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# JSSI MANUAL FOR BUILDING PASSIVE CONTROL TECHNOLOGY PART-7 STEPPING COLUMN SYSTEM

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## SUMMARY

Connection between upper structure and foundation is one of the most vulnerable and damaged locations during and after earthquake. This work introduces a system using stepping column to palliate or retrofit such disadvantage. Stepping column is designed to substitute rigid connections between upper structure and foundation by passive damper, as a consequence it allows parts of structure to uplift or stand on the ground alternatively depending on direction of ground acceleration. A single degree-of-freedom (SDOF) analytical model of stepping column system is established, in which the upper structure is modeled as a cantilever with lumped mass at top, the stepping column consists a friction damper and a contact element. General form of cyclic force-deformation behavior of the system is derived, important properties of the system in vibration also discussed. The hysteresis loop obtained from the model analytically is confirmed in good agreement with time-history analysis, and it clearly shows that amount of structure bending as well as energy dissipated depend on damper and vertical load posed on the system. Comparison responses of a 4-story building with its simplified SDOF model indicates that the proposed SDOF stepping column system can be used for calculating peak responses of the structure.

## INTRODUCTION

Many metropolitan areas have been severely struck by damaging earthquakes recently, taking along with a number of serious socio-economical problems. Many buildings ceased functioning and required costly structural and nonstructural repairs, although they may successfully protect the occupants' lives. Given the large number of buildings affected or will affect by strong earthquakes, many retrofit techniques were practically used for their seismic upgrading. Some popular solutions used for retrofitting buildings, for example, are the stiffening and strengthening of buildings with concentric diagonal steel bracing, or the use of energy dissipation devices. Because of the abundantly different forms of structure, however, it is always necessary for many other effective methods to be proposed and developed.

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Prominent characteristics of medium to high-rise structures are large tension force in columns due to high overturning moment, and their deformation primarily due to overall bending of structure rather than shear deformation, especially during seismic excitation. Consequently, connection between upper structure and foundation becomes one of the most vulnerable and damaged location during or after earthquake. This study proposes a so called 'stepping column system', which can be employed to palliate or retrofit damaged structures caused by those disadvantages. Stepping column [1,2] is designed to substitute rigid connections between upper frame and foundation by passive damper, in a way completely different from base-isolated system. The system allows parts of the structure to uplift or stand on the ground alternatively depending on direction of the ground movement.

**Stepping Column System**

The stepping column (SC) is in fact a substitution for rigid connections at base location by dampers, Fig. 1a is an example of an actual building having SC made by visco-elastic damper [2]. In this study, however, we consider a SC made by elasto-plastic (EP) damper for the purpose of generalizing the problem. The considering SC sketched in Fig. 1b consists an EP damper whose force-deformation relation is shown in Fig. 1c. In modeling the contact and uplift of this column with foundation, a contact element with force-deformation relation drawn in Fig. 1d is used. The SC system is called, in our current study to represent for any frame with SC inserted.

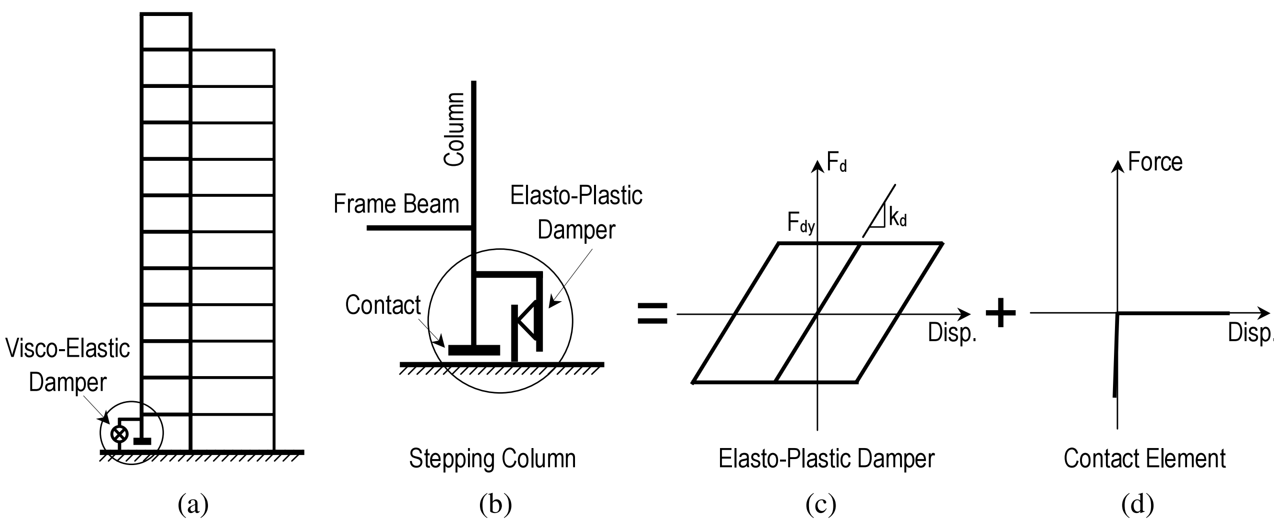



**Figure 1. Example of a Stepping Column.**

**Objectives**

The response of structures using stepping column system can always be obtained by nonlinear time history analysis of the full structure. However, such method is time consuming and lack of a clear understanding and controlling the problem. Moreover, it is useful to have some simple methods for evaluating peak responses of the structure at preliminary stage of design. The objective of this study, therefore, is to establish a SDOF analytical model for SC system and derive the general form of cyclic force-deformation behavior of the structure. Then assessing the feasibility of the model in simulation of real structures.

The hysteresis behavior obtained directly from the proposed SDOF model is confirmed in good agreement with time history analysis. Comparison responses of a 4-story building designed to use stepping column with its simplified SDOF model indicates that SDOF stepping column system can be used for calculating peak responses of the structure.

