



SUBSTRUCTURE PSEUDO DYNAMIC TEST ON RC BUILDING WITH SOFT STORY CONTROLLED BY HPFRCC DEVICE

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SUMMARY

Collapse or severe damage has been observed in many RC residential buildings with soft story at the previous earthquake disasters. Hence development of an effective structural control technology is one of the urgent tasks for residential apartments in the urban area. However most of the conventional structural control methods for soft story buildings stuff a space with structural elements such as RC walls or response control dampers. This paper proposes a new technique for structural control of RC buildings with soft story by using ductile short columns as response control devices placed beside the existing columns at the soft story. This device is made by High Performance Fiber Reinforced Cementitious Composite (HPFRCC), which exhibits multiple cracking and strain-hardening characteristics in the uni-axial tensile stress. It has been found that the HPFRCC device shows a very good ductile structural manner in conjunction with a high resistance capacity against the axial force. Since these characteristics give a high potential in the improvement of the structural performance of buildings, it can meet the multi-purpose performance requirement from the society. In this paper the authors was conducted a substructure pseudo-dynamic test in order to investigate the feasibility and advantages of the structural control by HPFRCC devices, and to confirm effectiveness of the seismic response analyses. As the result of the experiment, the seismic response of the RC buildings with soft story was successfully controlled as expected by using HPFRCC device, and the reliability of the analytical tool has also been clarified.

INTRODUCTION

When the 1995 Kobe Earthquake hit the southern part of Hyogo Prefecture in Japan, the first floor of multi-story reinforced concrete (RC) residential buildings with independent columns on the first floor and shear bearing walls on the second floor or above were heavily damaged. The first floor of these types of buildings, commonly referred to as 'soft story buildings', has much lower yield strength and stiffness than the second and higher floors. When these buildings are subjected to lateral loads such as large-scale earthquake, most of the input energies are dissipated through plastic drifts on the first floor. Therefore, it is postulated that by placing an element to control the response displacement (hereinafter referred to as

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‘seismic response control element’) on the first floor, the concentration of deformation on the first floor at the time of large-scale earthquake can be reduced, thus preventing the collapse of the first floor and mitigating the damage to the columns.

This paper discusses a substructure pseudo dynamic test carried out on a 12-story soft story RC building with seismic response control elements placed beside the existing columns on the first floor. The test results confirmed the usefulness of this approach to seismic strengthening of soft story buildings. In addition, by comparing the experimental results with analytical results, the accuracy of the seismic response analytical method was verified.

A PROPOSED SEISMIC RESPONSE CONTROL ELEMENT FOR MITIGATING DAMAGE

Seismic strengthening of a soft story building

A widely used method for reducing response displacements of soft story buildings is to install bearing walls in the shorter span direction. Such a wall would provide a continuous multi-story bearing wall (from the first floor), and would serve to take up most of the horizontal forces. However, soft story buildings are purposely designed to have wide spaces on the first floors for parking lots and stores and the installation of bearing walls may severely restrict or affect this functionality.

In situations when placing bearing walls do not limit the intended function of the first stories of soft story buildings, the addition of bearing walls represent a structurally reliable and desirable seismic response control method. However, if securing wide spaces on the first stories of soft story buildings is important, it may not be possible to install bearing walls or for that matter braces and structural control devices. In such situations, the response displacement can be controlled by strengthening the columns on the first floors to the required strength and ductility through the following methods: (1) adding shear reinforcing bars to RC columns, (2) lining steel tubes around RC columns, and (3) using SRC columns. Another effective method is to use high-strength concrete to resist high axial forces. The above strengthening schemes are options that are available for new constructions but for existing soft story buildings there are a number of difficulties that limit their wide applicability. For instance, it is difficult to incorporate structural steel into existing RC columns. One method that is feasible is to wind RC columns by additional reinforced concrete, steel tubes or continuous fiber sheet but these winding methods are problematic, as they require large-scale construction work to anchor the end of the longitudinal reinforcing bars that are added. The required strength and ductility on a soft story can be attained using these winding methods but the shear span ratio of the strengthened column becomes relatively small. In addition, a large fluctuating axial force still acts on the column, causing horizontal displacements larger than on the upper stories. Therefore, it is inevitable that the first story columns will be damaged.

A proposed response control method for mitigating damage

In order to address this issue, it is imperative to use the latest structural technology to control damage to existing structural members, including columns, by minimizing both the response displacement and axial force. In this paper a response control method is proposed to achieve this goal easily by installing short-span columns with ductility (hereinafter referred to as ‘additional columns’) next to the existing columns as shown in Figure 1. The stiffness and strength of such additional columns can be changed simply by changing their shapes, bar arrangement, and materials used. Moreover, as cement materials used for these columns can be formed in any desired shape, it is possible to obtain response control elements with property and configuration suitable for most structures. As shown in Figure 1, by using short-span columns with stabs, it is not only possible to obtain large stiffness and strength from a relatively small element but also bear large axial forces, which are major characteristics of this element. In addition, as

bearing walls support the upper floors, the additional columns can function effectively without requiring any special reinforcement on beams to prevent yielding at early stages as in the case of frame structures.

