SEISMIC RETROFIT BY CONNECTING TO ADJACENT BUILDING

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ABSTRACT :

Feasibility of retrofitting seismically vulnerable buildings, designed in accordance with out-dated building codes, was studied by externally inter-connecting to an adjacent building, constructed in accordance with present building codes, at floor levels with connecting elements at floor levels, so that the use or the occupancy will not be interrupted during the work. The reliability of this retrofit method was examined by nonlinear earthquake response analysis of multi-story frame buildings. The lateral resistance and deformation relation of columns was selected on the basis of experimental data, in which shear strength, loss of gravity load carrying capacity and loss of lateral load resistance were recognized.
An old five-story building, subjected to a series of code level earthquake motions, is shown to collapse at a specific story. Therefore, the five-story seismically vulnerable building is connected to a new ten-story building, designed to develop the weak-beam strong-column performance, at floor levels by connecting elements. It is shown that the collapse of the vulnerable building can be prevented by the connection, although significant damage developed in the five-story building and the deformation of the ten-story building slightly increased.

KEYWORDS : Seismic retrofit, Seismically vulnerable building, Collapse, Interconnecting buildings, Viscous damper connecting elements, Nonlinear response

1. INTRODUCTION

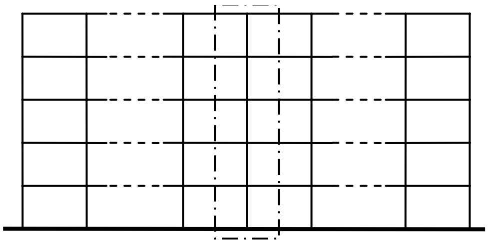
During the 1995 Hyogo-ken Nanbu Earthquake, many reinforced concrete buildings (hereafter called “old buildings”), designed and constructed before the 1981 revision of building codes in Japan, collapsed at a specific story due to the loss of gravity load carrying capacity of columns after brittle shear failure. It is urgent to strengthen these existing old buildings for earthquake resistance. Such rehabilitation work, however, does not progress as fast as it should due to the fact that the rehabilitation often requires the interruption of the use or occupancy of the building. This paper studies the feasibility of strengthening an old building by externally connecting to an adjacent new building constructed in accordance with the present building codes (hereafter called a “new building”) at floor levels by connecting elements so that the occupancy or the use will not be interrupted during the work.

The effectiveness of controlling seismic response by connecting single-story or multi-story linearly elastic buildings has been shown by many researchers in the past. Few studies in the past dealt with inelastic buildings. Fujii et al. (2004) studied the feasibility of retrofitting existing buildings by connecting to adjacent better performing buildings of different height. Friction dampers were used as connecting members. The story-shear and drift relation of a building under monotonically increasing load was idealized by a trilinear skeleton curve. Different failure modes were considered depending on construction year. It was generally observed that the response could not be reduced by coupling structures of similar height, but the response amplitudes could be reduced when buildings exhibited different story-shear and drift relations or failure modes.

This paper studies the earthquake response of multi-story buildings in an inelastic range, especially considering the collapse of an old reinforced concrete building by the loss of vertical load carrying capacity of columns after shear failure.

2. ANALYTICAL MODEL

A multi-story uniform moment-resisting frame as shown in Figure 1 is represented by a series of columns on a single vertical column line and adjacent beams (fish-bone model) as shown in Figure 2, by cutting the beams at the mid-span of the beams. The beam ends of the fish-bone model correspond to the beam inflection points in the frame under earthquake loading, and are supported by horizontal rollers. The stiffness properties of the two adjacent beams can be combined and represented by those of a single beam. The number of bays was considered comparable for the two buildings. The idealized frame models are connected at each floor level by connecting elements (Figure 3).