## STEPPING COLUMN SYSTEM

### SDOF Model of the System

SDOF model of a SC system is drawn in Fig. 2a. The model consists two stepping columns used in one span of a structure that is designed or retrofitted with SC. $L$ is the span length, and $H$ is *effective height* of the structure. *Effective stiffness* of the upper frame, which can be determined by performing a pushover analysis (details shown in later sections), is represented as bending stiffness $K_{\mathrm{SC}}$. *Total effective mass* of the structure is lumped as $M$, while amount of vertical load $W$ applied to each SC is determined as the portion of vertical load transfer to this span only. For SC, $F_{dy}$ is yield force of damper and initial elastic stiffness $k_d = \infty$ is assumed, e.g. lead damper or friction damper (Fig. 2c), for simplification of derivation process. The stiffness of contact element in compression (Fig. 1d) $k_c = \infty$ also assumed.

### Force-Displacement Relation

Under an applying force the SC system vibrates in different stages, which are classified as follows

- Stage 0: both stepping columns fully contact with the ground.
- Stage 1: either left column (Stage 1L) or right column (Stage 1R) uplifted (Fig. 2b).
- Stage 2: both columns uplifted.

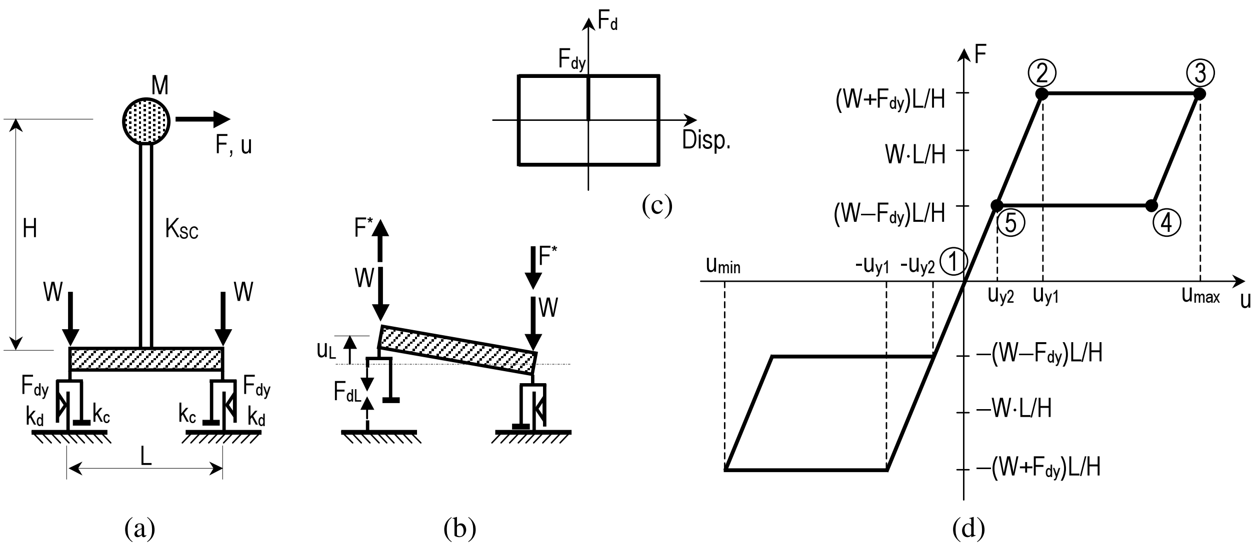



**Figure 2. SDOF Model of the SC System.**

From Fig. 2a the relation between applying force $F$ and displacement $u$ can be written generally as

$$u = \frac{F}{K_{\mathrm{SC}}} + u_d \frac{H}{L} \tag{1}$$

here $u_d$ stands for $u_{\mathrm{L}}$ or $u_{\mathrm{R}}$: deformation of left or right SC depending on respective stage.

In order to establish the $F$-$u$ relationship, we consider a portion drawn in Fig. 2b, in which the upper structure temporarily taken out and $F$ is replaced by equivalent couple $F^*$, thus

$$F^* = F \frac{H}{L} \tag{2}$$

Assuming $F$ applied from left to right (Fig. 2a) at the beginning, to some extent left SC will be uplifted while right SC stands firm on the ground. Equilibrium of the forces in Fig. 2b is written as

$$F^* = W + F_{d\mathrm{L}} \tag{3}$$

Now using Eqs. 1-3, the force-displacement relation of SC system is obtained by reasoning as follows
- Path ① – ②, $F < (W + F_{dy})L/H$ (or $F^* < W + F_{dy}$): 2 SCs stand on the ground, $u_\mathrm{L} = u_\mathrm{R} = 0$ (Fig. 2c),

$$u = \frac{F}{K_{\mathrm{SC}}} \tag{4}$$

- Path ② – ③, $F > (W + F_{dy})L/H$ (or $F^* > W + F_{dy}$): left column is to be uplifted $u_\mathrm{L} \neq 0$, $u_\mathrm{R} = 0$. We have

$$u = u_{y1} + u_L \frac{H}{L}, \quad u_{y1} = \frac{(W + F_{dy})L/H}{K_{SC}} \tag{5a,b}$$

Keep increasing $F$, damper in the left column will be stretched to its maximum displacement $u_{d\mathrm{L}0}$, and peak displacement at the mass level is $u_{\max}$, in correspondence.

- Path ③ – ④, the force is reduced $F < (W + F_{dy})L/H$ (or $F^* < W + F_{dy}$): there is no change in $u_\mathrm{L}$ ($u_\mathrm{L} = u_{d\mathrm{L}0}$) because the damper is of friction type (see Fig. 2c). We have

$$u = u_{\max} - \frac{(W + F_{dy})L/H - F}{K_{\mathrm{SC}}}, \quad u_{\max} = u_{y1} + u_{d\mathrm{L}0}\frac{H}{L} \tag{6a,b}$$

- Path ④ – ⑤, $F$ continues to be reduced so $F < (W - F_{dy})L/H$ (or $F^* < W - F_{dy}$): $u_\mathrm{L}$ starts to reduce. Then

$$u = u_{d\mathrm{L}0}\frac{H}{L} + u_{y2} - (u_{d\mathrm{L}0} - u_\mathrm{L})\frac{H}{L} = u_{y2} + u_\mathrm{L}\frac{H}{L}, \quad u_{y2} = \frac{(W - F_{dy})L/H}{K_{\mathrm{SC}}} \tag{7a,b}$$

Keep decreasing $F$, left column can be compressed back to its initial configuration ($u_\mathrm{L} = 0$) and the mass move back to its initial position. Repeat this reasoning process for the opposite side we will come up to the $F$-$u$ hysteresis behavior as drawn in Fig. 2d.