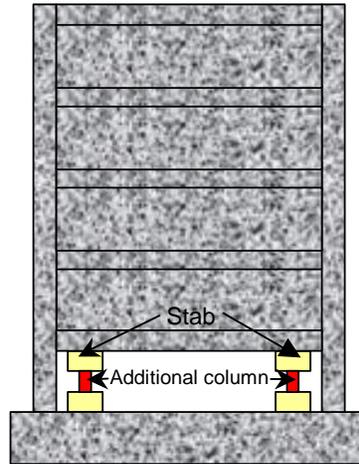
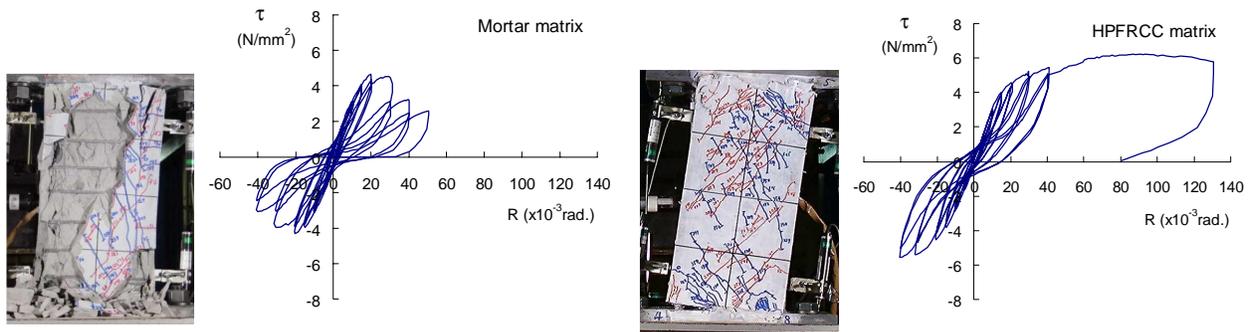


Figure 1. Conceptual drawing of additional columns for structural control of soft story building

Construction of an additional column

The additional columns can be constructed in many different ways, using steel encased reinforced concrete (SRC) or reinforced concrete filled tube. In the tests concrete is substituted by a high performance fiber reinforced cementitious composite (HPFRCC) developed recently, having a strain hardening property under tensile stress. The structural performance of seismic response control elements using HPFRCC has been proven by static loading tests conducted by Fukuyama [1]. As seen Figure 2, the results of their tests clearly shows that sufficient deformation capacity was not obtained when conventional mortar and concrete were used because the seismic response control elements were largely damaged and finally collapsed due to large shear and compressive forces. However, when HPFRCC was used, the deformation capacity of the seismic response control elements was confirmed for displacements as large as 13% rad. under the very large shear force in average of 5 to 6N/mm², and the degree of damage was reduced.



a) In case of mortar matrix
(Severe damage and small deformation capacity are shown.)

b) In case of HPFRCC matrix
(Large deformation capacity without failure until deformation of over 13% rad. can be seen.)

**Figure2. Result of the static loading tests of response control elements by Fukuyama [1]
(Damage properties and average shear stress (τ) – deflection angle (R) relationships)**

SUBSTRUCTURE PSEUDO DYNAMIC TEST OF A SOFT STORY RC BUILDING WITE ADDITIONAL COLUMNS

Test structure

Soft story structure

Figure 3 shows a floor plan and elevation of the soft story building used in the test. The building is a 12-story RC building that was designed using seismic standards stipulated before the 1995 Kobe Earthquake, when story yielding on the first floor was allowed. The test structure is the lower two stories with a single plane of the structure in the direction indicated in the figure. Figure 4 shows the shape of the test structure without additional columns, and the bar arrangements. Dimensions and material properties of the test structure are shown in Tables 1 and 2, respectively.

