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DAMAGE BY THE 2011 EAST JAPAN EARTHQUAKE OF REINFORCED CONCRETE SCHOOL BUILDING IN TOCHIGI RETROFITTED BY STEEL-BRACED FRAME

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ABSTRACT: A reinforced concrete three-story school building which was retrofitted by steel-braced frames suffered moderate damage such as column shear failure for the top story without any seismic retrofits under the 2011 East Japan Earthquake. Although the damaged story has possessed sufficient lateral-force resisting capacity to be required by the current seismic codes, irregular stiffness distribution over the height of the building, generated by the seismic retrofits except for the top story, caused the concentration of a large deformation to the top story without retrofits.

Key Words: 2011 East Japan Earthquake, reinforced concrete building, seismic retrofit, steel-braced frame, damage, seismic capacity evaluation, lateral stiffness

INTRODUCTION

The 2011 East Japan Earthquake caused a huge disaster in Tohoku and Kanto areas due to mainly both tsunami and serious accident at a nuclear power plant. In such a social atmosphere, it is inevitable that people does not pay attention to damage of reinforced concrete (R/C) buildings induced by shaking due to earthquake ground motions. However, damage of R/C buildings retrofitted by a steel-braced frame, a R/C shear wall, and so on should not be overlooked.

Seismic capacity evaluation and seismic rehabilitation of existing R/C buildings are carried out actively in Japan after the 1995 Kobe Earthquake. Many school buildings especially have been retrofitted to enhance earthquake resistant performance because of its functions required as both a daily educational and an emergency evacuation facility. Tremendous ground shaking by the 2011 East Japan Earthquake was a new experience for these retrofitted buildings in a very extensive area of East Japan. Therefore, adequacy of recent seismic retrofit design method for existing R/C buildings should be verified through damage survey on R/C school buildings after the 2011 East Japan Earthquake.

This paper describes damages of a R/C three-story school building retrofitted seismically by multi-story steel-braced frames in Tochigi prefecture. Sufficient seismic lateral capacity required by the Japanese building law was provided to the building by seismic rehabilitation conducted at the first, second and penthouse stories in 2009. However four R/C columns at the third story without any seismic retrofits failed in shear although it was judged that the enough lateral capacity was kept for the third story.

Seismic resisting performance of the building during the earthquake is discussed through the seismic capacity index evaluated according to "Standards for seismic performance evaluation of existing reinforced concrete buildings" comparing with observed damages. A problem concerning the seismic retrofit design method for existing R/C buildings is pointed out in the paper.

OUTLINE OF SCHOOL

A junior high school in Tochigi prefecture was surveyed, which is located at a small mound encroached by small valleys in the east of Utsunomiya City where the JMA intensity of the main shock was 6+. A part of school buildings were built on a reclaimed land from a valley. This school consists of a class-room building, a special class-room building, a gymnasium, a facility for a school lunch, a technical-subject-room building, a musical-subject-room building and a martial arts building as shown in Fig. 1.

This paper focuses on a reinforced concrete (R/C) three-story class-room building which suffered moderate damage although retrofitted by two-story steel-braced frames. Hereafter, damages to other buildings are mentioned briefly except for the martial arts building without any damages.

