STRENGHTENING EFFECT OF ECCENTRIC STEEL BRACES

TO EXISTING REINFORCED CONCRETE FRAMES

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SUMMARY

One of the main buildings of the Tohoku Institute of Technology in Sendai, which was damaged by the '78 Miyagi-ken-oki earthquake, was restored. The eight storied reinforced concrete frame construction was strengthened in the longitudinal direction by means of steel cross braces which were installed with eccentricity in both façades from outside of the building.

In this paper, the scheme of the bracing is described and the results of experimental works on the behavior of the eccentric cross braces, strength of brace-to-frame connection and on spandrel weakening device are presented. Also, the aseismic effect of the bracing system to the building is evaluated.

INTRODUCTION

Two of the main buildings of the Tohoku Institute of Technology in Sendai, Japan, were seriously damaged by the Miyagi-ken-oki earthquake of June 12, 1978 [1]. One of them, Building No.3, four storied reinforced concrete frame construction, was judged irrecoverable, demolished and being reconstructed. The other, Building No.5, which was eight storied R.C. frame construction (Figs.1 and 4) was repaired, strengthened and resumed its service in ten months after the earthquake.

The damage to both the buildings is characterized by the same mode of failure, i.e. shear and bending-shearing failure of columns in the north side frame under the action of horizontal force in the longitudinal direction (Figs.2 and 3). One cause of the destruction is supposed to be the deficiency of ultimate strength of the frames, none of or very few shear walls existing in this direction. Another and more important factor is the influence of in-fill spandrel wall. The walls, being cast-in-place only in the north frame as shown in Fig.1, made the stiffness of the frame about four times greater than those of the other two frames and gave rise to concentration of shearing force , thus resulting the brittle shear failure of the columns.

The latter factor had been ignored in the design of buildings until the dangerous effect of in-fill spandrel walls was drastically recognized by the destruction of Hachinohe Technical College in 1968 Tokachi-oki earthquake [2],[3] (the Buildings No.3 and No.5 were built in 1966 and 1968 respectively).

For the Building No.5, not only the failed and cracked columns, beams, walls and slabs were repaired, but the strengthening of the original frames together with the weakening of the spandrel wall was done to improve the resistance against big earthquakes expected in the future.

Strengthening of the building in the transverse direction was made by installing additional R.C. shear walls as shown in Fig.4. As the strengthening in the longitudinal direction, steel cross braces were attached to both faces of the building from outside. Rigid connection between

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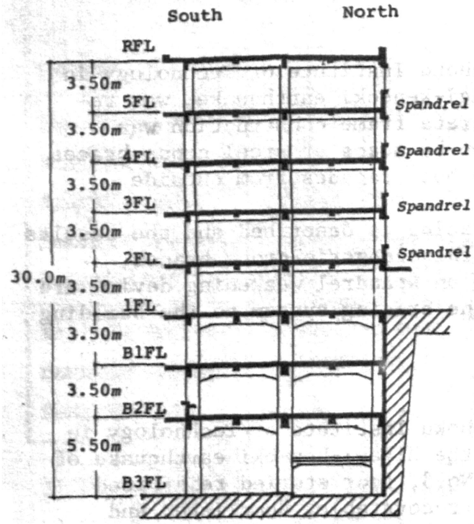


