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CASE STUDIES ON PERFORMANCE BASED SEISMIC DESIGN USING CAPACITY SPECTRUM METHOD

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SUMMARY

This research aims to show the procedures and results of Performance Based Seismic Design using Capacity Spectrum Method (CSM) for Ductile Steel Structures.

Two 9-story buildings, A and B for examples, which are rigidly jointed steel structures with the same configuration were designed under different seismic performance criteria.

Building A was designed to satisfy the minimum requirements, which are described as "Life-safety against severe EQ (earthquake)" and "Serviceability after moderate EQ".

Building B was designed to have one more requirement as "Repairability after severe EQ", which indicates higher seismic performance.

CSM is the procedure where the demand spectra and the capacity of the structure met in the Acceleration Displacement Response Spectrum (ADRS) format at the performance point,

which describes a different level of seismic design criteria.

The results were evaluated by the elasto-plastic multi-mass time-history seismic response analyses using real EQ records.

The building performance improvement can be done with a slight expense of steel weight.

INTRODUCTION

"Life-safety against severe EQ (earthquake)" and "Serviceability of the structure after moderate EQ" were the standard seismic design criteria.

After the 1995 Kobe EQ, it becomes widely recognized that some particular building should aim to higher levels of seismic design criteria such as “Repairability ” or “Serviceability ” when severe EQ occurs.

Responding to these current demands, the Performance Based Seismic Design is intended to be introduced to the Japanese Building Standard Law as a near future design procedure, however, details are not yet clear.

This paper aims to show the procedures and results of Performance Based Seismic Design using Capacity Spectrum Method (CSM) for ductile moment resisting steel frames.

CONCEPT OF PERFORMANCE BASED SEISMIC DESIGN USING CSM

Capacity Spectrum Method (CSM)

The Capacity Spectrum Method is a graphical and approximate method used to compare the building capacity and an earthquake demand, as shown in Fig. 1.

An earthquake demand is represented by a response spectrum curve.

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Conventionally it is represented by S_a (acceleration response)-T (period) relationships for different damping levels. However, in CSM it is expressed by an Acceleration (S_a)-Displacement (S_d) Response Spectrum (ADRS) format, in this the T-axis is converted to the S_d-axis where the radial lines from the origin in the ADRS format represent the period in the form of ω^2 ($=(2\pi/T)^2$).

The lateral force resisting capacity of the building is represented by the story shear-force to inter-story drift relationships ($Q-\delta$ curve) obtained from pushover analyses.

The secant modulus (the radial line from the origin) in the $Q-\delta$ curve represents an equivalent linear system with an equivalent hysteresis damping (substitute damping) which reflects the effect of inelastic behavior.

The substitute damping h caused by inelastic behavior is indicated here by Shibata method [2] as follows;

$$h = 0.2\left(1-\frac{1}{\sqrt{\mu}}\right)+h_e \tag{1}$$

where, $h_e = 0.02$: elastic damping for steel moment frame ,and, μ : ductility ratio

When both curves are plotted on ADRS format, the point of intersection approximates the response and the performance of the structure for a particular earthquake, this is called a "performance point".