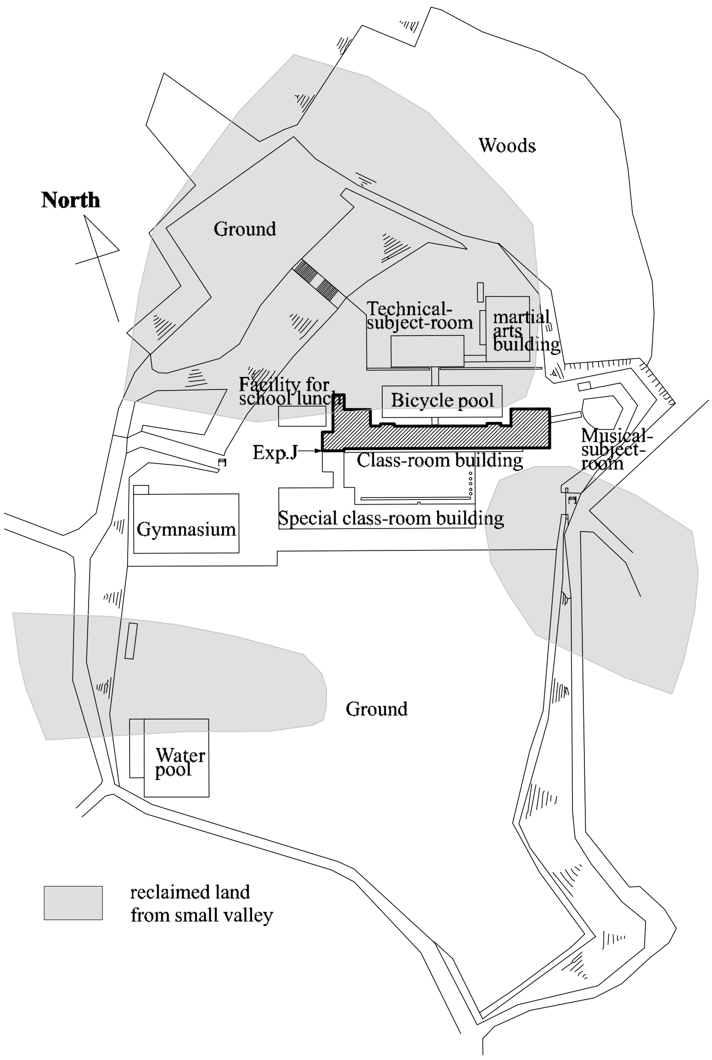


Fig. 1 Location of school buildings

Special class-room building

The special class-room building with a total floor area of 2,235 m^2 is a two-story R/C structure designed according to old seismic code and constructed in 1973. The building is supported by R/C pile foundations. The first story of the building was retrofitted in 2009 by three steel-braced frames in a longitudinal direction to enhance the seismic capacity index *Is* of 0.58 to 0.72. The index *Is* for the second story without any seismic rehabilitation was 1.00. The residual seismic capacity after the earthquake was estimated to be 0.96 times the capacity before for the first story in a longitudinal direction, by which the damage level of the building was regarded as slight.

Gymnasium

The gymnasium, supported by R/C pile foundations, is a single-story steel structure constructed in 1975. A roof shell consists of steel members with a L-shaped section which are combined like diagonal grids, forming a low-rise arch. Lattice girders are spanned in a transverse direction. A lattice column, which consists of steel members with a L-shaped section, is fixed by a steel plate with anchoring bolts embedded in a reinforced concrete stub. Four vertical braced frames, where steel members of L-65x65x6 are used for an upper-story and L-75x75x6 for a lower-story as a diagonal chord, are placed in a longitudinal direction.

The seismic capacity index *Is* and the strength ratio *q* of the building is 0.36 and 0.89 respectively in a longitudinal direction, and 0.38 and 0.77 respectively in a transverse direction, regarded as insufficient for criteria, i.e., 0.7 and 1.0 respectively. Seismic retrofit, however, was not yet carried out to the building at the time of the event.

All diagonal chords in vertical braced frames for an upper-story were ruptured at the weld zone between a gusset plate and a column or the vicinity of a bolt hole on the gusset plate, because of which the building lost completely the lateral capacity. Diagonal chords in braced frames for a lower-story buckled heavily. Predicted earthquake resistant capacity in a transverse direction was dominated by the flexural buckling of bottom chords in a lattice girder by the seismic evaluation in 2010, whereas buckling of those members was not observed for this field survey. Jacketing concrete at the bottom of a column spalled off at both gable frames, which caused exposure of the base-plate of a column.

The damage rate of structural members and non-structural members was Grade 5 and Grade 3 respectively in a longitudinal direction, and Grade 1 and Grade 3 respectively in a transverse direction, by which the damage of the building was classified into the severe level for a longitudinal direction and the minor level for a transverse direction.

Facility for a school lunch

The facility for a school lunch with a total floor area of 162 m^2, supported by spread foundations, is a single-story steel structure with poor details constructed in 1975. A diagonal chord of a vertical brace fell off due to shear rupture of the bolt on the gusset plate at the end of the chord. The building inclined westward, resulting in an inclination of 2.7 %. Many cracks occurred for exterior and interior finishes. Ground fissures developed near the building.

The damage rate of structural members and non-structural members was Grade 5 and Grade 1 respectively in a longitudinal direction, and Grade 2 and Grade 1 respectively in a transverse direction, by which the damage of the building was classified into the severe level for a longitudinal direction and the minor level for a transverse direction.

Technical-subject-room building

The technical-subject-room building with a total floor area of 396 m^2, supported by spread foundations, is a single-story steel structure constructed in 1975. The building consists of moment-resisting frames for both directions, however with only one vertical brace in an east end frame of a transverse direction which yielded during the earthquake. One bay of the north frame sank 75 mm to 115 mm, whereas obvious settlement of exterior foundations was not observed. Since the north frame leaned westward to approximately 1 % and on the other hand, the south frame did eastward to 0.5 %, the building seemed to twist three-dimensionally.

