

SEISMIC DAMAGE OF A NINE-STORY RC RESIDENTIAL BUILDING IN SENDAI DESIGNED BY OLD SEISMIC CODES

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ABSTRACT: Structural damage is reported for a nine-story reinforced concrete residential building which experienced the 2011 Tohoku-chiho Taiheiyo-oki Earthquake in Sendai. The structural system is moment resisting frames in longitudinal direction and coupled shear wall system in the transverse direction. The failure mode of beam-column joints are compared to the prediction by the current seismic design guidelines. The validity of their provisions on joint shear strength and the other specifications are discussed.

Key Words: reinforced concrete, residential building, seismic retrofit, beam-column joint, shear strength, structural damage, seismic codes, seismic design

INTRODUCTION

Investigation on damage of buildings following strong earthquakes provides an invaluable opportunity for structural engineers to validate the current seismic codes as well as performance based design methodology having been developed for the control of seismic damage. Types of failure of members, such as flexural or shear are concerns of structural engineers who decide the detailing of structural members. So keeping records of damage and structural detailing of members are of great importance. In this report, a case of a nine-story reinforced concrete (RC) residential building in Sendai (N-building) are described. The building was constructed in 1969 and suffered extensive damage by the 2011 Tohoku-chiho Taiheiyo-oki Earthquake on March 11 and its aftershocks. The overviewing photo of the building is shown in **Fig. 1**.

Although this building escaped from a structural collapse, the all the leaseholder in the offices at the first and the second floor and the apartments at the other floors had evacuated. Damage occurs to structural members including columns, beams, shear walls and beam-column joints. They suffered

extensive damage particularly at the first story. Extensive shear cracks, concrete crush and buckling of longitudinal reinforcements were observed in many columns and shear walls. Many visible diagonal cracks were found from outside on the exterior beam-column joints at the fourth floor through 9th floor. Major shear cracks on coupling beams were also found in transverse direction and non-structural reinforced concrete walls in longitudinal direction.

The extent and distribution of damage are reported and some preliminary result of correlation with seismic vulnerability assessment done in 2011 are discussed. The failure mode of beam-column joints are compared to the prediction by the current seismic design guidelines. The validity of the recent provisions for joint shear strength are discussed.

DAMAGED BUILDING

The location of the N-building is in Taihaku ward, Sendai City. It is the neighborhood of JR Nagamachi Station which is about 5 km south of JR Sendai Station. Seismic intensity in JMA scale was 5+ (five major) and 5- (five minor) for the main shock on Mach 11 and the largest aftershock on April 7 respectively. No death or injury is reported due to this building. Taihaku ward is commercial and residential area and densely populated. The death toll in Taihaku ward is 8, while the population is 218,704 (as of April 1, 2010). So the percentage of death is very small. There was no tsunami inundation in Taihaku ward.

The building site is flat and on an alluvial plain. Design and construction was completed in 1969. It experienced 1978 Miyagi-ken oki Earthquake as well as recent major earthquakes in 2003 and 2005. It is reported that it had slight damage due to the past earthquakes but has been functional. So the building has been continuously used. No repair work nor major seismic retrofit has been done to this building before March 11, 2011.

The elevations and the plans are shown in **Figs. 2** through **6**. The building is a nine story reinforced concrete building. The use of the first and second floor is municipal office and medical office, while the third floor through ninth floor are apartment units for rent. A corridor are placed on each floor above 3rd floor at the mid bay of the transverse direction which accommodates for the access to each apartment unit.

The story height is 3.6m, 3.3m and 2.75-2.6m for the 1st, the 2nd and the the upper floor respectively. It has a one story in the basement and a three-storied penthouse. The height of the roof floor level is 26.15 meter from the ground level. The building has a foundation supported by reinforced concrete piles of 5 meter length. The type of supporting soil is of category II designated by the building standard law enforcement orders.

The structural system consists of four regular moment resisting frames in longitudinal direction with seven bays with span length of 5.4 m. The transverse direction are coupled shear walls at third story and above with wall length of 8 m and coupled beam length of 2m. The structural plan and elevation is regular at the third story and above, whereas the floor of the first and second story are extended one span to outside at three sides and provide open space for the municipal or medical offices. There are some reinforced concrete shear walls and non-structural concrete block masonry walls at these two stories but the direction and the placement of the walls are not in consistent to the placement of the coupled shear walls above the 3rd story.