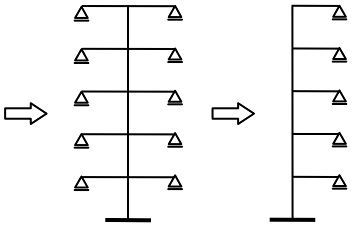


Figure 1 Structural model for independent building   Figure 2 Analytical model

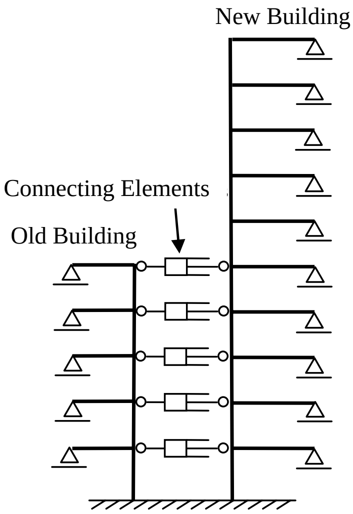


Figure 3 Analytical model for interconnected buildings

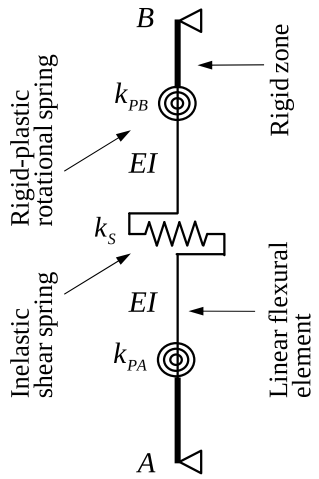
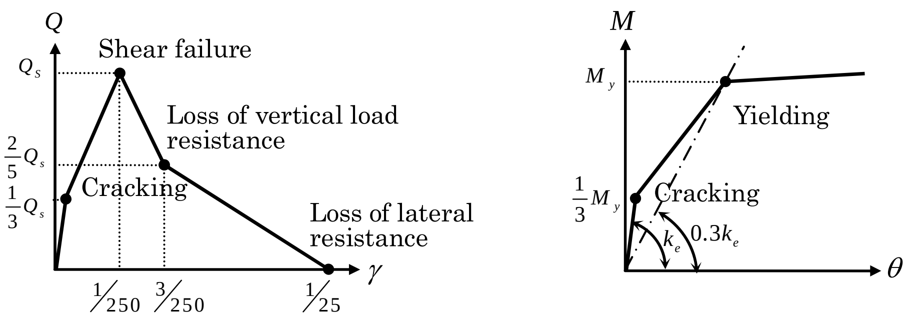


Figure 4 Member model



(a) Shear spring of column   (b) Bending spring at member end

Figure 5 Restoring force and deformation relation of members

A structural member is represented by a linear flexural element with rigid zones at the member ends (Figure 4). Inelastic flexural rotation is assumed to concentrate at the ends of the linear flexural element, represented by rigid-plastic rotational springs. Inelastic shear stiffness is represented by a lateral spring.

The shear and lateral deformation relation (skeleton curve) of a shear spring under monotonically increasing load is shown in Figure 5(a). The four points represent the cracking, shear failure, collapse (loss of gravity load carrying capacity) and loss of lateral load resistance, respectively. The shear deformation angle was 1/250 at shear failure, 3/250 at collapse and 1/25 at loss of lateral load resistance, respectively (Yoshimura et al. 2005). A bending moment-rotation skeleton curve of a bending spring is shown in Figure 5(b). The first and second points represent flexural cracking and yielding, respectively. Before shear failure or flexural yielding, the response point follows hysteresis rules given by the peak-oriented hysteresis model. Once the response point passed the shear failure point or the flexural yielding point, the response point follows hysteresis rules given by the Pivot hysteresis model (Dowell et al. 1998).

3. GROUND MOTIONS

Three ground motions which satisfied a target design response spectrum (Figure 6) of the present Japanese building codes were simulated. The phase angle characteristics of the simulated ground motions were obtained from the following observed records;

(a) El Centro (NS) record of the 1940 Imperial Valley Earthquake,
(b) Hachinohe (EW) record of the 1968 Tokachi-oki Earthquake, and
(c) JMA Kobe (NS) record of the 1995 Hyogo-ken Nanbu Earthquake.

The phase angle characteristics control the distribution of acceleration amplitudes along the duration of an earthquake motion. The phase angle characteristics of a near field motion such as the JMA Kobe record cause large acceleration amplitudes in a short duration.

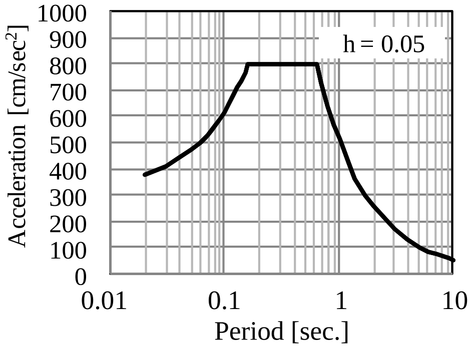


Figure 6 Target design response spectrum

4. PROPERTIES OF OLD BUILDING

4.1. Properties of Building

The old building was assumed to have 5 stories with uniform span of 6.0 m. Floor weight of 353 kN was concentrated at each beam-column joint. Floor weights, story heights and dimensions of column and beam sections are listed in Table 1. The first-mode elastic period was 0.62 sec. The damping coefficient varied proportional to instantaneous (tangent) stiffness; the first-mode damping factor was 0.05 at the initial elastic stage. The nominal concrete strength was 18 N/mm^2.