Fig. 1 Section of Building No.5

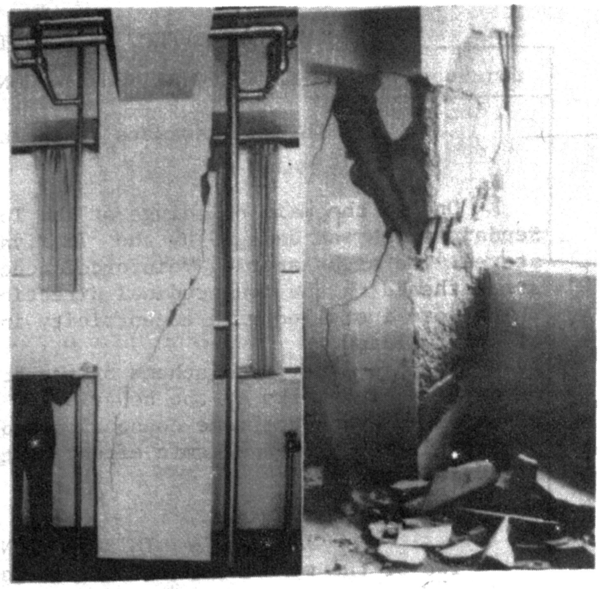


Fig.2 Typical cracks of columns

Fig.3 The most heavy damage to column

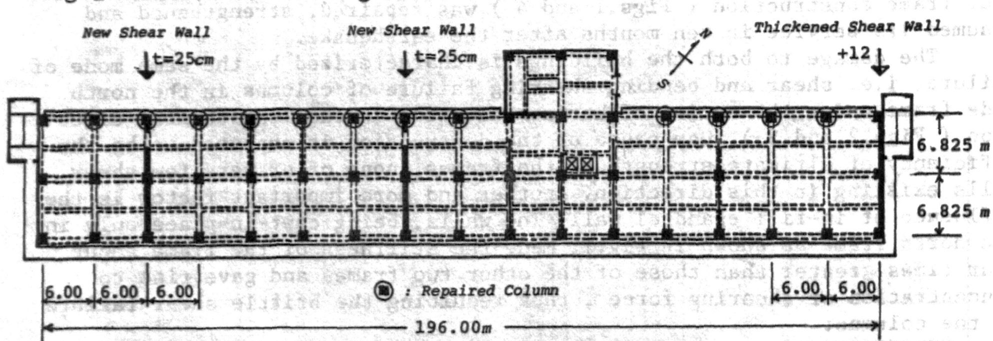


Fig. 4 Repaired columns and added shear walls (plan of 3F)

steel braces and the existing R.C. frames was secured by a kind of post-tentioning technique. Further, the ends of the in-fill spandrel wall were continuously perforated by core boring so that the columns may not encounter any more shear failure.

The authors were in charge of the restoration works as the members of a committee set in the Institute, and as such strengthening of existing R.C. frames by steel braces was a new technique which had not been experienced, a series of experiments was needed to verify the aseismic effect of the system.

In this paper, the design of the bracing system is described and results of tests on the behavior of braces, brace-to-frame connection and the weakened spandrel beams are presented. Basing on the experimental data, the strengthening effect of the bracing system is evaluated.

1. DESIGN OF STRENGTHENING BRACE SYSTEM

Basic Concept Though the installation of shear walls is the most common practice as aseismic strenghening for R.C. frame construction, a