The types of structural members are reinforced concrete (RC) but steel reinforced concrete (SRC). SRC is also used. SRC is a type of member with composite section of structural steel lattice encased in reinforced concrete. The beams and columns of the first story (above the 1st floor level) through the third story are SRC members. The basement and the fourth story and above as well as the penthouse are of RC construction. The girder for the exterior frame in longitudinal direction is of wall girder type, i.e. with narrow and deep section for the third story and above.

The concrete is normal weight with design compressive strength of 18MPa or 21MPa. The longitudinal reinforcement is of Grade SD35 with specified yield point of 345MPa. The concrete core

sample tests for compressive strength were carried out in 1999, the concrete retained the design strength and were in sound condition reportedly.

SEISMIC EVALUATION

A seismic vulnerability assessment of the building was carried out in 1999. Then reassessment was done in 2011 to consider the influence of 2003 Sanriku-minami Earthquake and 2005 Miyagi-ken Nambu Earthquake preparing for seismic retrofit work. The assessments were based on the Seismic Evaluation Standards¹⁾ of the Japan Building Disaster Prevention Association (BDPA).

The method of the Japanese Seismic Evaluation Standards are briefly explained here. It provides a predefined procedures to calculate the value of seismic index (I_s), which is a value given for each story of a building. The I_s value for the first story is a sort of base shear coefficient at collapse mechanism; calculated as the sum of the lateral capacity of the lateral resisting elements such as columns and shear walls which support the weight above the story. But the lateral capacity of each element are factored by a prescribed modifier considering the ductility. The factor ranges from approximately 0.8 to 3.0 given as tabulated values or calculated based on its a) dimension, b) type of member, c) shape factor such as shear-to-span ratio, d) failure type and e) the ratio of shear capacity to shear strength demand. So, the I_s value is intended to define such that seismic vulnerability could be represented in terms of maximum base shear coefficient of a linearly elastic system subjected to a base motion to which the story of the building remains within life safety limit. The I_s values of 0.6 or less is a recommended criteria of seismic retrofit otherwise special performance objectives are preferred. There are three options of modeling preciseness for the analyses. Level I is the simplest and the lateral capacity is roughly estimated based on the type and horizontal sectional area of lateral resisting members. Level III is the most sophisticated considering full failure mechanism of a structural system, whereas Level II is a compromised version of Level III by neglecting the beam sway mechanism.

The seismic index I_s for the N-building calculated in 2011 are listed in Table 1. They adopted the modeling of level II for the analysis. The results of the assessment of the N-building are summarized as follows. For the longitudinal direction (X-direction), the I_s value for 2nd through 9th stories are less than 0.60 while the I_s value for the first story is larger than 0.60. One of the reasons of the lower I_s value for the upper floors is that the most of the columns in Y_1 and Y_4 frames are short column with shear-to-span ratio of 1.6 and categorized as non-ductile to which modification factors of 0.8 is applied. The columns in Y_2 and Y_3 frames are identified as shear failure column because they have reinforced concrete monolithic wing walls and categorized as non-ductile due to small shear-to-span ratio. The I_s value for the first and the second story is relatively larger. Most of the columns are of shear failure type. But RC shear wall are placed randomly and provide large lateral capacity. The shear wall in the second floor provide eccentric lateral stiffness and penalty factor S_D is applied; i.e. a penalty factor considering the irregularities of stiffness eccentricity. Thus the I_s value for the 2nd story is lower than that of the first story.

For the transverse direction (Y-direction), the I_s value for the first, second and fourth story are less than 0.60 whereas the other stories have the I_s value larger than 0.6. Most of the columns are of shear failure type and eccentric placement of shear wall are attributed to the lower I_s value for the first and the second story. Combination of high shear force and high tensile axial force are expected to the columns locating beneath the boundary column of the rocking shear walls in the transverse direction from 3rd to 9th story. In addition to that, the SRC column of the first story have deficiency that the bottom of the steel shape is not continuous into the foundation but just ends at the first floor level. As the bottom section of the column has only longitudinal reinforcement, damage concentrates to this locally weak section when it is subjected to bending and high axial tension.

It is summarized by the discussion above that the most probable scenario of seismic damage derived from the seismic vulnerability assessment were; (1) the shear failure of column above the 3rd story in the longitudinal moment frames and (2) local damage of SRC column at the first story in the

transverse direction.

