On the Seismic Response of the Building of the Faculty of Architecture and Engineering at Tohoku University

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The building of the Faculty of Architecture and Engineering at Tohoku University survived two strong ground motions. Its survival is not surprising because the structure was stiff and strong. What is more surprising is the fact that the first ground motion did not cause severe structural damage while the second motion caused so much structural damage that the building had to be evacuated and demolished. The damage occurred despite two key facts: 1) the intensities of the mentioned ground motions are inferred to have been similar, and 2) the building was strengthened after the first motion (and before the second) following stringent standards.

INTRODUCTION

The building of The Faculty of Architecture and Engineering at Tohoku University is an ideal study case. It had a fairly regular structural system, its blueprints were clear and well preserved, it was instrumented and its instruments were well maintained, and it experienced two strong ground motions with similar intensities (one in 1978 and another in 2011). Between these two ground motions, the building was repaired and strengthened and this work was documented carefully. To have conceived and executed a full-scale experiment to produce the information produced by this building and the dedicated researchers who studied and monitored it through decades would have taken not only much time and effort but also a prohibitive amount of money. This paper reviews the properties and history of the building and focuses on the damage caused by the Tohoku Earthquake of 2011 and plausible explanations for it.

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BRIEF DESCRIPTION OF THE STRUCTURE

The building was a nine-story composite building built in 1969 (Figures 1 and 2). It had a two-story podium with a seven-story tower above it. The structural system consisted of a combination of structural walls and three-dimensional frames. The building was instrumented with accelerometers installed in the first and ninth stories (Shiga et al., 1981).

STRUCTURAL LAYOUT

In the lower two stories, the building had eight bays and two overhangs in the East-West (E-W) direction, and four bays in the North-South (N-S) direction. The floor plan of these two stories had the shape of the letter H, with outer dimensions of 72 m (E-W) by 36.6 m (N-S). The floor plans of the upper seven stories were rectangular, with five bays in the EW direction, two bays and two overhangs in the NS direction, and outer dimensions of 40 m (E-W) by 17.2 m (N-S). The total floor area was 9200 m^2. Story heights measured from top-of-slab to top-of-slab were 5 m for the first story, 4.3 m for the second story, 3.8 m for the third story, and 3.3 m for the rest. Figure 2 shows floor plans and elevations of the building.

The lateral-load resisting system was a combination of frames and structural walls. Frames were not discontinued where structural walls were present. Instead, frame elements (both columns and beams) with the same dimensions as frame elements elsewhere were cast integrally with the walls. This arrangement resulted in wall boundary elements with the same dimensions of columns located away from walls. The layout of walls and columns is shown in Figure 2. Details about the structure are given in Appendix A and nees.org/warehouse/experiment/3641/project/1122.

In each plan direction, there were two parallel walls. They were continuous from foundation to roof. In the E-W direction both of these walls were located along the mid column line (axis C). In the N-S direction these walls were located along the exterior column lines of the upper seven floors. In addition, C-shaped walls were located next to the northernmost column line (axis D).

The floor system consisted of flat slabs supported by frame girders and intermediate beams framing into these girders at their midspans.

The foundations of the building were spread footings connected by grade beams.

STRUCTURAL DETAILS

Beam, column, and wall dimensions and reinforcement are listed in Appendix A. Beams and columns were reinforced with steel angles, plates, and reinforcing bars in both their transverse and longitudinal directions. Essentially, the angles and plates formed a lattice frame that was cast within a concrete frame.

The specified compressive strength of the concrete was 210 kgf/cm^2 (21 MPa, 3000 psi). The mean compressive strength of samples extracted from the building in April 2011 was 180 kgf/cm^2 (18 MPa, 2600 psi) (Kuji, 2011). The mean compressive strength of cores extracted from the third story was 150 kgf/cm^2 (15 MPa, 2100 psi).

The longitudinal reinforcing steel bars were specified to meet Japanese Standard SD35 (nominal yield stress of 3500 kgf/cm^2-345 MPa, 50 ksi-, expected yield stress of 4000 kgf/cm^2-390 MPa, 57 ksi-). Steel angles were specified to meet Standard SS40 (nominal yield stress of 2400 kgf/cm^2-235 MPa, 34 ksi-, expected yield stress of 3000 kgf/cm^2-295 MPa, 43 ksi-). Transverse reinforcing bars were specified to meet SR24 (nominal yield stress of 2400 kgf/cm^2-235 MPa, 34 ksi-).

MEASURED PERIOD AND ESTIMATED BASE SHEAR STRENGTH

The fundamental or first-mode period of the building has changed over the years. The stiffness of the structure has changed because of cracking caused mainly by earthquakes and because of strengthening done in 2001. Numerical analyses of a linear-elastic model of the as-built building made ignoring the flexibility of the foundation soil indicate that, in the longitudinal direction (E-W), its initial period was approximately 0.5 seconds while in the transverse direction (N-S) it was 0.4 seconds. Motosaka et al. (2004) report initial periods approximately equal to these values for displacement amplitudes not exceeding approximately 1/10^5 times the building height.

Published limit analysis results (Suzuki et al., 2013) and (Kimura et al., 2012) show that the base shear strength of the building was 1) likely to have been between 0.3 and 0.5 times its weight[a] and 2) controlled by a flexural failure mechanism with hinges in columns and walls at the base of the third floor.

[a] The variation being related mainly to differences in the assumed distribution of lateral forces.

RESPONSE TO THE EARTHQUAKE OF 1978

One of the major earthquakes that affected this building was the 1978 Miyagi-Ken-Oki. Figure 17 shows acceleration records obtained by Tsamba and Motosaka (2011) at the first story. The peak ground acceleration was 0.26 g in the N-S direction, 0.21 g in the E-W direction, and 0.16 g in the vertical direction. The peak ground velocity was approximately 0.35 m/s in the N-S direction, and 0.25 m/s in E-W direction.

The earthquake caused shear and flexural cracks (with thicknesses not exceeding 1mm) in the exterior shear walls (Figure 4), short beams, and a few columns of the third and fourth stories (Shiga et al., 1981). Approximately 2.5% of the windows broke and furniture overturned.

THE RETROFIT OF 2001

In 2001 the building was retrofitted to reduce torsion and increase the shear strength of the exterior walls in the N-S direction. The retrofit was limited to the upper seven stories. The concrete of the webs of the exterior walls in the N-S direction (Axes 2 and 7) was replaced with thicker cast-in-place webs made with concrete with a cylinder compressive strength of 300 kgf/cm^2 (29 MPa, 4.3 ksi). Steel jackets were mounted on short beams in the interior frames (Axes 3 to 6 between Axes C and D) in the N-S direction. These beams were expected to be vulnerable to shear. Steel braces were fitted into two bays of the southernmost frame (Axis B between Axes 3 and 4 and Axes 5 and 6) in the E-W direction to reduce torsion, and portions of the floor slabs (between Axes 2 and 3 and Axes 6 and 7) were thickened and reinforced with additional steel welded wire. Figure 5 shows photographs taken during the retrofit.

Figure 6 shows details of the reinforcement used in the replaced wall panels. As the existing concrete of the webs of exterior walls (oriented in the N-S direction) was removed, the existing reinforcement was cut 0.20 m away from beams and columns. Post-installed anchors (deformed headed studs with 13-mm shafts and spaced at 0.10 m) were glued 110 mm into beams and columns, and were embedded 260 mm in the new webs. New web longitudinal and transverse reinforcement was provided in two layers (see Table 1 for details). Spiral reinforcement, with a bar diameter of 6 mm, a pitch of 50 mm, and an outer diameter of 120 mm, was provided around the perimeter of the new webs.

Table 1: Retrofit details for the walls on Axes 2 and 7

Story	Web Thickness (mm)	Web reinforcement		Anchors	
		Layers	Bar diameter (mm) and spacing (mm)	Layers	Bar diameter (mm) and spacing (mm)
9	180	2	Longitudinal D10 @200, Transverse D13 @200	1	D13 @ 100
8	180	2	Longitudinal D10 @200, Transverse D13 @200	1	D13 @ 100
7	180	2	Longitudinal D10 @200, Transverse D13 @200	1	D13 @ 100
6	180	2	Longitudinal D10 @200, Transverse D13 @200	1	D13 @ 100
5	200	2	D13 @ 200	1	D13 @ 100
4	200	2	D13 @ 200	1	D13 @ 100
3	250	2	D13 @ 150	2	D13 @ 150

RESPONSE TO THE EARTHQUAKE OF 2011

The building withstood the March 11th 2011 earthquake but not without heavy structural damage. It was demolished because the cost of repair was deemed too high for a structure nearing the end of its expected life span.

Figure17b shows acceleration records obtained at the base of the first story. Peak ground acceleration was approximately 0.34 g in both the N-S and E-W directions, and 0.26 g in the vertical direction. Peak ground velocity was approximately 0.45 m/s in the N-S direction, and 0.50 m/s in the E-W direction. At the ninth-story the maximum acceleration recorded in the transverse direction was 0.93 g (Figure 18b).

The damage concentrated at the base of the third floor in the exterior walls oriented in the N-S direction (Figure 7). The concrete in the boundary elements of these walls disintegrated along heights of up to 0.8 m. Longitudinal reinforcement buckled and/or fractured. The joint between the original beams and the web of the walls cast in the retrofit of 2001 was damaged: the concrete at the top of the beams spalled and top beam reinforcement was exposed. The interior C-shaped walls in the third floor had shear and flexural cracks, and spalling at the level of the 4th floor.

Surveys of the building were done in October 2011, March 2012, and June 2012. Figure 8 shows crack maps obtained in those surveys for Axes 2 and 3.

KEY OBSERVATIONS AND INFERENCES

The structure was shored snug during the retrofit of 2001 to avoid large increases in the axial loads in columns along the retrofitted axes (2 and 7). The top portions of the replaced web "panels" were cast using a concrete mix designed to reduce shrinkage, which could also have caused increases in column axial load. No clear signs of shrinkage in the replaced webs were observed in the inspections made after the 2011 earthquake.