The hypothesis of small displacements was used in conducting above reasoning, axial deformation of the column as well as $P$-$\Delta$ effect were also assumed to be insignificant

As can be seen in Fig. 2d, increase the vertical load $W$ applied on the system results in increasing of yield force of the system thus some reduction of peak displacement should be expected. On the other way, increase yield force $F_{dy}$ of damper results in considerable gain of energy dissipated per cycle, therefore the peak displacement should also be reduced.

**Vibration Features**

One prominent characteristic of SC system is that its ability to change between stages whose dynamic properties can be very different. In previous section the hysteresis behavior of the system was derived using static analysis, yet it is expected that under dynamic loading the system also conform to this hysteresis loop. This section investigates how the dynamic properties become different when the system changing from stage to stage.

Considering a particular case in which no damper in SC, i.e. in analysis model the damper is substituted by an elastic spring with negligible stiffness. Also denote the total stiffness of left and right SC as $k_1$ and $k_2$, respectively, their values written in terms of stiffness of damper (negligible) $k_d$ and contact element $k_c$ depend on particular stage at shown in Table 1.

**Table 1. Stiffness of SC System in Different Stage.**

| SC Stiffness | Stage 0 | Stage 1L | Stage 1R | Stage 2 |
|---|---|---|---|---|
| $k_1$ | $k_d + k_c$ | $k_d$ | $k_d + k_c$ | $k_d$ |
| $k_2$ | $k_d + k_c$ | $k_d + k_c$ | $k_d$ | $k_d$ |

The single mass system has many degrees-of-freedom but only two distinct vibration frequencies, i.e. those in accordance with horizontal and vertical displacement of mass $M$ (Fig. 2a). Utilizing the direct stiffness method to establish the equations of motion, then the closed-form solution for free vibration frequencies of the system in different stages can be obtained as below

$$\omega_1^2 = \frac{A_1 - \sqrt{A_2}}{A_3}; \quad \omega_2^2 = \frac{A_1 + \sqrt{A_2}}{A_3} \tag{8}$$

with

$$\begin{aligned} A_1 &= 12MEIHL^2k_1k_2 + EA[4H^3L^2k_1k_2 + 3EI(k_1+k_2)(4H^2+L^2)]M \\ A_2 &= M^2E^2\{-48IAH^2L^2k_1k_2[4H(3EI(k_1+k_2)+HL^2k_1k_2)+EA(12EI+HL^2(k_1+k_2))]+ \\ &\quad [12IHL^2k_1k_2 + A(4H^3L^2k_1k_2 + 3EI(k_1+k_2)(4H^2+L^2))]^2\} \\ A_3 &= 2M^2H^2\{4H[3EI(k_1+k_2)+HL^2k_1k_2]+EA[12EI+HL^2(k_1+k_2)]\} \end{aligned} \tag{9a-c}$$

here $E$, $A$, $I$ = Young's modulus, cross sectional area, moment of inertia of the column ($K_{SC} = 3EI/H^3$).

Now investigating a SC system with the following set of data: $L$ = 9.6m, $H$ = 28m, $M$ = 6250ton, $K_{SC}$ = $2.47\times10^5$kN/m (fundamental vibration period of the system is 1s), $k_d$ = 2.1kN/m, $k_c$ = $10^8$kN/m, and $A = \infty$ is assumed. Using Eq. 8, the exact frequencies for all vibration modes of the system in different stages are obtained as in Table 2.

**Table 2. Vibration Frequencies of the System in Different Stages.**

| | **Stage 0** (both columns sitting) | | **Stage 1** (one column uplifted) | | **Stage 2** (both column uplifted) | |
|---|---|---|---|---|---|---|
| | Freq. (rad/s) | Period (s) | Freq. (rad/s) | Period (s) | Freq. (rad/s) | Period (s) |
| 1st mode | 6.1553 | 1.0208 | 0.0063 | 999.7869 | 0.0044 | 1413.8786 |
| 2nd mode | 4466.7016 | 0.0014 | 880.0750 | 0.0071 | 0.6481 | 9.6952 |

In each stage, 2nd mode (vertical vibration) frequency is much greater than 1st mode (horizontal vibration) frequency, thus the effect of 2nd mode is very insignificant. Moreover, although considered in the formulation the possibility for stage 2 to occur can be disregarded because of the vertical load $W$ applied on each SC. Therefore, it is reasonable to believe the system has only two main vibration modes, which are equivalent to 1st mode of stage 0 and stage 1 (shaded cells in Table 2), respectively.

Normally, in dynamic time history analysis the viscous damping matrix is formed proportionally to mass or stiffness matrix or both as in the case of Rayleigh damping, the proportional factors ($\beta_M$ or $\beta_K$) are determined base on assumed modal damping ratios. Also note that the frequencies of significant modes (e.g. 6.1553 and 0.0063 rad/sec) are very different. Therefore in case of mass proportional damping, whose damping ratio is inversely proportional to the frequency ($\xi_n = 0.5\beta_M/\omega_n$), if the proportional factor $\beta_M$ is calculated using the fundamental vibration period of initial state (stage 0) only, this factor will create an incorrectly excessive amount of damping once the system vibrating into second mode. Taking the considering case for example, 1st mode is used to obtain proportional factor for 2% damping ratio: $\beta_M = 2\omega_1 \cdot \xi_1 = 2 \times 6.1553 \times 0.02 = 0.2462$. Using this factor the damping ratio of the second significant mode turns out to be $\xi_2 = 0.2462/(2 \times 0.0063) = 19.5$ or 1950%, which means response of the structure will be quickly damped out once one of the SC uplifted (stage 1).