The damage level of the building was regarded as moderate for both directions, because the damage rate of structural members and non-structural members was Grade 3 and Grade 2 respectively.

Musical-subject-room building

The musical-subject-room building with a total floor area of 251 m^2, supported by spread foundations, is a single-story steel structure constructed in 1990. The building consists of moment-resisting frames for both directions. Although exterior finishes and ceiling boards fell off widely, any structural damages were not observed. The damage of the building was classified into the minor level due to the damage rate of Grade 4 for non-structural members.

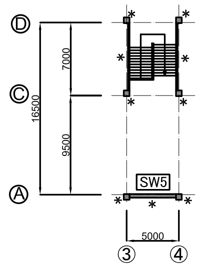
OUTLINE AND DAMAGE FOR CLASS-ROOM BUILDING

Building description

The class-room building with a total floor area of 3,276 m^2 is a three-story R/C structure with a penthouse designed according to old seismic code and constructed in 1974. The building is supported by R/C pile foundations, whose length was supposed to be approximately 6 m due to a boring investigation after-mentioned. The building consists of moment-resisting frames with 19 bays for a longitudinal direction and one-bay moment resisting frames with shear walls with an opening for a transverse direction. A seismic joint with a gap of 100 mm is placed between the building and the special class-room building located at the south side. Overall view of the building is shown in Photo.1.

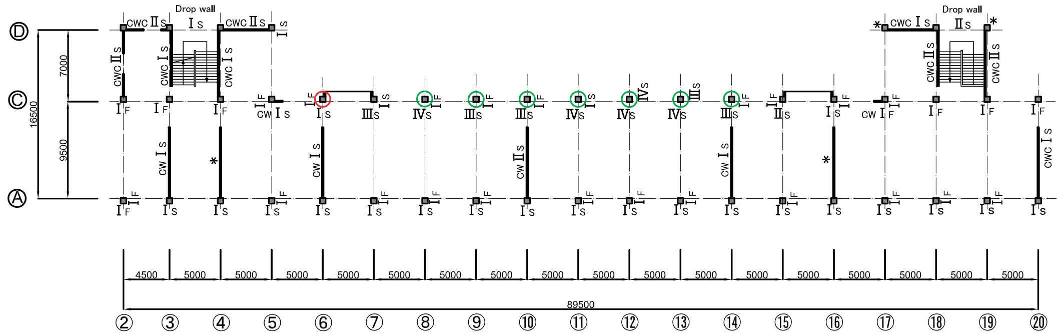
Photo. 1 North side view of the class-room building

The first, second and the penthouse stories of the building were retrofitted seismically in 2009 by steel-braced frames in a longitudinal direction, which consisted of steel members with a H-shaped section specified as H-200x200x8x12. A R/C shear wall with an opening was installed to the twentieth frame at the first story in a transverse direction to dissolve a pilotis frame with shear walls in upper stories. No retrofits were conducted at the third story in a longitudinal direction because the seismic capacity evaluation judged that sufficient lateral capacity was already provided for the story. Each