Figure 9 shows one of the columns that disintegrated during the 2011 earthquake. All deformed bars buckled. Some steel angles also buckled and remained buckled but, interestingly, other angles did not, and instead they remained straight and had fractures.The angles that were not buckled during the earthquake are likely to have fractured at welds or rivet holes. The angles that remained buckled must have either 1) fractured at rivet holes in the beam-column joint or 2) accumulated large plastic tensile deformations followed by compression. Studies of the steel samples extracted by Takenaka Corporation indicate that, away from rivet holes and welds, at least some of the reinforcing angles in the damaged columns did not reach large plastic deformations. The observed buckling may have resulted from a sequence of events starting with fracture in the beam-column joint followed by pullout and ending with buckling (Figure 11). The buckling of vertical reinforcing angles may have triggered the observed spalling of the column concrete shell. Wall sliding is unlikely to help explain the combination of buckling and fractures observed. The following observations provide more insight:

a) the shape of the cross sections of interior (C-shape) and exterior walls would indicate that, as can be confirmed by analysis, the webs of interior walls were at least as vulnerable in compression (caused primarily by flexure) as the boundaries of the exterior walls under southward inertial forces. The webs of interior walls did show cover spalling at their connections with 4^{th}-floor beams. Nevertheless, the level of damage was not comparable with the damage observed in the boundary elements of exterior walls.

b) Figure 7b shows the damage caused by the 2011 earthquake to the intermediate column (at intersection 2C) in the exterior structural wall on the east elevation of the building. The deformed reinforcing bars buckled despite the fact this column was unlikely to experience large compression forces during ground motion. Vertical splitting cracks in the column may have been the result of reinforcement buckling. Such cracks were not visible in the web, where damage tended to concentrate around the bottom ends of the anchor bolts installed in 2001.

Observations a and b above support the idea that spalling was caused by the following process: 1) the angles fractured in tension either in the beam-column joints or in the bottom region of the columns, 2) the deformed bars had enormous tensile strain near the fractured angles, 3) the deformed bars buckled when the tensile strain decreased, and 4) the angles fractured in the beam-column joints buckled as shown in Fig. 11 while those fractured in the columns did not.

Figure 12 shows the top of a 3rd–floor beam after the 2011 earthquake. The figure shows:

1) Anchor bolts that pulled out of the beam forming "conical" failure surfaces in the concrete
2) Buckled plain vertical bars which were part of the original web reinforcement
3) Beam stirrups
4) Beam top longitudinal reinforcement

The buckled plain bars were embedded in the webs cast in 2001 approximately 20 bar diameters. They are unlikely to have developed their strength in such a short length. Again, the observed buckling may be the result of pullout failures followed by compression. Figure 13 shows a cross section of the web-beam joint as modified in the retrofit. Observe that the dotted line does not cross any reinforcement anchored effectively to resist large tensile forces. At this location, the exterior walls were essentially unreinforced. A tensile failure at this location is likely to have occurred at a small wall drift and may have altered drastically the response of the wall as discussed later. Pullout of the anchor bolts explains the concentration of damage seen in the web. This type of failure could have been avoided had the concept of "capacity design" been followed closely (Sullivan, 2010). The anchorage of reinforcement needs to be stronger than the reinforcement itself.

The cracking away from the 3rd floor beams is also revealing and shows that shear deformations (and cracks) were larger in 1978 than in 2011, when the deformations seem to have concentrated at horizontal wall-beam joints.

THE CONUNDRUM

The key question is why was there more damage in 2011 despite the strengthening done in 2001? We see two plausible explanations 1) the demand was higher, and/or 2) the retrofit made the building more vulnerable.

WAS THE DEMAND AT THE SITE HIGHER?

It is clear that the earthquake of 2011 (The Great East Japan Earthquake, Mw 9.0) was a larger event than the 1978 earthquake (The Miyagi-Oki Earthquake, Ms 7.7). The critical question is whether the intensity at the site was larger.

Within 0.5 km, at least four other buildings that survived the 1978 event were evacuated and demolished after the 2011 event. We do not know all the details about their state after the 1978 earthquake so we cannot make direct comparisons on this basis. Nevertheless, we do know that 1) 4 out of 13 buildings inspected in the area in 1978 had 1.5-mm cracks in structural walls and were deemed to require structural repairs, and 2) in two of the buildings surveyed in 1978, the damage in 2011 took place mostly in coupling beams, which were not inspected in 1978.

In terms of PGA, the ground motion of 2011 was 30% more demanding. In terms of PGV, the increase in demand was also 30%. Nevertheless, PGA and PGV are far from being infallible intensity indices. A better index is linear spectral displacement. Figure 14 shows that, for practical purposes, the linear spectra from 1978 and 2011 were essentially equal. This coincidence is remarkable and makes this case extraordinary. Because linear spectra are not the only vehicle available to estimate displacement demand, we also considered nonlinear spectra. Nonlinear spectra are not as crisp as linear ones in that they depend on many parameters in addition to the ratio of mass to initial stiffness (i.e. post-cracking stiffness, strain-hardening stiffness, unloading stiffness, reloading stiffness, etc.). Nearly 700 dynamic analyses of nonlinear SDOF systems were conducted to try to understand to what extent nonlinear oscillators may have been more sensitive to the 2011 ground motion. The oscillators considered are described in Table 2 and Figure 15. The analysis results are summarized in Figure 16.

Table 2:

Oscillator Parameter	Bilinear	Trilinear	Simplified Takeda (Otani, 1974)
k2/k1	1	0.5	1
k3/k1	0.05 to 0.1	0 to 0.1	0 to 0.1
k4/k1	1	1	$(\Delta y/\Delta max)^{0.5}$
Fy/Weight[a]	0.3 to 0.4	0.3 to 0.4	0.3 to 0.4
Fcr/Weight	0.5 x Fy/Weight	0.5 x Fy/Weight	0.5 x Fy/Weight
Viscous Damp. Coeff.	0.02	0.02	0.02

On the basis of the nonlinear-analysis results obtained, it seems reasonable to conclude that the 2011 record did not produce consistently larger displacement demands in structures with the required toughness.

From the evidence presented we cannot conclude that the demand in 2011 was drastically larger than in 1978. We now turn to the recorded response to try to infer the displacement demand at which the failure of 2011 may have started.

THE NS RECORDS

Figures 17 and 18 show segments of acceleration records obtained at the site and generously provided to us by Prof. Motosaka of Tohoku University (2011). A quick inspection reveals a large difference in duration. The 2011 motion lasted more than 4 times the duration of the 1978 motion. We modified the acceleration records by removing signals with periods exceeding 6s (for records from 1978) and 16s (for records from 2011). The velocity records obtained by integrating the resulting acceleration records were also modified by removing the mean velocity. The modified velocities were integrated to obtain estimated displacements. Figure 19 shows segments of relative displacement histories estimated from the 1978 and 2011 records. They were computed subtracting computed base displacements from computed 9th-floor displacements. We concentrate on the NS direction because this was the direction of the walls that failed.

The acceleration and relative-displacement records are rich with information. Among other things they show that the peak relative displacement in 1978 was approximately 21 cm. They also

[a] If one increases the base- shear coefficient beyond 0.4, the results are expected to get closer to the linear results.

show that the effective period of the building was approximately 1 sec. in the same ground motion. The same is true for the initial part of the 2011 record[a]. Nevertheless, 82.2 sec. into the 2011 motion something occurred to the structure. The 2011 acceleration record reached a plateau (Figure 20) indicating that yielding may have occurred[b] as the displacement reached the maximum displacement reached in 1978 (21cm). As the building swayed in the other direction a radical event took place. The acceleration reached another plateau at approximately 82.7 sec. and soon after it decreased abruptly. The acceleration plateau was reached at an estimated displacement of approximately 21 cm. The abrupt acceleration drop occurred at an estimated displacement of nearly 23 cm. We do not claim to have the accuracy to estimate these displacements within 1 cm. We display the unwarranted number of significant figures simply to stress that the observed drop in acceleration occurred at a displacement comparable to the maximum displacement reached in 1978 confirming what the linear displacement spectra suggested (i.e. that the displacement demands were similar in 1978 and 2011). Additional abrupt drops in acceleration took place at 83.2 sec. and 83.7 sec. From that instant on, the structure had an increased effective period of approximately 1.2 sec.

The fluctuations in acceleration could be attributed to 1) the observed fractures in the boundary wall reinforcement, 2) the effects of higher modes of vibration, or both. We find option 1) more likely because:

a) the change in period that took place after the acceleration drops indicates a large and abrupt change in stiffness that cannot be explained by referring to higher modes, and
b) analyses of MDOFs and the shape of the computed response spectra (which show high amplifications for periods close to 1 sec.) tell us that the response of the structure was dominated by its first mode.

It is also reasonable to expect the observed fractures to have taken place when peak accelerations were reached (i.e. when the lateral forces peaked).

[a] Keep in mind that between 1978 and 2011 the building 1) was strengthened and 2) experienced several smaller earthquakes.

[b] Analyses of MDOFs and the shape of the response spectra (which show higher amplifications for periods close to 1 sec.) tell us that the response of the structure was dominated by its first mode, which allows us to infer from acceleration-displacement plots how the stiffness and the strength of the structure varied over time.

Notice that before 82 sec. the relative displacement computed from the 2011 record did not exceed 10 cm more than 3 or 4 times, and that it never reached 15 cm. This observation suggests that the increase in duration is unlikely to have been the cause of the failure as most of the cycles seem to have taken place well within the linear range of response.

Because the response of the structure was dominated by its first vibration mode, we examine next the relationship between absolute acceleration measured at the 9th floor and the relative displacement estimated for the same level. We do so expecting this relationship to provide us with information about how the stiffness and the strength of the structure varied over time.