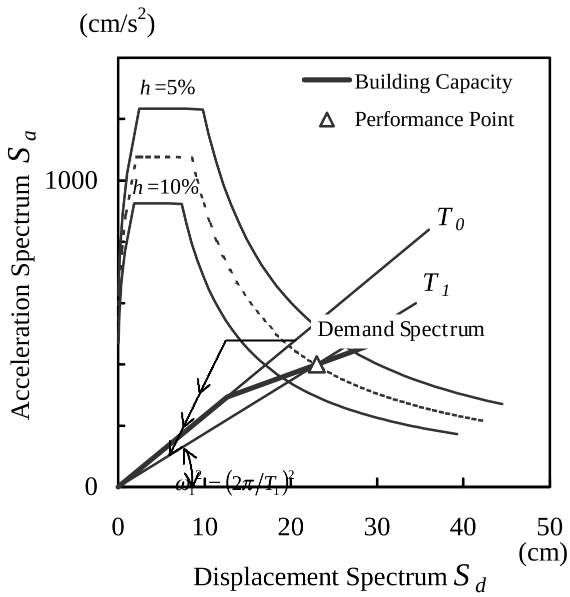


Fig. 1 : Concept of CSM

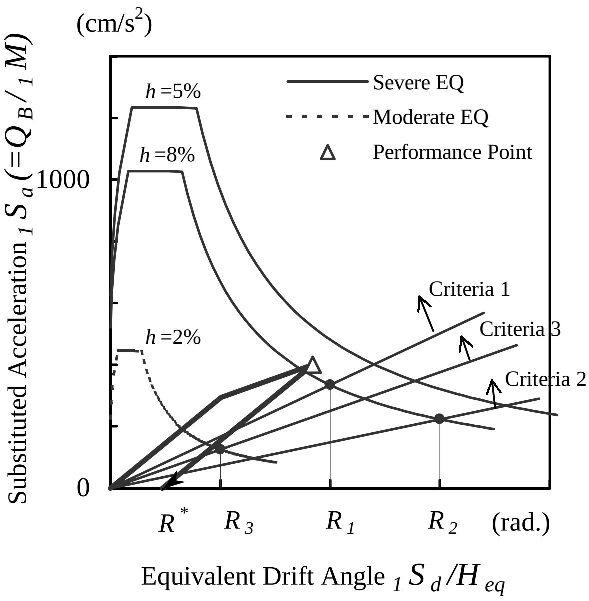


Fig. 3: Performance Based Seismic Design using CSM

Substitution from Multi-Degree of Freedom system to Single-Degree of Freedom system

As shown in Fig.2, a Multi-Degree of Freedom system (MDF) is substituted by a Single-Degree of Freedom system (SDF) under the following assumptions.

Firstly, the distribution pattern of the lateral displacements (drifts) of MDF resembles to its first natural mode of vibration, and secondly the equivalent height of the substitute SDF is determined from the height at ${}_1\beta\cdot\{{}_1u_i\}=1$ of the MDF, where, ${}_1\beta$ indicates the participation factor of the first natural mode, and $\{{}_1u_i\}$ indicates the first natural mode of vibration.

The set of $Q-\delta$ curves of MDF determined from pushover analysis is substituted to the global force-displacement relationship of SDF using the following equations [5].

$$ {}_1T \quad 2\pi\sqrt{\frac{\sum_{i=1}^{N} m_i \cdot \delta^2}{\sum_{i=1}^{N} P_i \cdot \delta_i}} \tag{2}$$

$$ {}_1M = \frac{\left(\sum_{i=1}^{N} m_i \cdot \delta_i\right)^2}{\sum_{i=1}^{N} m_i \cdot \delta_i^{\,2}} \tag{3}$$

Table 1: Seismic Design Criteria

Performance Levels of Building	**For Rear EQ. (500 years of Return period)**	**For Moderate EQ. (50 years of Return period)**
Special level (Grade 1)	Serviceable (1)	Serviceable / Non-damaged
High level (Grade 2)	Repairable (2)	Serviceable
Standard level (Grade 3)	Life-safe (3)	Repairable / Serviceable

(1) "Serviceable" level is that when the particular building systems are able to operate, therefore the structural elements had lightly-damaged or non-damaged.

(2) "Repairable" level is that when the buildings are able to be reused, therefore the structural elements behave in the inelastic region but within the limited deformation causing slight residual deformations.

(3) "Life-safe" level is that when the buildings do not collapse, therefore the structural elements are heavily damaged up to its deformation capacity.

