REPORT ON SEVERELY DAMAGED REINFORCED CONCRETE BUILDINGS BY 2016 KUMAMOTO EARTHQUAKE, JAPAN

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Abstract

2016 Kumamoto Earthquake with an M_{JMA} = 6.5 foreshock and an M_{JMA} = 7.3 main shock rocked the Kumamoto Prefecture on Kyushu Island of Japan on April 14 and April 16, 2016 respectively. Structural observation of four reinforced concrete moment resisting frame buildings suffered significant structural damage are reported as selected benchmarks for the seismic performance of low-rise reinforced concrete building structures in Japan. Two of them are two-story corridor structures of school building, connecting two buildings at two levels, remained standing but were left with extremely large residual lateral drift in the short span direction more than five percent at the first story after the main shock. The lateral load capacity are checked and it is revealed that they satisfied the level of the current requirement of the code. Nonlinear time history responses were calculated to base ground motions recorded at the near site of the buildings to evaluate the actual seismic demand of the buildings and to correlate the level of lateral capacity and the level of the damage, to assess the validity of the lateral strength demand requirement of the current code. The other two structures are two-way moment resisting frame building structures which nearly collapsed and demolished with severe structural damage to beam-column joints. To both of the buildings, story drift are concentrated at one of intermediate stories and the beam-column joints, which located at the top and the bottom of the intermediate story, the core concrete was fallen and the buckling of the longitudinal rebars in column were observed. Due to the failure of the beam-column joints, the vertical load support capability seemed to be lost and anchorage of longitudinal beam bars anchored in the beam-column joint may be lost, which caused the partial collapse of the buildings. The conformity of the detailing of the beam-column joint to the current seismic provisions are checked and the seismic demand and lateral capacity are compared by push-over analysis. It is revealed that the transverse reinforcement for the beam-column joints are minimal in both cases. The capacity of the gravity load carrying system without the contribution of the failed beam-column joints of one of the buildings are calculated by the yield line theory to assess the gravity collapse potential and compared with the observed damage to evaluate the validity of the current seismic provisions for reinforced concrete beam-column joints.

Keywords: Reinforced concrete, Moment resisting frame, Beam-column Joint, Partial collapse, Gravity load capacity

1. Introduction

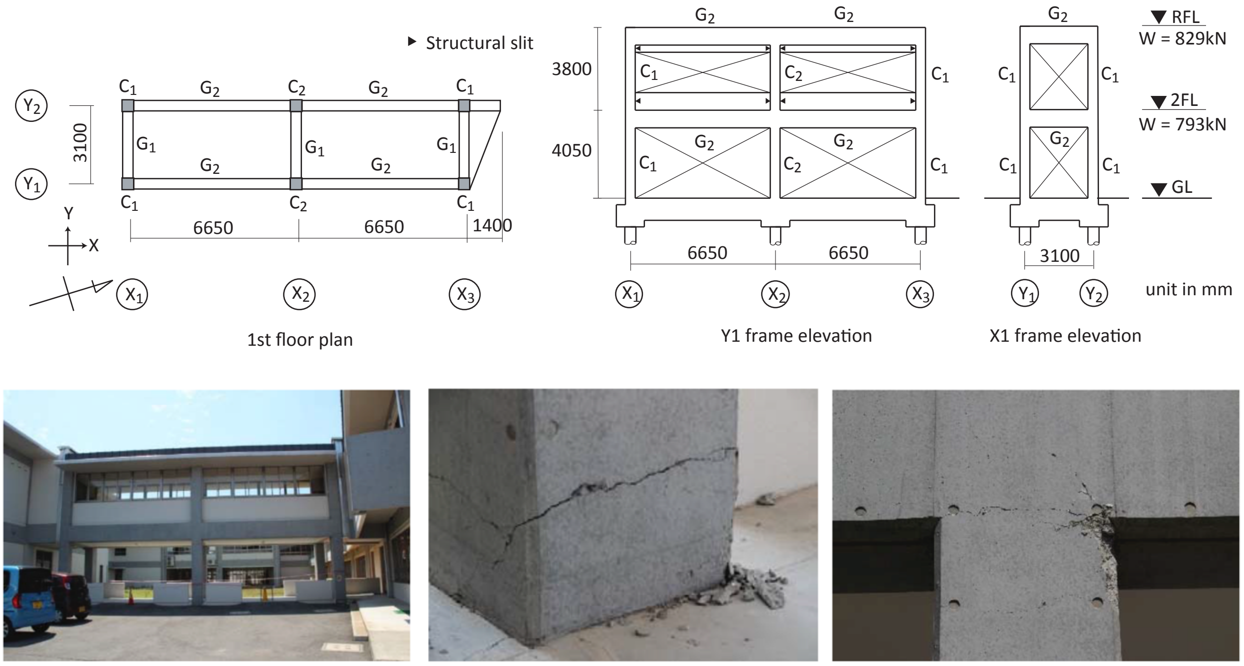
2016 Kumamoto Earthquake with an M_{JMA} = 6.5 foreshock and an M_{JMA} = 7.3 main shock rocked the Kumamoto Prefecture on Kyushu Island of Japan on April 14 and April 16, 2016. Fifty people were killed in the Earthquake. While various types of structural damage to Reinforced Concrete (RC) buildings were observed to new and old buildings [1,2], structural damage observation and the lesson of selected low-rise reinforced concrete buildings are reported here.

2. Collapse of two-story corridor structures of school buildings

One of the building type reported here is two-story corridor structures of school buildings, connecting two main school buildings for class at the second floor level, which barely escaped collapse but extremely large residual lateral drift were left of more than five percent at the first story in the direction of short span. Whereas the main buildings has little damage. The lateral load capacity of the first story are revealed that they satisfied the code minimum requirement. Nonlinear time history responses were calculated to evaluate the actual seismic demand of the buildings subject to base ground motions recorded at the near site of the buildings. Correlation of the level of lateral capacity and the level of the damage is compared. The validity of the lateral strength demand requirement of the current code are examined to know the reason of the peculiar type of buildings.