Obviously the Rayleigh damping is the most accurate damping for current system. However, stiffness proportional damping is still appropriate if only one vibration period is known at first, because damping ratio calculated from this type of damping directly in proportion with the frequency ($\xi_n = 0.5\beta_K \cdot \omega_n$), thus it does not create unrealistic damping ratio for the other modes of the current problem.

**Numerical Example**

This example illustrates the discussions in previous sections. It uses data of a SC system from experimenting structure, which will be introduced in next section. The data include: $L = 3.2$m, $H = 6.085$m, $M = 155$ton, $W = 430$kN, $K_{SC} = 4.29 \times 10^4$kN/m ($T = 0.378$sec), $k_d = 3.3 \times 10^5$kN/m, $F_{dy} = 400$kN and $k_c = 10^7$kN/m. Earthquake input is BCJ-L2 [3]. A finite element program called PC-ANSR [4] is used for analysis, the model uses nonlinear truss element for modeling dampers, gap element as contact element. The system is analyzed with initial damping ratio of 2%, which computed by utilizing stiffness proportional damping. The ground acceleration time history of BCJ-L2 earthquake, displacement response time history and base shear-displacement relation are plotted in Fig. 3.

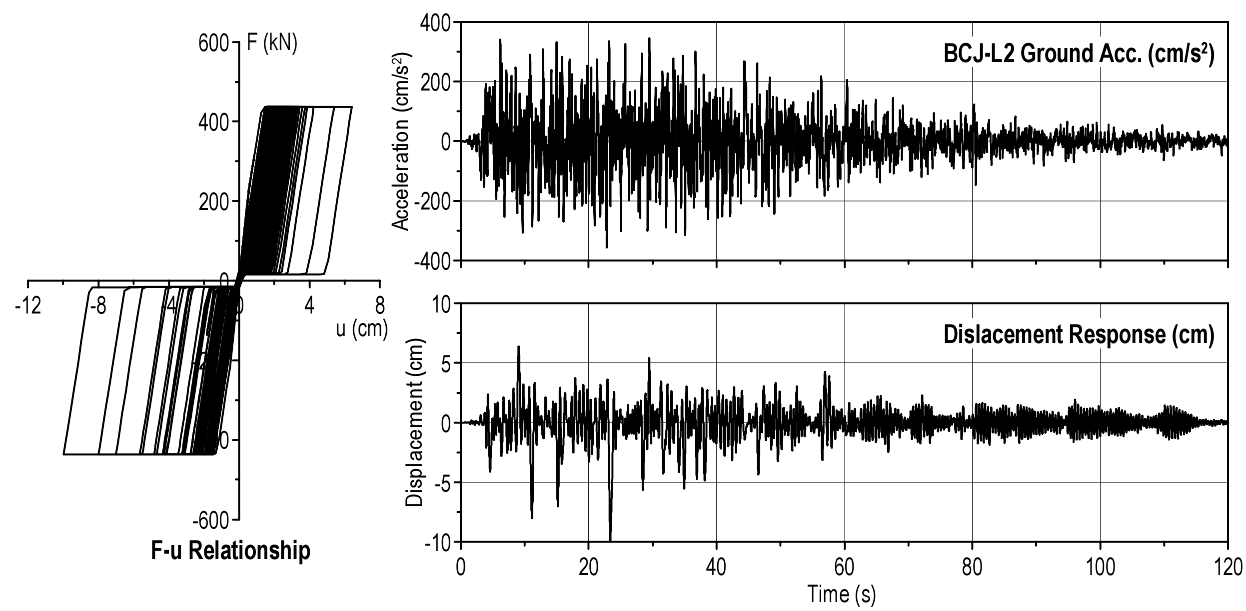



**Figure 3. Responses of SC System Under BCJ-L2 Earthquake.**

It is clearly that the hysteresis loop obtained by time history analysis has been well predicted in Fig. 2d. Elastic response spectrum of BCJ-L2 earthquake gives peak displacement of the similar structure without SC is only 5cm, which means about half of peak displacement of SC system. However, the maximum base shear force of SC system is 440kN, only about 20% of peak base shear of the system without SC.

## SC SYSTEM IN MODELING 4-STORY STEEL BUILDING

### 4-Story Steel Frame and SDOF Model

This section describes a steel frame that will be fabricated for the shake table test, the frame is 1/2 scale of a 4-story steel building planned to use stepping column. Then the SDOF model for the frame using SC system is presented.