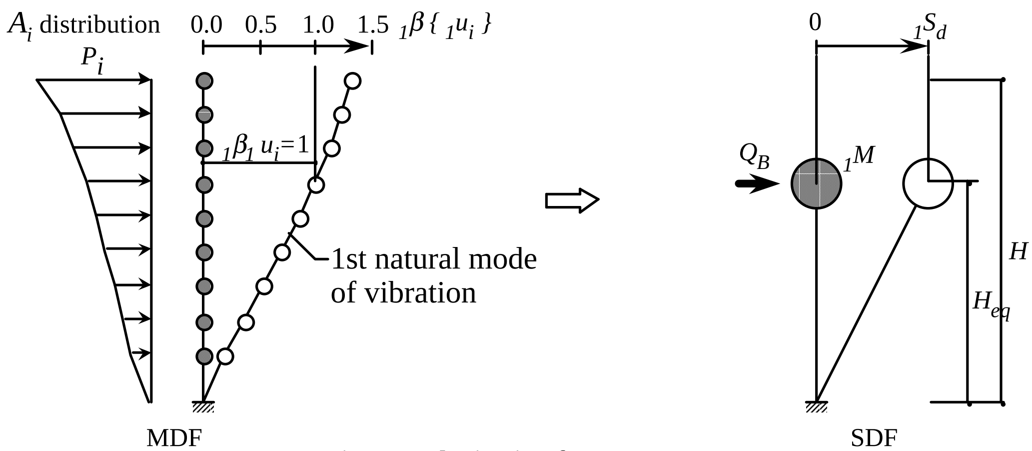


Fig. 2: Substituting from MDF to SDF

$$ {}_1S_a = \frac{Q_B}{{}_1M} \tag{4}$$

$$ {}_1S_d = \frac{{}_1S_a}{{}_1\omega^2} \tag{5} $$

$$ A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2{}_1T}{1+3{}_1T} \tag{6} $$

where, ${}_1T = 2\pi / {}_1\omega$: fundamental period of the substituted structure (SDF)

${}_1M$: substituted mass of the SDF which corresponding to effective mass of the 1st mode of MDF
Q_B : base-shear force
${}_1S_a$: substituted acceleration response (SDF)
${}_1S_d$: substituted displacement response of SDF which corresponds to the displacement at H_{eq} of MDF
δ_i : drift of the i -th story of MDF
P_i : lateral force applied to the i -th story of MDF which is determined from A_i distribution
${}_1\omega$: fundamental circular velocity of the substituted structure (SDF)
N : number of stories of MDF
H_{eq} : equivalent height of SDF corresponding to the height ${}_1\beta \cdot \{{}_1u_i\} = 1$ of the MDF
A_i : horizontal shear force distribution determined in current Japanese Seismic Design Code (MDF)
$\alpha_i = \sum_{i=i}^{N} w_i / \sum_{i=1}^{N} w_i$: the non-dimensional mass of i -th floor, where w_i indicates weight of i -th floor

Target Performance:

Performance levels of buildings are conventionally classified into 3 categories, as shown in Table 1, which can be defined numerically by the maximum displacement response [4]. Substituted damping is selected from “5 to 8%” for severe EQ, and “2 to 5%” for moderate EQ in the CSM. For ductile moment resisting steel frames, “Repairability after severe EQ (criteria 1)” is represented as the maximum displacement response less than *R1*, as shown in Fig.3, which is assumed here as 0.01 rad. of substituted drift angle (S_d / H_{eq}) by severe EQ with 8% damping to realize a slight residual displacement. “Life-safety after severe EQ (criteria 2)” is represented less than *R2* (assumed here 0.015 rad.) by severe EQ with 8% damping, and “Serviceability after moderate EQ (criteria 3)” is less than *R3* (assumed here as 0.005 rad.) by moderate EQ with 2% damping.

Structural members were selected by trial-and-error method until meeting the target performance through the fundamental period, the substituted drift angle, and the substituted acceleration response.

CASE STUDY

Model Frame :

Two 9-story office buildings A and B, which are rigidly jointed steel structures with the same configuration, were designed under different seismic performance criteria.

Building A(W=71kg/m) was designed to satisfy the minimum requirements, which are described as "Life-safe against severe EQ" and "Serviceable after moderate EQ", corresponding to “Standard level” in Table 1.