2.1 General information

Building A is of a two-story reinforced concrete. The structural system is moment resisting frame (RCMF) with 1 bay 2 frames with two spans completed in 2013 in Town of Otsu, Kikuchi-gun, Kumamoto Prefecture. The first story is open frame while the frames in span direction are with non-structural RC spandrel walls isolated to column with gap to avoid interaction on the second story. Figure 1 shows the plan and the elevation while Table 2 lists the member section detail. The type of soil is grade II and the super structure is supported by bearing piles. The design compressive strength of concrete is 24MPa, the grade of longitudinal reinforcements are SD345 (JIS G 3801). Although beam sway collapse mechanism are predicted according to



(a) overview (b) column base (c) column top

Fig. 1 – Building A

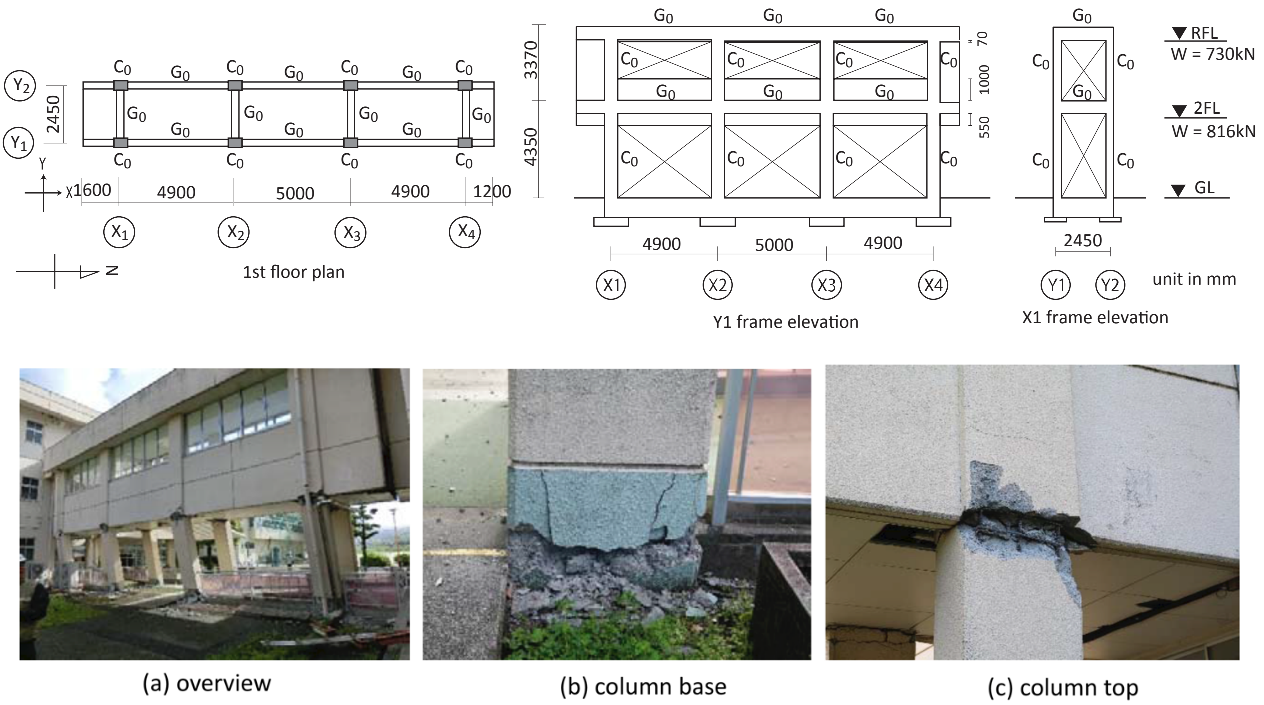


Fig. 2 – Building B

Table 1 – Member section and reinforcing detail of Buildings A and B

(a) Building A

section	column		beam	
2	C_1	C_2	● D22 G_1	G_2
1	C_1	C_2		
dimension	450 × 450		400×800	
	hoop D13@200		stirrup D13@200	

unit in mm

(b) Building B

section	column	beam
1 & 2	C_0	○ D19 G_0
dimension	600 × 400	300 × 600
	hoop D10@100	stirrup D13@100

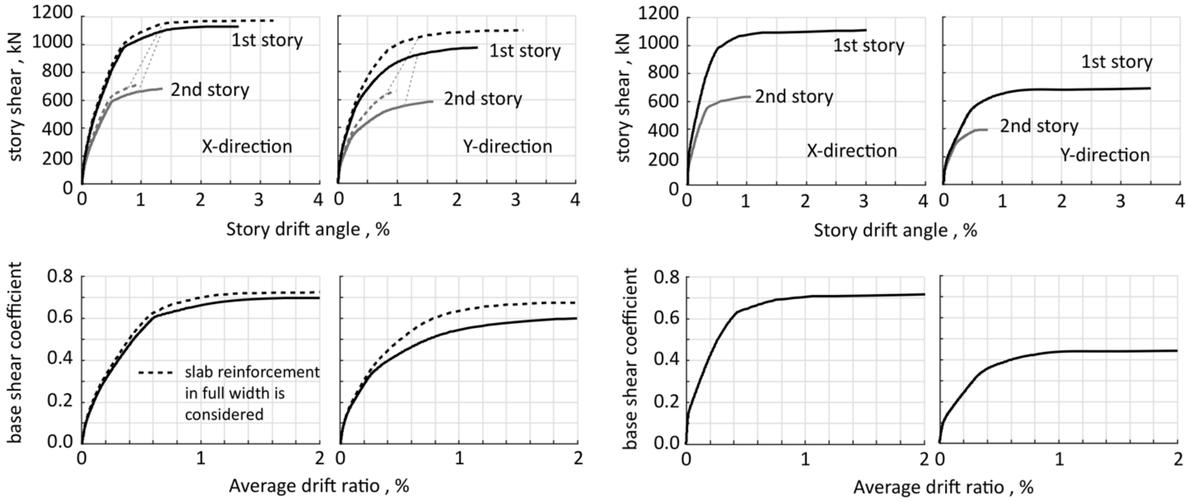
unit in mm

the design document, flexural yielding at the top and bottom of the first story column dominated the structural behavior, minor residual story drift of 1/500 remained after the earthquakes.