ABSOLUTE ACCELERATION – RELATIVE DISPLACEMENT RECORDS

Figures 21 and 22 show absolute acceleration – relative displacement curves estimated based on the records obtained on the 9th and 1st floors in the NS (transverse) direction. Interpreting these plots is not simple because they are sensitive to 1) the effects of higher modes and 2) the modifications made to the records to obtain sensible displacement estimates. With these limitations in mind we do notice in them the following consistent trends:

1) Wide hysteretic loops were observed only in the first 15 sec. of the 1978 motion (Fig. 21a)
2) After that instant, the structure responded nearly as an SDOF showing no clear evidence of stiffness decay or yielding until 82.2 sec. into the 2011 motion (Figs. 21b-d)
3) Yielding was first reached at a displacement of 20 to 21 cm (points A, A', Figs. 20, 22a)
4) At 23 cm the first large drop in acceleration (or strength) took place (point B, Figs. 20, 22a)
5) Two consecutive acceleration drops took place between 83 and 84 sec. (points C, D, Figs. 20, 22b)
6) After these drops, the structure was softer and retained the resulting stiffness for the rest of the motion
7) The peak acceleration in 1978 was higher than the peak acceleration in 2011 indicating that the strength of the structure may have decreased because the failure of anchor bolts occurred.

SYNTHESIS

The bulk of the evidence presented points in a single direction: the column failures of 2011 are likely to have taken place at a displacement similar to the maximum displacement reached in 1978. Despite this inferred similarity, and despite the strengthening done in 2001, the damage caused by the 2011 motion was dramatically different. If the demands were not higher, it is reasonable to conclude that the structure was more vulnerable in 2011. The observed damage hints that the source of the vulnerability was the web-beam connections modified in the strengthening of 2001. It is plausible that the discontinuity in the vertical web reinforcement created during the retrofit of the exterior walls led to a weak (essentially unreinforced) plane at the base of the third story exterior wall webs. Wall deformations seem to have concentrated at this level during the earthquake of 2011. The concentration of deformations must have in turn led to larger unit tensile strains in the reinforcement (Wang, 2014). This plausible increase in strain could have resulted in the fractures observed and higher probability of buckling of longitudinal column reinforcement. If "low-cycle fatigue" problems commenced in 1978, the increased strains must have accelerated them. Had the connection between the reinforcement replaced in 2001 and the rest of the structure not failed, it is likely that the response of the structure in 2011 would have been more similar to the response observed in 1978. Boundary element confinement could also have helped reduce the damage seen in 2011.

CONCLUSIONS

The available evidence suggests that discontinuities in reinforcement introduced during retrofit work done in 2001 caused concentration of deformations that led to the failures of wall boundary elements in the Building of the Faculty of Architecture and Engineering at Tohoku University during the Tohoku Earthquake of 2011. Reinforcement discontinuities ought to be avoided at critical sections of elements expected to resist lateral forces induced by earthquakes.

ACKNOWLEDGMENTS

We are grateful for the support received from the Japan Science Foundation and the National Science Foundation of the U.S.A. We also would like to express our gratitude to Takenaka Corporation, in particular to Dr. H. Kimura, and to all the engineers who helped inspect the building described here and provided us with feedback about our interpretations. The insight and guidance provided by Professors A. Shibata and Mete Sozen are invaluable to us. Our work would not have been possible without the generosity of Professor M. Motosaka.

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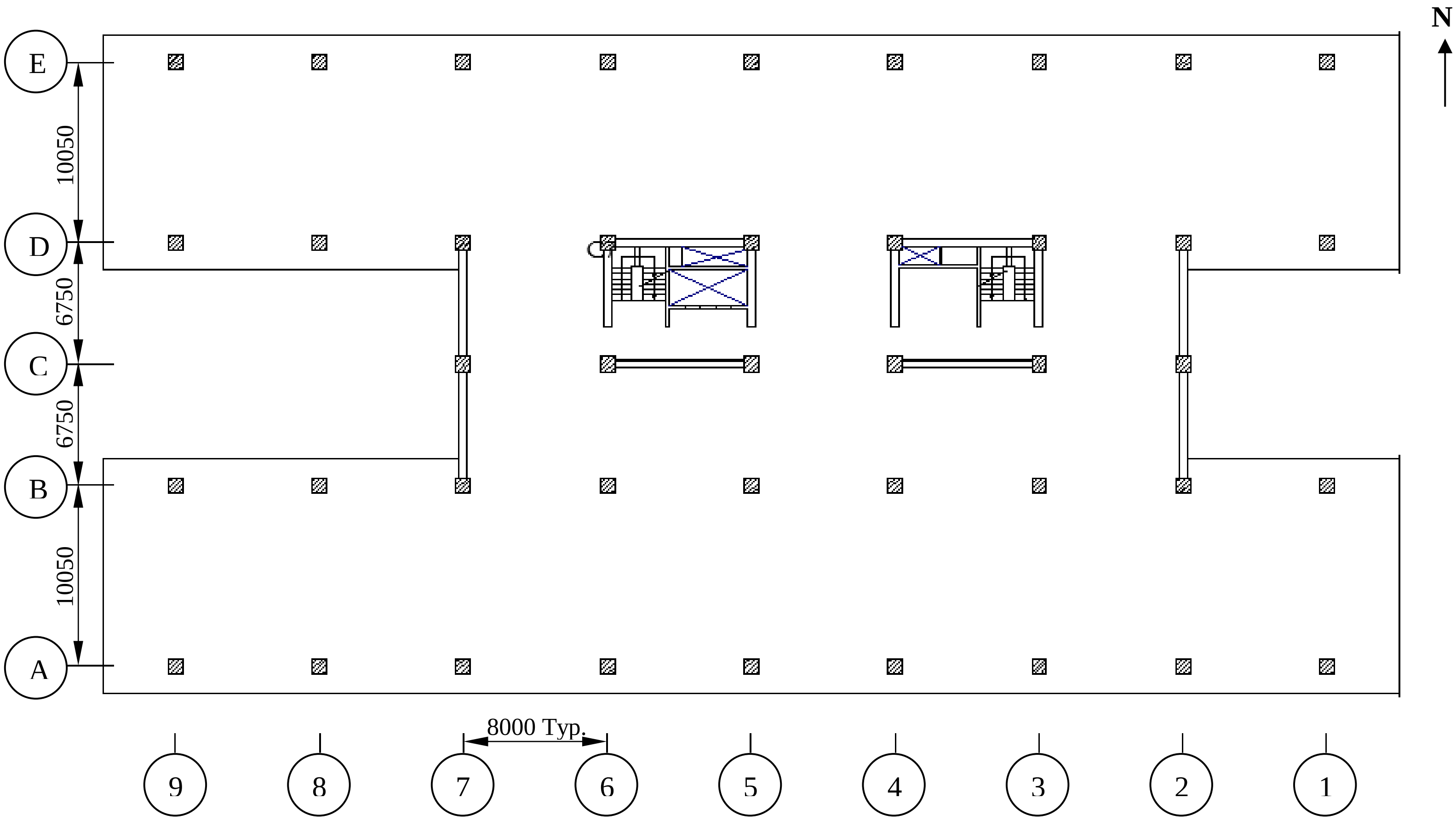
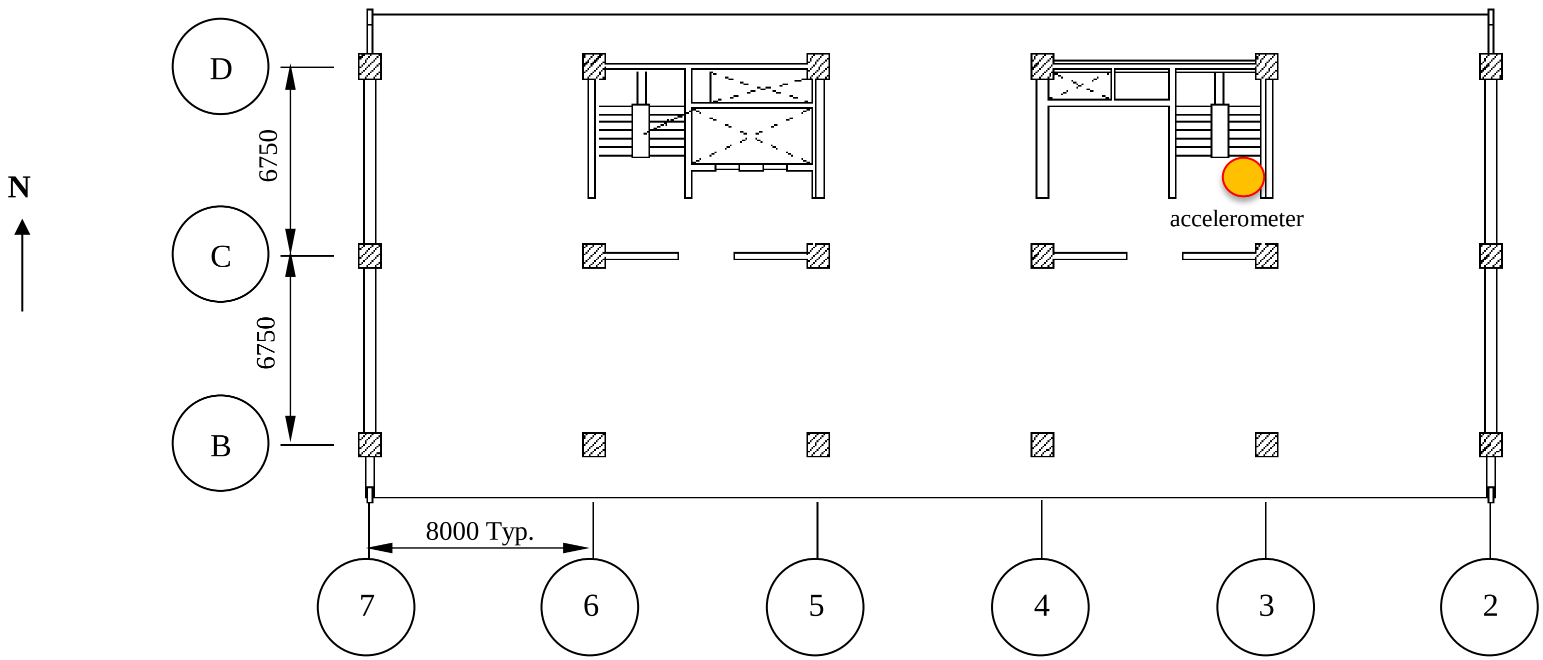
Figure 1: Building of the Faculty of Architecture and Engineering at Tohoku University