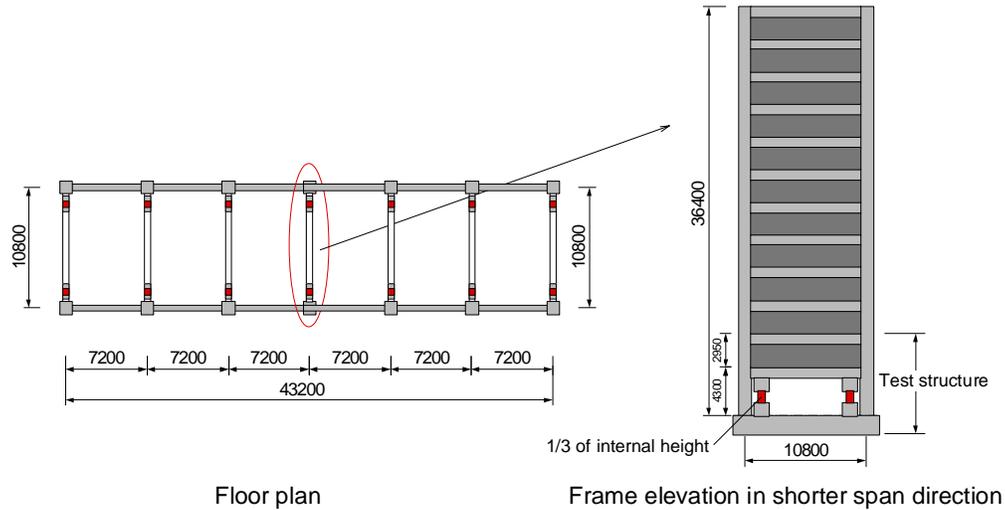


Figure 3. Outline of soft story building (unit: mm)

Table 1. Dimensions of test structure (Scale=1/2.5)

Floor	Column		Wall		Beam	Concrete	Reinforcing Bar
	Section	Bar arrangement	Thickness	Bar arrangement	Section	$F_c(N/mm^2)$	Yield stress (N/mm^2)
6-12F	400×400	20-D13 (4-D6@40)	100	D10@120 Double	260×120	240	345
3-5F						270	
2F					400×120	300	
1F							

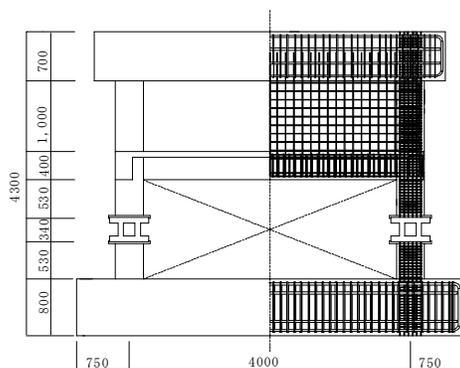


Figure 4. Shape and bar arrangement of test structure (Scale: 1/2.5 unit: mm)

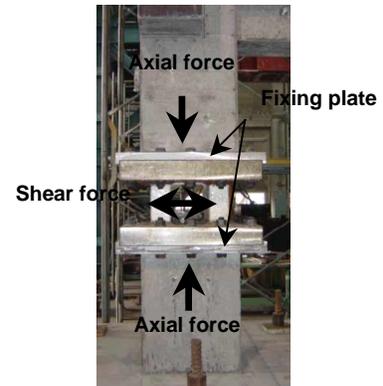


Photo1. Load cell for measuring stress on column

Table 2. Material properties of test structure

Concrete		
Compressive strength (N/mm ²)	Strain under compressive strength (%)	Initial stiffness (N/mm ²)
34.4	0.20	2.50×10 ⁴
Reinforcing bar		
Type of material (Application)		Yield strength (N/mm ²)
D6	(Shear reinforcing bar)	378
D10	(Wall reinforcing bar)	379
D13	(Longitudinal reinforcing bar)	392



a) Before installation of additional columns



b) After installation of additional columns

Photo 2. Shape of test structure

The test structure is a 1/2.5 scale model of the actual building, having stabs on the upper and lower ends. The first floor consists of columns, while the second floor consists of bearing walls with columns, similar to the test structure used in the tests by Haruta [2]. The span between columns is 4000 mm, and the internal height of the column on the first floor is 1400 mm. Columns on the first and second floors have cross sections of 400 x 400 mm. A load cell (see Photo 1) is installed at the center of the column on the first floor for measuring axial force and shear force. A plate is welded to the longitudinal reinforcement of the column using a fixing plate, and the load cell is bolted to the plate. The additional columns are installed near the columns on the first floor in order to mitigate the concentration of deformation on the soft story and thereby reduce damage to the soft story (see Photo 2). The additional columns are installed as close as possible to the existing columns and to achieve this the spacing between the center of the existing column and that of additional column is set to 600 mm.

Additional columns

Bar arrangement and dimension of the additional column are shown in Figure 5 and Table 3, respectively. The additional column used in the test has a cross section of 200 x 200 mm and the height of the additional column is 450 mm, which is one-third the internal height of the first story. The addition column's bending strength set to one-fourth that of the existing column by adjusting the quantity of the cross section of longitudinal reinforcement. The ratio of shear strength between the additional column and the existing column (shear strength of the additional column at the time of yielding (Q_{yd}) / shear strength of the existing column at the time of yielding (Q_{yc})) is about 0.75. The ratio of shear strength between the first story and the second story (C1/C2) increases from 0.66 to 0.85 after the installation of additional columns.