The old building was assumed to fail by shear failure of columns. Therefore, the shear strength of each column was determined equal to shear force calculated by linear frame analysis under the design seismic force in the

1970s; i.e. the design lateral force at a floor level was equal to 0.2 times the floor weight. In order to force the shear failure of columns, the yield moment of rotational springs in columns and beams was made equal to 1.3 times calculated bending moment.

Table 1 Floor weights, story heights and dimensions of column and beam sections (Old building)

Story	Floor weight [kN]	Story height [mm]	Column [mm]	Beam [mm]
5	353	3500	600 × 600	350 × 700
4	353	3500	600 × 600	350 × 700
3	353	3500	650 × 650	400 × 750
2	353	3500	650 × 650	400 × 750
1	353	4000	650 × 650	400 × 750

4.2. Response of Independent Buildings

When the independent old building was excited under the three ground motions, the damage of the building concentrated in a specific story and the building collapsed under all three ground excitations. After reaching the collapse point of the building, the response calculation was continued for the reference. Figure 7 shows maximum shear deformation angle. No flexural yielding took place at the member ends.

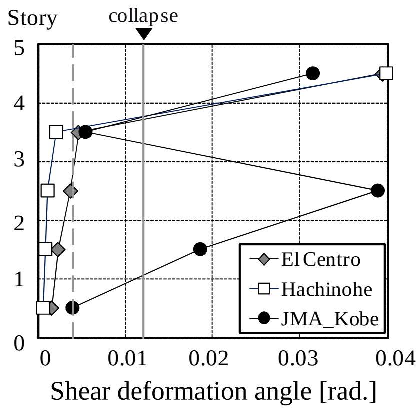


Figure 7 Maximum response of shear springs in the old building (Independent system)

5. PROPERTIES OF NEW BUILDINGS

5.1. Properties of Buildings

The new building was assumed to have 10 stories with uniform span of 8.0 m. Floor weight at each floor level was 691 kN. Floor weights and story heights and dimensions of column and beam sections are listed in Table 2. The damping coefficient varied proportional to instantaneous (tangent) stiffness; the first-mode damping factor was 0.05 at the initial elastic stage. The nominal strength of concrete was 28 N/mm^2.

Flexible and stiff buildings were considered herein;

(a) A flexible and ductile weak-beam strong-column building having elastic fundamental period of 0.90 sec (hereafter called a “new flexible building”), and

(b) A stiff and less ductile building having elastic fundamental period of 0.61 sec (hereafter called a “new stiff building”).

The stiffness of the new stiff building is increased by the use of large beam and column sections rather than providing structural walls or bracing members. It must have been more desirable to use structural walls rather than to use large frame members to control the response deformation of old building.

Table 2 Floor weight, story height and dimensions of column and beam sections (New buildings)