(a)

Figure 2: Plan views: (a) stories 1 and 2, (b) stories 3 to 9 (dimensions in mm)



(b)

Figure 2: Plan views: (a) stories 1 and 2, (b) stories 3 to 9 (dimensions in mm)

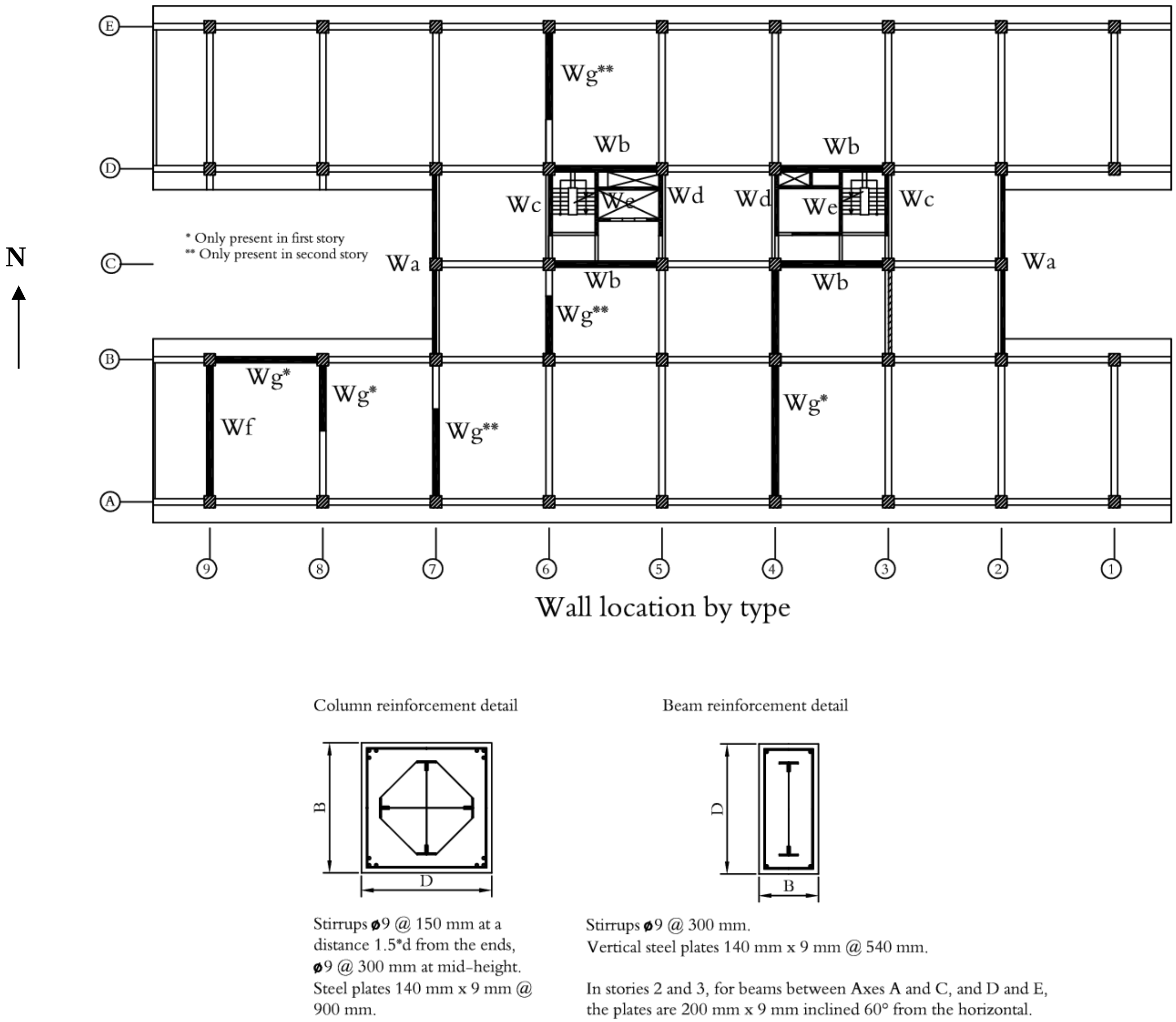
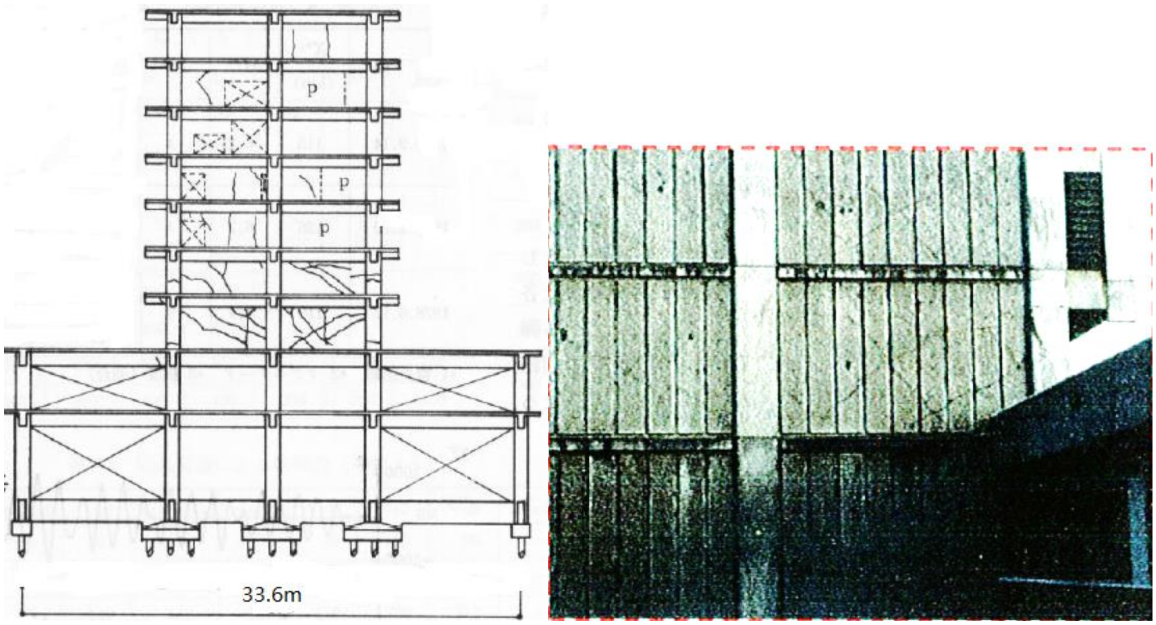


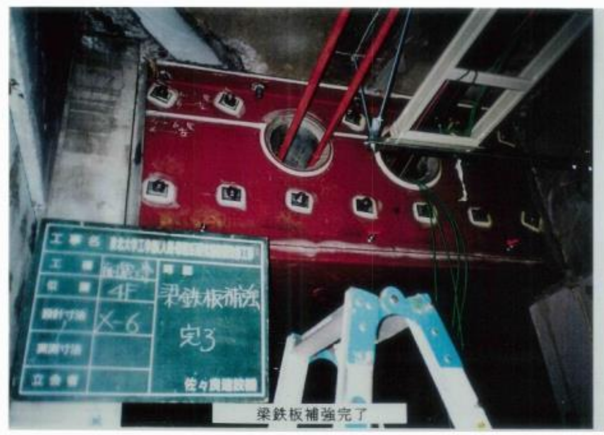
Figure 3: Wall layout, and beam and column reinforcement details. See Appendix A for details.



(a) (b)

Figure 4: Damaged observed in 1978: (a) crack map of the exterior wall along axis 2 (Shiga et al., 1981), (b) photo taken after the earthquake of 1978 showing cracks in second, third, and fourth stories.

(a) (b)



(c) (d)

(e)

Figure 5: Retrofit of 2011: (a) reinforcement at wall web-beam connection, (b) replaced wall web reinforcement, (c) exterior steel braces in South facade, (d) steel jacket on short beams, and (e) slab thickening (Courtesy of Tohoku University)