Table 3. Dimension of additional column

Section		200x200mm	
Bar arrangement	Longitudinal reinforcement	8-D10 (SD345)	$P_g = 1.42\%$
	Shear reinforcement	2-D6@40 (SD345)	$P_w = 0.80\%$
Material		PS-HPFRCC ($F_c = 50\text{N/mm}^2$)	

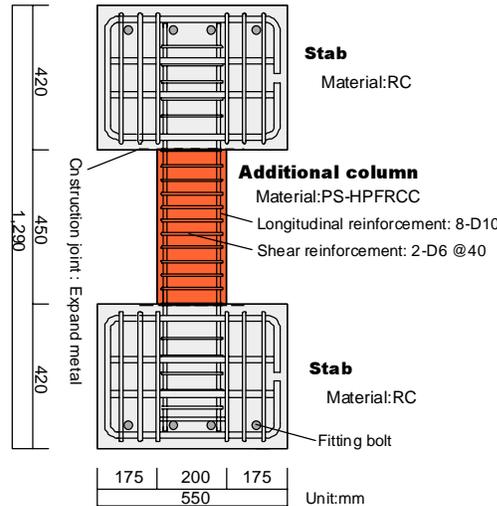


Figure 5. Bar arrangement of additional column

HPFRCC used in the test is composed of mortar matrix (water-cement ratio 45 % and sand-cement ratio 40 %) mixed with polyethylene fibers and steel cords (specific volume of 1 % for both) and is referred to as 'PS-HPFRCC'. The polyethylene fibers are 15 mm long with a diameter of $12 \times 10^{-6}\text{mm}$, and the steel cords are 32 mm long stranded fibers with a diameter of $405 \times 10^{-6}\text{mm}$ ($\phi 130 \times 10^{-6}\text{mm} \times 5$). Figure 6 shows the stress-strain relation of PS-HPFRCC. The stress-strain behavior of PS-HPFRCC does not decrease until strain increases to about 1 to 2 % even after formation of the first crack under a tensile stress. Moreover, after reaching the maximum compressive stress, the strength of PS-HPFRCC does not decrease rapidly, showing a material property similar to confined concrete.

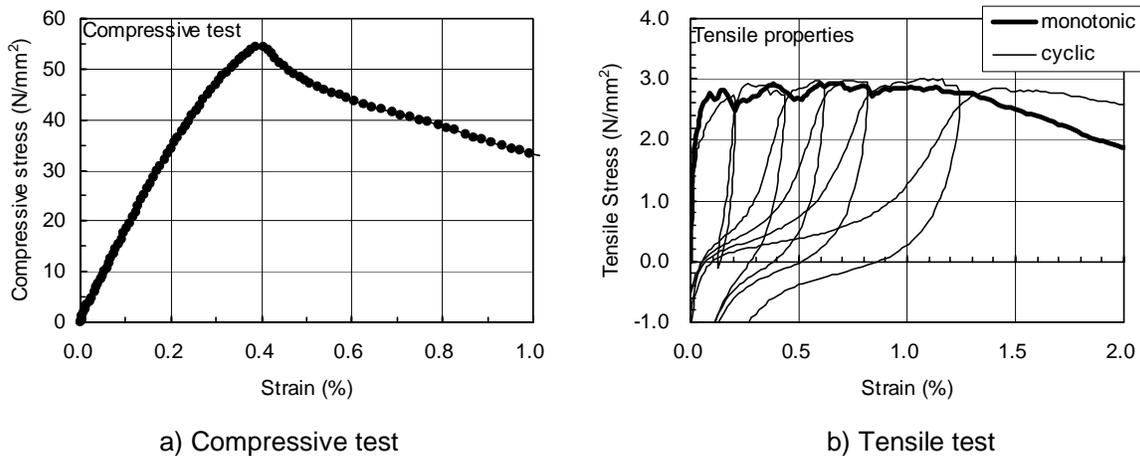


Figure 6. Stress - strain relationships of PS-HPFRCC

Installation of Additional columns

Before placing the additional columns, post-cast anchor bolts were installed on the slab under the test structure, and the additional columns were then bolted to the frame using steel fitting equipment (see Photo 3). The fitting equipment has loose holes in the vertical direction for ease of construction. The additional columns installed in this test are considered not to resist tensile force because the shear resistance of the additional columns under tensile axial force is small as they do not bear much tensile stress.



Photo 3. Shape of fitting equipment

Test method

The substructure pseudo dynamic test method described in Tsutsumi [3] is used in this study. Using this method, the seismic behavior of the upper ten stories are calculated at each loading step, while simultaneously applying force partially to the lower two stories. This way the seismic response of the 12-story building can be reproduced pseudo-dynamically.

