13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3406

SEISMIC STRENGTHENING USING PRECAST PRESTRESSED CONCRETE BRACES

Fumio WATANABE[1], Satoru MIYAZAKI[2], Masanori TANI[3], Susumu KONO[4]

SUMMARY

Two identical half-scale portal frames were constructed based on the pre-1980 Japanese Building Standard and strengthened with two types of proposed reinforced concrete brace. Both frames showed more than 70% increase in lateral load carrying capacity without excessive frame deformation. In an analytical program, the nonlinear and dynamic frame analyses have been conducted to evaluate the effect of end fixity and out-of-plane deformation of braces on the performance of braced frames. The failure modes of braced frames involved the tension failure of the first story column due to the cantilever action, buckling or crushing of concrete braces, axial tensile failure of beams, and the bearing failure at joints. The experimental and analytical results showed the efficiency of the proposed retrofit strategy.

INTRODUCTION

After major earthquakes such as the Northridge and Kobe earthquakes in 1990's, the seismic upgrading of existing buildings have been attracting more attention than ever. Upgrading of seismic performance of a building can be achieved by increasing the strength or ductility. However, many existing buildings have not had seismic upgrading since construction cost is high due to intensive labor work and long suspension of service. This research aims to develop a simple seismic strengthening method which satisfies a) no wet concrete work, b) no rebar or bolt anchorage, c) short construction period, and d) low construction cost. For this purpose, an X-shaped precast prestressed concrete brace system was developed and its efficiency is shown experimentally and analytically.

During the 1995 Kobe Earthquake a large number of reinforced concrete buildings suffered serious structural damage and some of them collapsed. Most of those damaged buildings were designed according to the pre-1980 Japanese Building Standard [1]. Since then the seismic upgrading of existing buildings have been major concerns to minimize the earthquake disaster in urban areas of Japan. Upgrading of

[1] Professor, Dept. of Architecture, Kyoto University, Japan. E-mail: nabe@archi.kyoto-u.ac.jp
[2] Structural Engineer, The Kansai Electric Power Co. Inc.
[3] Structural Engineer, Mitsubishi Heavy Industries, Ltd.
[4] Associate Professor, Dept. of Architecture, Kyoto University, Japan. E-mail: kono@archi.kyoto-u.ac.jp

seismic performance of existing buildings can be achieved by several construction methods such as construction of new in-filled walls, strengthening of existing walls, diagonal steel bracing, column jacketing by steel or fiber sheet. These upgrading methods give buildings higher strength and/or ductility. However, seismic upgrading makes little progress due to several reasons such as suspension of building services for long construction period, noisy and troublesome construction works, high construction cost, and etc. The purpose of this research is to develop a simple seismic strengthening method which satisfy a) no re-bar connection or bolt anchorage between the brace and existing frames, b) easy construction, c) short construction period and d) low construction cost including the out-of-service loss.

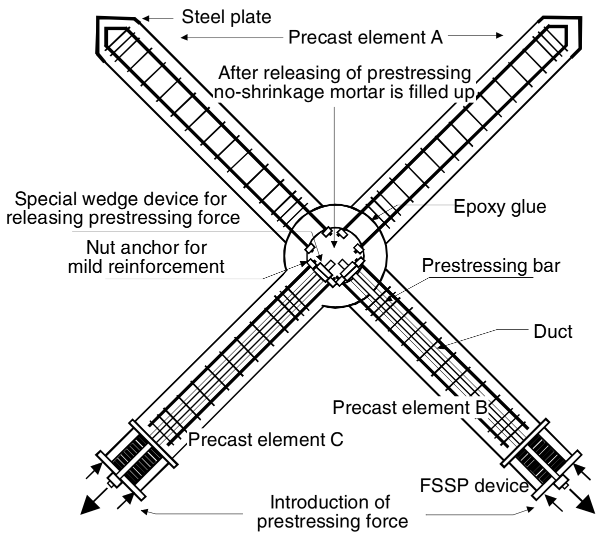


Fig. 1 Assembled precast concrete brace and its reinforcement

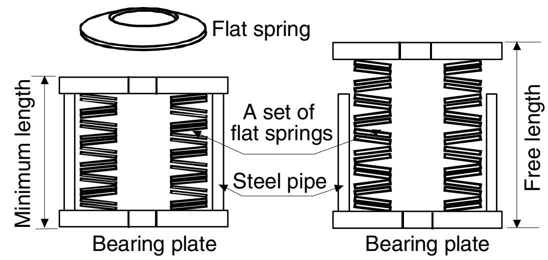


Fig. 2 Flat spring and steel pipe (FSSP) device

METHOD OF SEISMIC STRENGTHENING

An X-shape precast prestressed concrete brace consists of four precast units. They are assembled at construction site and prestressing force is introduced to two bottom legs as indicated in Fig. 1. Gaps between brace ends and frame corners are filled with high strength no-shrinkage mortar. After hardening of grout mortar, the prestressing force is released. Then the X-shape brace extends by itself and is fixed to a boundary frame.