Building B is another two-story RC moment resisting frame with identical structural system with 1 bay 2 frames with three spans, completed in 1981 in Town of Mashiki, Kami-mashiki-gun, Kumamoto Prefecture. The plan and the section detail are shown in Fig. 2 and Table 2 respectively. The compressive strength of concrete is 21MPa and the grade of the reinforcement is SD30 (JIS G 3801). The verdict of the seismic vulnerability assessment in 2007 is seismic retrofit is not necessary. The sample core compressive strength of concrete was 22.8MPa and 20.9MPa, which is very close to design strength 21MPa. Failure of fist story columns in flexure at the bottom and the top was severe. Concrete crushing and local buckling of longitudinal bar were observed. Residual story drift at the first story was 1/17 and quite large.

2.2 Pushover analysis and collapse mechanism

The buildings conformed to new seismic provisions introduced in 1981 are called Shin-taishin and are required to determine the collapse mechanism and the assessment of lateral capacity. Member strength such



(a) Building A (b) Building B

Fig. 3 – Pushover curves

as shear should be larger than shear demand at the collapse mechanism to guarantee prevention of premature failure to assure ductile behavior of the structural system. To determine the collapse mechanism, pushover analysis are common in practical design recently using package software available in market. The reliability of the push over analysis depend on the assumption of non-linear structural modeling parameters.

Non-linear pushover analysis was carried out using a commercial software CANNY99 [3] under lateral force distribution of A_i; standard distribution factors in Building Standard Law. The corridor building and the adjacent main buildings were confirmed to be isolated by expansion joints, so the structure is modeled as independent building. The software incorporates the one component model for beam; two non-linear rotational springs at member ends and connecting elastic line element for beams, where tri-linear primary curve are calculated for cracking point and yielding point by flexural analysis and an assumption of yield stiffness, while the column is modeled by fiber model, assuming concrete constitutive model and bi-linear model for steel. Shear deformation is included in linearly elastic member. The beam-column joint is rigid. Concrete strength is from the design strength in Building A and the sample core strength in Building B. Yield point of reinforcing bar is assumed to be110% of nominal yield point to consider the expected average strength.

The relation of story drift ratio and story shear as well as the relation of base shear and the ratio of top floor displacement to total height of the building are shown in Fig. 3. The base shear coefficient at mechanism are calculated to be 0.68 and 0.58 in frame direction and transverse direction respectively for Building A. The coefficient for Building B is 0.71 and 0.42, both of which is much larger than the current requirement of seismic provision of 0.30. So they satisfy the current seismic code in Japan with margin.

Predicted collapse mechanism is beam sway mechanism. Observed collapse mechanism was story sway mechanism at the first story. They are contradicted each other. By considering the upper bound of beam flexural yield moment by assuming all the slab reinforcement is effective in T-slab section, the base shear at collapse mechanism increases to 0.70 and 0.64. Even in this case, the column-to-beam strength ratio in transverse direction decreased to between 1.27 and 1.80, which is contradict to the observed collapse mechanism. The reason of the difference of collapse mechanism shed the reliability of current modeling practice and need further investigation of the reason.

2.3 Non-linear Static Procedure for estimation of maximum response

Equivalent SDOF models are determined from the pushover curve of the structures. Three site response spectra of observing stations near the buildings are used. They are 1)the municipal building of Otsu;1.8km south west

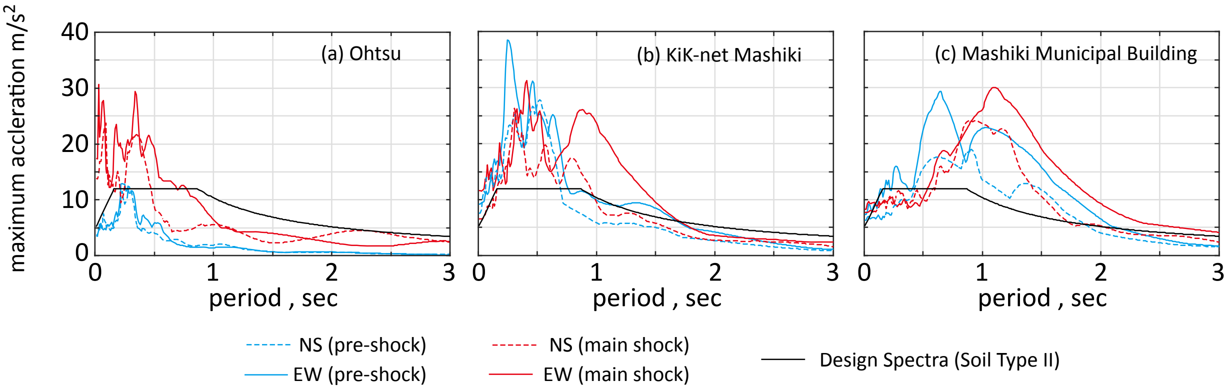


Fig. 4 – 5% damping acceleration response spectra of observed base motion at near sites