In order to appropriately model the upper ten stories, a detailed preliminary analysis was conducted using a plane frame model of the 12-story frame. The analysis confirmed that the bearing walls on the upper ten stories did not show any bending and/or shear cracks. Therefore, in the response calculation for the upper ten stories in the pseudo dynamic test, the wall resisting the lateral force is modeled as a column with equivalent elasticity stiffness. The mass of the upper ten stories is concentrated at the floor of each story, while that of the test structure is concentrated at the top of the test structure. As for degrees of freedom, horizontal and vertical degrees of freedom are assigned to the members and rotational freedom is permitted in the connections of columns and beams. Damping is modeled as an instantaneous stiffness proportional damping and the damping coefficient is set to 3 % at the natural period of the elastic model. In the response analysis, the operator-splitting (OS) method is used for integration, with an integrant pitching of 0.01sec.

Seismic waves used in the test are the El-Centro NS waves normalized to 10 cm/sec, 25 cm/sec, 50 cm/sec, and 75 cm/sec. The four seismic waves (referred to as 'Elc_10, Elc_25, Elc_50, and Elc_75') are input in increasing order of the maximum acceleration, starting from the smallest one. The input period for each wave is set to seven seconds; however, the test is ended if the test structure appeared to have experienced its maximum response displacement.

Photo 4 shows how the force is applied to the test structure. A horizontal force (seismic force) is applied using two hydraulic actuators with maximum capacities of 1000 kN to the center of the top of the test

structure. After an initial axial force of 892 kN is applied on each column from the top of the test structure using two hydraulic actuators with maximum capacities of 3,000 kN, the effect of overturning moment on the upper stories of the building is reproduced. More specifically, the seismic response of the frame (from 3 to 12 stories) was calculated at each loading step. Then, the reaction force obtained from the calculation (shear force that satisfies the value of the target horizontal displacement and fluctuating axial force on the third story floor) is applied to the test structure.