When a frame with an X-shape brace is subjected to lateral seismic load, only one of diagonal members works effectively in compression. However, the remaining diagonal member becomes free because concrete does not carry tension force. This may results that the tensile diagonal comes off from a frame. To avoid this phenomenon, a special device with a flat spring and steel pipe (FSSP) in Fig. 2 is installed at the bottom end of each diagonal member. This device makes possible to maintain a certain amount of compressive force in the diagonal member even if it experiences elongation under tension.

Figure 3 shows the lateral force resisting mechanism and design items. Lateral seismic force is resisted by the horizontal component of compression force induced in one of the diagonal members. Another diagonal member does not resist the horizontal force but a small compression force can be maintained due to the spring reaction. Total axial response of a diagonal with FSSP device is indicated schematically in Fig. 4, where the abscissa indicates the axial shortening and the ordinate indicates the axial compressive force. When a prestressing force is released, the response point is given by Point C. This response point moves from Point C to D due to creep and shrinkage. During an earthquake a response point is to move from D to B in compression or from D to F in tension. Even in tension, a certain amount of compressive force can be maintained since FSSP device is still under compression. Therefore both diagonal members stay in compression during full period of earthquake response. This is a lateral load resisting mechanism of the X-shaped brace retrofit scheme with FSSP devices. In practice, following design considerations need to be checked as shown in Fig. 3.

A) Loss of prestressing force due to concrete creep and shrinkage should be minimized and the response point D in Fig. 4 should not go down beyond the point E.
B) Buckling of diagonals should be considered in the calculation of axial strength of a brace.
C) Concrete bearing failure at the frame corners should be avoided since the section size of a brace is generally smaller than that of beams and columns.
D) Direct shear failure at beam and column end sections should be avoided.
E) Joint shear failure should be avoided.
F) Beam tension should be calculated considering the stress pass of a whole structure and should be less than the beam axial strength.

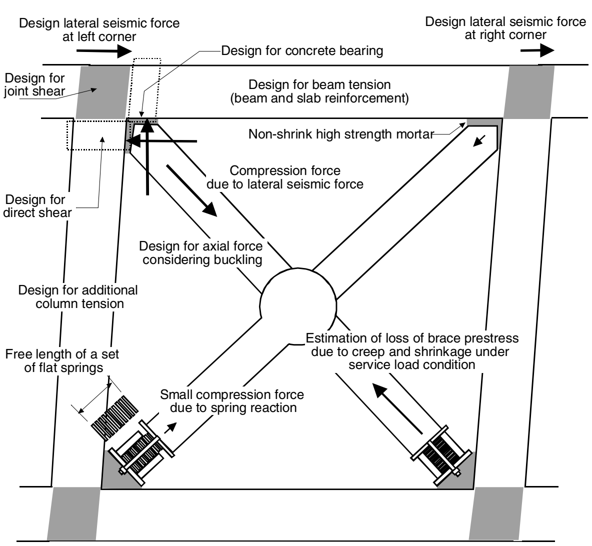


Fig. 3 Lateral force resisting mechanism and design items

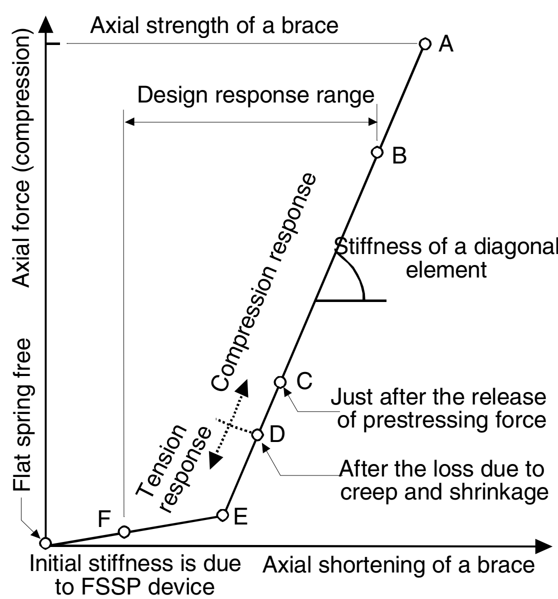


Fig. 4 Axial response of a brace

TEST ON HALF-SCALE SPECIMENS

Frames

Two half-scale specimens (No.1 and No.2) were constructed with different sectional configurations. The boundary frames are identical as a sub-assemblage of a four-story reinforced concrete building, which was designed following the pre-1980 Japanese Building Standard [1]. The column section was 300 x 300 mm and had sixteen longitudinal D10 deformed bars. The beam section was 325 x 275 mm and had four longitudinal D10 deformed bars at the top and bottom. Mechanical properties of materials are shown in Table 1. Before installing a brace system column axial force of 324 kN was applied with internal unbonded prestressing steel bar. This column axial force corresponds to the axial force induced to the first story column due to the gravity loading. Beam is also prestressed at 300 kN with unbonded prestressing steel bars to avoid the tension failure of the beam section.