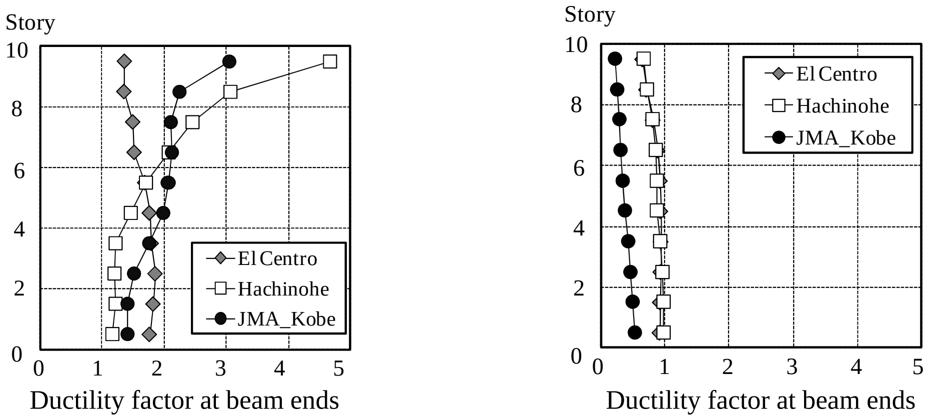
Story	New flexible & stiff buildings		New flexible building		New stiff building	
	Floor weight [kN]	Story height [mm]	Column [mm]	Beam [mm]	Column [mm]	Beam [mm]
10	691	3500	800 × 800	500 × 900	1000 × 1000	650 × 1050
9	691	3500	800 × 800	500 × 900	1000 × 1000	650 × 1050
8	691	3500	850 × 850	500 × 900	1050 × 1050	650 × 1050
7	691	3500	850 × 850	500 × 900	1050 × 1050	650 × 1050
6	691	3500	900 × 900	500 × 900	1100 × 1100	650 × 1050
5	691	3500	900 × 900	550 × 950	1100 × 1100	700 × 1100
4	691	3500	950 × 950	550 × 950	1150 × 1150	700 × 1100
3	691	3500	950 × 950	550 × 950	1150 × 1150	700 × 1100
2	691	3500	1000 × 1000	550 × 950	1200 × 1200	700 × 1100
1	691	4000	1000 × 1000	550 × 950	1200 × 1200	700 × 1100

The new building was designed to develop the weak-beam strong-column performance and the ultimate lateral load resisting capacity (story shear) Q_u at the formation of collapse mechanism was made equal to the required capacity Q_{un} prescribed in the present Japanese building codes. For this purpose, the linear frame analysis, considering stiffness degrading at beam ends, was carried out under the lateral load which gives story shear force equal to the required ultimate lateral load resisting capacity Q_{un}. In order to form the collapse mechanism as planned in the new building, yield moment at member ends, where plastic hinges were planned to be formed, was determined equal to bending moment calculated by the linear frame analysis, whereas other yield moment and shear strength of members were made equal to 1.35 times calculated shear forces and bending moments.

Hence, the required ultimate lateral load resisting capacity Q_{un} was determined for the story shear coefficient Q_{ud} required for the building if it were to remain at the elastic stage and for the structural characteristic factor D_s reducing the story shear force at the elastic stage according to damping of the building and ductility at the plastic stage. The story shear coefficient Q_{ud} was determined for the base shear coefficient corresponding to the design response spectrum S_A (Figure 6) and for the story-shear coefficient distribution factor A_i. The structural characteristic factor D_s was assumed to be 0.3 for the new flexible building and 0.55 for the new stiff building.

5.2. Response of Independent Buildings

Flexural yielding took place at all beam ends of the independent new flexible building as planned in the weak-beam strong-column design under the three ground motions (Figure 8(a)), but the ductility factors at beam ends were generally less than 3.0 for the El Centro and JMA phase earthquake motions; the Hachinohe phase earthquake motion caused the ductility factor greater than 4.0 at the end of the roof level beam.



(a) New flexible building (b) New stiff building

Figure 8 Ductility factor of beams in the new building (Independent system)

The beam ends of the independent new stiff building did not yield in flexure under the three ground motions because the building was provided with high resistance (Figure 8(b)).

6. PROPERTIES OF CONNECTING ELEMENTS

The old and new buildings were connected at each floor level by two types of truss elements with mechanical hinges at both ends;

(a) Rigid elements, and
(b) Viscous damper elements.

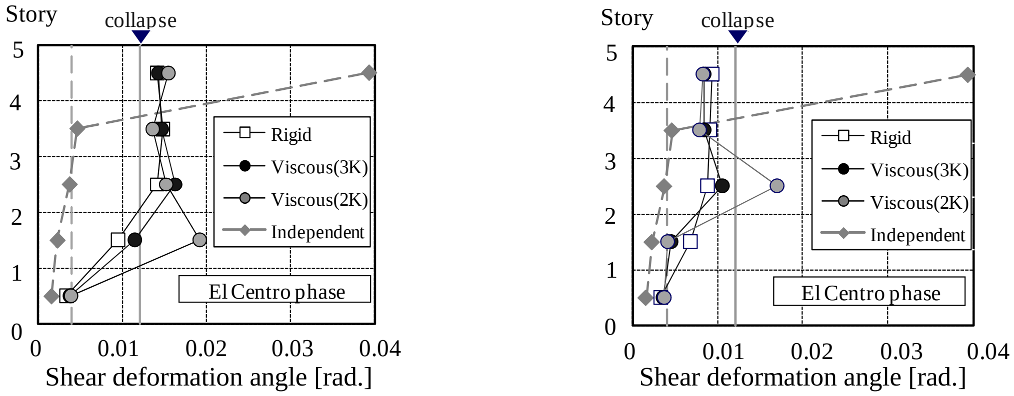
The damping coefficient of a viscous damper element was selected to be 2000 or 3000 kN/sec/m (hereafter designated “2K” and “3K”).