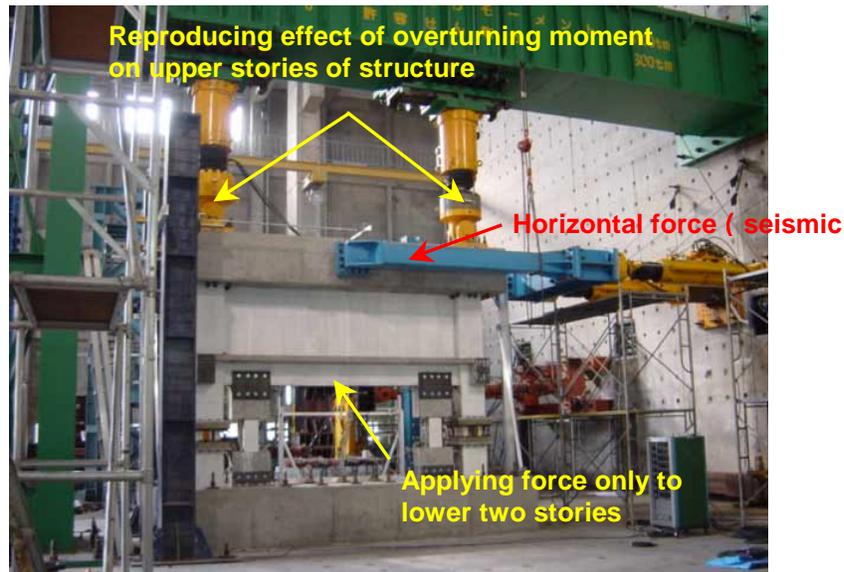


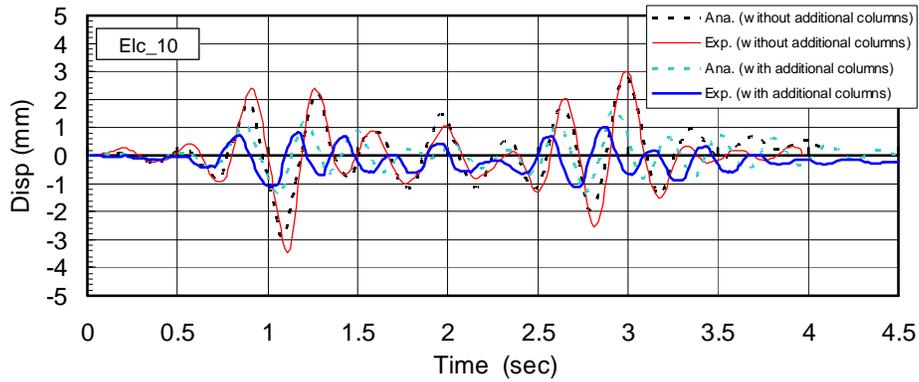
Photo 4. Outline of force application

Test results

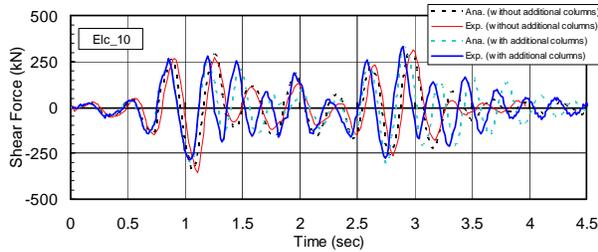
Figures 7 to 9 show the experimental (solid lines) and analytical (dotted lines) time histories of horizontal displacement and story shear force at the floor level on the third story, and the axial force on the northern column on the first story, for the four seismic input waves. These figures, also, show the analytical and experimental values reported in Haruta [2] for the case without additional columns.

Experimental values for the input wave Elc_50 are obtained only through halfway point as the test had to be suspended when the welded part between a fixing plate for the load cell and the longitudinal reinforcement of the column was destroyed just prior to the maximum response displacement. Though the test was resumed after repairing the broken part, this paper discusses the results obtained before the fracture of welded part as the fracture exerted too large an effect on the northern column.

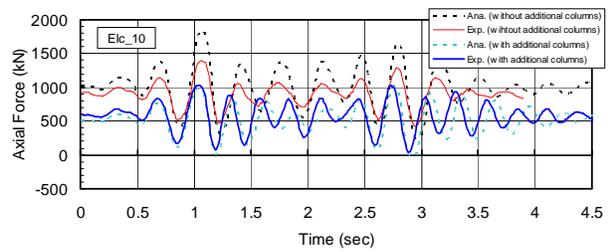
A comparison of the experimental values with and without additional columns reveals that response displacement decreased by about 40 % as a result of installing the additional columns. The maximum response displacement with additional columns is $R=1/250\text{rad}$. within the elastic range for input wave Elc_25. Though some plastic deformation on the existing column is observed during the input wave Elc_50, the maximum response displacement is restrained to $R=1/125\text{rad}$. within the elastic range as well. These test results confirm that the use of additional columns in soft story buildings leads to reduced damages.



a) Displacement at the third story floor level

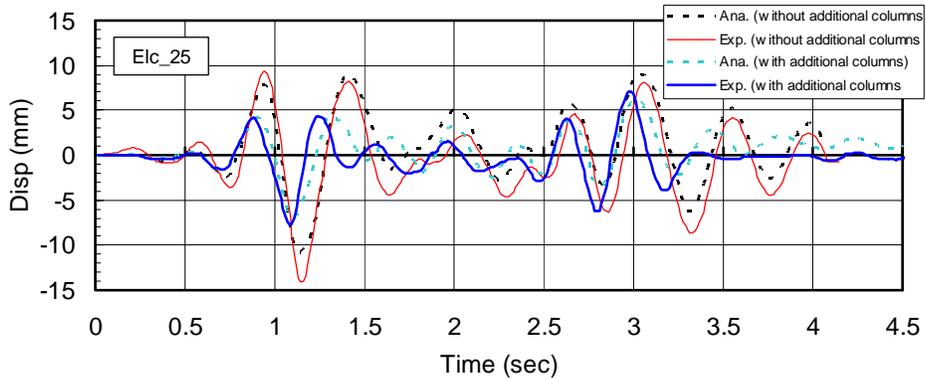


b) Shear force at the third story floor level

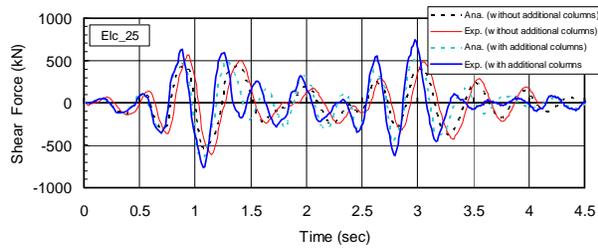


c) Axial force on the northern column on the first floor

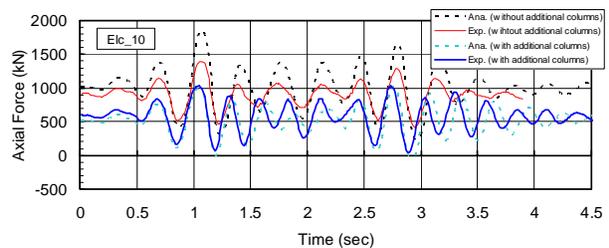
Figure 7. Time history of response for seismic input wave Elc_10



a) Displacement at the third story floor level



b) Shear force at the third story floor level



c) Axial force on the northern column on the first floor

Figure 8. Time history of response for seismic input wave Elc_25

The axial force of the existing column decreases in 1.0sec which reaches the maximum response value of Elc_25 from 2500kN to 1500kN, and the additional column bears the decrease minute. The additional column would bear about 40% of all axial force approximately, and axial force ratio (axial stress / compressive strength of cementitious materials) of this time additional column is being achieved in about 0.3. The fluctuating axial force of the existing columns can be controlled by installing the additional columns which can bear axial force. In this case, the axial force bearing ratio is almost proportional to the ratio of areas of the existing and additional columns.