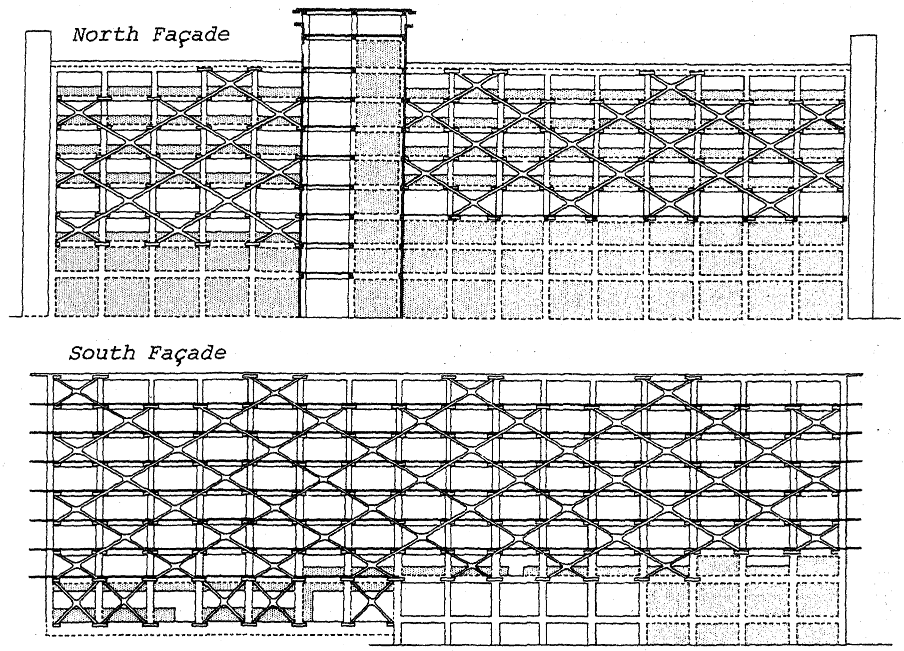


Fig. 5 Arrangement of cross braces

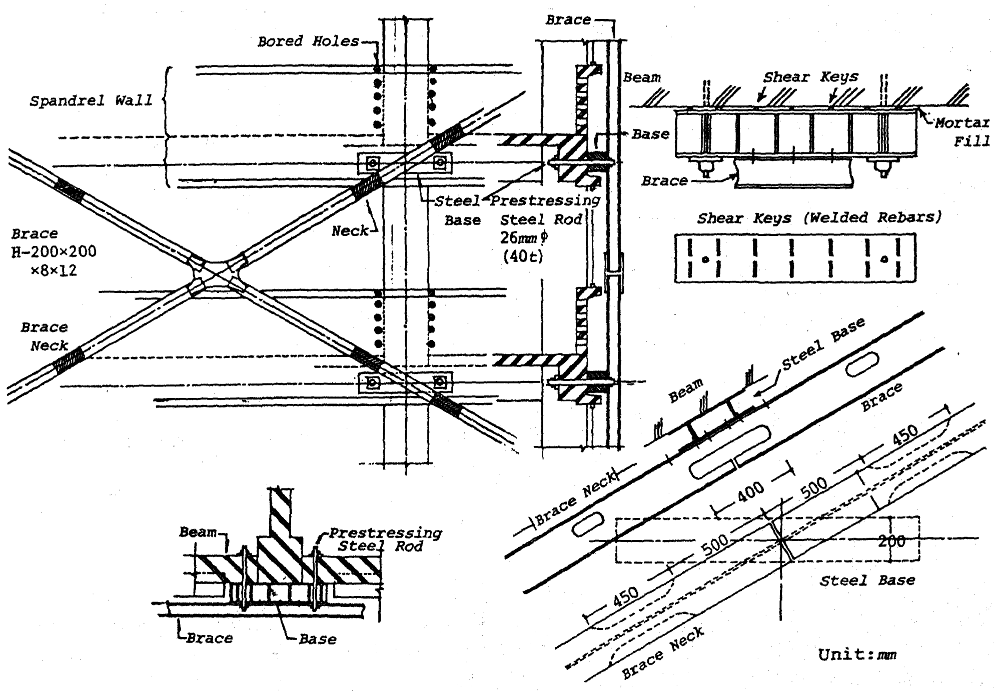


Fig. 6 Attachment of braces to existing R.C. frame

system of steel braces was adopted in the restoration work. The advantage of the system can be summarized in three points: a) natural lighting through windows is not intercepted, b) installation is approached from outside, thus facilitating the work and not making any obstacle in the interior of the building, and c) uiform distribution of braces is possible so that no concentration of shearing force occurs.

Brace to frame connection   As shown in Fig.6, brace members were fastened by friction bolts to steel bases which were set against the R.C. beam face and, after filling the gap with cement mortar, post-tentioned by prestressing steel rods inserted through bored holes.

Brace Members   H-section (H-200×200×8×12,*mm*) of weathering steel (JIS SMA41A, σ_Y=35*kg/mm²* and σ_B=49*kg/mm²*) was used with coating of a rust stabilizing agent because the braces were to face the open air.

In each brace member, the outer flange was cut at the nodal point, as shown in Fig.6, in order to derive fully eccentric property. Further, as shown in Fig.7, in B-3 braces used for 4th and 5th stories, the neck section having narrowed inner flange and perforated web was formed. B-2 braces for 1st to 3rd stories had narrowed inner flange and B-1 braces for B3 to B1 stories had not the neck section. These necks were for accelerating the yielding under axial forces.

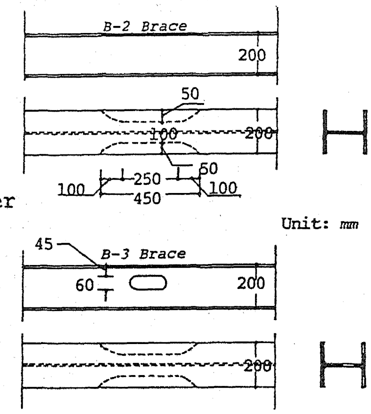


Fig. 7 Brace necks

Amount of Steel Used and Period of Execution

The total amount of steel sections and plates used for the bracing system was about 50 tons. Four months were needed for the fabrication and the installation.

