.0		
□－650×32	735.6		

Table 3: Comparison of optimum design and final design

Member	Group number	Optimum design	Final design
		Member	Member
Steel column	1	•-550×19	•-550×22
	2	•-500×19	•-600×22
CFT column	11	•-650×32	•-550×22
	12	•-650×32	•-650×25
	13	•-600×22	•-600×22
	14	•-650×28	•-650×25
	15	•-550×22	•-600×22
Beam	101	BH-700×350×12×32	H-700×300×13×24
	102	H-588×300×12×20	SH-600×300×12×22
	103	H-588×300×12×20	H-588×300×12×20
	104	H-700×300×13×24	H-700×300×13×24
	105	H-588×300×12×20	SH-600×300×12×22
	106	BH-600×350×12×32	BH-600×300×12×32
	107	BH-600×350×12×32	BH-600×300×12×28
	108	BH-700×350×12×32	H-700×300×13×24
	109	H-700×300×13×24	BH-700×350×12×32
	110	BH-600×350×12×32	SH-600×300×12×22
	111	BH-600×350×12×32	BH-600×350×12×32
	112	BH-600×350×12×32	SH-600×300×12×25
	113	BH-700×300×12×28	BH-700×300×12×28
	114	BH-700×350×12×32	BH-700×300×12×32
	115	SH-600×300×12×22	SH-600×300×12×22
	116	H-588×300×12×20	H-588×300×12×20

DYNAMIC BEHAVIOUR WITH HYSTERETIC DAMPERS

5.1 Results of Dynamic Response Analysis:

It can be observed from Figs.8-11 that the maximum responses satisfy design criteria for seismic response performances and the seismic safety of the building is guaranteed.

To check the seismic safety of the building's final design, time-history response analyses have been performed for the design earthquake ground motions defined in Section 3.2. Fig.8 shows the maximum story deformation angles from the dynamic response analysis under level 1 design earthquakes. Fig.9 illustrates those under level 2 design earthquakes. Fig.10 shows the maximum story ductility factors from the dynamic response analysis under level 1 design earthquakes. Those under level 2 design earthquakes are plotted in Fig.11.

5.2 Findings from Dynamic Response Analysis:

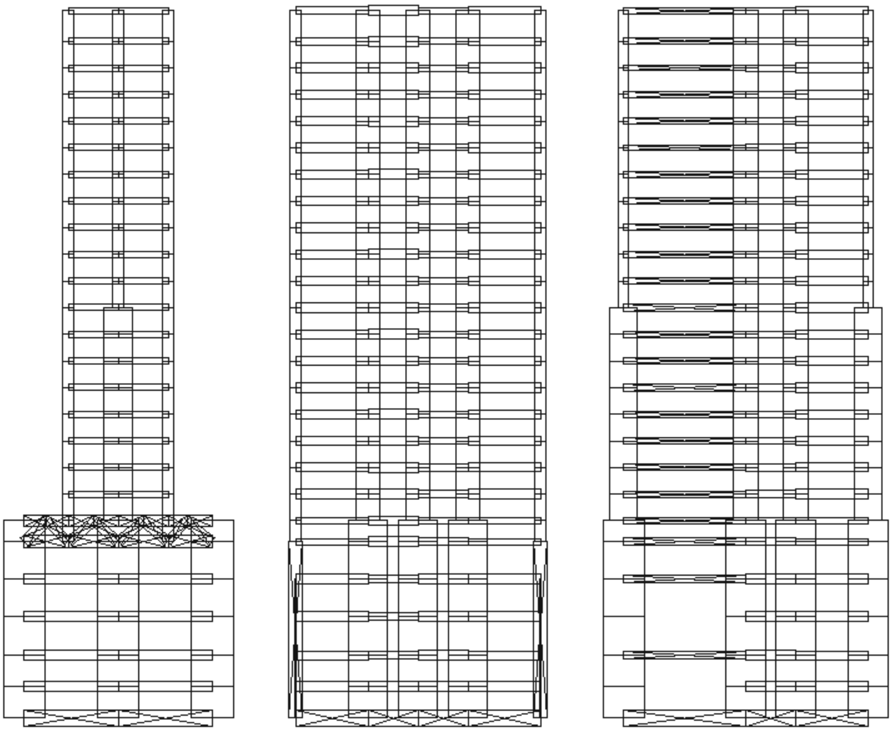
It can be observed from Figs.8-11 that the maximum responses satisfy design criteria for seismic response performances and the seismic safety of the building is guaranteed.