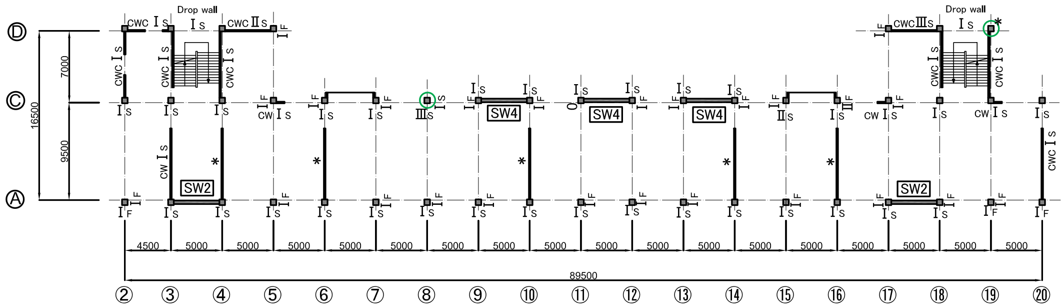


Penthouse

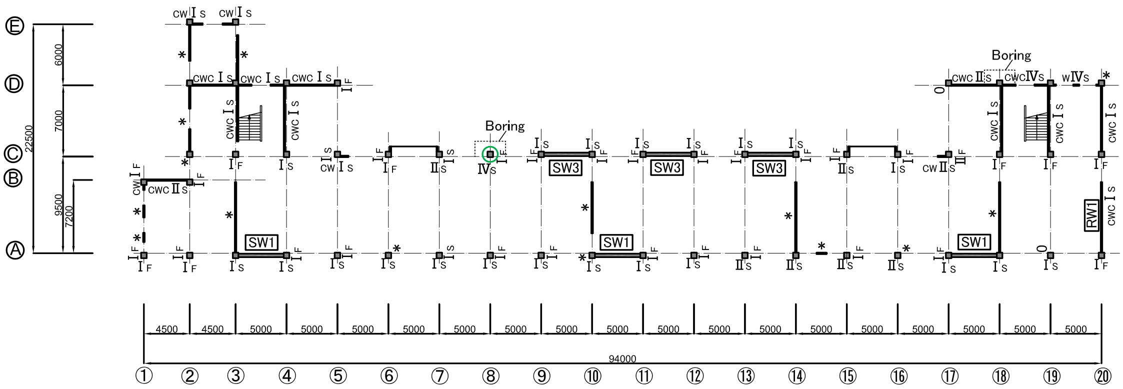
Legend:
SW1 to SW5 : steel-braced frame
RW1 : R/C shear wall placed after retrofit
Roman numerals I to V : damage rate of members
Suffix s : damage by shear crack or shear failure
Suffix f : damage by flexural crack
* : not observed members at field survey



Third Floor

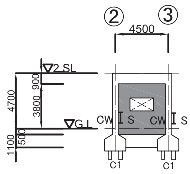


Second Floor

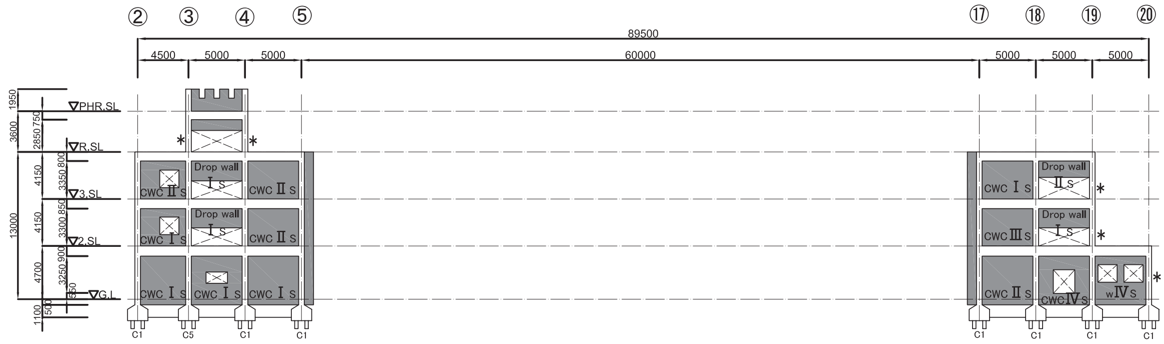


First Floor

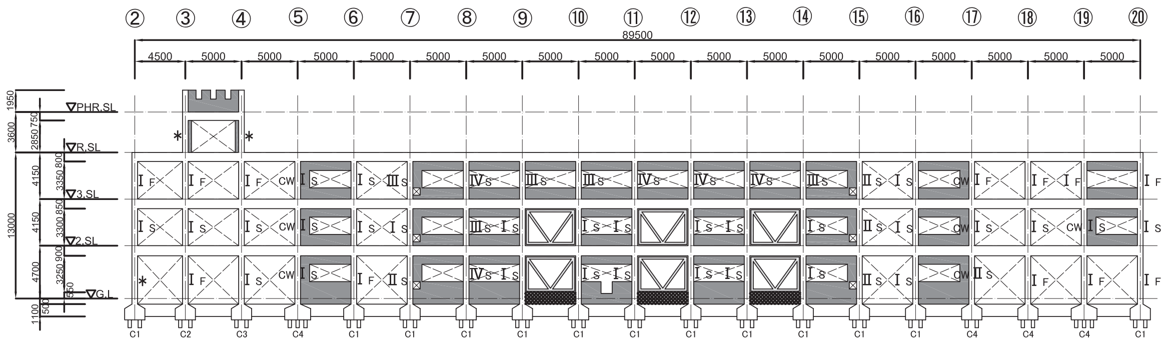
Fig. 2 Floor Plans of the class-room building