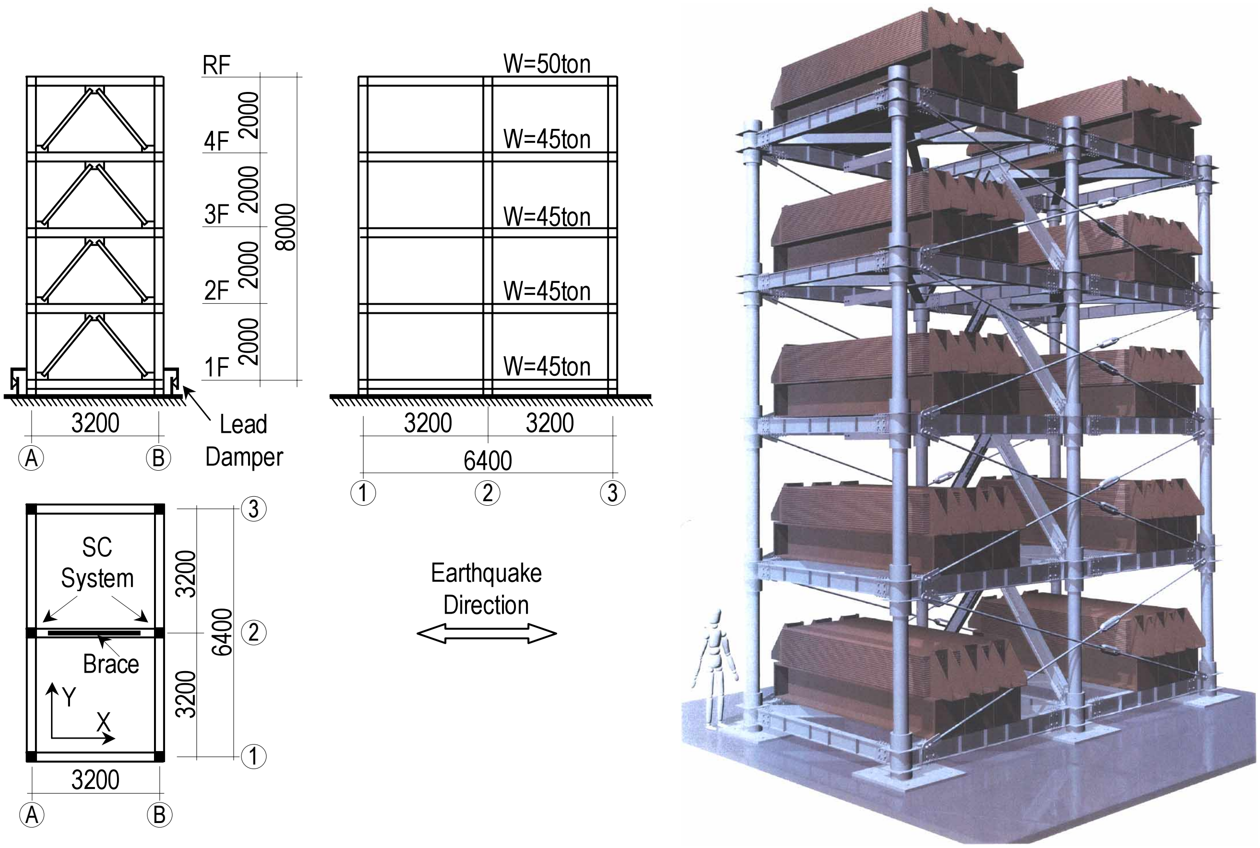



**Figure 4. Four Story Steel Frame.**

The testing frame's dimension and vertical load are given in Fig. 4. Plane frame No. 2 is designed to use SC system and will be called here as SC-frame, the vertical load is distributed so as about 38% goes to this SC-frame. External frames are fixed as usual and will be called here as FX-frames. The fundamental period of the original structure was about 0.5 s. In case of testing frame, vibration periods of the first three modes, which are all horizontal vibration modes, are 0.32, 0.11, 0.06 s, respectively; the 4$^{th}$ mode is vertical vibration mode, having period of 0.06s.

Lateral stiffness of each story is computed by pushover analysis of the structure subjected to a set of monotonically increasing lateral forces with an invariant height-wise distribution. When there is no uplift, lateral stiffness is of the whole structure; when the SC-frame uplifted, the remaining stiffness is of FX-frames only. Therefore, the order for calculating lateral stiffness of SC-frame and FX-frame is as follows: first the total stiffness of each story is calculated using results at initial steps (full structure is standing on the ground) of the pushover analysis. Second, the stiffness at each level of FX-frames (a small amount stiffness of connected beams also included) is determined using results of the pushover analysis after the SC-frame uplifted. Finally, the stiffness of each story in SC-frame is computed by subtracting total stiffness with stiffness of FX-frame. Results are listed in Table 3.

**Table 3. Horizontal Stiffness of Story (kN, m)**

| Frame | FL1 | FL2 | FL3 | FL4 |
|---|---|---|---|---|
| Total | $2.018{\times}10^5$ | $1.556{\times}10^5$ | $1.253{\times}10^5$ | $9.440{\times}10^4$ |
| FX-frame | $6.060{\times}10^4$ | $5.150{\times}10^4$ | $4.000{\times}10^4$ | $2.630{\times}10^4$ |
| SC-frame | $1.412{\times}10^5$ | $1.041{\times}10^5$ | $8.530{\times}10^4$ | $6.810{\times}10^4$ |

Having stiffness of each story, the effective mass and height [5] of fundamental mode is approximated from the static deflections (Eq. 10a-b) due to a selected set of lateral forces $\{F_i\}$, the forces $\{F_i\}$ can be computed in accordance with the so-called $A_i$ distributions specified in Japanese Seismic Design Code [6]. At the $i$th story, $A_i$ times total weight of the higher floors determines the corresponding story shear.

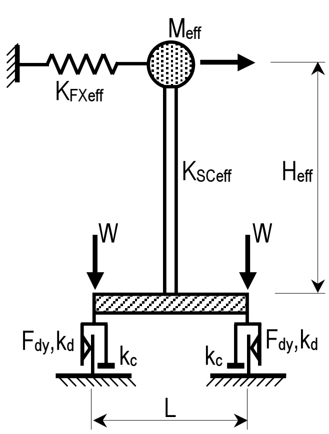
$$M_{eff} = \frac{\left(\sum_{i=1}^{4} M_i u_i\right)^2}{\sum_{i=1}^{4} M_i u_i^2}; \quad H_{eff} = \frac{\sum_{i=1}^{4} H_i M_i u_i}{\sum_{i=1}^{4} M_i u_i}; \quad \omega_1^2 = \frac{\sum_{i=1}^{4} F_i u_i}{\sum_{i=1}^{4} M_i u_i^2} \qquad \text{(10a-c)}$$

here $M_i$, $H_i$, and $u_i$ = mass, height, and displacement at i-th floor, respectively.
The effective stiffness is computed using effective mass $M_{eff}$ and the natural vibration frequency $\omega_1$, which can be estimated using Rayleigh's Quotient as in Eq. 10c. Table 4 summarizes all the outcome of above calculation process.