7. RESPONSE OF CONNECTED BUILDINGS

In the following, the response to the El Centro phase ground motion is mainly introduced; the duration of large acceleration amplitudes was medium in the three ground motions.

7.1. Effect of Adjacent Building

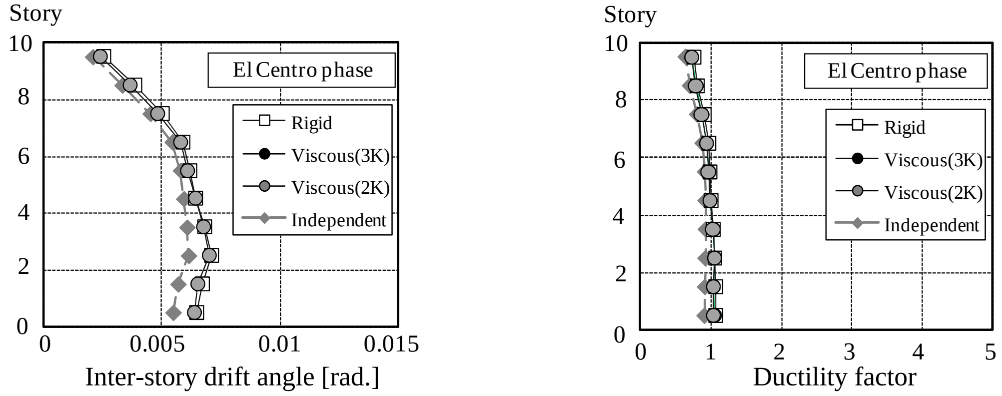
The collapse of the old building cannot be prevented by connecting to the new flexible building, using rigid or viscous damper elements (Figure 9(a)). When the old building was connected to the new stiff building, the response was reduced and the collapse was prevented except for the case using viscous damper elements 2K in the third story (Figure 9(b)). It is essential to increase the stiffness and the strength of the connected buildings and restrain the deformation in the old building to protect the old building.



(a) Connecting to new flexible building (b) Connecting to new stiff building

Figure 9 Maximum response in shear springs in the old building
(Connected system, El Centro phase ground motion)

The maximum inter-story drift angle of the new stiff building slightly increased by connecting (1.2-1.4 times the response of the independent case) using rigid or viscous damper elements, but was less than 0.7 % (Figure 10(a)). Most beam ends did not yield and significant damage did not develop (Figure 10(b)).



(a) Maximum inter-story drift angle (b) Ductility factor at beam ends

Figure 10 Maximum response of new stiff building (Connected system, El Centro phase ground motion)

7.2. Effect of Connecting Elements

The damage of the independent old building concentrated in a specific story and the old building collapsed, as mentioned above (Figure 7). By connecting to the new stiff building using rigid elements, the damage concentration was not seen, and the collapse could be prevented (Figure 9(b)). However, if viscous damper elements were used, the damage of the old building concentrated in the third story and the collapse could not be prevented when using viscous damper elements 2K (Figure 9(b)). Therefore, it is effective to restraint the deformation in the old building by the new stiff building using rigid elements to control response of the old building.

The energy dissipation by hysteresis damping at the beam ends of the new flexible and by viscous damper elements was studied (Figure 11). The dissipated energy by the viscous damper elements was small compared with the dissipated energy by hysteresis damping at the beam ends; the response of an old building could not be reduced through the energy dissipation by the viscous damper elements. This is the reason why the response control effect of the viscous damper elements did not become larger compared with the rigid elements.