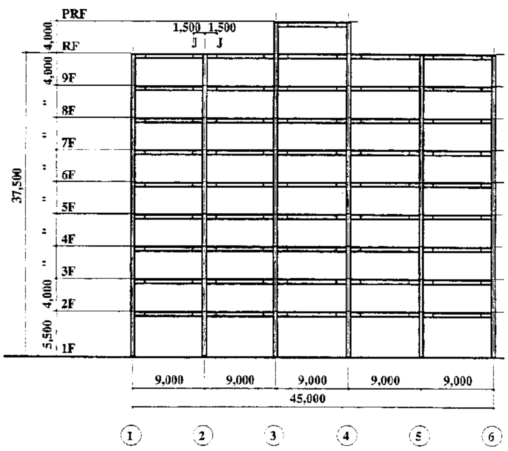
Building B(W=83kg/m) was designed to have one more requirement of "Repairable after severe EQ", which indicates higher seismic performance corresponding to “High level” in Table 1.

The structure is 5x5spans with 9mx9m spanning, as shown in Fig.4 [1], and average weight of the building (the dead +live load per unit floor area) is about 550kg/m . Columns are cold formed rectangular hollow sections, and beams are H-shaped rolled sections, as shown in Table 4 (material properties are both JIS SN490), which are selected to realize the weak beam type collapse mechanism with ductile behavior.

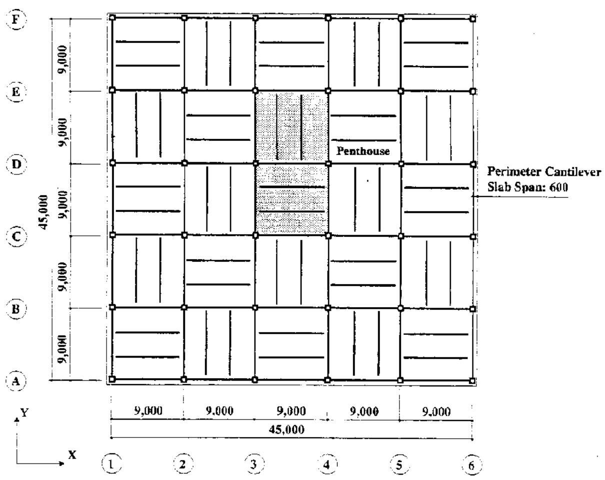
Table 2: Structural Members for Model Frame

Story	**Building A**		**Building B**	
	Column	**Beam**	**Column**	**Beam**
R/9	Box-450x16	H-500x250x9x16	Box-500x19	H-600x250x9x16
9/8				
8/7	Box-450x19	H-500x250x9x16(19)	Box-500x22	H-600x250x12x19
7/6		H-500x250x9x19(22)		H-600x250x12x19(22)
6/5				H-600x250x12x22(25)
5/4	Box-450x22	H-600x250x12x19(22)	Box-600x25	H-700x250x12x22(25)
4/3				
3/2				H-700x250x14x25(28)
2/1	Box-500x25	H-700x250x14x19(22)	Box-600x28	H-750x250x14x25(28)

( ) indicates the external end (at the connection to external column) of Beam member (Unit: mm)



(a) Structural Elevation



(b) Structural Plan

Fig. 4: Model Frame

Pushover Analysis:

Pushover analysis was performed under the following assumptions;