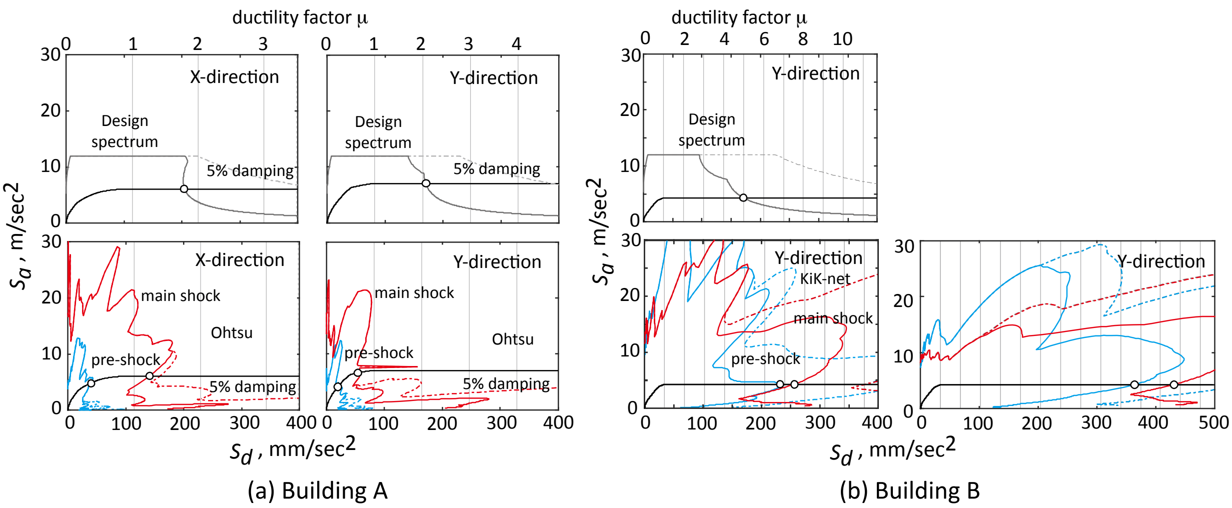


Fig. 5 – Response estimation by demand spectrum and capacity spectrum

to Building A, 2)KiK-net Mashiki; 1.4km northwest of Building B and 3)the municipal building of Mashiki;1.2km west of Building B). The site response spectra for 5% damping are shown in Fig. 4, where the code based design spectrum are overlaid. The demand spectra and the capacity spectra are overlaid in Fig. 5, where the intersection of the capacity and the demand spectra means the predicted response.

It is revealed that the estimated maximum displacement response of Building A is equivalent to ductility factor (μ) of 2 for code spectra, while response to the Otsu spectra gives μ = 1.2 in the transverse direction. The response is equivalent to 1.9% story drift if lateral drift distributed evenly at 1st and 2nd story, while it is 3.1% if all the drift occurred only at the first story. The estimated maximum ductility factor response of Building B in transverse direction (Y-direction) is μ=5.0 for seismic code demand, μ=7 and 7.5 at foreshock and main shock respectively for KiK-net Mashiki, and μ=11 and μ=12.5 at foreshock and main shock respectively for Municipal building of Mashiki, equivalent to first story store drift of 6.0% and 9.6%.The two building are confirmed to have code required lateral capacity. But the reality is the base motion level exceeded the required level and caused local collapsed of Building B. But the observed damaged to the main buildings of the both schools were minor. Further investigation is necessary to find rational explanation the strange contrast of damage to different type of structure.

3. Collapse of low-rise two way moment resistant frame structure

The seismic design provisions for RC beam-column joint has been introduced since 1991, where the first joint shear capacity equation was given in Japan. At 1995 Hyogoken-Nambu Earthquake, not a few new buildings are reported to have had some or severe damage to beam-column joints which conformed to the joint shear provisions. Two low rise RC buildings with damage to beam-column joints had been identified in Kumamoto. Both of Pre-Shintaishin Building C and Shintaisin Building D had suffered partial collapse and excessive residual drift. It should be noted that Japanese current seismic provisions for beam-column joint has a significant difference from those of the US and the NZ. That is no mandate confinement detailing provisions for joint hoops except minimum requirement of 0.2% and no mandate requirement for column-to-beam strength ratio.

3.1 General information

Building C is a five-story RC municipal building of Uto City completed in 1965. Seismic vulnerability assessment carried out in 2003 put a verdict of insufficient earthquake resistance, but no retrofit work was done. The verdict was mainly due to the brittle columns vulnerable to shear failure. The structural system is two way moment resisting frame structure with pentagon plan connected to auxiliary structure with rectangular plan as shown in Fig. 6. The two structures are continuous only by monolithic RC slab at each floor with thickness of 120mm and length of 745mm along the joint. The part of pentagon building toppled severely whereas the building of the rectangular plan was kept intact due to stiffness and strength of staircases incorporated in the structure. The connecting slab had fractured severely with concrete fell off after the main shock.

The use of the building was municipal office. The story height of the 3rd story or higher are 3600mm and the span length is 8900mm in the two perpendicular direction. The first story plan is extended to

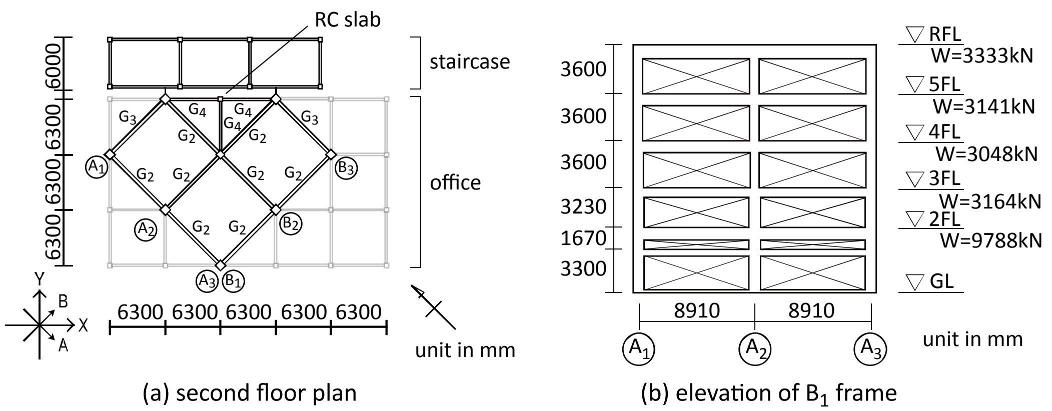


Fig. 6 – Plan and elevation of Building C

Table. 2 – Member section and reinforcing detail of Building C

(a) column

	5F	4F	3F	2F
A_2B_1	700×500	700×600	700×650	850×850
A_1B_1	200, 200; 700×700	100, 100; 700×700	50, 50; 700×700	850×850
hoop	9φ @120			