2. HYSTERETIC CHARACTERISTIC OF CROSS BRACES

One third scale models were used in order to investigate the hysteretic behavior of the units of cross braces of the three different types. Brace section of 75*mm*×75*mm* was built up from 4.2*mm* thick sheet (σ_Y=32*kg/mm²* and σ_B=48*kg/mm²*) for the flange and 3.0*mm* thick sheet (σ_Y=25*kg/mm²* and σ_B=34*kg/mm²*) for the web.

As shown in Fig.8, alternate horizontal load was applied through a hinged rigid frame. The specimens were brought to their ultimate state by 9 to 10 cycles of loading with monotonically incrasing amplitude. Fig.9 shows the load- horizontal displacement curves for the point E in Fig.8.

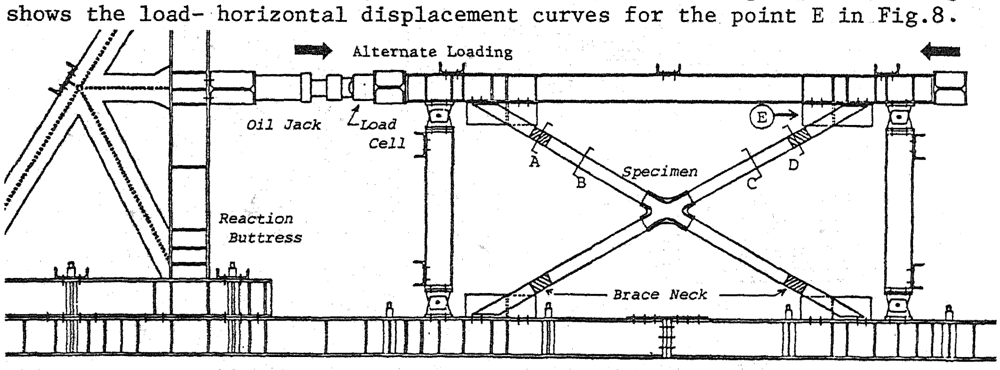


Fig. 8 Test setup for 1/3 scale models of cross braces

The behavior of the braces are characterized by the eccentric nature in the action of axial forces. As shown in Fig.10, the eccentricity is especially dominant in the neck sections (sections A and D in Figs.8 and 10) In the consequence of the eccentricity, the elastic stiffness to the lateral force was reduced to about 40% of the case in which the same members were subjected to concentric action of the axial force.

Yielding both in tention and compression initiated from the inner flange in the neck section because of the eccentricity, and gradually propagated outward up to the outer extreme of the web, the outer flange remaining almost stress free or well in elastic range even in the ultimate state.

The ultimate bearing capacity was determined by the buckling of the inner flange and web in the neck section. However, the reduction of the load level was very small as can be seen in Fig.9. This is due to the localized nature of the buckling. The estimation of ultimate loads *Pult*, which were obtained under the assumption that inner flange and web in the neck were in the stress level of tensile strength/yield stress in tension members and of yield stress in compression members, outer flange being assumed to be stress free, is in good coincidence with the experimental values.

The property of the braces is summarized in Table 1.