**Table 4. $F_i$ Distribution and Effective Values**

| Floor | $M_i$ (ton) | $H_i$ (m) | Total plane frames | | External plane frames (fix) | |
|---|---|---|---|---|---|---|
| | | | $k_i$ (kN/m) | $F_i$ (kN) | $k_i$ (kN/m) | $F_i$ (kN) |
| 4 | 50 | 8 | $9.440{\times}10^4$ | 752.0 | $2.630{\times}10^4$ | 798.2 |
| 3 | 45 | 6 | $1.253{\times}10^5$ | 444.6 | $4.000{\times}10^4$ | 445.2 |
| 2 | 45 | 4 | $1.556{\times}10^5$ | 349.7 | $5.150{\times}10^4$ | 333.6 |
| 1 | 45 | 2 | $2.018{\times}10^5$ | 266.7 | $6.060{\times}10^4$ | 236.0 |
| Effective values | | | $\omega_1$ = 20.0 rad/s<br>$T_1$ = 0.3 s<br>$M_{eff}$ = 154.9 ton<br>$K_{eff} = 6.202{\times}10^4$ kN/m<br>$H_{eff}$ = 6.073 m | | $\omega_{FX1}$ = 11.2 rad/s<br>$T_{FX1}$ = 0.6 s<br>$M_{FX\_eff}$ = 153.8 ton<br>$K_{FX\_eff} = 1.916{\times}10^4$ kN/m<br>$H_{FX\_eff}$ = 6.096 m | |

A simplified SDOF model for analyzing the frame is proposed in Fig. 5, using SC system for modeling SC-frame and a support spring for FX-frame. In this SDOF model $H_{eff}$, $M_{eff}$, $K_{SCeff}$, $K_{FXeff}$ are from the effective values in Table 4. The properties of SC and vertical load $W$ are also given in Fig. 5.
In considering effects of vertical vibration mode in the SDOF model, another dynamic analysis is also conducted in which vertical component of mass is included. $M_y = 0.5M_{eff}$, based on the tributary area that is half of the floor area (Fig. 4); axial stiffness of SC system is computed such as its vertical vibration period is similar to that of MDOF frame (0.06 s).



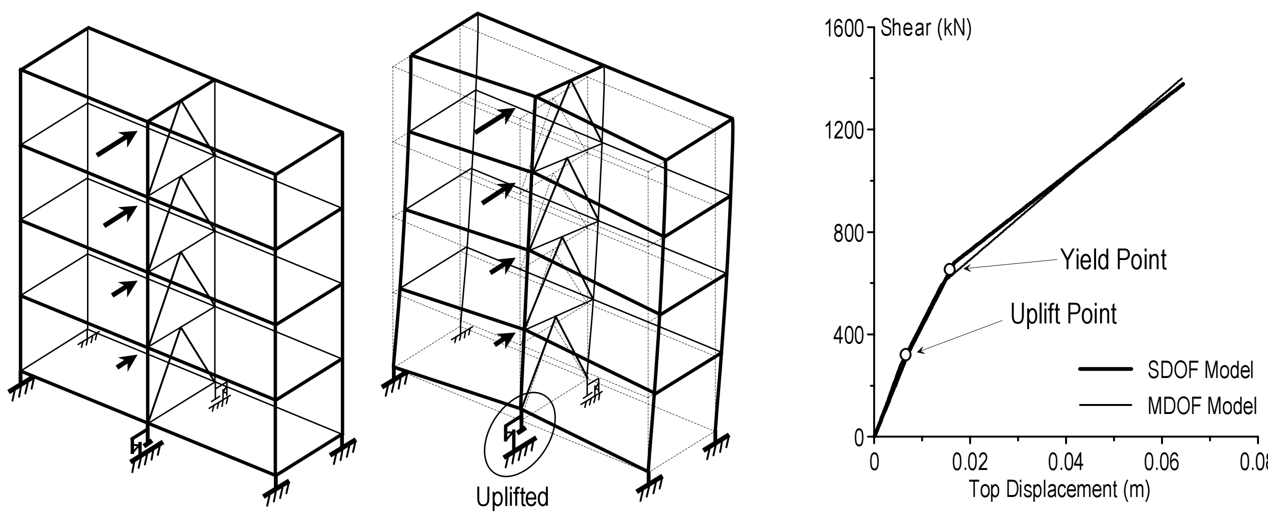

**SDOF Model Properties**
$H_{eff}$ = 6.085 m; L = 3.2 m
$M_{eff}$ = 155 ton
$K_{FXeff} = 1.916{\times}10^4$ kN/m
$K_{SCeff} = 4.286{\times}10^4$ kN/m
**SC Properties**
Contact: $k_c = 1.00{\times}10^7$ kN/m
Damper: $F_{dy}$ = 400 kN
$k_d = 3.30{\times}10^5$ kN/m;
$\alpha = 1.0{\times}10^{-4}$
**Vertical Load**
W = 430 kN

**Figure 5. SDOF Model of Testing Frame.**

## Comparative Evaluation of Analysis Models

Static analysis of the multi degree-of-freedom (MDOF) 4-story frame (Fig. 6a) and its SDOF model (Fig. 5) subjected to lateral forces that increases in small increments is implemented, deformed shape of the frame after uplifted can be seen in Fig. 6b. Distribution of lateral forces $\{F_i\}$ for MDOF frame is in accordance with $A_i$ distribution, the load step in SDOF analysis also taken same as the load step used in MDOF analysis, $\Delta F = 0.01\Sigma M{\cdot}g = 2.3$kN.

Resulting curves of base shear force vs. top displacement are shown in Fig. 7c. Refer to Fig. 2d, the shear force at which SC system starts to uplift and yield force of SC system (corresponding to yielding of damper) are $W{\cdot}L/H$ and $(W+F_{dy})L/H$, respectively. In case of the current SDOF model, stiffness $K_{\mathrm{FX}eff}$ of additional FX-frame needs to be accounted for (Fig. 5), the equivalent forces can be computed approximately as $W{\cdot}L/H_{eff}{\cdot}(1+K_{\mathrm{FX}eff}\ /K_{\mathrm{SC}eff}) \approx 330$kN and $(W+F_{dy})L/H_{eff}{\cdot}(1+K_{\mathrm{FX}eff}\ /K_{\mathrm{SC}eff}) \approx 630$kN, respectively. Observe that the pushover curves match each other closely.