(b) beam

	RF		5F		4F		3F
	in	out	in	out	in	out	
G_1	350×900						
stirrup	9φ @200						

○ D25 ● D22 △ D19 × D16

unit in mm

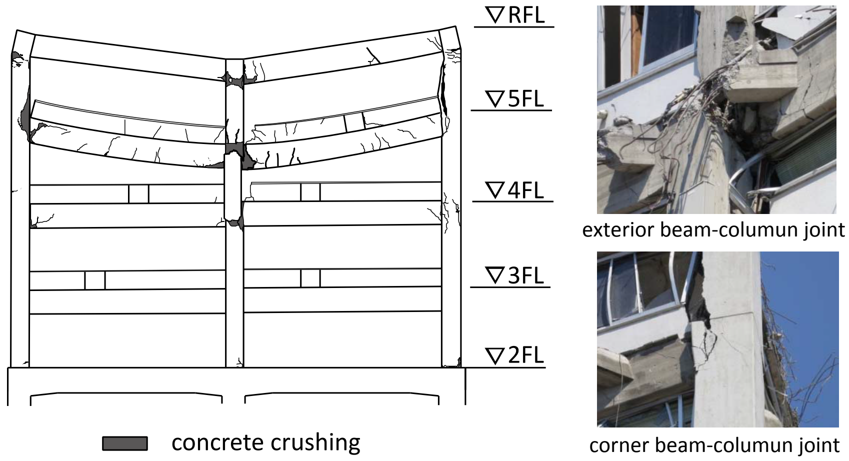
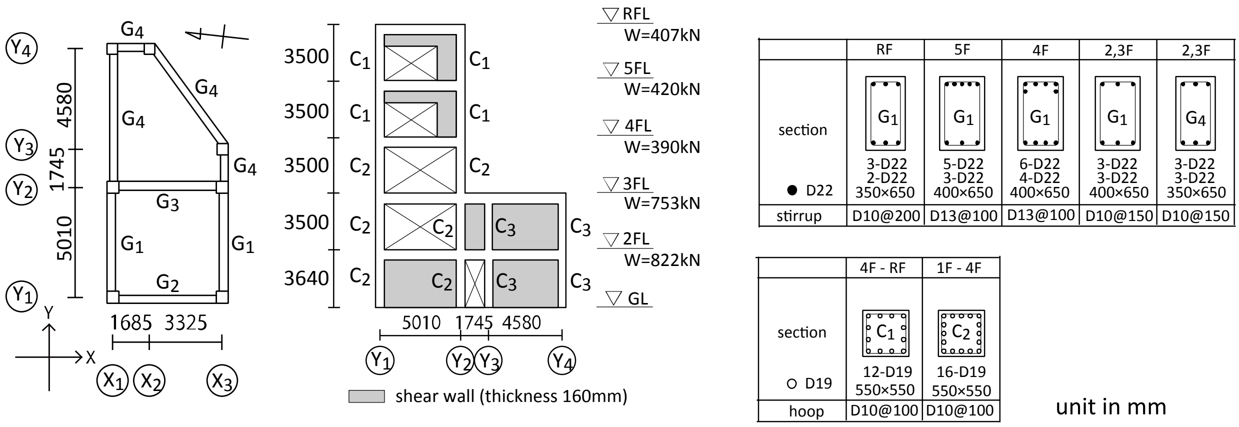


Fig. 7 – drawing of frame B_1 damage and photos of damaged beam-column joint

accommodate the space for reception. The plan, elevation are shone in Fig. 6 and typical member sections are listed in Table 2. The structure was constructed on a pile foundation. Concrete compressive strength records tested in 2003 for seismic assessment were larger than 180kg/cm^2; design strength of concrete. The tensile test of longitudinal coupon collected from collapsed building showed larger yield point of 399MPa, than the nominal yield point of 295MPa.

The damage of the outer frame is depicted in Fig. 7 which was drawn based on the photo image. The center column of the outer frame at the 4th story popped outside losing gravity supporting capability triggered by the failure of beam-column joint at the top and the bottom of the column. The concrete in the joint crushed and fell off completely, and the longitudinal reinforcements buckled. The cracks due to failure of anchorage of beam bar were developed in the joint panel at the side of the corner joint. No shear failure of columns or beams were not observed.

Building D is a five-story RC structure for a fire fighter training facility which belongs to a fire station completed in 1988 in Town of Mashiki, Kumamoto Prefecture. The first and second story is stiff and strong due to thick RC shear walls monolithically constructed with the frames with thickness of 160mm. From the 3rd to 5th floors are MRF with four columns and no shear wall. The plan, elevation and section reinforcing detail are shown in Fig. 8. The flexural reinforcement in beams in x-direction and y-direction is common from