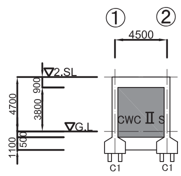
E frame



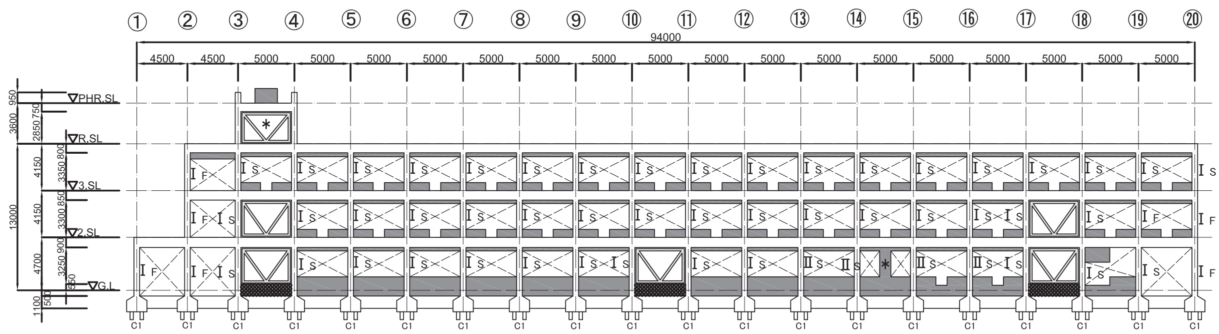
D frame



C frame



B frame



A frame

Fig. 3 Frame elevations of the class-room building

Designation	C1	C2	C3	C4	C5
Top	20-D22	24-D22	24-D22	20-D22	20-D22
Bottom	24-D22	30-D22	32-D22	28-D22	26-D22
Hoop	2-9Φ@165				
Width & Depth	550×500				

Fig. 4 Column sections at first story

Photo. 2 Shear failure of columns in C frame at third story

Photo. 3 Shear failure of column adjacent to unit bay strengthened by steel-braced frame

Photo. 4 Differential settlement of R/C floor slab at first floor

floor plans and frame elevations after the seismic retrofit are shown in Fig.2 and Fig.3 respectively.

Typical columns designated as type C1 at the first story, exhibited in Fig.4, had a rectangular cross section with depth of 500 mm and width of 550 mm to a longitudinal loading direction. Longitudinal steel bars of 24-D22 and 20-D22 were placed at a bottom and a top of the column respectively although the design drawings at construction pointed out that plain bars should be used as a column longitudinal bar. The hoops were 9 mm plain bars spaced at 165 mm on centers along the mid-height with a 90-degrees hook. Specified concrete compressive strength was 18 N/mm^2.

Damage of class-room building

The damage rate of columns and walls is shown in plans of Fig.2 and framing elevations of Fig.3 by Roman numerals. Damages of columns and differential settlement of floor slab at the first story are shown in Photo.2 to 7. Four R/C columns at the third story, where no retrofits were conducted because of having sufficient seismic capacity, failed in shear by bi-directional horizontal loading due to the earthquake motion. Some columns adjacent to a unit bay strengthened by a steel-braced frame also exhibited severe shear cracks of Grade 3 or shear failure of Grade 4 at the first and second stories shown in Photo.3. Some boundary R/C columns of a steel-braced frame had minor or moderate shear cracks at the first and second stories. A few R/C columns at the south frame designated A frame, whose clear height was larger than that of the north frame designated C frame, also had minor shear cracks of Grade 1 or 2 at the first story.

The first floor slab in a toilet room, which was projected to the north side of the building, sank 290 mm due to local settlement of the building since the whole building did not incline. Concrete brick walls and R/C shear walls were alternately placed between the class-rooms in a transverse direction.

Photo. 5 Shear failure of C-13 column at third story

Photo. 6 Shear failure of C-8 column at second story

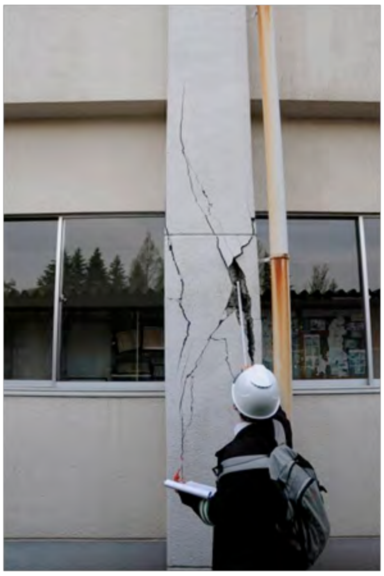


Photo. 7 Shear failure of C-8 column at first story

(a) Shear failure and bar buckling (b) Break-off and horizontal slip at pile top

Photo. 8 Damage of piles

Many concrete bricks fell off due to out-of-plane loading by the earthquake motion because the bricks were not anchored to a R/C girder at a top of the wall. Secondary walls beside a staircase failed heavily in shear.