Table 1 Mechanical properties of materials

(a) Concrete

Location		f'c (MPa)	Ec (GPa)	ft (MPa)
Original frame	No1	27.5	26.1	2.53
	No2	25.0	24.3	2.51
Brace	No1	24.9	23.3	2.58
	No2	64.1	33.3	4.52

(b) Reinforcing bar

Bar Type	fy (MPa)	Es (GPa)	ft (MPa)
D6	360	171	515
D8	576	215	645
D10	378	184	555

Table 2 Test variables

	Brace section (mm)	Expected failure mode
No1	100x120	Buckling of a brace
No2	120x150	Direct shear at beam or column end

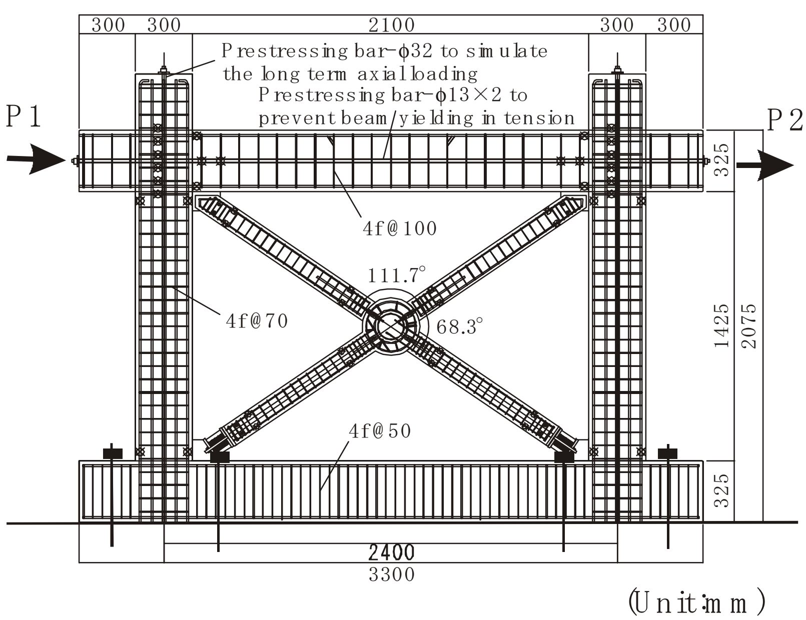


Fig. 5 Test specimen and reinforcement

Intended failure modes

Test variables for two specimens are shown in Table 2. In brace sections four D6 deformed bars were provided. In the design of specimens, ACI Code equations [2] or Navier's equation [3] were used to predict the buckling strength of a brace and ACI Code equation [4] or ACI Code Commentary equation [5] to predict the direct shear strength at beam and column end. In both specimens the bearing strength of frame concrete at the interface between brace and frame was sufficient to avoid the compression failure. Beam-column joint failure did not take place in the experiment.

FSSP device

The FSSP devices at the bottom of diagonal members consist of a steel pipe and multiple flat springs as indicated in Fig. 2. A set of flat spring had the stiffness of 2.5 kN/mm. When the axial force of 25 kN was applied to FSSP device a top bearing steel plate touched a steel pipe. The steel pipe had a diameter of 104 mm and a thickness of 20mm. This steel pipe section was designed to have enough strength for the axial compression even if a diagonal member of No. 2 specimen fails in axial compression.

Loading and measuring methods

Loading system is shown in Fig.6. Same amount of horizontal load (P1=P2) was applied at either end with two hydraulic jacks. After the beam yielded in tension, one hydraulic jack was kept constant to avoid the excessive beam elongation. Displacement controlled reversed cyclic loading was employed with 1 mm increment of lateral displacement and with two cycles at each prescribed displacement. Lateral displacement was computed by averaging displacements measured at north and south ends of the beam. In this paper the loading to south is expressed as positive. At the beginning of testing, axial load of 300 kN was introduced to each column with prestressing steel bars to simulate long-term load. No attempt was made to keep this axial load constant once the test started.