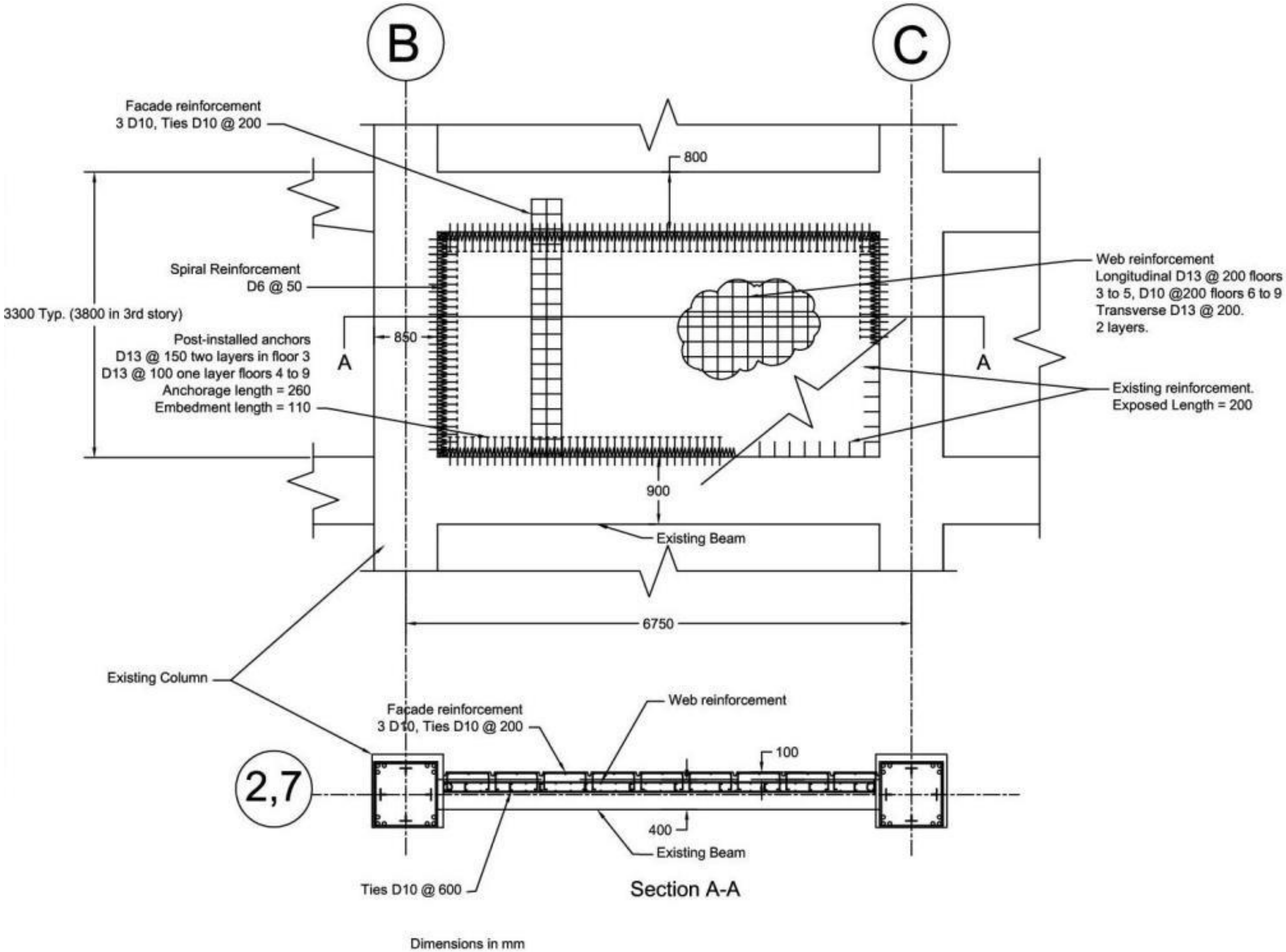


Figure 6: Details of the wall web reinforcement installed in 2001 (Appendix A)

(a)

(b)

Figure 7: Damage to exterior walls after the earthquake of 2011: (a) disintegrated boundary element, (b) damage to beam-web joint and intermediate column

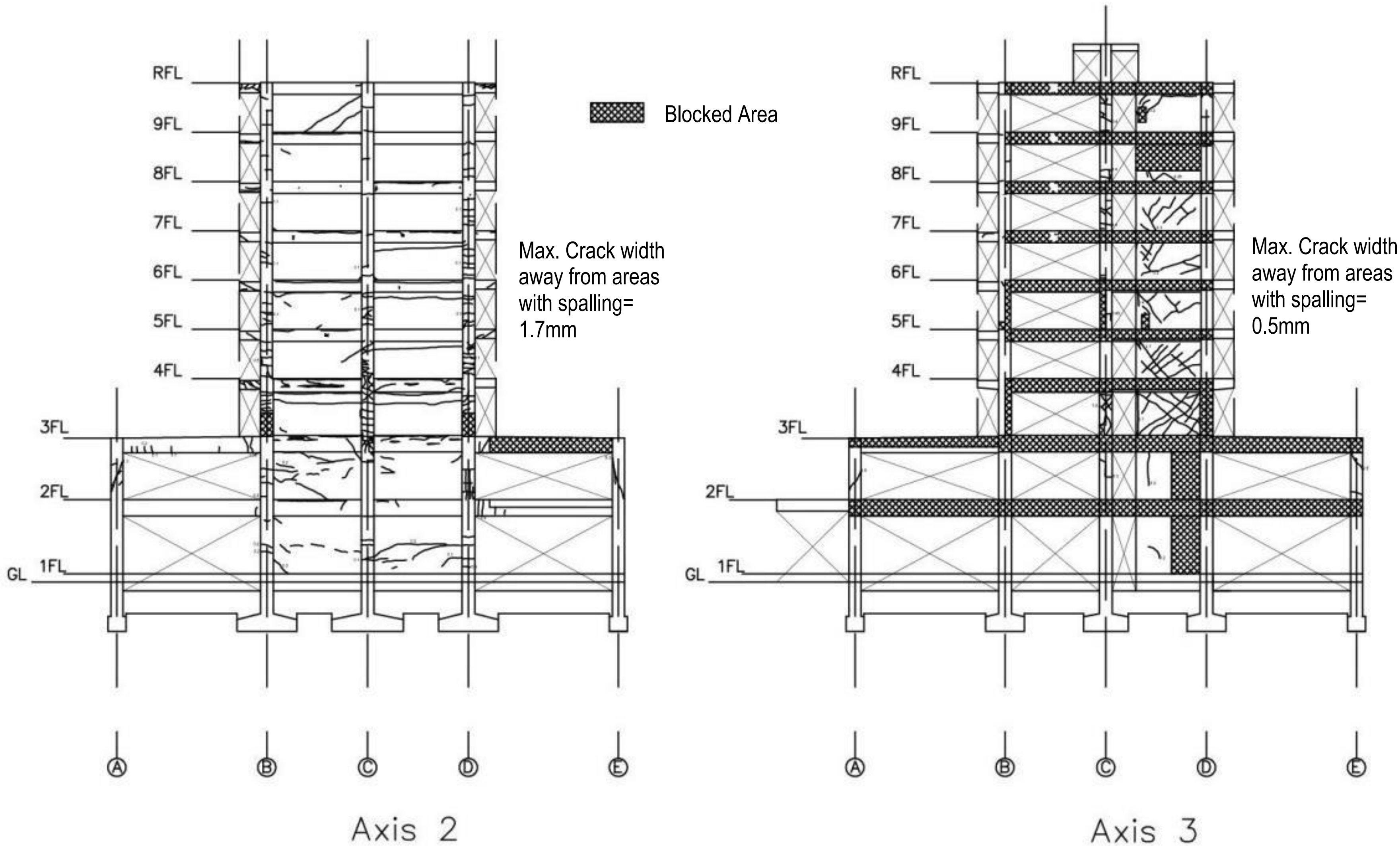


Figure 8: Crack maps obtained after the earthquake of 2011

Figure 9: Base of boundary element at third floor

Figure 10: Fracture at weld

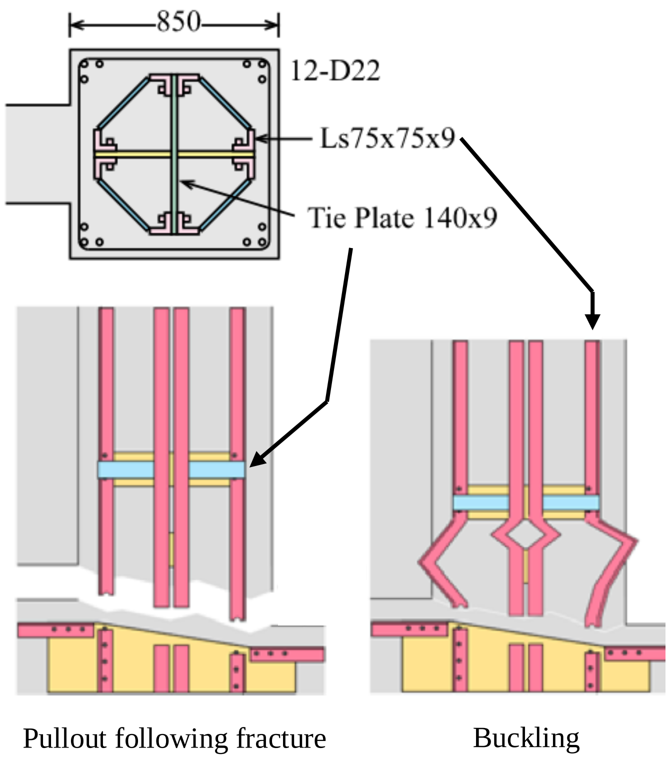


Figure 11: Plausible failure sequence (dimensions in mm)