Residual seismic capacity after the earthquake was estimated to be 0.78 times the seismic capacity before the earthquake for the third story in a longitudinal direction, which for the first and second stories was greater than that for the third story, i.e., 0.80 times and 0.87 times the original capacity respectively. In this estimation of the residual seismic capacity, the damage grade for a steel-braced frame was assumed to be similar to that for a shear wall with boundary columns at both ends, which seemed to underestimate the residual seismic capacity. Thus the damage of the building was classified into the moderate level at the third story without any seismic rehabilitations.

Residual seismic capacity in a transverse direction was estimated to be 0.95 times, 0.95 times and 0.83 times the capacity before for the first, second and third story respectively.

Additional survey

Additional field survey to the school was carried out in November, 2011, eight months later from the

Table 1 Concrete compressive strength by drilled cores (unit in N/mm^2)

Story	Average X	Standard deviation σ	X-σ/2
3	23.4	3.7	21.6
2	25.6	2.2	24.5
1	23.7	11.2	18.1

Table 2 Seismic capacity evaluation by second level procedure

(a) Before seismic retrofit

(a-1) Longitudinal direction

Story	*Eo*	*SD*	*T*	*Is*	*CTSD*
Penthouse	0.31	1.00	0.96	0.30	0.31
3	0.81	0.90		0.70	0.73
2	0.50	0.90		0.43	0.45
1	0.56	0.90		0.48	0.50

(a-2) Transverse direction

Story	*Eo*	*SD*	*T*	*Is*	*CTSD*
Penthouse	1.84	1.00	0.96	1.77	1.84
3	1.40	0.90		1.21	1.26
2	0.93	0.90		0.81	0.84
1	0.84	0.90		0.72	0.75

(b) After seismic retrofit

(b-1) Longitudinal direction

Story	*Eo*	*SD*	*T*	*Is*	*CTSD*
Penthouse	1.17	1.00	0.96	1.13	1.17
3	0.81	0.95		0.73	0.76
2	0.81	0.95		0.73	0.76
1	0.85	0.95		0.78	0.81

(b-2) Transverse direction

Story	*Eo*	*SD*	*T*	*Is*	*CTSD*
Penthouse	1.84	1.00	0.96	1.77	1.84
3	1.35	0.95		1.23	1.28
2	0.91	0.95		0.83	0.87
1	0.88	0.95		0.80	0.83

earthquake. It was found that the damage grade of many columns and walls more progressed than that at the survey in the end of April, 2011. This was caused by the after shocks of which the JMA intensity were equal to or less than 4; the moderate shock. Estimated residual seismic capacity after the earthquake declined to 0.65 times from 0.78 times the capacity before for the first story in a longitudinal direction. The damage of the building developed into the almost severe level. Columns inclined to a random direction with an inclination angle of 0.4 % to 0.6 %.

Boring investigation to some R/C piles of the class-room building was simultaneously conducted by the town hall. Four piles driven at a north side of the building were revealed under the eighth column in C frame and the eighteenth column in D frame shown in Fig.3. A top of one pile under the eighteenth column in D frame failed in shear and longitudinal bars buckled as shown in Photo.8(a). The other pile broke off and slipped horizontally as shown in Photo.8(b). Shear and flexural cracks and spall-off of concrete were observed at a top of the piles under the eighth column in C frame. The damage level of the pile foundation was regarded as severe from the observation.

It seems that R/C piles suffered severe damage above-mentioned by the main shock in March 11, 2011 and several columns inclined gradually during eight months after the main shock.

SEISMIC CAPACITY EVALUATION FOR CLASS-ROOM BUILDING

The detailed survey and the seismic capacity evaluation of the building were carried out by the town hall in 2009 before the earthquake. Concrete compressive strength which was obtained by a cylinder test using drilled cores from the building is shown in Table 1. Concrete compressive strength taken by subtracting a half of the standard deviation from the average compressive strength among three cores ranged from 18.1 N/mm^2 to 24.5 N/mm^2, exceeding the specified compressive strength of 18 N/mm^2 for all stories.