**(a)** **(b)** **(c)**

**Figure 6. Deformed Shape After Uplifted and Force-Deformation Curves.**

The responses of the MDOF frame and SDOF model to the ground motion of BCJ-L2 earthquake will be compared next. The response quantities of interest are floor displacements, base shear, and base overturning moment in the building. The time axis of input wave for these time history analyses was rescaled to $1/\sqrt{2}$ so that the similarity law satisfied. The duration of input wave is taken as 50 seconds, analysis time step for MDOF structure and SDOF model are $0.01/200\sqrt{2}$ and 0.001, respectively. Initial damping ratio is assumed to be 0.5% and input as stiffness proportional damping.

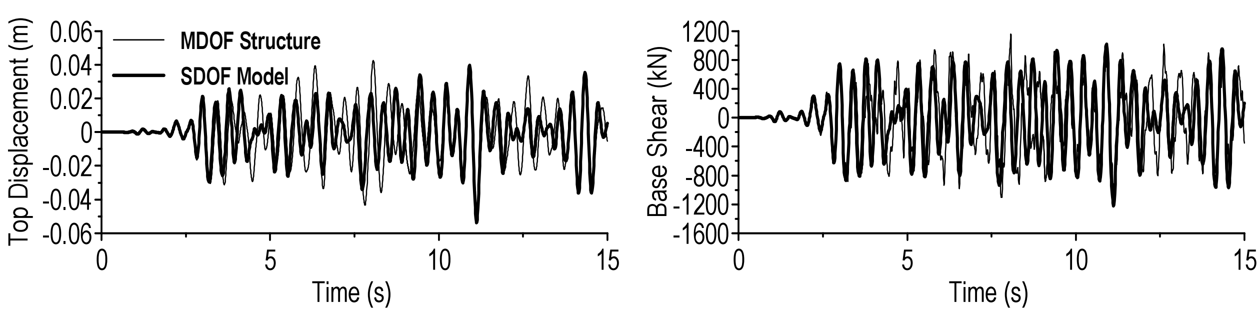



**Figure 7. Top Displacement and Base Shear Responses.**

Displacement response at top level of the building and base shear from MDOF frame and SDOF model are shown in Fig. 7 for the first 15 seconds. Note that displacement and also base shear are essentially in phase, thus it seems under BCJ-L2 earthquake the 4-story building mostly vibrate in fundamental mode; the peak values are also similar due to similar periods and identical damping ratio in the two analyses.

Fig. 8 plots the relations between the base overturning moment and roof drift angle $\theta$ (roof displacement divided by total height) obtained from time history analysis. The theoretical values of shear force at which SC system starts to uplift and yield force of SC system can also be confirmed in Fig. 8a (= $M/H_{eff}$). The smooth hysteresis of the SDOF model disappears when vertical vibration mode is considered, however the peak responses are almost identical in two analyses (Fig. 8b). The reason is identified because of impact on contact element when SC sitting back, causing large vertical acceleration of the mass and producing additional axial force in the column, which in turn effects on the base shear force and overturning moment. Fig. 8c ascertains that peak response can be accurately determined with this simple SDOF model. The $M$-$\theta$ relation of MDOF structure is quite fluctuated like the SDOF model accounted for vertical vibration, however it obviously resembles the smooth hysteresis of the simplified SDOF model. Sources of this fluctuation can be fingered out because the overturning moment of MDOF frame is a combination of some other vibration modes, in which mainly due to the vertical vibration mode.