OBSERVED DAMAGE TO STRUCTURAL COMPONENTS

The damage of members were photographed and each member was graded into five levels. The grading was based on the maximum crack width, the extent of concrete crushing as well as buckling of longitudinal steel. The criteria for the grading are explained in **Table 2**.

The evaluated grade of the columns and walls are shown in **Figs. 2** through **6**. The first story column at X_1 - Y_1 suffered the damage of the severest grade of V. The **Fig. 7** shows the photo of the column. Cover concrete spalled off at the bottom of the column and the longitudinal bars are buckled. Flexural yielding at the base occurred and high axial force variation due to rocking wall at the above level cause the severe damage. Possibility of this problem had been already pointed out by the vulnerability assessment done in 2011. Story drift at the first story seems to be the largest because the number of columns with grade III or severer were observed only for the first story. This interpretation may be endorsed by the fact that most of the gross fracture were observed only at the first story. The damage of columns at the second story is not significant.

The other severe damage was observed on the boundary beams which consist of coupled shear walls in transverse direction starting from the 4th floor level through the roof floor level. Damage grade of the beams were III from the fourth floor through seventh floor as shown in the elevation of X_1 Frame in **Fig. 6**. The section of the beams is 350mm wide and 600mm deep where the longitudinal reinforcements are 5-D22 bars for top and 3-D22 bars for bottom. The stirrups are 9mm diameter plain bar with spacing of 200mm. No shear failure was not observed but the stirrups were not sufficient for control the residual crack width less than 1.0mm.

Some of the exterior RC/SRC columns in Y_4 frames suffered a shear failure at the third, fourth and fifth story with damage grade of IV. **Fig. 9** shows the shear cracks on the columns.

By the image on the photo of the exterior side of Y_4 frames shown in **Fig. 10**, it is revealed that some beam-column joints have significantly cracked and concrete crushing was also found. The cracks originated from the corner of the window opening and proceeded into the center of joint. Crushing of concrete as well as tile peeled off at the crossing point of diagonal cracks were evident. It is presumed that longitudinal direction was imposed very large story drift, because the non-structural RC partitions along the corridors in the longitudinal direction between Y_2 frame and Y_3 frame had been significantly failed in shear.

CORRELATION OF DAMAGE AND SEISMIC VULNERABILITY ASSESSMENT

The N-building has no tilting remained and overall damage level of the building is moderate. But the building had been entirely evacuated in June 2011 when the author visited this building site. There is no plan of repairing at the time of this paper submission. Considering the seismic intensity of 5+, the damage level of this building is relatively higher than the similar buildings around this site in this area. So this building should be more vulnerable than the other similar buildings.

The distribution of damage in vertical direction is not in good correlation to the I_s distribution. The I_s value of the third story through 8th story in the longitudinal direction is minimum in this building and I_s value is relatively larger for the first story. But damage is severer particularly to the first story. This is partly due to the poor detailing of the base of first story column of SRC.

The coupled shear wall system in transverse direction seems to perform well except poor control cracking performance of coupling beams. Although the shear failure of column above the 3rd story in the longitudinal moment were predicted, the reality is the shear failed column are observed at the third and fourth story only. This may be attributed to the neglect of the lateral resisting contribution of auxiliary components of RC non-structural wall components abundant in longitudinal direction.

FAILURE MODE OF BEAM-COLUMN JOINT SUB-STRUCTURE

The seismic evaluation standard¹⁾ have no predefined equations to consider beam-column joints with poor performance. But as shown in **Fig. 5**, diagonal cracks were observed at many beam-column joints in the longitudinal frame of Y₄. So, the failure type is evaluated by the current seismic design.

The sections of beam-column joints are shown in **Fig. 8** for the beam-column joint at fifth floor level and seventh floor level taken from the structural drawing. They are crucial beam-column joints provided with joint hoops with common spacing to the hoops in the column according to the structural drawing. The flexural strength, shear strength and joint shear strength are calculated based on the design strength of material and design equations as follows,

Flexural ultimate moment M_u of a beam and column section are calculated by a equation (1) and (2) respectively,

$$M_u = 0.9a_t f_y d \quad (1)$$

$$M_u = 0.8a_t f_y D + 0.5ND \left(1 - \frac{N}{bDF_c} \right) \quad (2)$$

where, a_t : sum of the sectional area of tensile rebar in the beam, f_y : specified yield point of the tensile rebar, and d : effective depth of the beam, D : full depth of the column, N : axial force in compression, and F_c : concrete compressive strength. The axial force of the column is calculated considering the dead load of tributary area for gravity load.