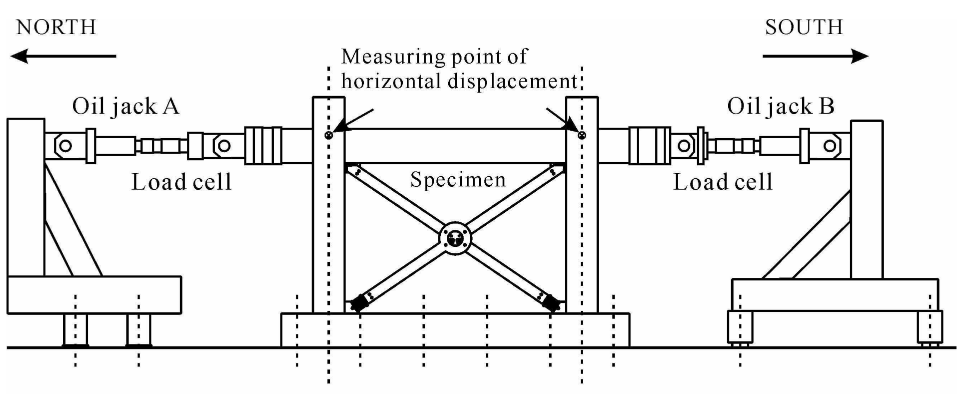
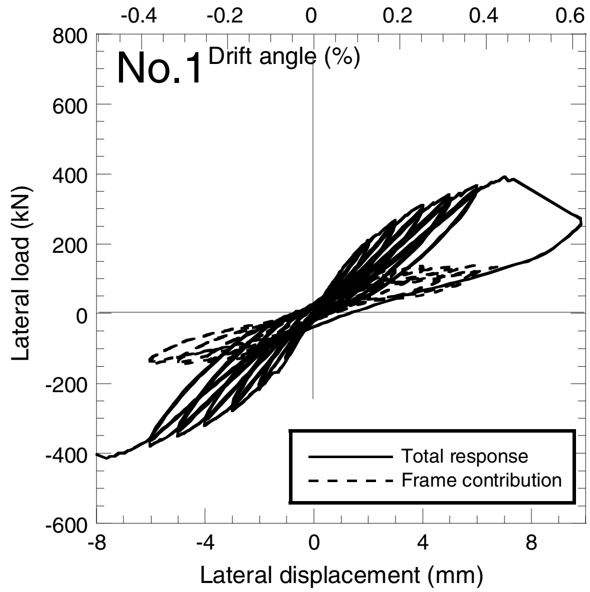


Fig. 6 Loading system

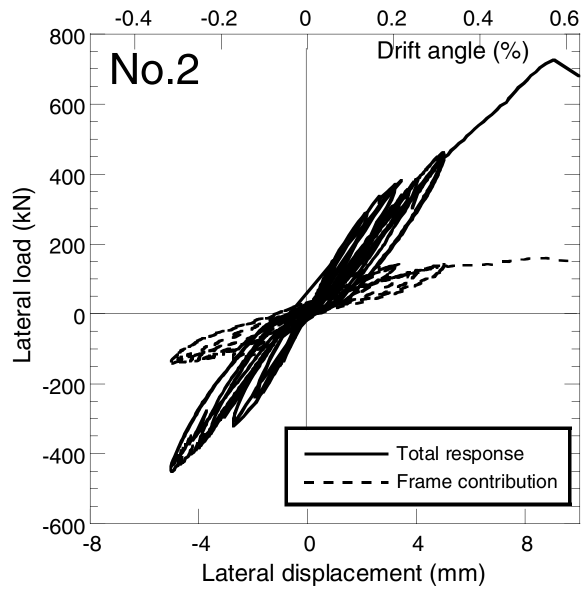
TEST RESULTS

Load-displacement relationships

Lateral load-displacement relationships are shown in Fig. 7. Both No.1 and No.2 failed with crushing of the brace concrete. The maximum capacity of No.1 was 397 kN in positive direction and 410 kN in negative direction. The maximum capacity of No. 2 was 726 kN at the lateral displacement of 9 mm in positive loading.