CONCLUSIONS

The conclusions are as follows:

1.The proposed design procedure incorporating an optimum design method is effective in the design of an actual high-rise building.

2.The proposed design procedure makes it possible to save structural cost and reduce computational cost than the conventional seismic resistant design method, including iterative dynamic response analysis.



(sectional areas of cross-marked members are not design variables)

Figure 7: Cross-sectional areas of optimum design (50 step)

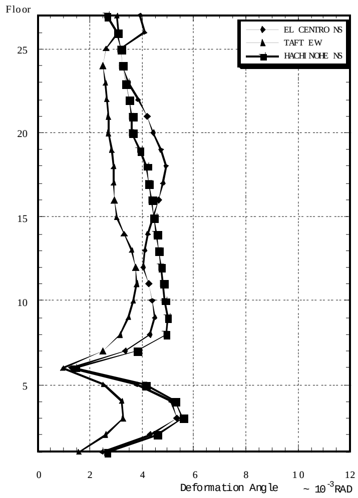


Figure 8: Maximum story deformation angles in B direction? Level 1?

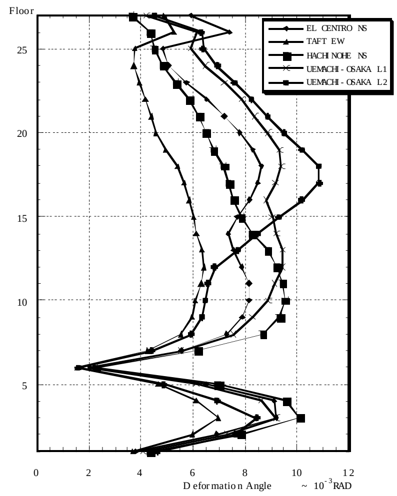


Figure 9: Maximum story deformation angles in B direction? Level 2?

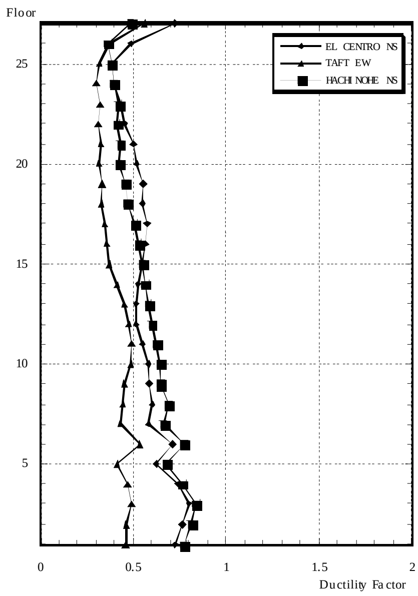


Figure 10: Maximum story ductility factors in B direction? Level 1?

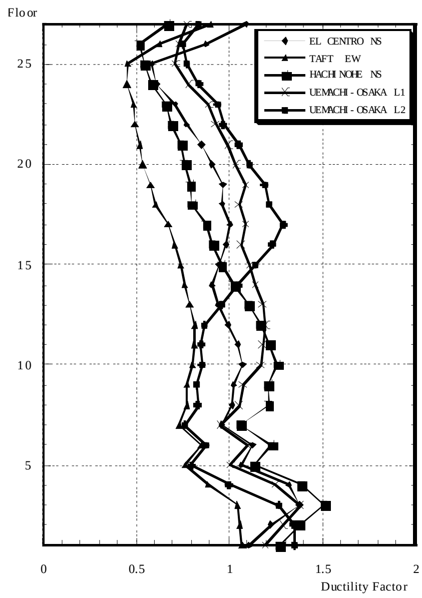


Figure 11: Maximum story ductility factors in B direction? Level 2?

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