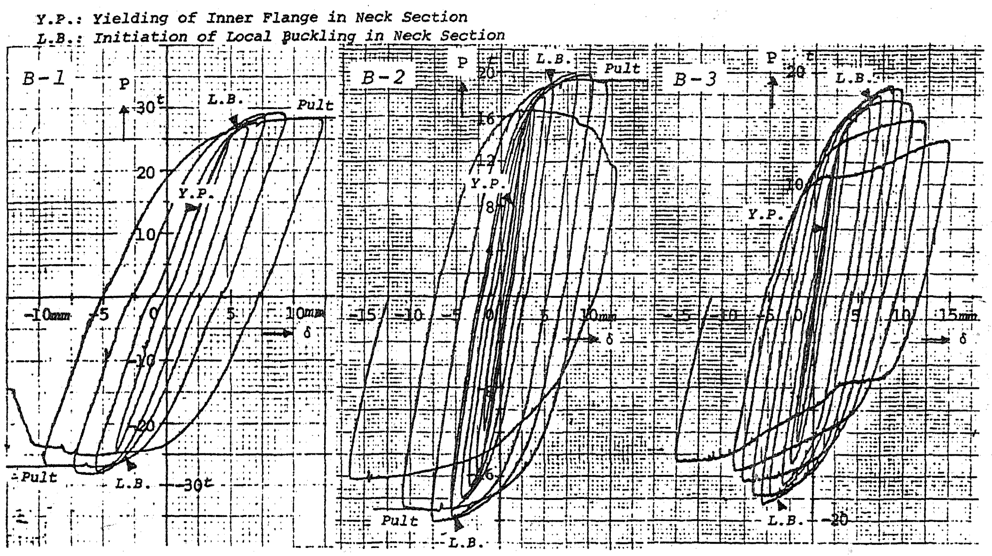


Fig. 9 Load-displacement curves (Point E in Fig.8)

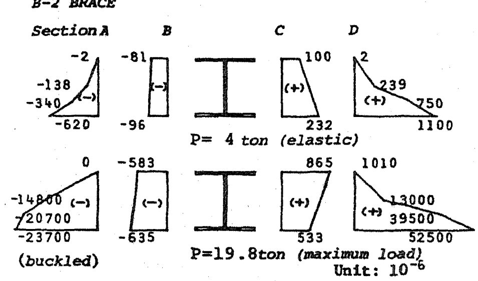
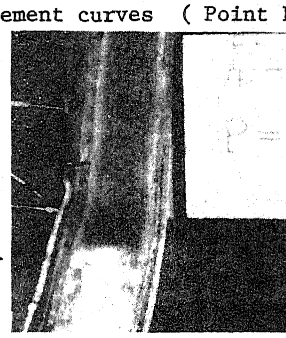


Fig.10 Measured axial strain



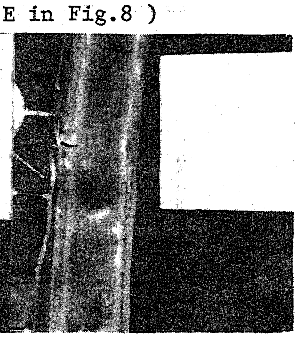


Fig.11 Buckling of Neck (B-2)

Fig.12 Fracture of Neck (B-2)

Table 1 Summary of Experimental Results of Cross Braces

Specimen	Initial Stiff-ness *1 %	Initial Yielding *2 Load P_y, *ton*	Displace-ment δ_y, *mm*	R $=\delta_y/H$ *3 10^{-3}*rad*	Max. Load *ton*	Ultimate State Load P_{ult}, *ton*	Displace-ment δ_{ult}, *mm*	R $=\delta_{ult}/H$ *3 10^{-3}*rad*
B-1	37	14. (117)	2.1 (6.3)	1.83	29.3	28.4 (255)	9.8 (31.2)	8.4
B-2	40	8. (71)	1.3 (3.9)	1.09	19.8	19.0 (181)	10.8 (32.4)	9.3
B-3	40	6. (54)	1.0 (3.0)	0.86	18.7	15.6 (156)	10.4 (29.4)	8.9

(----) : *Figure converted for the corresponding prototype braces*
*1 : *Lateral stiffness divided by theoretical stiffness for the case of concentric cross braces*
*2 : *The first yielding of inner flange in the neck section*
*3 : H *is the height of story*

3. STRENGTH OF BRACE-TO-FRAME CONNECTION

The strength of brace-to-frame connection is governed by the shearing capacity of the junction of steel base and R.C. beam face under the existence of the transverse prestress. In order to prove the reliability of the joint, a series of slip tests was carried out. Scale of the specimens was 1/2. Fig.13 shows the test setup. The tests were performed under two kinds of loading: the loading in the direction parallel to the axis of base and the one perpendicular to the axis.