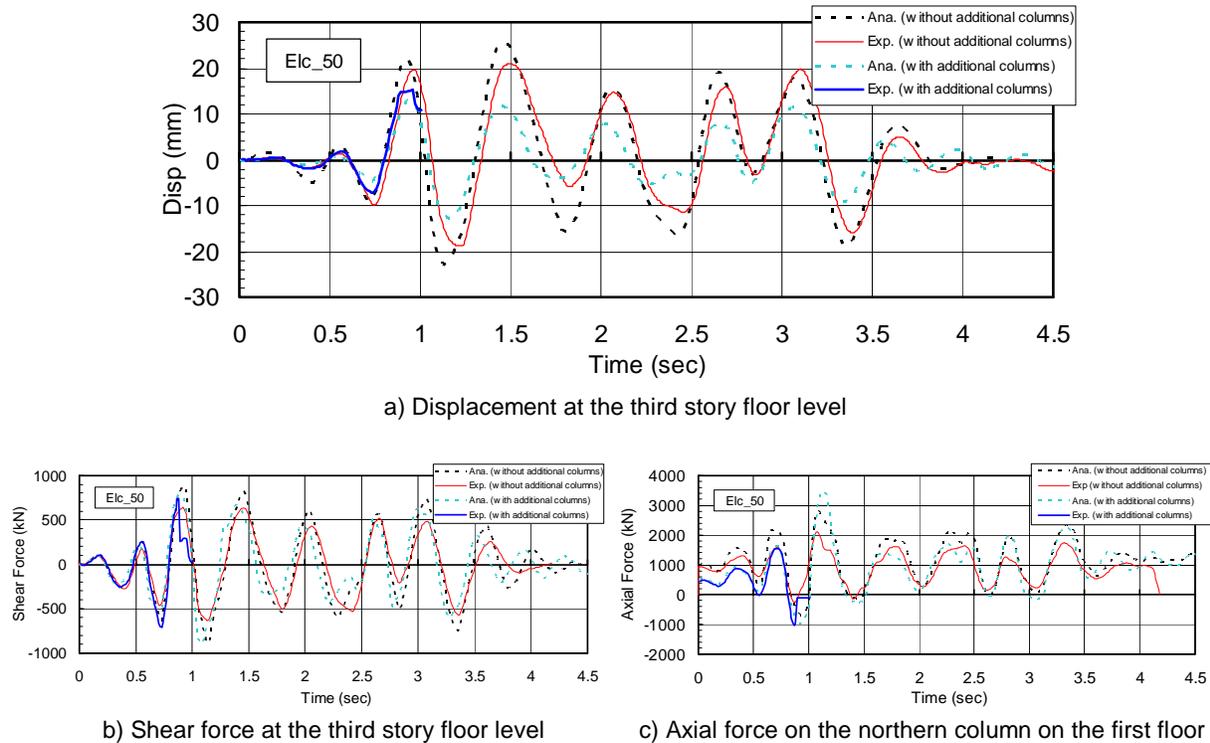


Figure 9. Time history of response for seismic input wave Elc_50

Photo 5 shows damage to the additional columns after the completion of seismic input wave Elc_25. The existing columns experience fluctuating tensile axial forces due to overturning moment of the structure, and tensile cracks occurred. However, the width of cracks is small, and the cracks closed after the completion of input waves. The existing column is hardly damaged, and there are no indications of plastic deformation and likewise there are no noticeable bending and shear cracks (though there are tensile cracks). The additional columns installed in the test do not have the capacity to resist tensile force; however, from the perspective of damage reduction, it is desirable to prevent these tensile cracks as well. If the additional columns can be designed such that they have the capacity to resist tensile forces, then it may be possible to control even more effectively the damage to the columns. However, as the bearing capacity of the additional columns under tensile stress is small, the matter to be resolved in the future includes the details of installation of the additional columns as well.