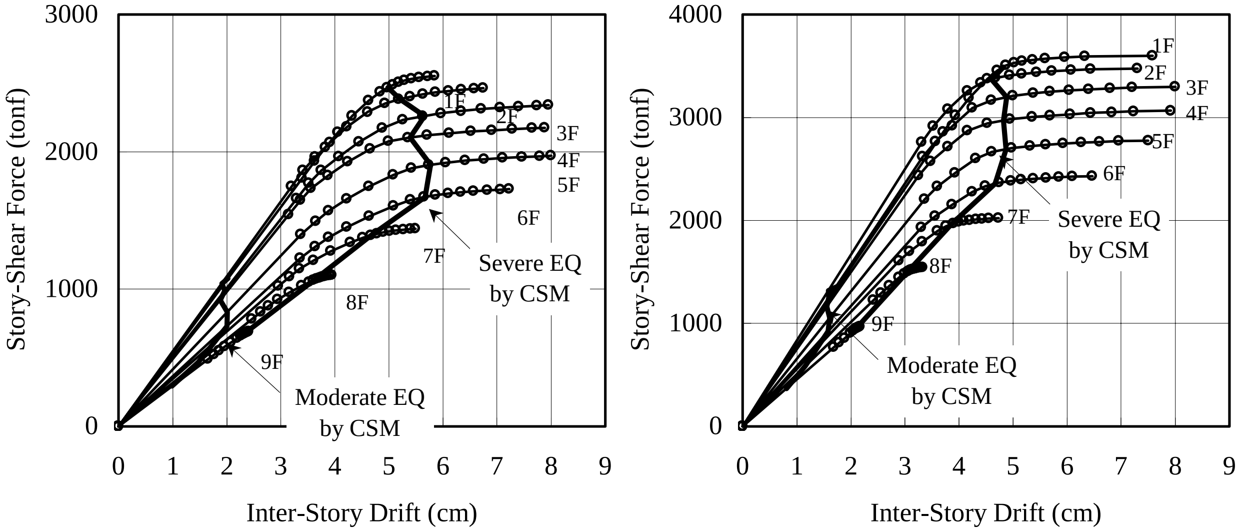
1) Lateral shear force distribution is A_i indicated in Eq(6). 2) Bending stiffness of beams are evaluated as composite beams with concrete slab. 3) Bending, shear, and axial deformations are considered in beams and columns. 4) Deformations of panel zone are neglected. 5) Column-bases are fixed. 6) Perfect bi-linear characteristics are assumed in the plastic hinges of beams at M_p (full plastic moment), and of columns at M_{pc} (reduced plastic moment due to axial force). 7) Yield strength (σ_y)=3.63 tonf/cm (10% increased from nominal value) was used for structural members.

Story shear force to inter-story drift relationships ($Q-\delta$ curves) for each story are shown in Fig.5.

Plastic hinges occurred, as shown in Fig.6, firstly at the beam-ends, and finally at the column bases, which indicates the weak beam type collapse mechanism.

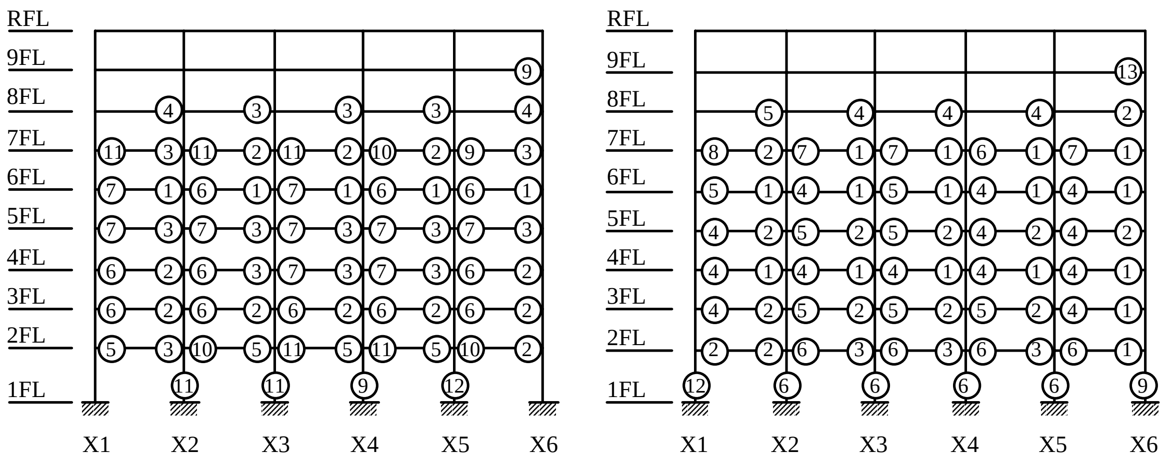
The hysteresis characteristics are modeled as tri-linear.