The shear strength of the beams and columns are calculated using the equations for shear adopted in the AIJ Guidelines for RC Buildings²⁾.

The joint shear demand V_j is calculated by,

$$V_j = a_t f_y + a'_t f_y - V_c \quad (3)$$

where, a_t : sum of the sectional area of tensile rebar for positive moment in the beam, a'_t : sum of the sectional area of tensile rebar for negative moment in the beam, f_y : yield point of the tensile rebar, and V_c : column shear at the ultimate flexural strength of beam at the column face.

The joint shear capacity V_{ju} in N is calculated by the equation²⁾,

$$V_{ju} = 0.8\kappa\phi F_c^{0.7} b_j D \quad (4)$$

where, κ : shape factor (=1.0 for crucial beam-column joints), ϕ : modification factor of transverse beam (= 0.85 for joint covered with beams at three sides), F_c : concrete compressive strength in MPa, b_j : effective width of the joint in mm and D : full depth of the column in mm.

Table 3 lists the story shear capacities in kN for the RC beam-column joints at X₅-Y₄ (common to X₅-Y₄), corresponding to a) shear failure of column b) shear failure of beam, c) shear failure of beam-column joint, d) column yielding mechanism, and e) beam yielding mechanism. The flexural capacity of the beams are always the smallest of the capacities. This means the analysis by modeling Level II is inappropriate to this building. The ratios of joint shear demand to joint shear capacity if more than 2.0. Thus these beam-column joint is lightly reinforced joint and has been considered to be safe to joint shear failure. But in reality, the joint shear failure were observed. It is obviously inconsistent with the joint shear design currently adopted to the seismic design codes for RC structures.

Three dimensional shaking table tests of four story full scale RC building at E-Defense has

confirms the importance of beam-to-column strength ratio recently³⁾. RC beam-column joints with the column-to-beam strength ratio closed 1.0 showed severe joint shear failure than beam yielding. The series of tests of the author on beam-column joints by authors also revealed the joint with beam-to-column strength ratio of 1.0 to 2.0 showed joint failure. The author are developing the theoretical model to explain the failure of beam-column joint with the strength ratio of 1.0.⁴⁾ The calculated value of column-to-beam strength ratio listed in Table 3 shows that the ratio is in the range of 1.26 to 1.48 if the column capacity is calculated using the axial force for column tributary area excluding the tributary area of the wall. It is confirmed that the column-to-beam strength ratio of the beam-column joint showing severe diagonal cracking ranges between 1.0 and 1.5 which is vulnerable to joint shear failure predicted by recent research results.

CONCLUDING REMARKS

The extent and distribution of damage are reported for a nine-story reinforced concrete residential building in Sendai (N-building) designed and constructed in 1969 by old design codes. It is revealed from the investigation that the N-building escaped from a collapse and no tilting remained.

The building had life safety performance and overall damage level of the building was moderate. Considering the seismic intensity of 5+, the damage level of structural members of this building is relatively higher than the similar buildings around this site in this area.

The first story SRC column subjected to bending and high axial tension by the rocking wall above showed the severest damaged in this building. It is due to the deficiency of steel lattice that is not continuous nor embedded into the foundation but just ends at the first floor level which allowed concentration of local deformation adjacent to the weak section at the bottom of the column.

The failure of beam-column joints were observed to the joints which conforms to the current seismic design codes. The calculated margin of joint shear strength is 2.0 or more for the beam-column joint, whereas the value of column-to-beam strength ratio is in the range of 1.26 to 1.48. It is confirmed that the column-to-beam strength ratio between 1.0 and 1.5 is vulnerable to joint shear failure.