(a) floor plan (b) X_3 frame elevation (c) section of beam and column

Fig. 8 – Plan and elevation of Building D

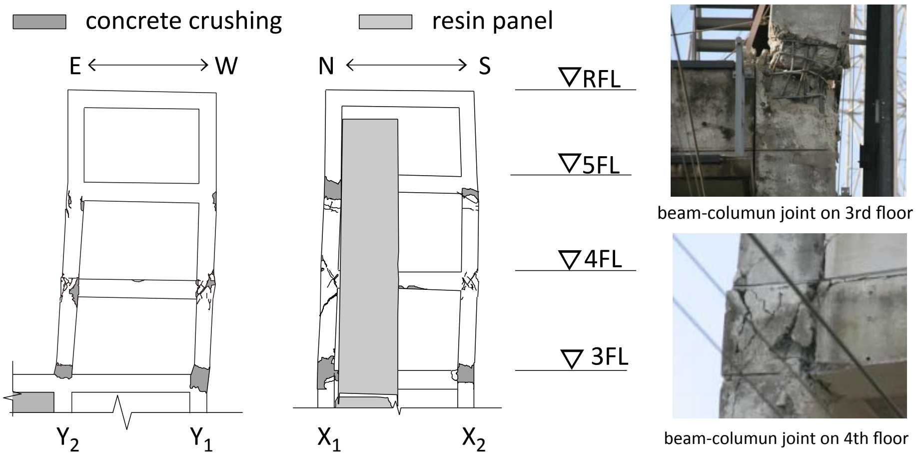


Fig. 9 – Damage of Building D

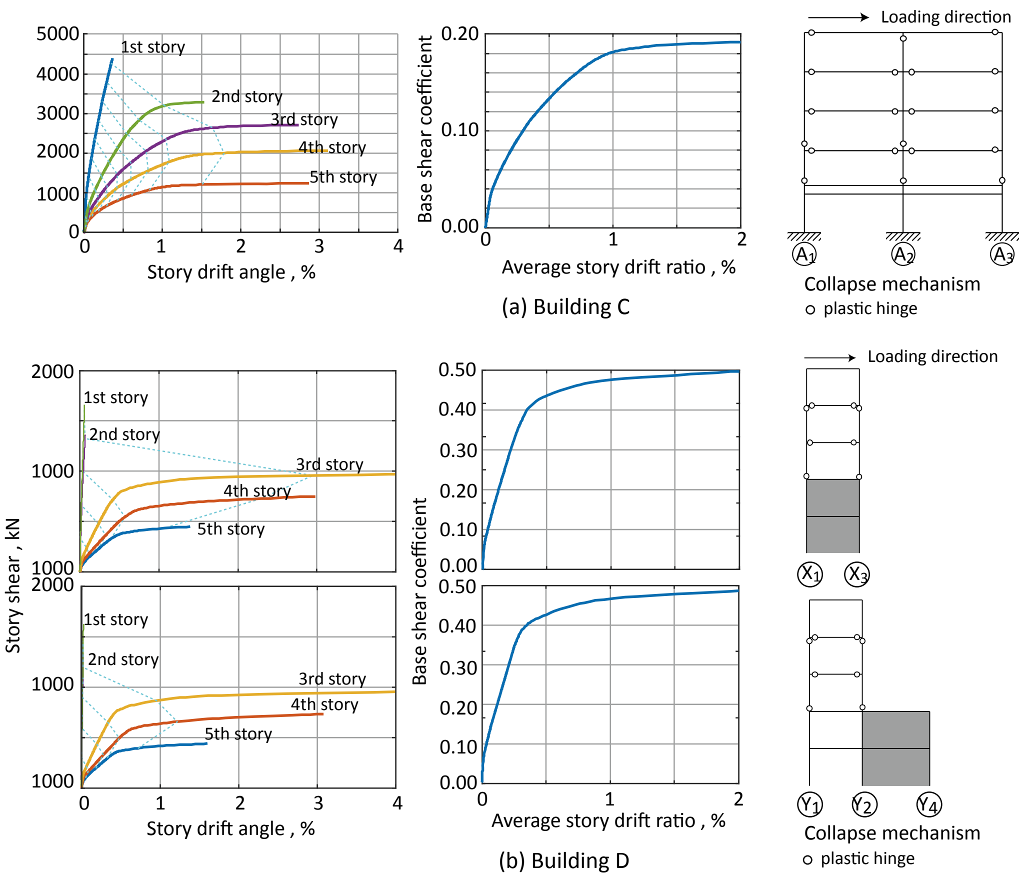


Fig. 10 – Pushover curves and collapse mechanism by non-linear static analysis

the 3rd to the roof floor, while the width of the beam in *x*-direction is 350mm which is smaller than 400mm of *y*-direction. The structure is supported by a pile foundation. According to the structural calculation document, the structural type is ductile moment resisting frame. The observed damage to the structural members in *x*-direction and *y*-directions are shown in Fig. 9. It is known that after the foreshock, the crush of beam-column joint concrete on 3rd floor severed particularly and fell off, while the damage to beam or column were minor and cover concrete did not fell off. After the main shock, damage to beam-column joints got severer with more cracks and crushing of concrete, lot of cover concrete fell off and reinforcing bar became visible. By the aftershocks, the residual story drift increased to 5% in *y*-direction and 1% in *x*-direction. The building was demolished in a few weeks after the main shock.

3.2 Pushover analysis and collapse mechanism

Non-linear pushover analysis was carried out using CANNY99 [3] to assess the collapse mechanism and lateral resistant capacity of Buildings C and D. The model are same as those used for the corridor building in Section 2.2. Each shear wall is modeled with a fiber model for flexure action and a non-linear shear spring for bending and shear deformation respectively. Shear cracking force and shear strength of shear walls were calculated by equations in AIJ Guidelines [4]. The flexural resistance of out-of-plane direction is also considered. The beam-column joint is assumed to be rigid and have infinitely strong to shear. Compressive strength of concrete used for the Building C is the strength of the sample core test listed in the vulnerability assessment report, while the strength of concrete used for Building D is design strength of 21MPa. Model for tensile yield point of reinforcing bar are from tensile test of coupon taken from Building C, while 110% of nominal yield point is assumed for Building D. Lateral load distribution coefficients A_i are assumed. The story drift-story shear relations and collapse mechanism are shown in Fig. 10, where occurrence of plastic hinges on the column means yielding of all the tensile reinforcing bars in the fiber model section.

The collapse mechanism of Building C is beam-sway mechanism along the stories on 3rd story and higher. The base shear coefficient at the collapse mechanism is 0.19, which is smaller than current code requirement. Building D shows collapse mechanism of local lateral sway mechanism at 3rd and 4th story and the base shear coefficient are 0.55 in both directions, which is much larger than required minimum value of 0.40 specified in the current code.

3.3 Evaluation of beam-column joint seismic design and detailing

The seismic design of beam-column joints of Building C and Building D are evaluated how they conformed to the recent provisions adopted by AIJ. The values calculated are for; a) ratio of joint shear demand to capacity, b) ratio of bond strength to bond capacity of longitudinal passing through the beam-column joint, c) ratio of development strength of longitudinal reinforcement in beam-column joint, d) column-to-beam strength ratio, and e) amount of joint hoops in transverse reinforcement ratio, are shown in Fig. 11. The values of a) b) c) are based on AIJ Guidelines [5], while d) are calculation based on reference [4] at axial force at collapse mechanism obtained by the pushover analysis above. The column-to-beam strength ration [4] are calculated as the ratio of nodal moments of beams and columns. The mechanical properties for steel and concrete is same as those used for pushover analysis. The actual anchorage length of beam bars in beam-column joint is not known so, they are assumed as 75% of column depth for Building C and column-depth minus 100mm for Building D respectively.

The beam-column joint of Building C satisfies the current Japanese seismic provisions for joint shear, anchorage in beam-column joint and development strengths, while joint hoop reinforcement ratio is 0.1% smaller than minimum requirement. The column-to-beam strength ratio is very close to 1.0, which may be the reason of severe joint failure which cause losing concrete in joint core, leading to lateral deformation concentration at the 4th story, increasing the contribution of vertical reinforcement in joint, causing buckling of longitudinal bars in beam-column joint. The loss of the beam-column joint core concrete triggered the loss of anchorage of the transverse beam longitudinal bars. So the failure of beam-column joint caused the loss of the integrity and failure of gravity support structure system at the 4th story near the central column on the outer frame.