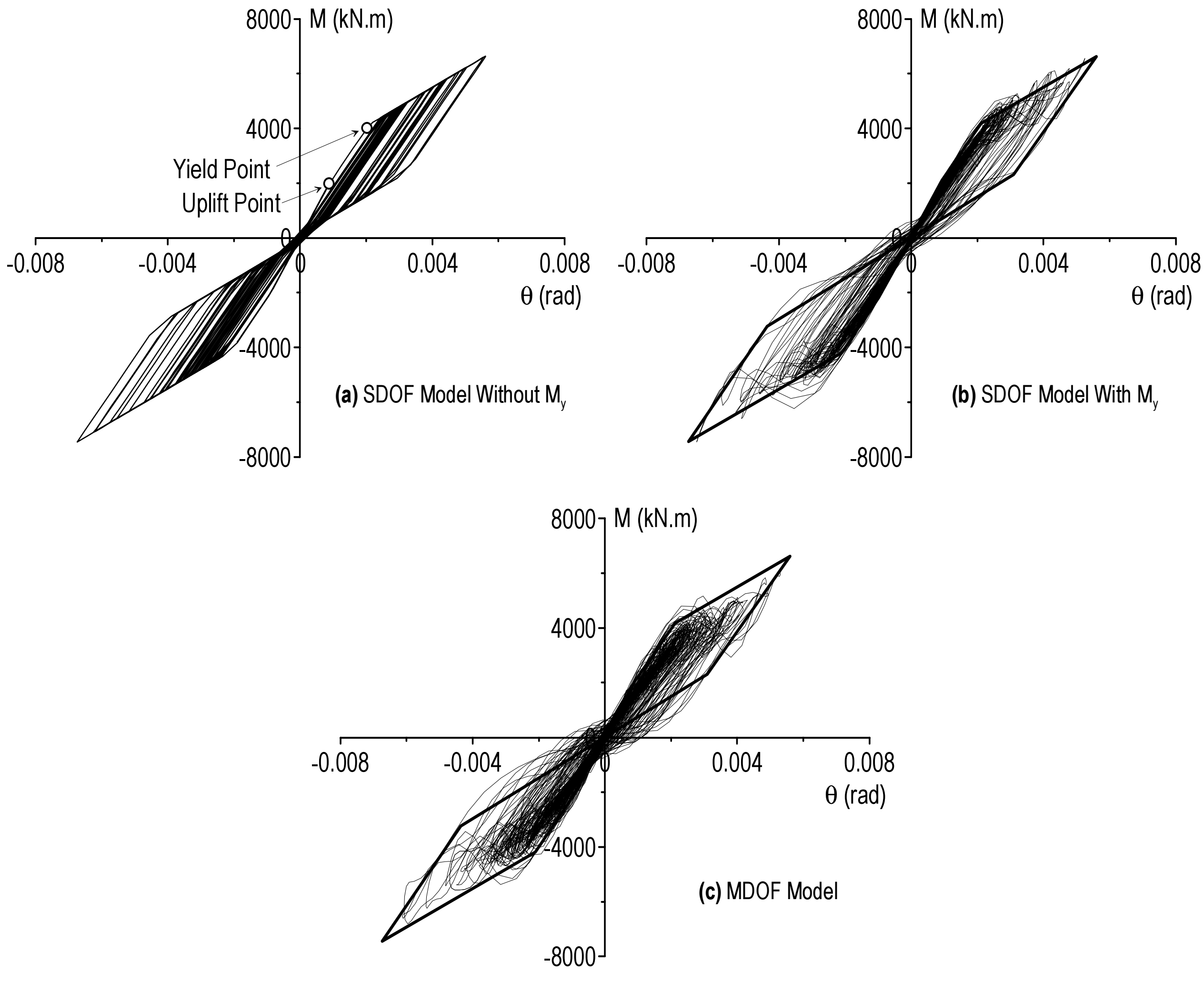



**Figure 8. $M$-$\theta$ Relations From SDOF Model (a-b), and MDOF Structure (c).**

In Fig. 9 the peak values of floor displacements, story drifts, shear forces and moment of the MDOF frame are compared with respective values interpolated from SDOF model assuming straight deformed shape. The drifts are somewhat underestimated in lower stories and overestimated up to 18% for the upper story.

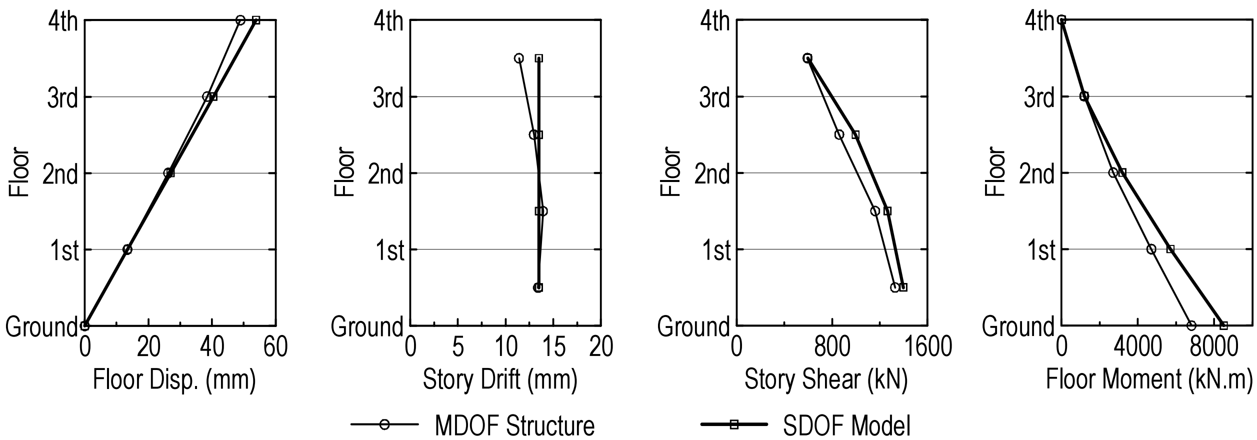



**Figure 9. Height-wise Variation of Displacement, Drift, Shear, and Moment.**

## CONCLUSIONS

This study proposes a new form of building's passive control using friction damper to substitute the rigid connection between upper structure and foundation. The study aimed to develop and evaluate the feasibility of a simplified SDOF model that can be used for analyzing stepping column structure. It has led to the following conclusions:

1) The behavior of the structure using stepping column is reasonably predicted by the derived hysteresis loop. The hysteresis clarifies that amount of structure bending as well as energy dissipated depends on damper and vertical load posed on the system. Therefore, it can serve as a useful tool for selecting important parameters in designing stepping column.

2) The ability of uplifting facilitates stepping column structure extending its vibration period, therefore reducing acceleration response, base shear and moment. When SC returning back to the ground causes some kind of impact forces but their effect on the peak responses is found insignificant.

3) Comparing the peak responses of a 4-story steel building with its simplified SDOF model using proposed SC system demonstrated that the simplified model provided very good results of floor displacements, story drifts, as well as shear force and overturning moment. Based on issues presented here the authors are persuaded to develop a procedure to estimate the peak responses utilizing only response spectrum without necessary to conduct time history analysis. Work is in progress.

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## REFERENCES


1. Kanada, M., Kasai, K., and Okuma, K. “Innovative Passive Control Scheme: a Japanese 12-Story Building with stepping Columns and Viscoelastic Dampers”, Proceeding of Structural Engineers World Congress (SEWC) 2002, Yokohama, JAPAN, CD-ROM, T2-2-a-5, 2002 (in Japanese).
2. Kasai, K., Kanada, M., and Okuma, K. “Real Example for a Passively Controlled Building with Stepping Column: Analysis and Full-Scale Damper Experiment”, Proceeding of Passively-Controlled Structure Symposium 2001, Structural Engineering Research Center, Tokyo Institute of Technology, 2001: 235-249 (in Japanese).
3. The Building Center of Japan. Available at http://www.bcj.or.jp/download/dl_index.html
4. Maison B.F. “PC-ANSR A Computer Program for Nonlinear Structural Analysis”, 1992 Version, National Information Service for Earthquake Engineering, California.
5. Chopra, A.K. “Dynamics of Structures: Theory and Applications to Earthquake Engineering”, Prentice-Hall, Inc., New Jersey, 1995.
6. Architectural Institute of Japan. “Ultimate Strength and Deformation Capacity of Building in Seismic Design (1990).” 1990 (in Japanese)