(a) Specimen No. 1



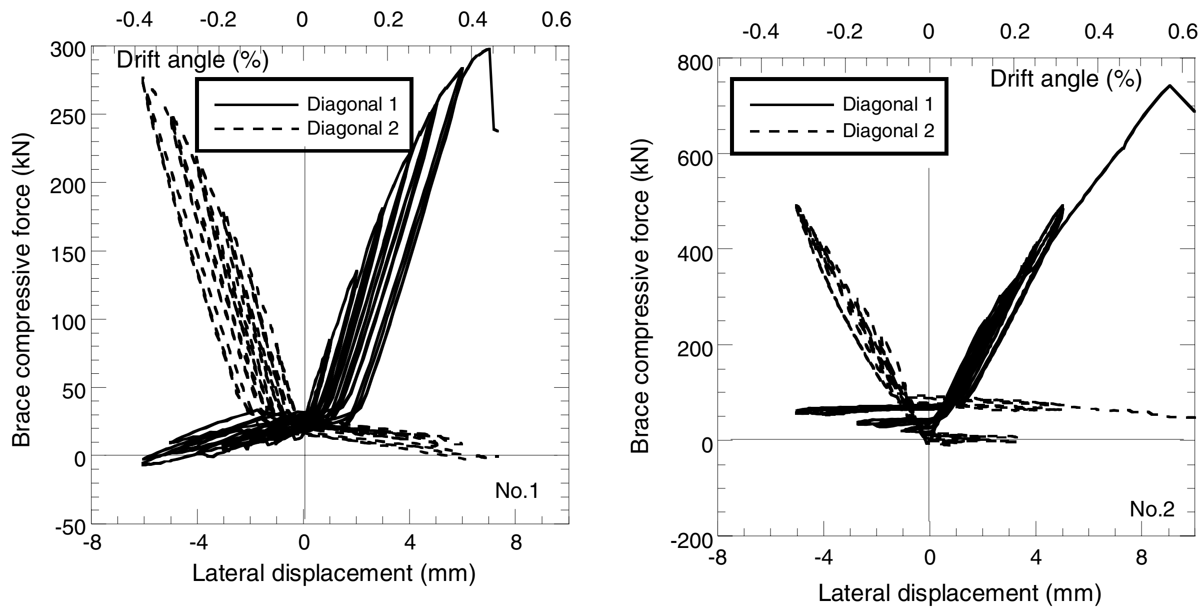
(b) Specimen No. 2

Fig. 7 Experimentally observed lateral load displacement curves and contribution of frames

Both specimens showed the almost elastic response with small energy absorption till the failure. Figure 7 indicates that cyclic load-displacement curves have small slip around the origin. As mentioned earlier the axial stiffness of a brace changes at the point E in Fig. 4. That is, the brace shows relatively low stiffness up to the re-contact of the steel bearing plate and the steel pipe of FSSP device. Observed slip deformation is due to this stiffness change of the brace and some residual elongation of beams and columns during cyclic loading. The amount of slip deformation depends on several parameters such as the initial prestressing force and the stiffness of FSSP device. At this moment the effect of this slip deformation to the seismic performance has not been examined. However for elastic responding systems (strength resisting systems) a small slip deformation can be permitted to some extent. In Fig. 7 the lateral load carried by frame is also indicated. The contribution of the original frame was obtained by deducting the lateral component of measured brace compressive force from total lateral force (P1+P2) where the brace compressive force was calculated by concrete strain measurements.

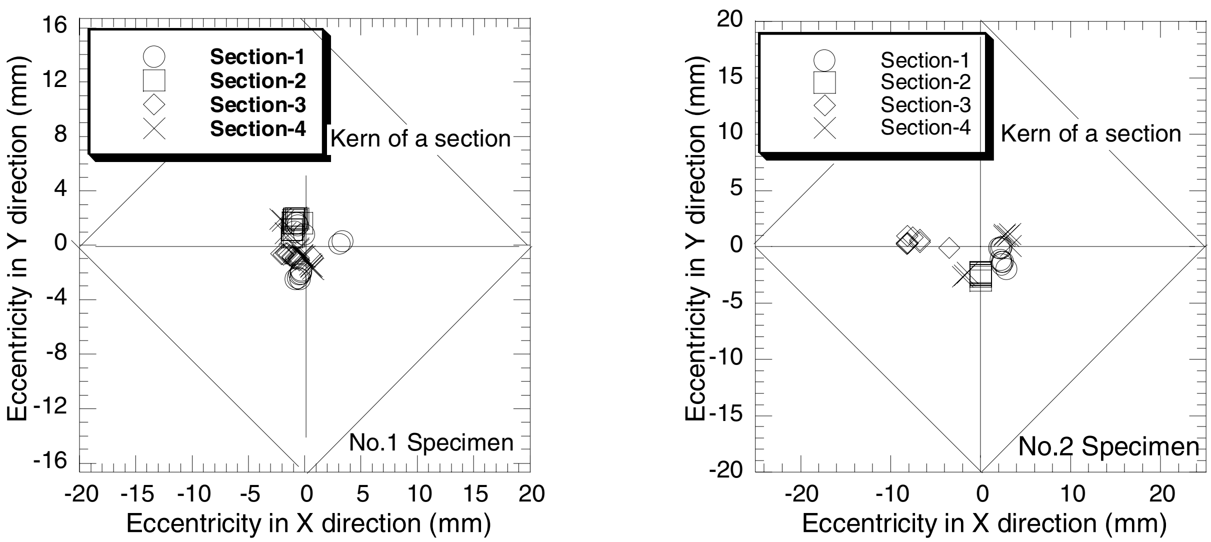
Response of each diagonal member

A variation of the axial force of the diagonal member with respect to the lateral displacement is shown in Fig.8. In positive loading Diagonal-1 works in compression and Diagonal-2 in tension. Kinking point in the cyclic curve corresponds to the point E in Fig. 4. It is seen that the brace force was kept in compression during almost all loading cycles. To investigate the bending effect induced to a brace concrete section axial strains were measured at four faces of each brace. From strain measurements the brace axial force and the eccentricity of axial force in a brace section were calculated. In Fig. 9 calculated eccentricities for braces of No.1 and No.2 specimens are indicated for four sections of a brace during the tests. The area inside a lozenge indicates the kern of a section of diagonal members. Calculated eccentricities are very small and all of response points dropped within a border of radius of the corn. From these observations it can be stated that braces were almost concentrically loaded.