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Table 1 Values of Seismic Index I_s

Story	Longitudinal direction	Transvers direction
9	0.37	1.49
8	0.27	1.04
7	0.23	0.82
6	0.20	0.70
5	0.21	0.62
4	0.19	0.51
3	0.20	0.71
2	0.44	0.44
1	0.62	0.39

Table 2 Damage grading criteria of RC components

Damage grade	Criteria
0 No damage	No damage
I Slight	Structural concrete cracking of width less than 0.2mm
II Minor	Structural concrete cracking of width larger than 0.2mm and less than 1.0mm.
III Moderate	Structural concrete cracking of width larger than 1.0mm and less than 2.0mm.
IV Major	Structural concrete cracking of width larger than 2.0mm, with cover concrete spalling and visible reinforcement
V Severe	Cover concrete spalling off, with some concrete crushes and longitudinal reinforcement buckling

Table 3 Calculated story shear at mechanism of beam-column joints, column-to-beam strength ratio and joint shear strength margin

	Shear failure in kN			Flexural hinge in kN			Column-to-beam strength ratio		Joint shear strength margin*
	column	beam	joint	column case 1*	column case 2*	beam	case 1	case 2	
9FL	544.4	858.7	863.4	522.5	396.7	231.7	2.25	1.71	3.73
8FL	555.0	929.1	984.0	650.8	454.0	320.2	2.03	1.42	3.07
7FL	589.2	1043.2	1112.3	751.5	496.7	335.0	2.24	1.48	3.32
6FL	799.7	1148.5	1150.1	906.6	574.8	432.2	2.10	1.33	2.66
5FL	907.8	1162.5	1624.0	1082.9	664.0	528.3	2.05	1.26	3.07

* case 1: axial force of column is considering tributary area for both column and wall as shown in the figure below, case 2: axial force of column is considering tributary area for column only, Joint shear strength margin = joint shear demand V_j / joint shear capacity V_{ju}