The beam-column joint of Building D almost satisfied the seismic provisions of anchorage, while the safety margin of bond capacity and joint shear strength is barely1.0 or a little bit smaller than 1.0 at some exterior beam-column joints in loading direction of slab tension. The joint hoop ratio of beam-column joint having large damage is 0.36% and satisfied the minimum requirements of seismic provisions [5].

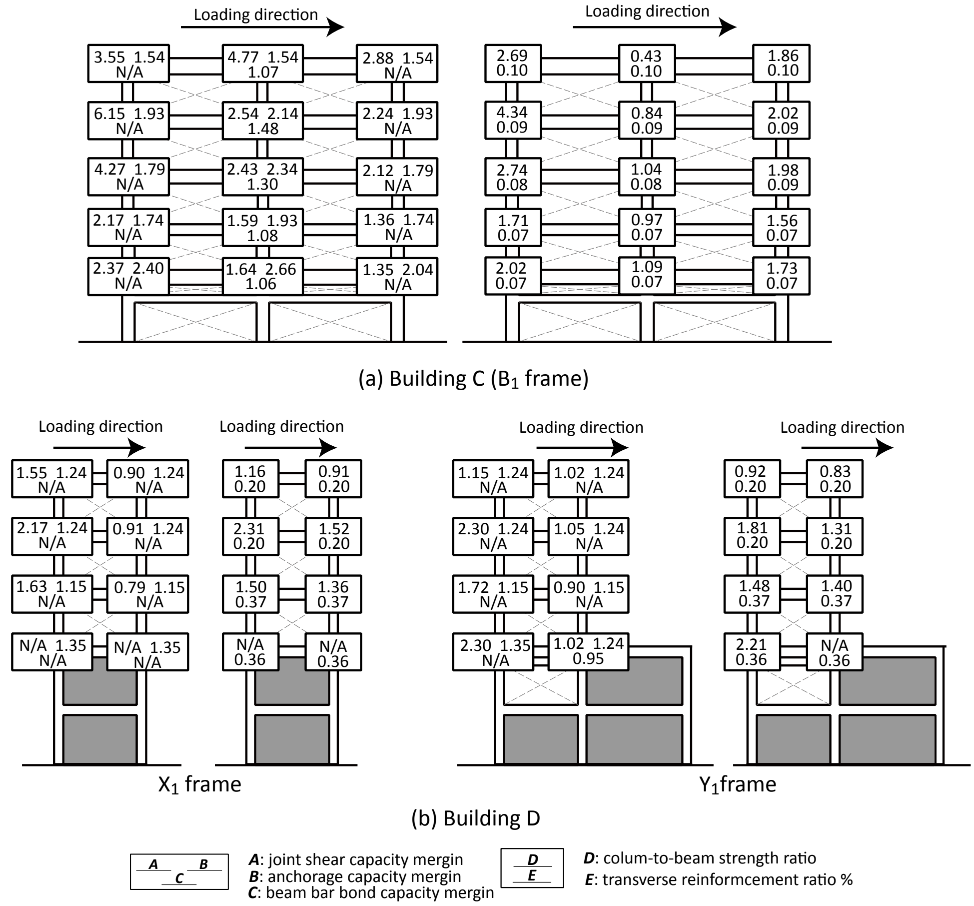


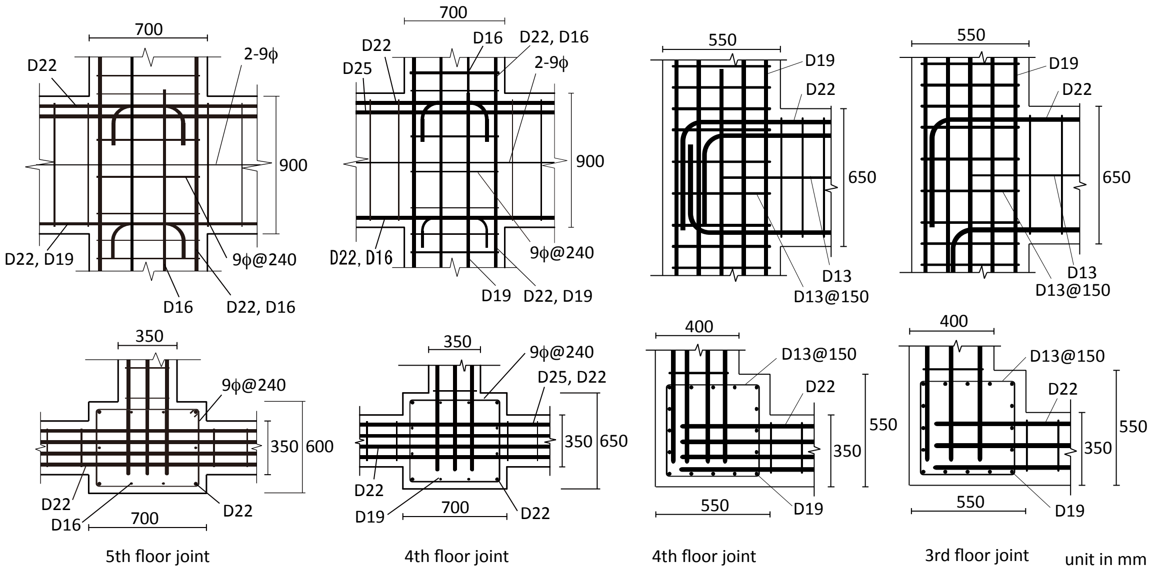
Fig. 11 – Performance index of beam-column joints of Building C and Building D

Reinforcing detail found in the structural drawing are shown in Fig. 11. The amount of longitudinal reinforcement at the bottom of beam is less and some of them are not passing through the joint but anchored with 90 degree bending downward in the joints. When the beam-column joint failure begins, the anchorage was lost and pulled out, which jeopardize the beam capability to back up the vertical load redistributed to resist the downward movement.