Figure 12: Close up of web-beam joint

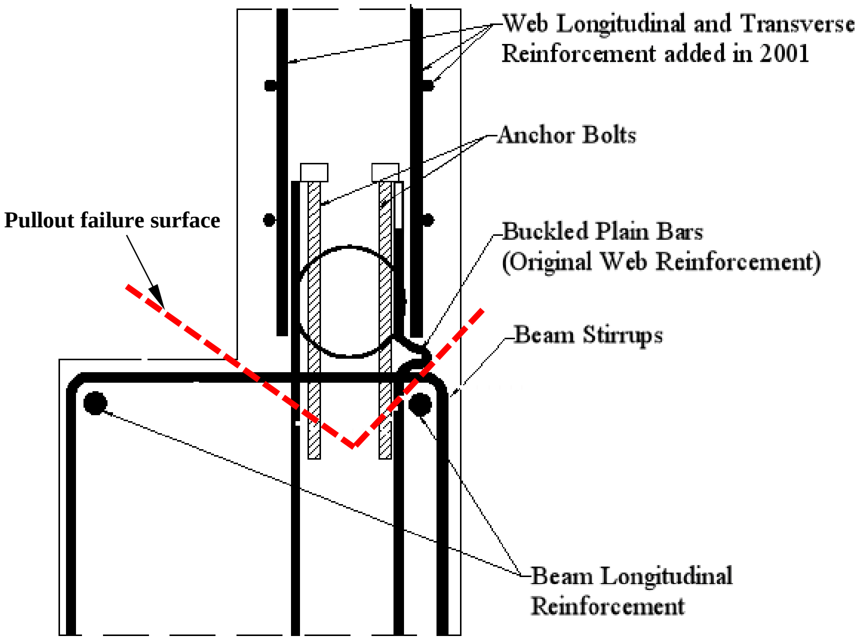


Figure 13: Cross section of web-beam joint

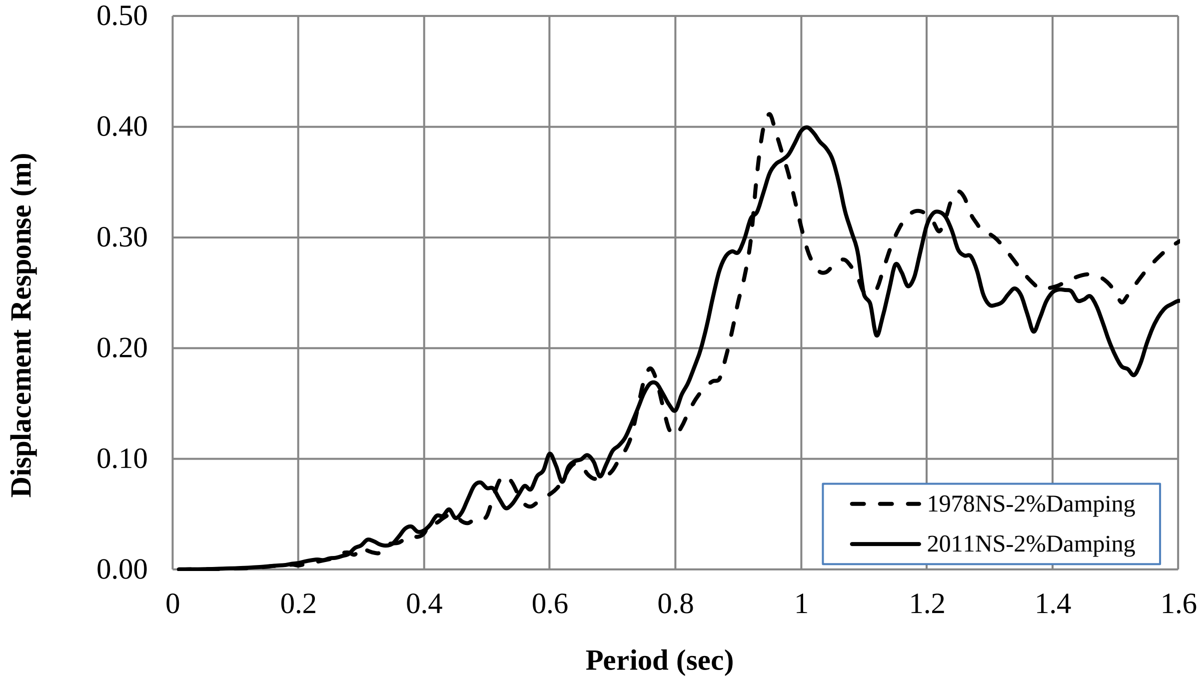


Figure 14: Linear displacement spectra

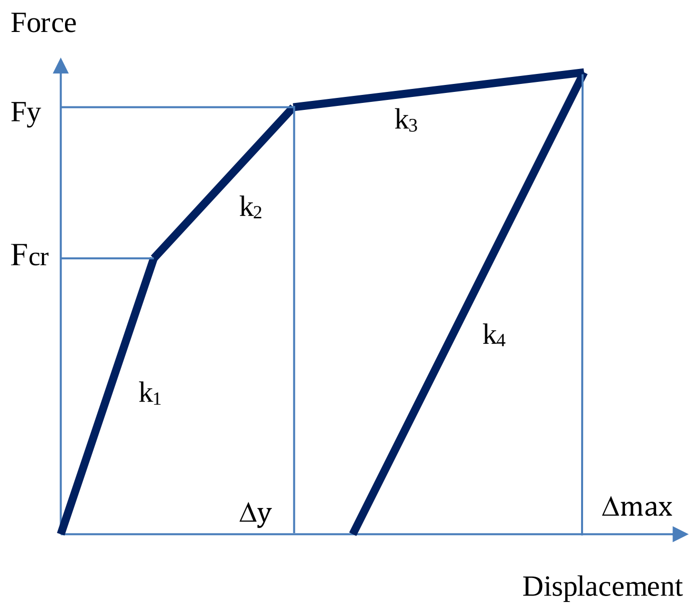


Figure 15: Load-displacement curve for nonlinear oscilators

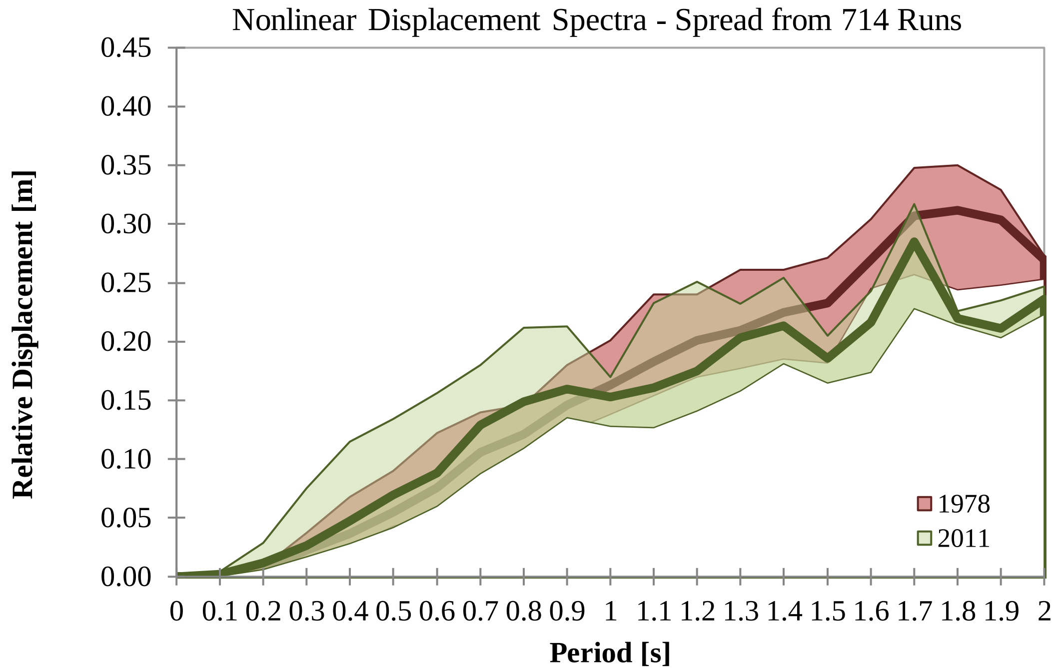
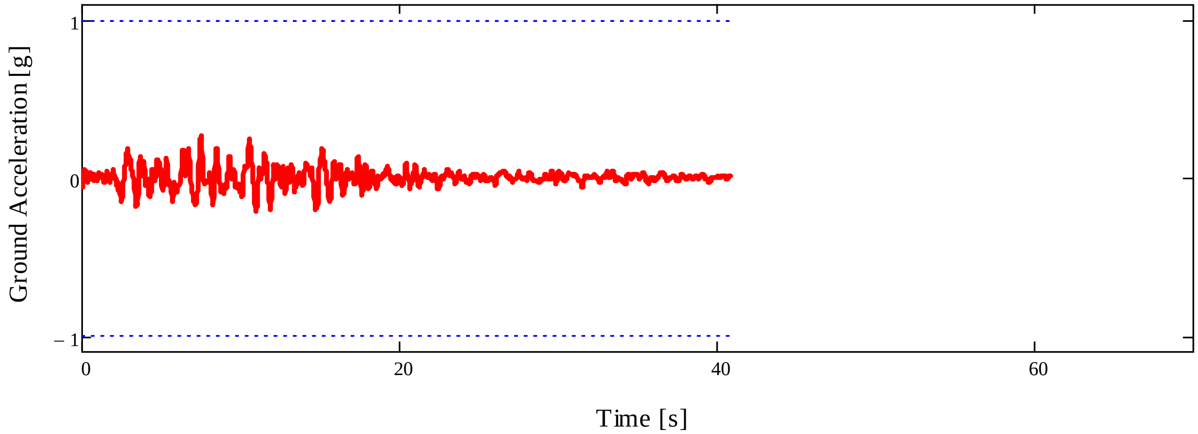
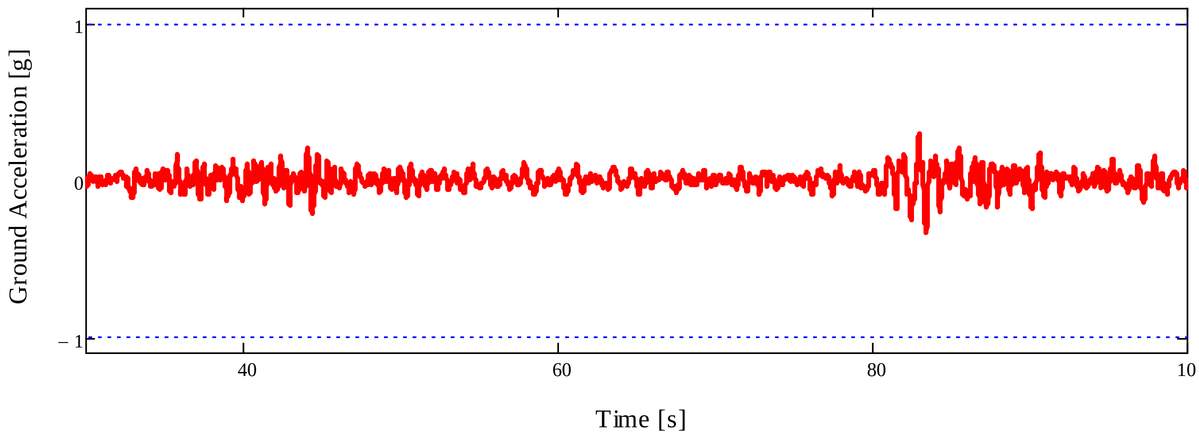