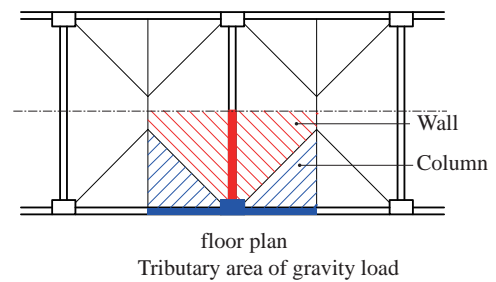


Fig. 1 Overview of the N-Building from the west side

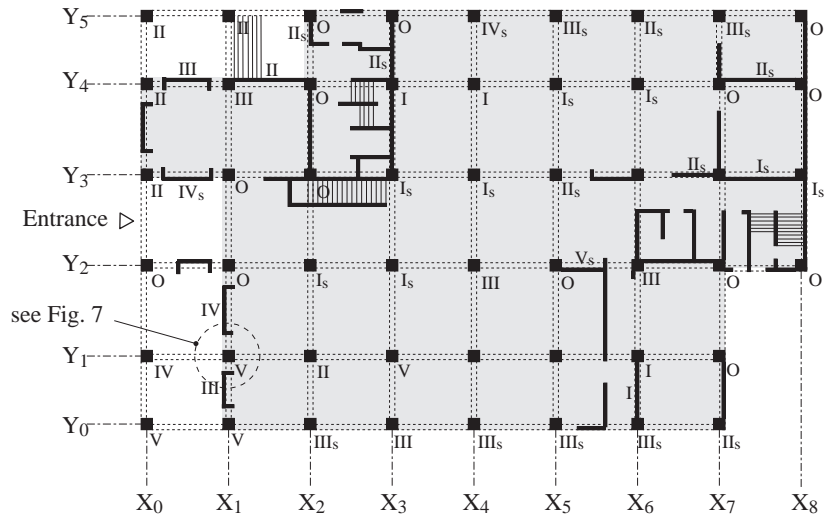


Fig. 2 Plan of 1st floor and damage grade of structural members

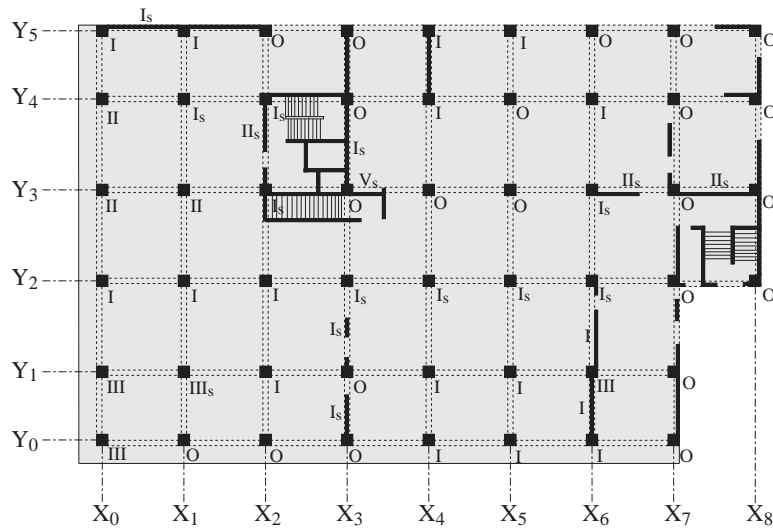


Fig. 3 Plan of 2nd floor and damage grade of structural members

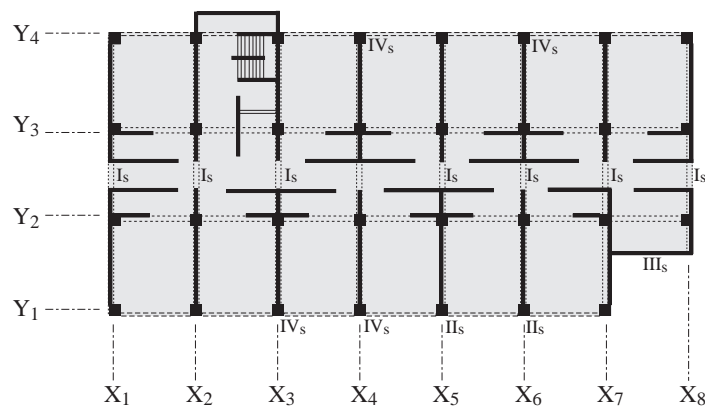


Fig. 4 Plan of 3rd floor and damage grade of structural members

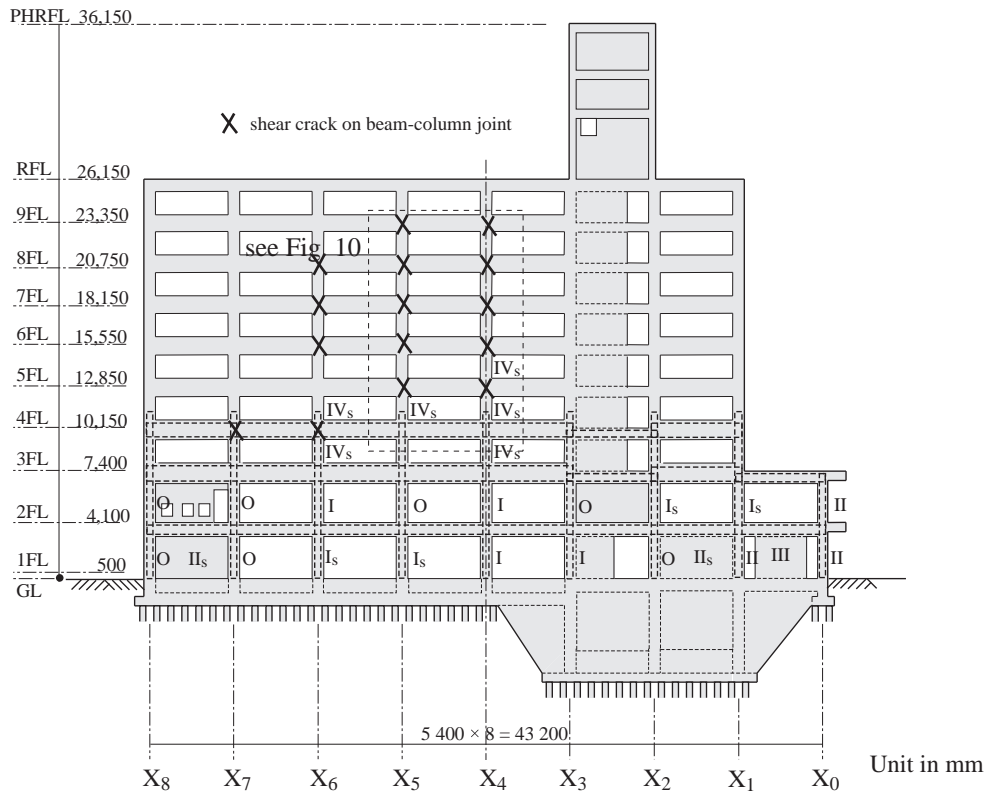