(a) No. 1 (b) No. 2

Fig. 8 Compressive force in diagonal members– lateral displacement relationships



(a) No. 1 (b) No. 2

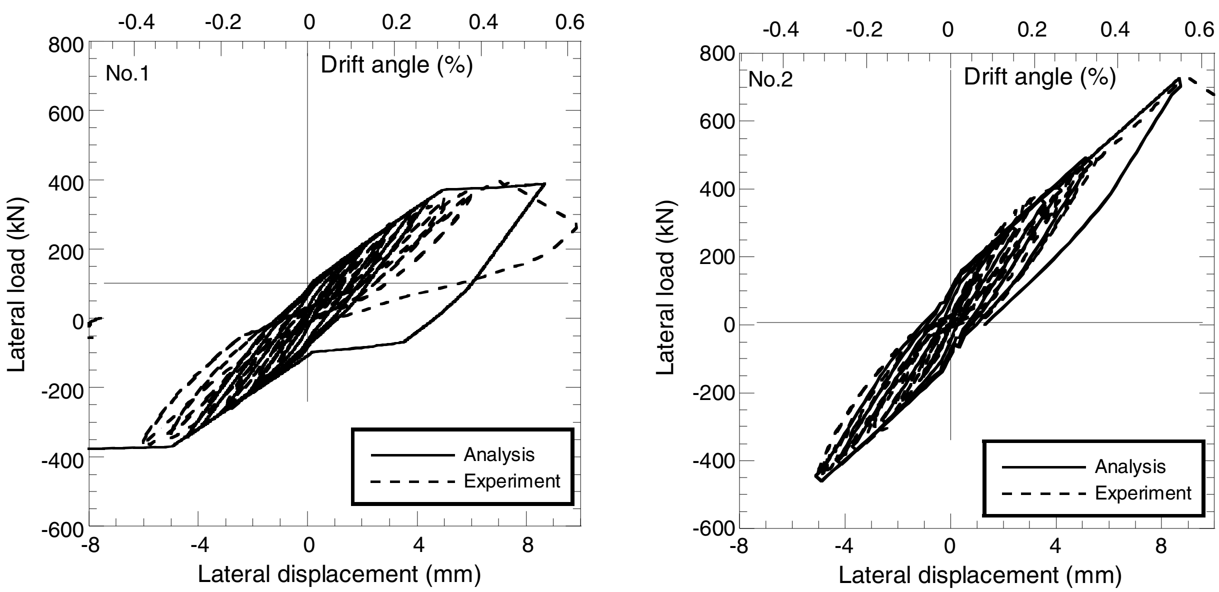
Fig. 9 Eccentricities of axial force induced to brace sections

Strength enhancement

In Fig. 7 the lateral load resisted by the frame is indicated by dotted line. At the maximum load carrying capacity, 64 % was sustained by a brace in No.1 and 79 % in No.2, respectively. The base shear coefficients at ultimate limit state were 0.57 for No.1 and 0.84 for No.2, where the base shear coefficient of the original frame was 0.2.

NUMERICAL SIMULATION

In the analysis, a commercial three-dimensional nonlinear frame analysis program was used. The beam and column were modeled as a combination of elastic linear elements (bending and shear) with two non-linear rotational springs at member ends. The rotational spring had a trilinear moment-rotation relation with two characteristic points of flexural cracking and flexural yielding where moment-axial force interaction is considered. The diagonal members were simulated with an axial spring element with a bi-linear force – elongation model. The beam column connection was assumed to be rigid and a brace was connected with a pin joint to a frame at the center of beam-column joint. Numerical calculation was conducted applying gradually increasing lateral loads at the both beam-column joints at which the experimentally obtained lateral displacement was enforced. Solid lines in Fig. 10 indicate the predicted load displacement responses. Both specimens reached the ultimate limit states by compressive yielding of brace concrete. Experimental and theoretical curves showed good agreement except the slip behavior near the origin.



(a) No. 1 (b) No. 2

Fig. 10 Theoretically predicted responses of two specimens

A TYPICAL RETROFIT EXAMPLE USING THE PROPOSED BRACE

Original building

The building is a 4-story office building in Osaka. Total building area is 3508m^2 and height is 14.65m. It was designed in 1965 and the construction was completed in 1966. It consists of a moment resisting frame with shear walls in both longitudinal and transverse directions. Frames in longitudinal direction had shear walls with eccentricity at the first and second floors where the plan view of the second floor is shown in Fig. 11 as an example. The third and fourth floor had sufficient shear walls without much eccentricity and no retrofit was necessary.