Fig.14 shows a load-slip displacement curve for a specimen loaded longitudinally. The junctions behaved very well both in strength and in ductility in virtue of the prestress. In the ultimate state, the coefficient of friction with respect to the induced prestress reached to 1.6-2.0 in the cases of longitudinal loading and to 1.1 in the cases of transverse loading. In the context of the prototype, the maximum hrizontal force which can be transmitted from frame to brace is estimated to be 120*ton*

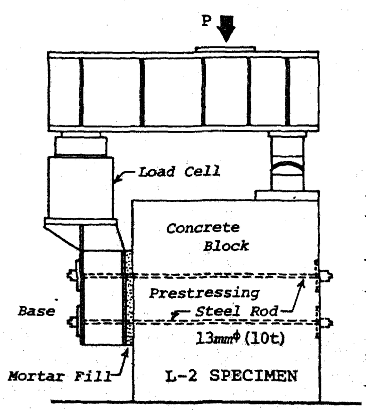


Fig.13 Slip test of steel base

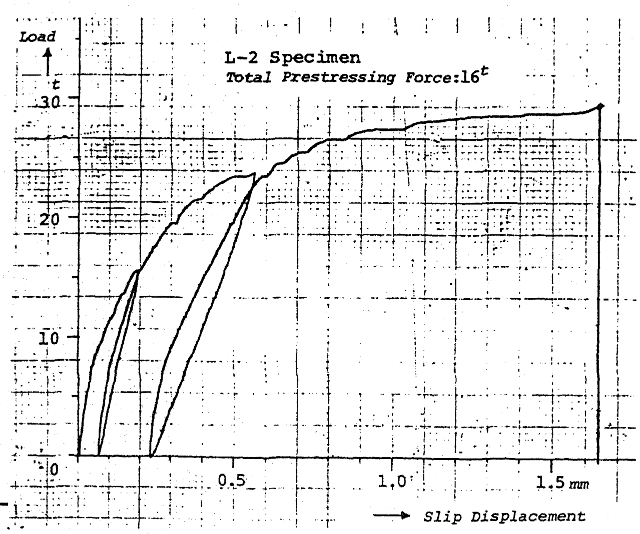


Fig.14 Load-slip displacement curve

in each of the general nodes (2 prestressing rods) and 240*ton* in the end anchorages (4 prestressing rods), which mean enough resistance in view of the ultimate capacity of braces indicated in Table 1.

4. BEHAVIOR OF WEAKENED SPANDREL WALL

In order to observe the performance of the spandrel walls whose ends were weakenend by bored holes, a pure beam and beams having spandrels with and without the holes were subjected to alternate bending moment and shear in the form of simple beam test. Scale of the specimens was 1/3 (Fig.17).

Fig.15 shows load-deflection curve of the beam with weakened spandrel(A-2) compared with the backbone curves of the other two.

For the bending moment in which the spandrel wall was in compression, the resistance of the weakened spandrel beam was reduced to about 1/3 of the unweakened one as the result of crushing of concrete remaining between the holes.*1

In the context of the framed configuration, it can be proved that the level of horizontal force corresponding to the spandrel crushing is well under the level of the one of the shear failure of the column, thus enabling us to avoid the latter.