Figure 16: Nonlinear displacement spectra

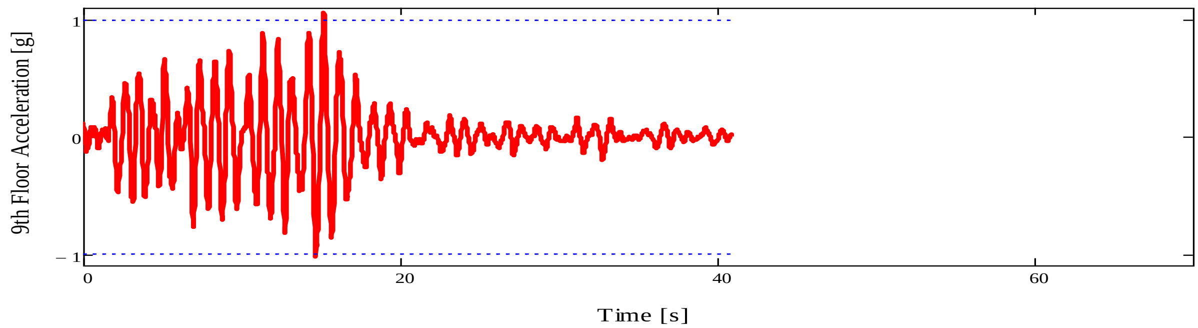


(a) 1978

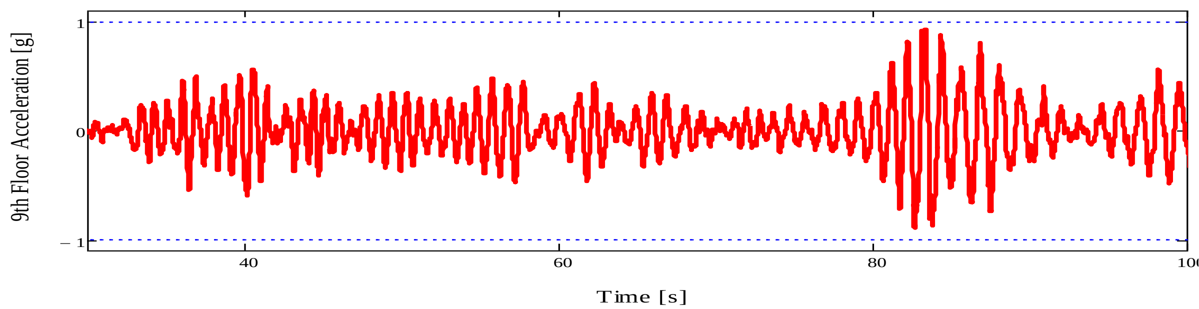


(b) 2011

Figure 17: Ground acceleration records

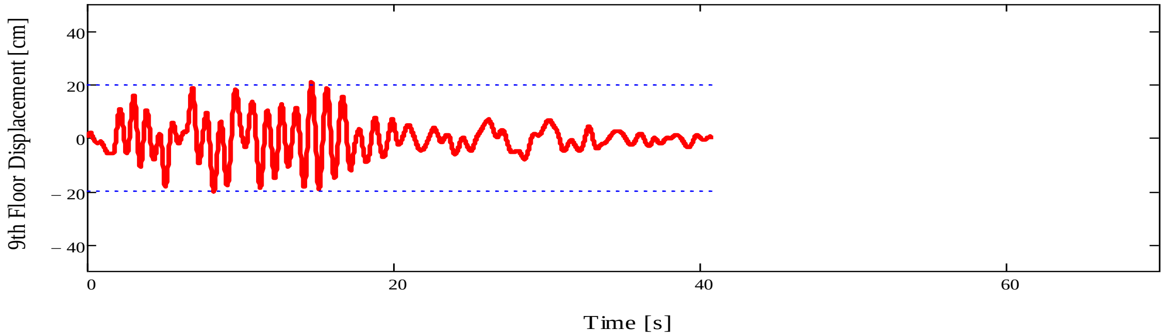


(a) 1978

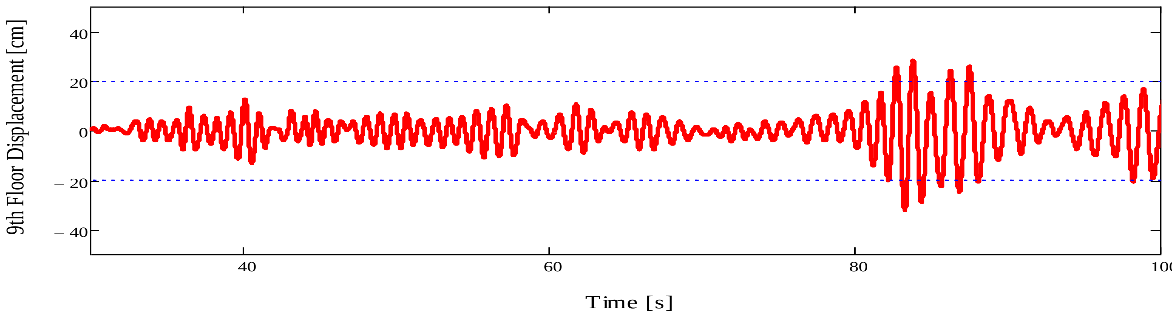


(b) 2011

Figure 18: Ninth-floor acceleration records

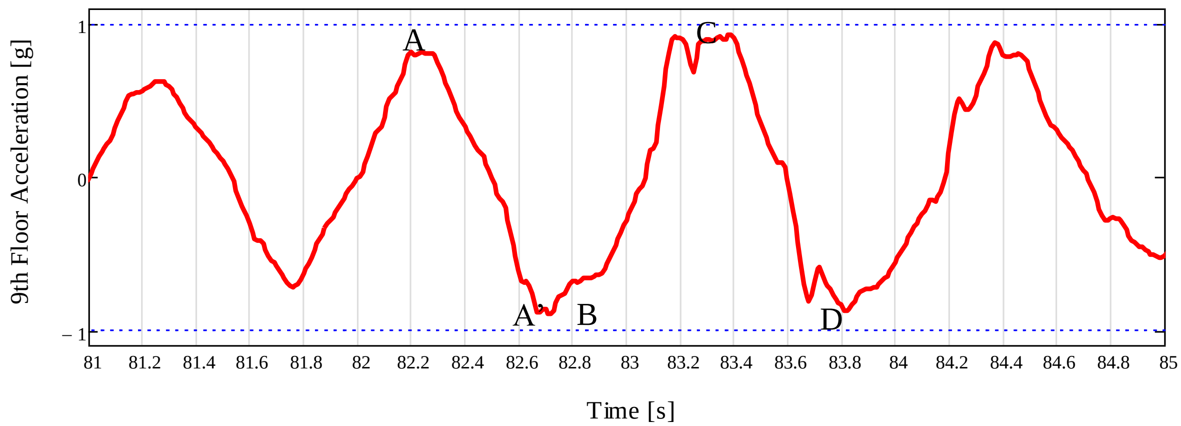


(a) 1978

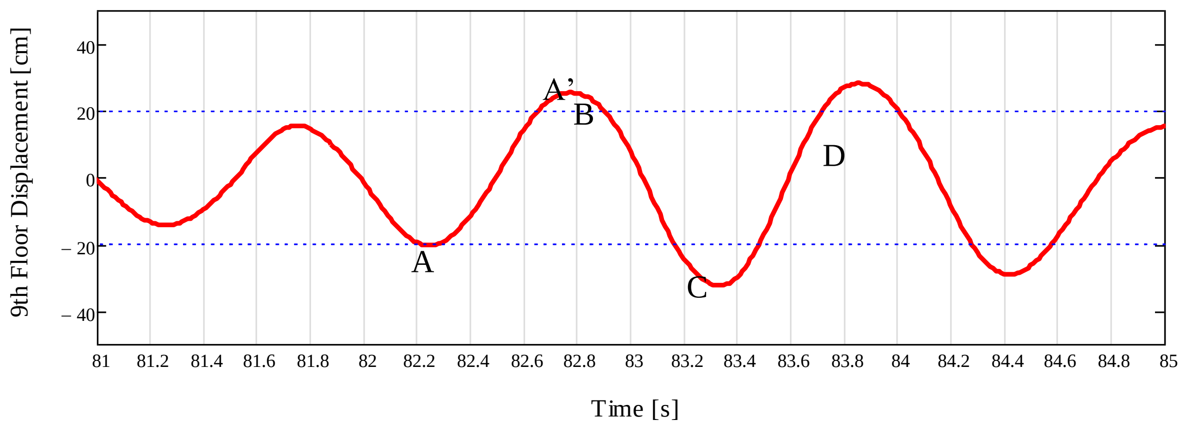


(b) 2011

Figure 19: Ninth-floor relative displacement

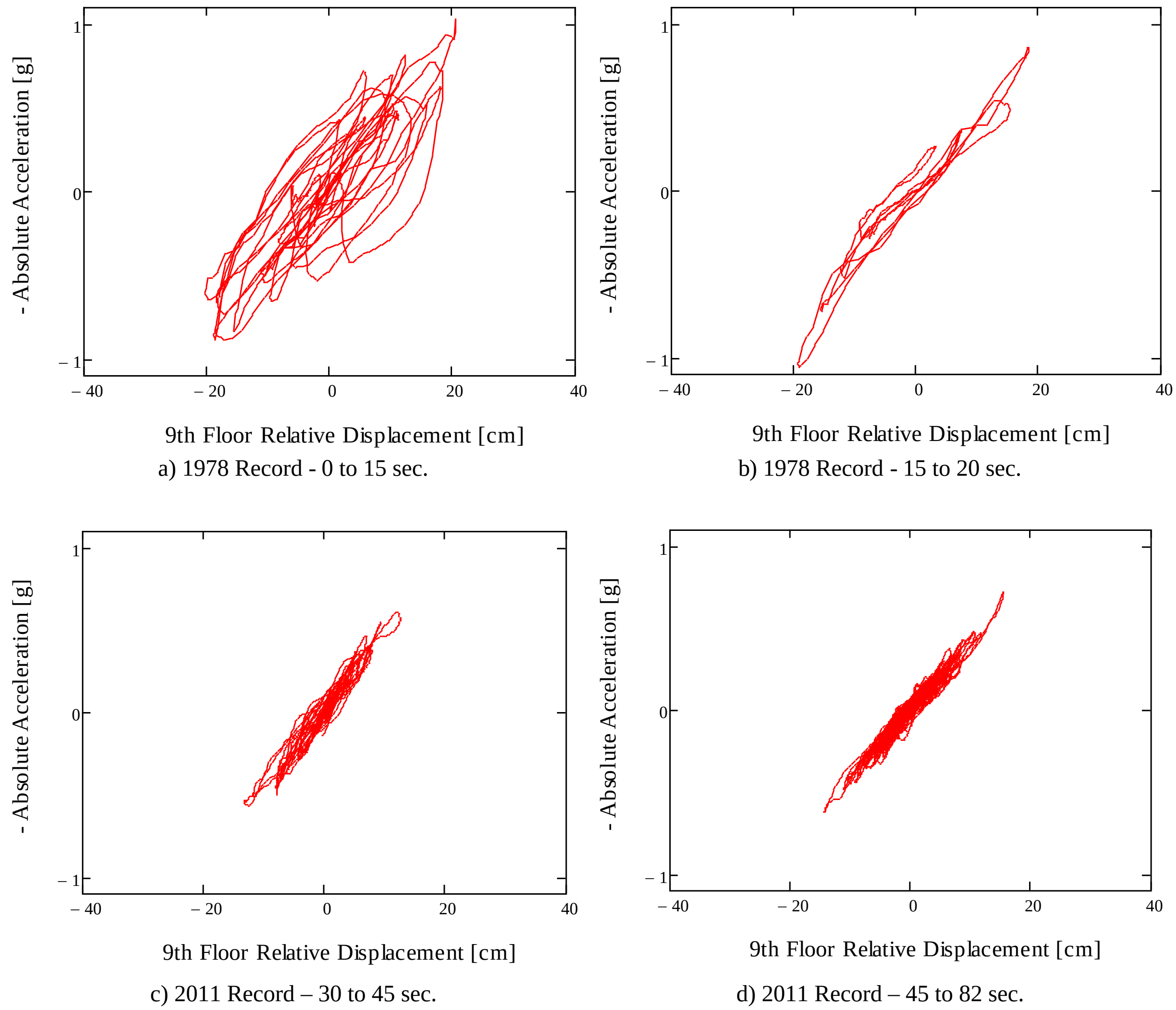


(a) acceleration



(b) displacement

Figure 20: Close-up views of ninth-floor acceleration and relative-displacement histories (2011)



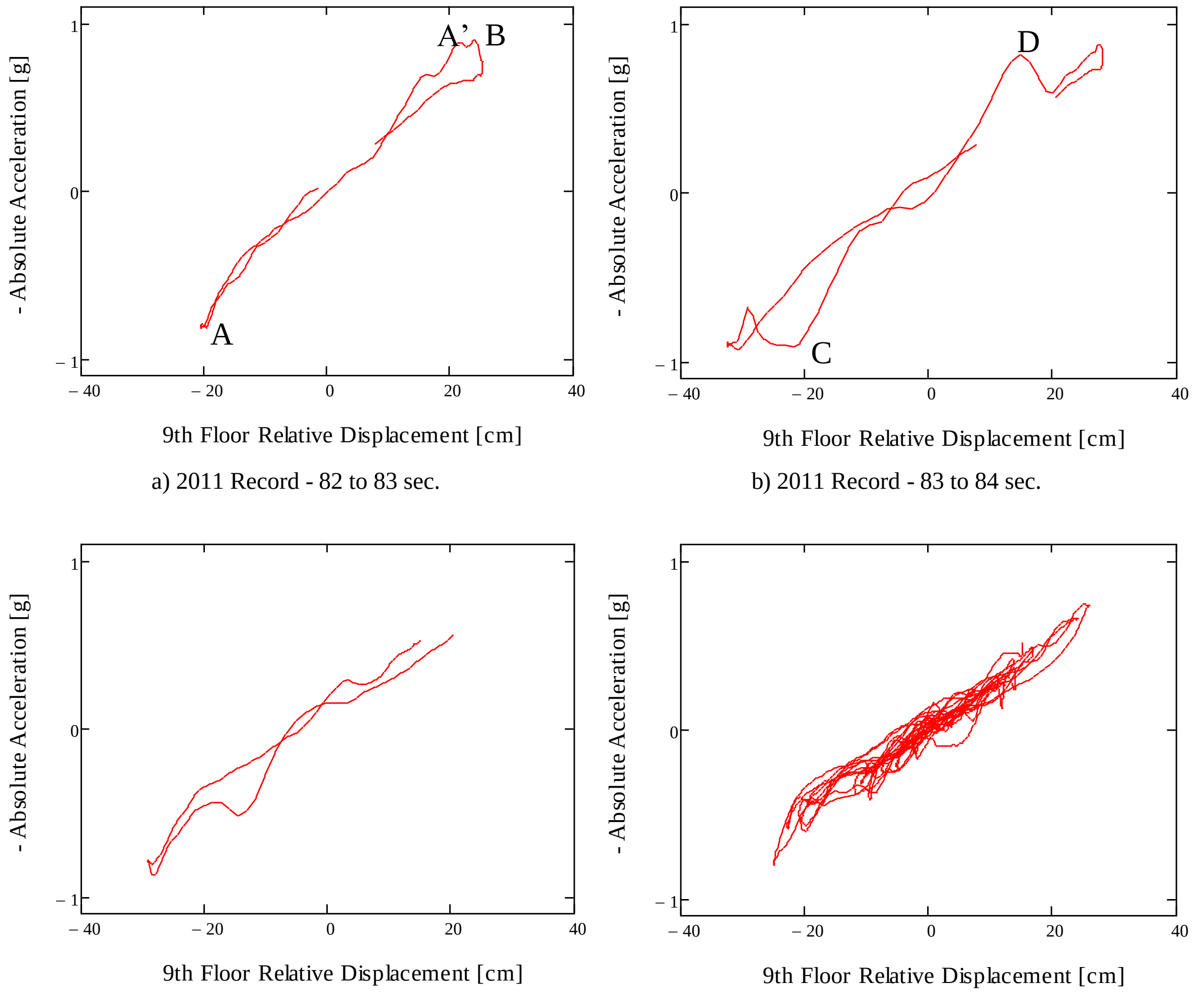
a) 1978 Record - 0 to 15 sec.

b) 1978 Record - 15 to 20 sec.

c) 2011 Record – 30 to 45 sec.

d) 2011 Record – 45 to 82 sec.

Figure 21: Initial 9th-floor acceleration-displacement response



a) 2011 Record - 82 to 83 sec.

b) 2011 Record - 83 to 84 sec.

c) 2011 Record - 84 to 85 sec.

d) 2011 Record - 85 to 100 sec.

Figure 22: Final 9th-floor acceleration-displacement response

APPENDIX A

Table A1: Size and reinforcement of beams, columns, and walls (Shiga et al., 1981)

Table A1.1: Wall reinforcement and thickness (all dimensions in mm)

Story	Wall type / Thickness						
	Wa	Wb	Wc	Wd	We	Wf	Wg
9	150	150	200	150	200		
8	150	150	200	150	200		
7	150	200	200	150	200		
6	150	200	200	150	200		
5	200	300	200	200	200		
4	200	300	200	200	200		
3	250	300	250	250	200		