The first inflection point of this model (here defined as μ =1) is determined as the first yielding of a particular beam, especially at the rear end of the long-span beam caused by vertical load, and the second inflection point is determined at a stage close to the collapse mechanism (the third stiffness of this model is almost 0).



(a) Bldg. A **(b) Bldg. B**

Fig.5 : Story-Shear Force to Inter-Story Drift Relationship



(a) Bldg. A **(b)Bldg. B**

Fig. 6: Plastic Hinge Pattern

Demand Spectra and Capacity of Buildings

The demand spectra and the capacities of the buildings are described in ADRS format (S_a - S_d diagram), as shown in Fig.7. The demand spectra for severe EQ (500years of return period) with 5 to 8%t damping and moderate EQ (50years of return period) with 2%damping are derived from reference [3], which are applied to Tokyo area with medium soft soil condition. The capacity of the buildings are determined as global load-deformation relationships through Eq(2) to Eq(5).

The performance points obtained from CSM are summarized in Table 3.

The load-deformation curve of Bldg. A satisfies the criteria 2, however does not satisfy the criteria 1.
The Bldg.B satisfies the both criteria.
Although the substitute drift angles ($_1S_d$ / H_{eq}) of the Bldg. B corresponding to severe EQ is over 1/100, the residual drift angle (R^*) is less than 1/500 (0.02rad.), which is considered to maintain the "Repairbility".
The differences between Bldgs. A and B in the terms of unit steel weight is only 15%.

Table 3: Results of CSM

	Unit steel weight (W) *1)	**Severe EQ (500years of return period)**			**Moderate EQ (50years of return period)**		
		$_1S_a$ (gal) $(=Q_B/_1M)$	$_1S_d/H_{eq}$ (rad.)	R^* (rad.)	$_1S_a$ (gal) $(=Q_B/_1M)$	$_1S_d/H_{eq}$ (rad.)	R^* (rad.)
Bldg. A	71 kg/m	316	0.0127	0.0022	137	0.0046	0.
Bldg. B	83 kg/m	431	0.0110	0.0015	168	0.0037	0.

*1) W : total steel weight of columns and beams divided by the total floor area of the building