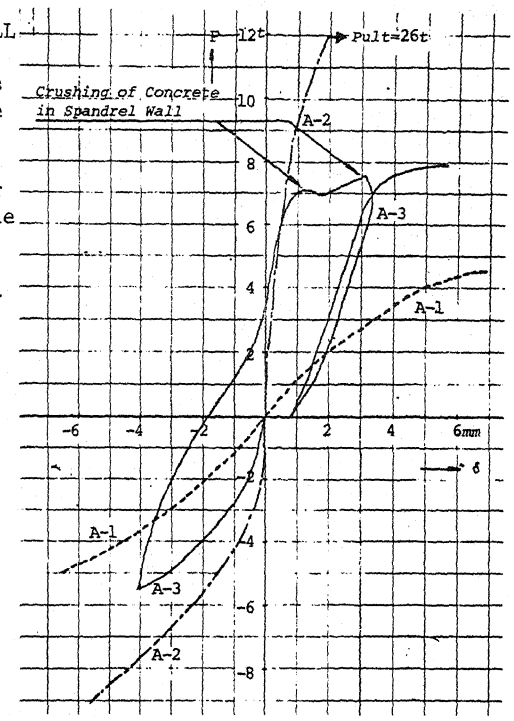


Fig. 15 load-deflection curve

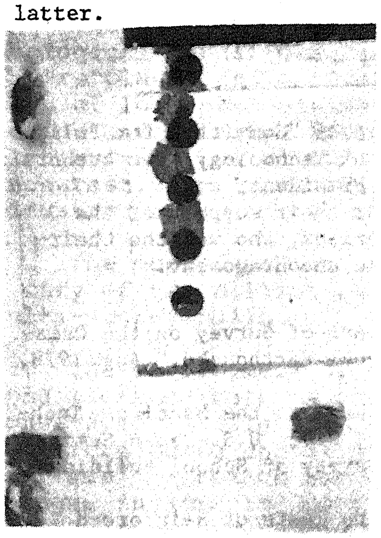
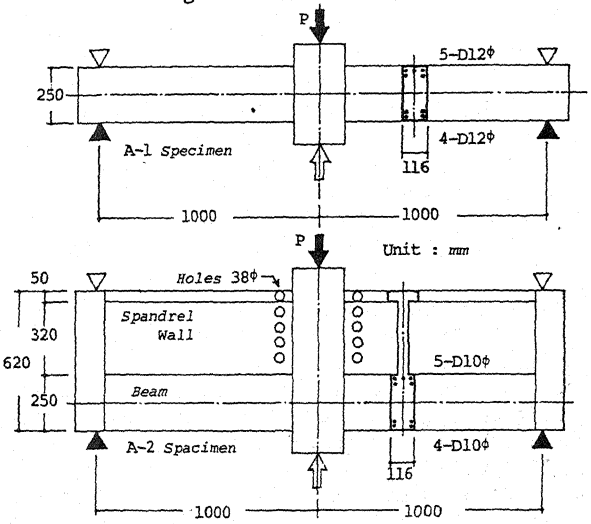


Fig. 16 Crushing of concrete around holes



A-3 Specimen is the same as A-2 but without holes

Fig. 17 Test specimens

*1 The wall concrete between holes was grooved with only outmost layer of 1cm thick (3cm in prototype) being left.

5. EVALUATION OF STRENGTHENING EFFECT OF BRACES : CONCLUSION

Table 2 shows the variation of natural period of vibration of Building No.5 before and after the earthquake obtained from Y. Abe's microtremor measurements [4]. The stiffness of the building in the longitudinal direction, which was once reduced to 40% of the original value, was almost totally recovered. Shortening of the period from 0.49*sec* in Feb. 1979 to 0.35*sec* in Apr. 1979 means that the R.C. frame stiffness became twice by installing the steel bracing system.

As the ultimate deformability of braces of about 1/100 of the story height is probably similar to the one of the R.C. frames, the summation of strength of the frames and braces seems to be reasonable. As shown in Table 3, the ultimate shearing force coefficients also became twice by the bracing.

This level of earthquake responce will hardly be reached in view of such large capacity of energy dissipation of braces as exhibited in the experiments.

Table 2 Natural Period of Building No.5

Date of Observation	Direction T	Direction L
Apr., 1975	0.39	0.34
Feb.22, 1978: after an earthquake of scale IV	0.39	0.44
June 28,1978: after Miyagi-ken Oki earthquake	0.43	0.53
Feb.13, 1979: after restoration of columns	0.36	0.49
Apr. , 1979: after installation of braces	0.36	0.35

unit: sec T : Transverse, L : Longitudinal

Table 3. Estimation of Ultimate Shearing Force : Longitudinal Direction

Story	Weight		Before Earthquake		After Strengthening			
			Shear.F.	Coeff.	Frames	Braces	F + B	Coeff.
	W, t	ΣW, t *1	Q_c, t *2	k_c *3	Q_F, t *5	Q_B, t	(Q_F+Q_B)	k *4
5	1,593	1,979	1,550	0.78	1,648	1,092	2,740	1.39
4	1,417	3,396	1,710	0.50	1,755	2,496	4,251	1.25
3	1,440	4,836	1,899	0.39	2,131	2,534	4,665	0.96
2	1,447	6,283	2,414	0.38	2,496	2,896	4,012	0.86
1	1,820	8,103	2,962	0.37	2,962	2,534	5,498	0.68

*1 including weight of penthouses
*2 shearing force to whole building corresponding to shear failure of north frame columns, including resistance of shear walls
*3 $k_c = Q_c / \Sigma W$, *4 $k = (Q_F + Q_B) / \Sigma W$
*4 shear failure of columns assumed not to occur

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