Fig. 5 Elevation of Y₄ Frame and damage grade of structural members

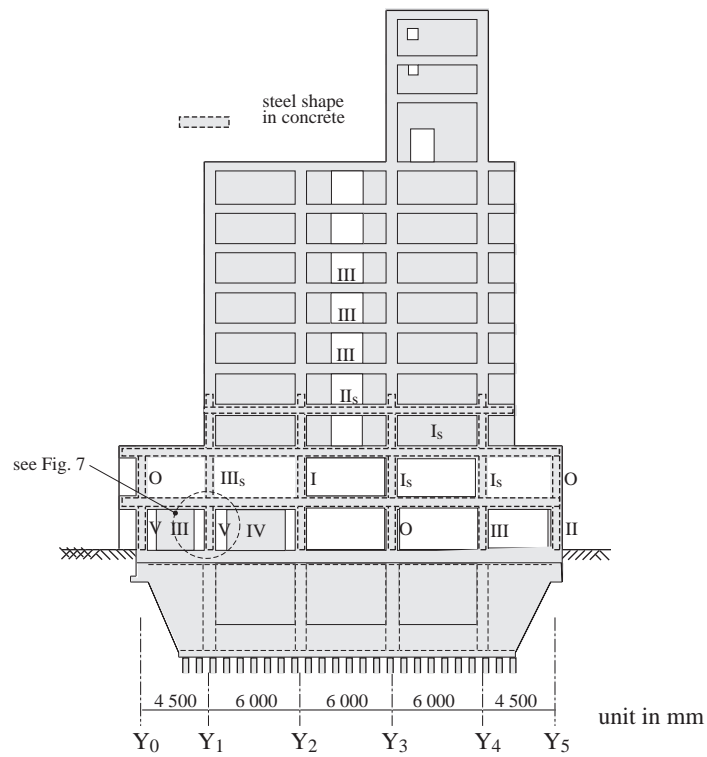


Fig. 6 Elevation of X₁ Frame and damage grade of structural members



Fig. 7 Column X_1-Y_1 at 1st floor with concrete crushing and rebar buckling

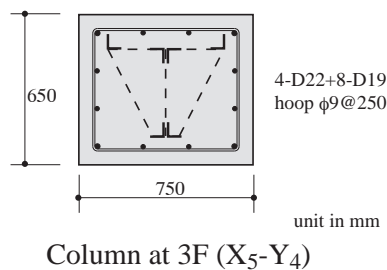
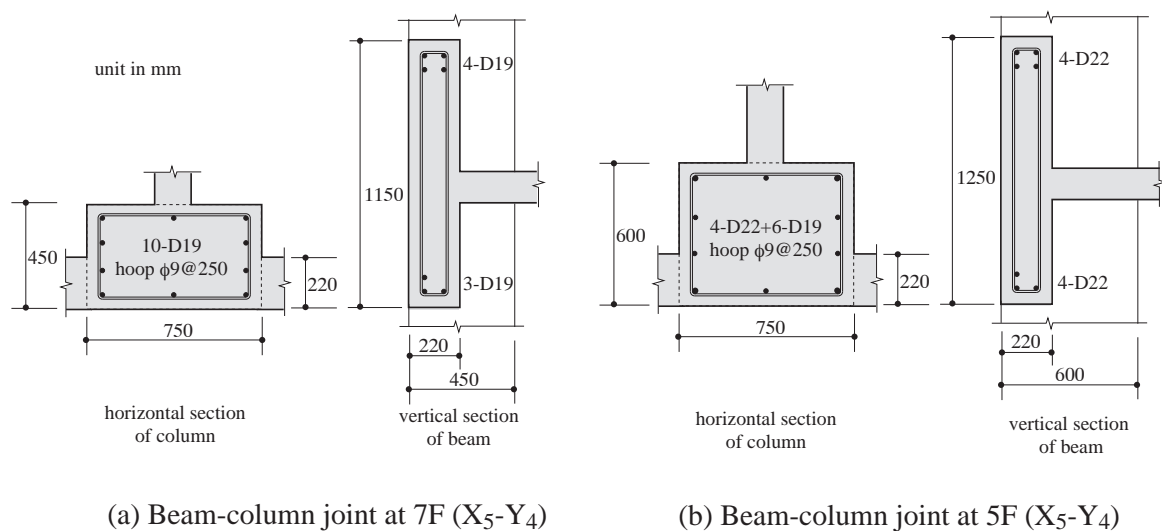


Fig. 8 Detail of a typical SRC section of columns in Frame Y_4



Fig. 9 Shear failure of exterior column in Y₄ frame



Fig. 10 Joint shear failure of beam-column joints