The seismic capacity indices *Is* of the building before and after the seismic retrofit were computed according to the second level procedure in "Standards for seismic performance evaluation of existing reinforced concrete buildings," which was issued from Japan Building Disaster Prevention Association (JBDPA) in 2001, and are shown in Table 2, where *Eo* is the basic seismic capacity index, S_D is the structural design index, *T* is the time index, *Is* is the seismic capacity index obtained as the product of indices *Eo*, S_D and *T*, and C_TS_D is the cumulative strength index determined corresponding to the ductility index *F* multiplied by the index S_D. In the second level procedure, flexural and shear strengths of columns and walls are evaluated using a set of practical equations. Beams at a top and bottom of a column are assumed to be infinitely stiff and strong.

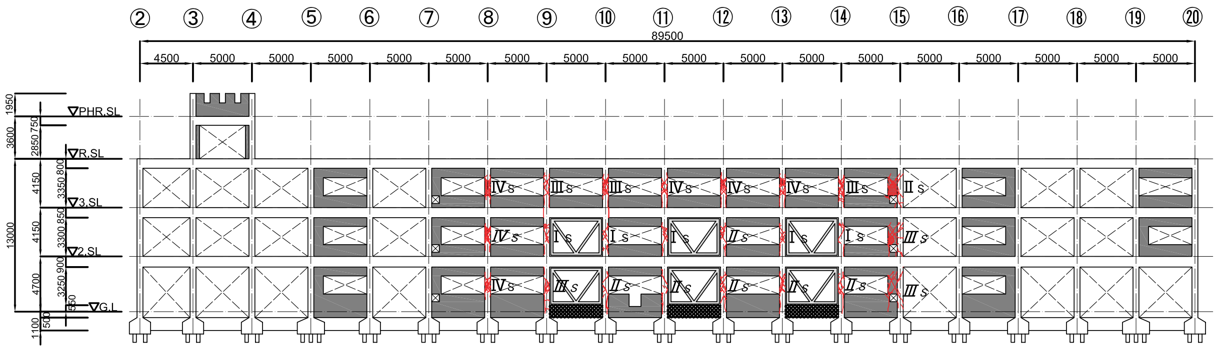
Floor weight was evaluated to be 12.0 kN/m^2 for the first story, 11.8 kN/m^2 for the second story, 9.0 kN/m^2 for the third story and 13.1 kN/m^2 for the penthouse. Specified compressive strength of 18 N/mm^2 for concrete and yield strength of 344 N/mm^2 for longitudinal bars of columns were used to evaluate the shear and flexural capacity for vertical members. The structural design index S_D was improved to 0.95 from 0.90 by widening a seismic joint between the building and the special class-room building. The time index *T* was evaluated to be 0.96. The story index, which relates the single-degree-of-freedom response to that of multi-story structures, was assumed to be one-third for the penthouse and n+1 divided by n+i for other stories, where n is the number of stories and i is the number of the i-th story concerned.

The seismic capacity index *Is* in a longitudinal direction for the original building was 0.48 for the first story, 0.43 for the second story and 0.30 for the penthouse, which were less than the standard requirement of 0.70. Therefore, these stories were retrofitted by a steel-braced frame consisting of members with a H-shaped section specified as H-200x200x8x12. The seismic capacity index *Is* increased by the retrofit to 0.78 from 0.48 for the first story, 0.73 from 0.43 for the second story and 1.13 from 0.30 for the penthouse. The seismic capacity index *Is* for the third story without any seismic rehabilitation was 0.70 almost equal to that for the second story after the retrofit. Judging from both vertical distributions over the stories of the seismic capacity indices *Is* and the cumulative strength indices C_TS_D modified by the structural design index, it is not necessarily inadequate that no seismic retrofit was provided to the third story.

A pilotis column with a short one-sided wall before the retrofit, located at a cross point of the grid line A and 20 for the first story, was estimated to be critical for axial capacity in a transverse direction because the column can not sustain axial compressive force induced by lateral loading during earthquake attacks. Therefore, the seismic capacity index *Is* for the first story in a transverse direction for the original building was evaluated to be 0.72, which was rather less than that for other stories as shown in Table 2(a-2). Thus installation of a R/C shear wall with an opening at the twentieth frame for the first story was conducted to cancel the pilotis column, and enhanced seismic lateral capacity of the building in a transverse direction, as is indicated by the seismic capacity index *Is* of 0.80 after the seismic retrofit.