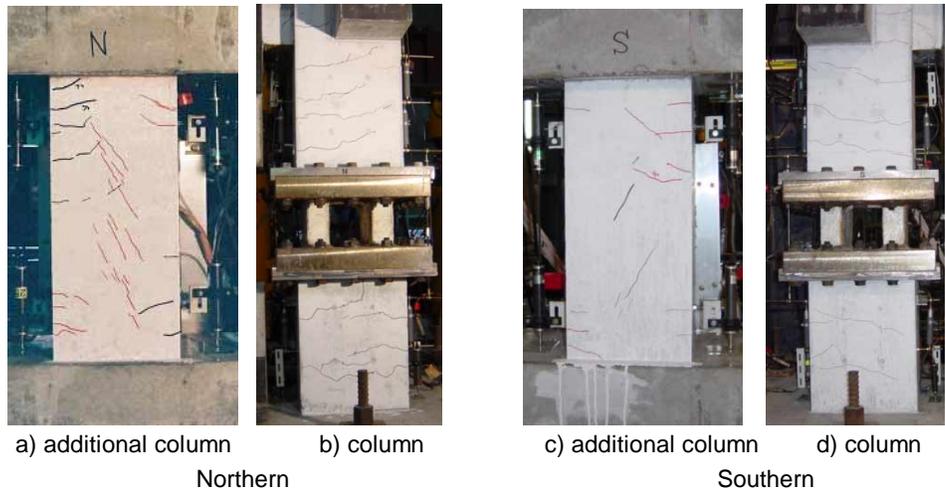


Photo 5. Destruction of columns after completion of seismic input wave Elc_25

Validity of seismic response analytical method

By comparing the experimental results with the results of preliminary analysis carried out prior to the test, the validity of the seismic response analytical method using the analytical assumptions and modeling of members is verified. In the preliminary analysis, members are modeled as follows: (1) columns are modeled as linear members with elasto-plastic flexural springs at the end and a vertical spring in the middle; (2) walls are modeled with three linear members, as shown in Figure 10, in the center of the wall there is an elasto-plastic spring and on both sides of the wall there are springs in the axial direction; (3) additional columns are modeled as linear members with elasto-plastic flexural springs and a vertical spring, as shown in Figure 11, while the slab part is treated as a rigid member. The restoring force models used for each member are: a TAKEDA model, Takeda [4] for a flexural spring, an axial-stiffness model, Kabeyazawa [5] for a spring in the axial direction, and an origin-oriented model for a shear spring. The additional columns are assumed to have properties similar to normal reinforced concrete columns that undergo large bending deformations. Therefore, as in the case of the RC columns, a TAKEDA model is used as the restoring force model for the spring at the end of members. A skeleton curve is obtained based on cracking strength, yield strength, and the stiffness decreasing ratio at the yield point, using a conventional equation for reinforced concrete members. The stiffness after reaching the flexural yield strength is set to be 0.001 times the initial stiffness.

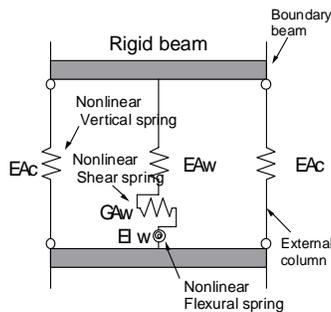


Figure 10. Modeling of wall

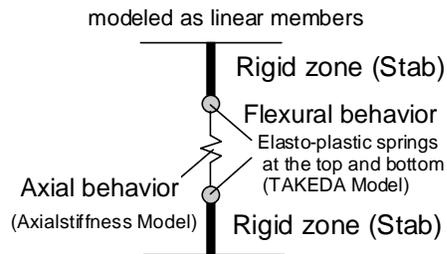


Figure 11. Modeling of additional column

Comparing the experimental and analytical values in Figures 7 to 9, for the case with additional columns, the following observations can be made:

- (i) The largest differences between the analytical and experimental values for seismic input wave Elc_25 are +0.7 % at 0.9 sec, +3.3 % at 1.2 sec, and -7.9 % at 2.7 sec, or about 10 % at the largest. This indicates that the analytical results reproduced the experimental values quite accurately.
- (ii) The largest differences between the analytical and experimental values for seismic input wave Elc_50 are -13.1 % at 0.9 sec when showing the maximum response displacement in the positive direction. In this case, too, the analytical results reproduced the experimental values well within an error of 15 %.
- (iii) The analytically calculated shear force and axial force compare favorably with the respective experimental values.

The above results confirm the validity of the seismic response analytical method as an analytic tool.

CONCLUSION

Additional columns were installed in an existing 12-story soft story RC building and tested to investigate their efficacy in reducing response displacement of soft story buildings subjected to lateral loads of the earthquake. The substructure pseudo dynamic tests carried on this building reveal the following:

- * The installation of additional columns proposed in this paper can lead to reductions in response displacement of about 40 % when compared to the response displacement of a soft story building without additional columns.
- * The fluctuating axial forces of the existing columns can be controlled by installing the additional columns which can bear axial force. In this case, the axial force bearing ratio is almost proportional to the ratio of areas of the existing and additional columns.
- * The analytical results for the case with additional columns can reproduce the experimental values quite accurately. Thus, the validity of the seismic response analytical method using a plane frame is confirmed.

ACKNOWLEDGEMENT

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