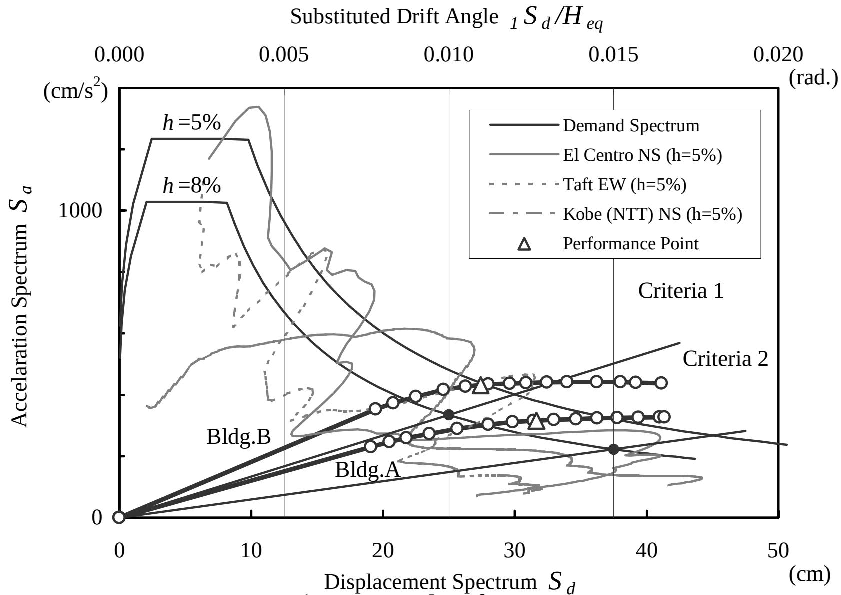


Fig. 7: Results of CSM

Table 4: Input EQ Waves

Input EQ Waves	**Recorded Max. Acceleration**	**Recorded Max. Velocity**	***Time Duration***	**Used Max. Acceleration Corresponding to Vmax=50cm/sec**
El Centro 1940 NS	341.7	33.65	30.0	510.8
Taft 1952 EW	175.9	17.12	30.0	496.6
Kobe (NTT) 1995 NS	330.7	88.95	20.0	185.9

Time History Dynamic Response Analysis:

The results obtained through CSM (a method of approximate analysis) were evaluated by a more precise elasto-plastic multi-mass time-history seismic response analysis using real EQ records.

The restoring force characteristics are assumed for normal-tri-linear, and the EQ records are, as shown in

Table 4, scaled to 50cm/sec as of maximum velocity (V_{max} =50cm/sec) corresponding to severe EQ.

To evaluate the characteristics of the input EQ waves, as shown in Fig.7, the response spectra with 5% damping were plotted in ADRS format (S_a - S_d diagram). In spite of randomness of their spectra, their envelope is similar (however quantitatively larger) to the smoothed design spectrum used in CSM.

Performance points obtained by CSM are shown in Fig.5.

The maximum response of displacement (drift angle) and ductility, shown in Fig.8, are larger than the results obtained from CSM, because the CSM represents the averaged behavior of MDF as SDF.

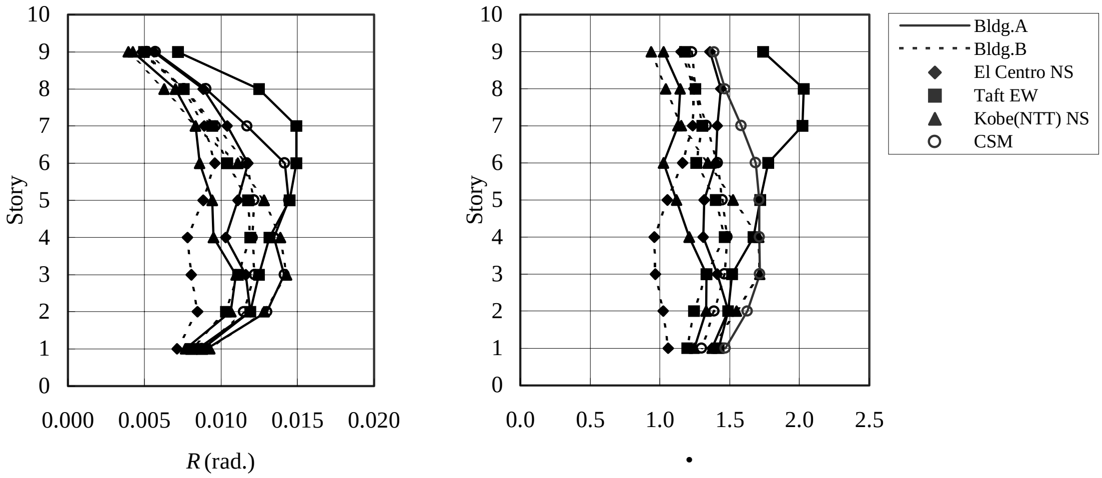


Fig. 8: Maximum Drift Angle (R) and Ductility Ratio (μ) Response

CONCLUSION

Conclusions are summarized as follows;

1. Performance based seismic design can be attained through CSM under appropriate assumptions.
2. Up-grading of building performance can be obtained by a slight expense of steel weight (only 15% of steel weight increase was enough in this case).
3. Results obtained through CSM are consistent with the results using more exact time-history seismic response analysis.
4. CSM shows the averaged response of Multi-Degree of Freedom system, therefore, the response concentration to the particular story should be avoided in the design procedures such as a control of collapse mechanism, a control of strength ratio between beam to column, etc.

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