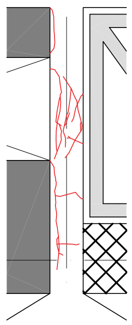
BEHAVIOR OF STEEL-BRACED FRAME

Crack patterns of columns neighboring to three steel-braced frames in C frame are shown in Fig.5. Shear cracks with a width of 0.2 mm to 1.5 mm were observed by the additional survey for the boundary R/C columns of steel-braced frames at the first and second stories shown in Photo.9. This indicates that behavior of two-story steel-braced frames during the earthquake was dominated by such a manner that lateral shear force was shared between two boundary R/C columns and diagonal steel chords in each unit. Moreover, flexural cracks developed at critical sections of boundary beams and transverse beams framing into the steel-braced frame and shear cracks did in a plastic hinge region of

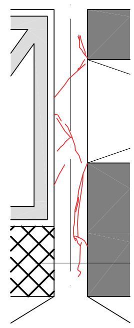


Crack patterns in C frame

Italic Roman numerals : damage rate observed at November 2011



Crack pattern of ninth column



Crack pattern of tenth column

Fig. 5 Crack patterns of columns neighboring to steel-braced frames in C frame

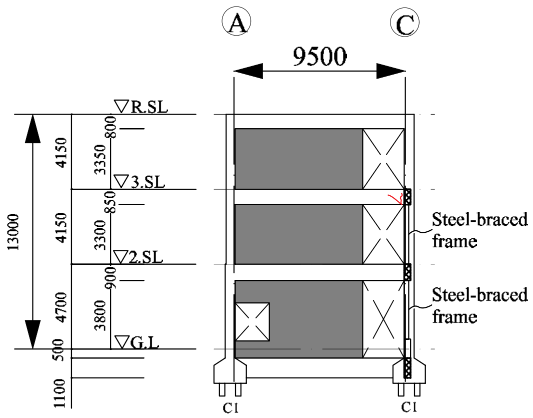


Fig. 6 Frame elevation of tenth frame in transverse direction

Photo. 9 Shear cracks of C-14 column at first story

the transverse beam in the third floor, which was a short span beam in the tenth frame for a transverse direction because a large opening was placed on a shear wall for a passage way as illustrated in Fig.6. This is a symptom that these beams resisted an uplift rotation of the rigid body consisting of a two-story steel-braced frame and two boundary R/C columns.

Predicted lateral strength was 2926 kN for a strengthened unit bay consisting of a steel-braced frame and both boundary R/C columns at the first story in C frame, which was computed on the assumption that one of diagonal steel chords in a steel-braced frame yields in tension and the other buckles in compression with shear failure of both boundary R/C columns. On the other hand, predicted lateral strength on the assumption that a foundation of the rigid body above-mentioned uplifts and rotates was 2968 kN, which is slightly greater than the former strength of 2926 kN. This computation result agrees approximately with the observation on the field survey.

Steel-braced frames for the first and second stories were installed into the C frame removing both drop and spandrel walls as shown in Fig.5. Drop and spandrel walls adjacent to a steel-braced frame, however, remained without a seismic slit so as to separate a short boundary column from these secondary walls. It seems that this resulted in greater enhancement of the lateral stiffness in the first and second stories than expected for the seismic retrofit design. Since the lateral inter-story displacement for the third story became larger during the earthquake than that for the first and second stories, eventually short columns with a shear span ratio of 1.6 in C frame failed in shear in the third story. This points out that the estimation of the story stiffness according to the current standards for seismic performance evaluation of retrofitted R/C buildings, i.e., the structural design index S_D, was likely to be inadequate when a seismic retrofit was carried out using steel-braced frames in such a manner that adjacent drop and spandrel walls were in service without any seismic slits, attributed to more stiffen the steel-braced frame.

CONCLUSIONS

The reinforced concrete class-room building with three stories, which was retrofitted by steel-braced frames for the first, second and the penthouse stories, suffered moderate damage for the third story, where no seismic retrofits were conducted and four columns failed in shear subjected to the 2011 East Japan Earthquake. Shear failure also occurred in a few columns at the first and second stories with seismic retrofits. The vertical distribution of the seismic capacity indices in a longitudinal direction, which seems to be adequate, can not explain the reason why shear failure of columns concentrated on the third story. It is probable that the actual lateral stiffness in the first and second stories after the retrofit was greater than that predicted according to the evaluation standards because existing drop and spandrel walls adjacent to a steel-braced frame remained without any seismic slits. In other words, irregular story stiffness distribution over the height of the building, generated by the seismic retrofit except for the third story, resulted in the concentration of a large deformation to the third story and caused shear failure of some columns.

R/C piles under footings of the building suffered severe damage such as shear failure, buckling of a longitudinal bar, and many cracks. An inclination of columns which was not observed at the survey in April, 2011 grew gradually during eight months.

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