By assuming only the longitudinal reinforcing bar can support the axial load after the beam-column joint core concrete fell off, the axial stress level in reinforcing bars normalized with yield point due to gravity load supported by the column are estimated as an index relating vulnerability of buckling. The index is 0.68 and 0.41 for 4th and 5th story columns respectively for central column on the outer frame B_1 of Building C. The values for Building D is 0.18 and 0.21 at the beam-column joint of 3rd floor level which are much smaller than those of Building C. In case of Building C, the index is larger and the buckling of vertical reinforcing bars triggered the partial collapse at 4th story. On the contrary, the index is smaller for Building D and no buckling occurred in the beam-column joint, which may have prevented from partial collapse of Building D.

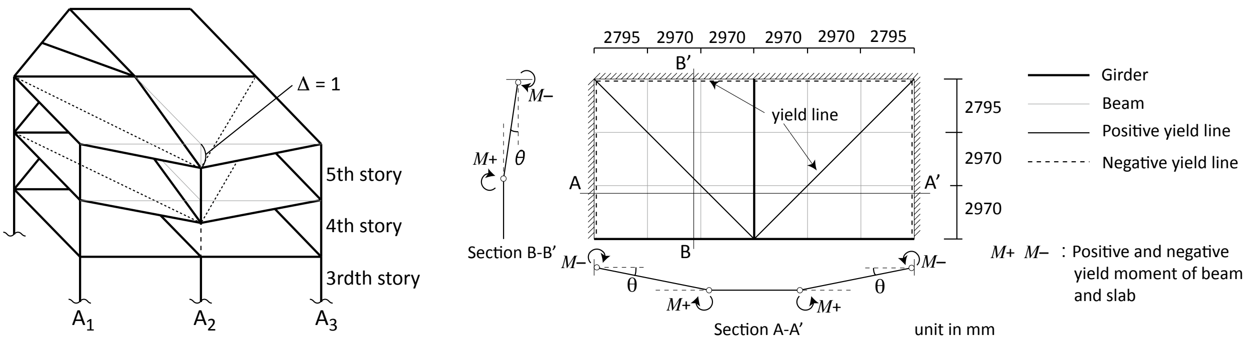
3.4 Ultimate gravity support capability by yield line theory

Upper bound theorem and yield line theory are applied to the fifth and the roof floors system of Building C to investigate the gravity collapse scenario. The mechanism shown in Fig. 13 are assumed to calculate external work by the gravity and the inner work dissipated by the plastic deformation caused by a unit vertical displacement. They are compared to assess the floor system capacity to back up the loss of axial force in the central column on the outer frame. External work considers the contribution of the distributed weight of slab,



(a) Building C (b) Building D

Fig. 12 – Reinforcing detail of beam-column joint



(a) Assumed mechanism at ultimate load (a) Yield line on the slab

Fig. 13 – Assumed yield lines for estimation of load at collapse mechanism

beam, girder and column whereas the internal work considers the plastic deformation of slabs, beams and girders. The external work is 948.2kNm and the internal work is 826.6kNm [6]. So it is interpreted that the loss of the central column on 4th story causes the local collapse at 4th story after the failure of beam-column joints. In reality, the anchorage of beam reinforcing bars pulled out and vulnerability to the collapse is higher.

4. Concluding remarks and recommendations

2016 Kumamoto Earthquake left collapsed or severe damage in some reinforced concrete building. Among those buildings, low rise building; a two two-story buildings which had excessive residual story drift and two five story buildings with significant damage to beam-column joint are selected and investigation was made to obtain lessons to improve the current seismic design provisions.

Building A and Building B experienced a very large residual story drift concentration at the first story. The lateral capacity of collapse mechanism is larger than code provisions, two times for Building A and 1.6 times for Building B. In spite of such large margin of the strength, the maximum residual story drift at the first story were estimated as 3% and 6% for Building A and B respectively. One reason is very strong ground motion recorded, which is larger than design spectrum adopted in the current seismic code, which have been implicitly considered maximum story drift of 2% for RC ductile frame structure by experts. So it is expected the buildings may collapsed if they had minimum capacity by seismic code specifies. But still there is a question the main building of the school class, which is usually three or four story moment resisting frame structure did not suffer such damage anywhere. The corridor building should have some special factors which caused the response of the buildings quite different from other buildings on the same site. The other lesson is moment resisting frame with two story need special attention to strength, ductility and collapse mechanism by preparing for unexpected stress magnification due to slab effect in T-beam section to secure the intended collapse mechanism and preventing failure of beam-column joint.

Building C and Building D was designed as beam-sway mechanism had suffered significant failure to some beam-column joints, in spite the dimension of the joint is large enough to conform to the joint shear seismic provision. The problem was insufficient joint confinement with column-to-beam beam strength ratio close to 1.0. It caused joint shear failure and the concentration of lateral story drift at a particular story. In case the failed exterior beam-column joint do not have bearing sufficient vertical reinforcement and no backup gravity supporting structural system, the vertical longitudinal bar passing through the joint buckled and triggered the partial collapse and structural instability. The lessons from the buildings are, lightly reinforcing beam-column joint needs confining joint hoops to prevent joint shear failure. In particular the building with exterior beam-column joints supporting large gravity load is essentially vulnerable to gravity collapse of the building if there is no redundancy in gravity support system.

5. Acknowledgement

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6. References

[1] Architectural Institute of Japan (2018): Report on the Damage Investigation of the 2016 Kumamoto Earthquakes. (in Japanese)

[2] Building Research Institute and National Institute for Land and Infrastructure Management (2016): Quick Report of the Field Survey on the Building Damage by the 2016 Kumamoto Earthquake, Technical Note of NILIM No. 929, Building Research Data No.173. (in Japanese)

[3] Li Kang-Ning (1996): CANNY99, Manual for Three Dimensional Nonlinear Dynamic Structural Analysis Computer Program Package.

[4] Architectural Institute of Japan (2016): *AIJ Standard for Lateral Load-carrying Capacity Calculation of Reinforced Concrete Structures* (*Draft*). (in Japanese)

[5] Architectural Institute of Japan (1999): *Design Guidelines for Earthquake Reistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept*. (in Japanese)

[6] Saito M, Shiohara H, Tajiri S and Mukai T. (2018): Structural Observation of Reinforced Concrete Buildings Collapsed or Severely Damaged by 2016 Kumamoto Earthquake, *Proc. of the 15th Synpoium on Earthquake Engineering*, 1864-1873, Sendai, Japan (in Japanese)