Retrofit strategy

Since the lateral load carrying capacity of the original building was relatively high although not sufficient, it was determined to increase the lateral load carrying capacity of the first and second story rather than the ductility. For example at the second floor, the thickness of the internal shear wall was increased and the X-shape brace was placed at five locations of the peripheral fame so that rooms can obtain enough external light. Fig. 12 shows the results of pushover analysis for the first and second story before and after the retrofit where Q is the story shear force in MN and W is the total structural weight. The analysis was terminated when the short columns at the corner of the first story reached its ultimate deformation for shear failure. Both the first and second stories achieved about 20% increase in story shear force. The construction of X-shape brace is shown in Fig. 13.

CONCLUSIONS

Loading tests and structural analysis were conducted on two frame specimens strengthened with a proposed strengthening method with the X-shape concrete brace. Obtained conclusions are summarized below.

1) A new seismic retrofit method by X-type concrete brace was proposed and its efficiency was shown. Its simple construction process was confirmed.
2) The X-type concrete brace system largely enhanced the lateral load carrying capacity of a frame.
3) The special FSSP device (a set of flat springs and steel pipe) maintained the brace in compression during simulated earthquake loadings. It prevented a diagonal member to come off from an existing frame during an earthquake.
4) Non-linear frame analysis program was able to predict the seismic response of the frame strengthened by the proposed brace system.
5) Some slip deformation was observed in experiments and its effect on a dynamic response behavior should be examined further.

ACKNOWLEDGMENTS

Authors express hearty thanks to Mr. Y. Okuno who conducted tests and analysis. Takenaka Corporation, Daiwa Corporation and Nagai Design Company financially supported this research.

REFERENCES

[1] Architecture Institute of Japan, Japanese Building Standard, 1950
[2] ACI Committee318, *Building Code Requirement for Reinforced Concrete*, 1989, pp.121-124
[3] S. P. Timoshenko, *Theory of Elastic Stability*, 1961, pp.310
[4] ACI Committee318, *Building Code Requirement for Reinforced Concrete*, 1989, pp.159-163
[5] ACI Committee318-R, *Building Code Requirement for Reinforced Concrete*, 1989, pp.159-163