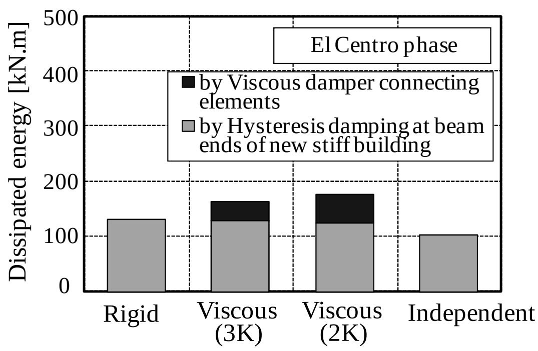


Figure 11 Energy dissipation by beams of new stiff building and viscous damper elements
(Connected system, El Centro phase ground motion)

7.3. Effect of Input Ground Motions

Nonlinear response of the inter-connected buildings is considerably different under ground motions having different phase angle contents, although the ground motions satisfied the same target design response spectrum (Ohami et al. 2005). Figure 12 shows maximum response in shear springs in the old building connected to the new stiff building under the three ground motions.

Under the JMA Kobe phase ground motion with large acceleration amplitudes over a short duration in the three ground motions, no shear failure took place in all stories and then the effectiveness of the response control was high, using rigid or viscous damper elements. Under the El Centro phase ground motion with large acceleration amplitudes over a medium duration, the effectiveness of the response control was reduced. The effectiveness becomes lower under the Hachinohe phase ground motion with large amplitude acceleration over a long duration. Therefore, the effectiveness of strengthening by inter-connecting buildings is low under ground motions with large acceleration amplitudes over a long duration.

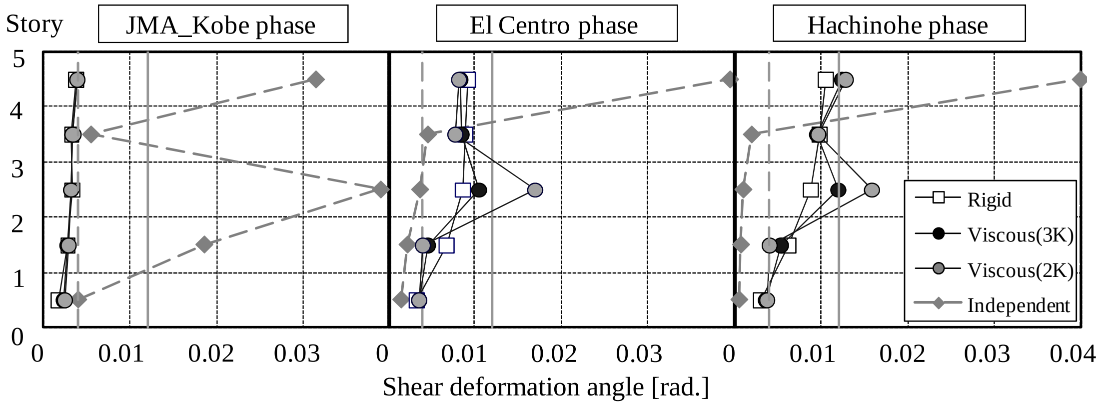


Figure 12 Maximum response in shear springs in the old building connected to the new stiff building

8. CONCLUSIONS

Retrofit feasibility was studied by externally inter-connecting a seismically vulnerable old building designed and constructed according to out-dated building codes to an adjacent new building constructed according to the present building codes at floor levels with connecting elements. The reliability was examined by nonlinear earthquake response analysis. Followings may be pointed out.

The collapse of an old building can be prevented by connecting to a new stiff building using rigid elements, although the deformation of the new building slightly increased and significant damage developed in the old building. However, even if viscous damper connecting elements are used, the response of an old building could not be reduced through the energy dissipation; damage of the old building concentrated in a specific story. The effectiveness of strengthening by inter-connecting buildings was considerably different under ground motions having different phase angle contents although their acceleration response spectra were comparable. The effectiveness is low under ground motions when large amplitude acceleration last over a long duration.

REFERENCES

Dowell, R.K., Seible, F. & Wilson, E.L. (1998). Pivot Hysteresis Model for Reinforced Concrete Members, *ACI Structural Journal*, Title No. 95-S55, **95-5**, 607-617.

Fujii, S., et al. (2004). Regeneration of Town by Interconnecting Old Pencil Buildings (in Japanese), Report of Research Committee on Regeneration of Town by Interconnecting Existing Buildings, Architectural Institute of Japan.

Ohami, K., Otani, S. & Omiya, M. (2005). Influence of Earthquake Phase-angle Characteristics on Nonlinear Response (in Japanese), *Proc. of the 4th Annual Meeting of Japan Association for Earthquake Engineering*, 476-477.

Yoshimura, M. & Takaine, Y. (2005). Formulation of Post-peak Behavior of Reinforced Concrete Columns Including Collapse Drift (in Japanese), *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, **No.587**, 163-171.