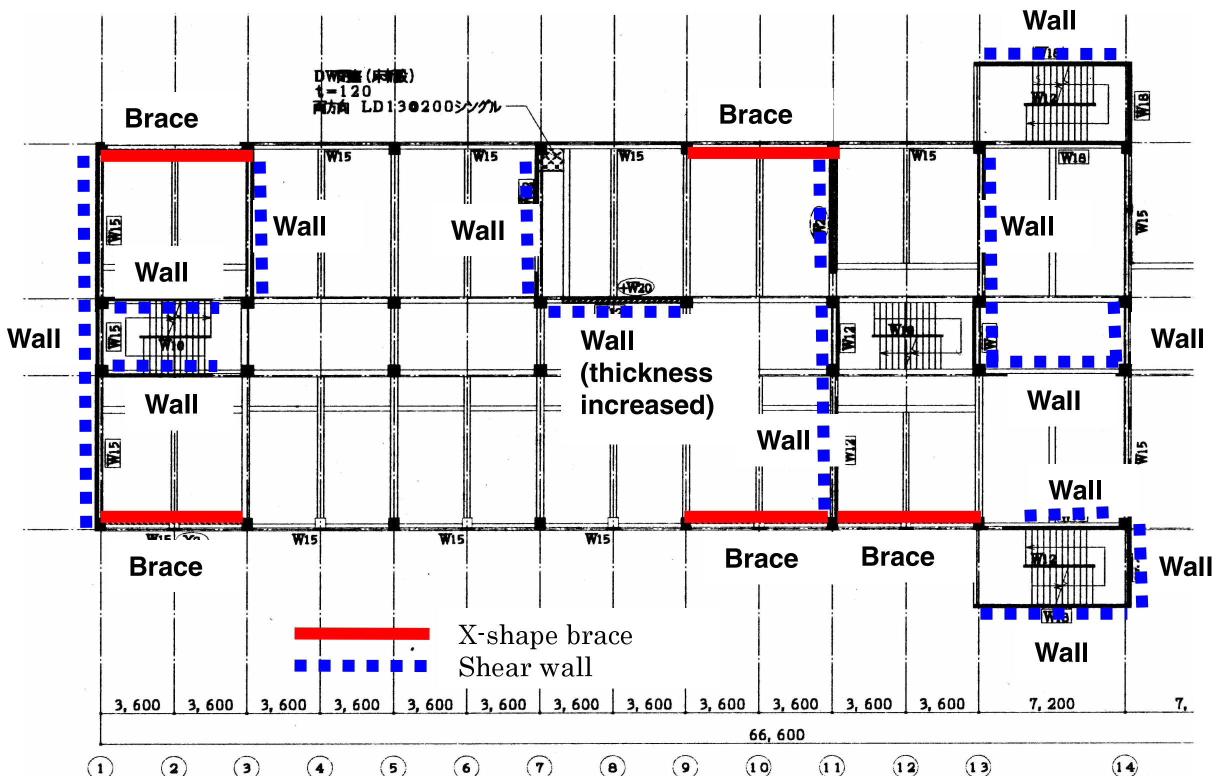


Fig. 11 Location of X-shape braces and shear walls at the second story

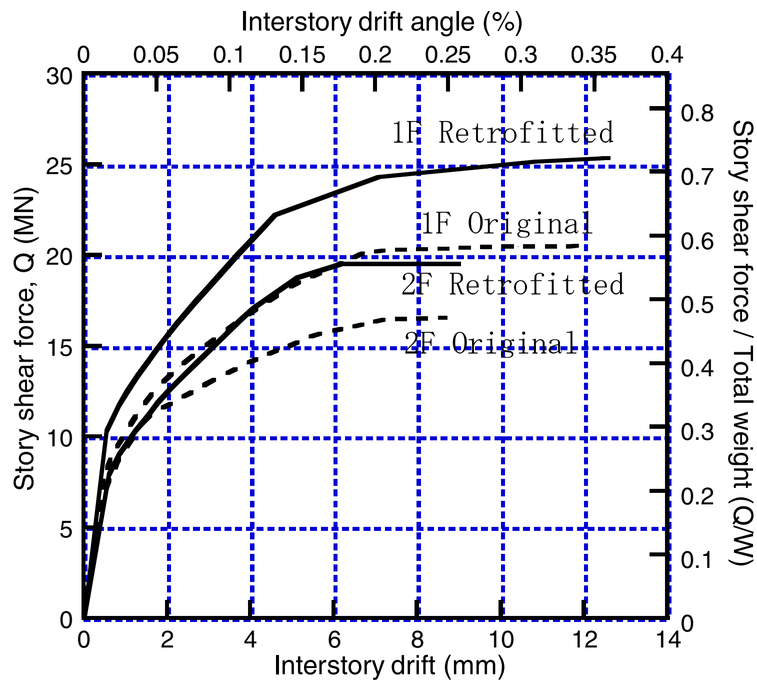
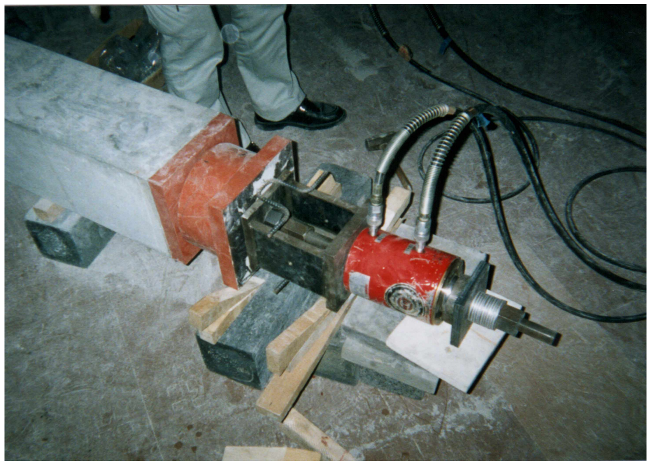
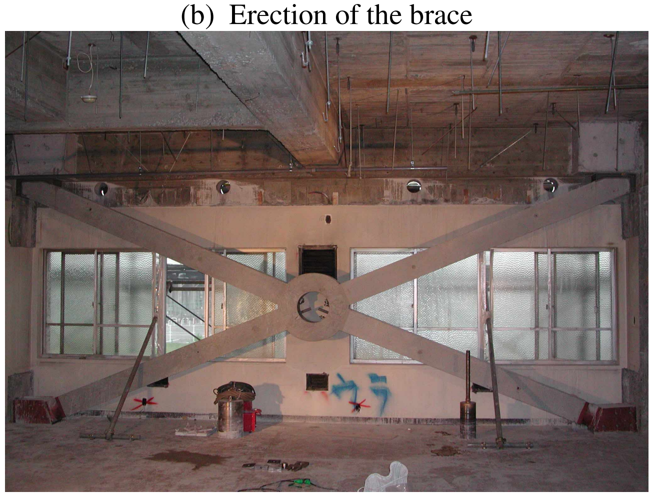


Fig. 12 Results of the pushover analysis for the original and retrofitted building



(a) Introduction of prestressing force to the lower part of a diagonal member

(b) Erection of the brace



(c) The X-shape concrete brace placed in the existing frame (Enough openings still exist.)

Fig. 13 Retrofit construction of the proposed X-shape brace system (Courtesy of Takenaka Corporation)