2	300	400	300	300	200		150
1	400	500	400	400	200	250	150

Reinforcement	
Thickness	Reinforcement
150	φ9 @ 200, one layer
200	φ9 @ 200,two layers
250	φ9 @ 200,two layers
300	φ13 @ 200, two layers
400	φ13 @ 200, two layers
500	φ13 @ 200, two layers

Table A1.2: Column reinforcement (all dimensions in mm)

Story	Axes B, C, D between Axes 2 and 7	Axes A and E between Axes 1 and 9, Axes B and D with Axis 1	Axes B and D with Axes 9 and 9
9 8 7	BxD: 800x850[a] 8Ls-65x65x6[b] 12-D19[c]		
6 5 4	BxD: 800x850 8Ls-75x75x6 12-D19		
3	BxD: 800x850 8Ls-75x75x9 12-D22		
2	BxD: 800x850 8Ls-75x75x9 12-D22	BxD: 800x800 8Ls-75x75x9 12-D22	BxD: 850x850 8Ls-75x75x9 12-D22
1	BxD: 850x850 8Ls-75x75x12 12-D25	BxD: 800x800 8Ls-75x75x9 12-D22	BxD: 850x850 8Ls-75x75x12 12-D25

[a] First line, BxD: North-South dimension x East-West dimension
[b] Second line, Steel angles
[c] Third line, reinforcing bars

Table A1.3: Beam reinforcement

Table A1.3.1: Beam reinforcement part 1 (all dimensions in mm)

Story	Width x Depth, Steel angles	Axis 2 through 7, Between Axes B and D			
			Axes B and D	Mid-span	Axis C
R	400x800 4 Ls-65x65x6	Top	2-D22	2-D22	2-D22
		Bottom	2-D22	2-D22	2-D22
9		Top	4-D22	2-D22	4-D22
		Bottom	2-D22	2-D22	2-D22
7, 8		Top	2-D25 and 2-D22	2-D25	2-D25
		Bottom	2-D22	2-D22	2-D22
6		Top	4-D25	2-D25	2-D25 and 2-D22
		Bottom	2-D25	2-D25	2-D25
5	400x800 4 Ls-75x75x6	Top	4-D25	2-D25	4-D25
		Bottom	2-D25 and 2-D16	2-D25	2-D25
4	400x900 4 Ls-75x75x6	Top	4-D25	2-D25	2-D25 and 2-D22
		Bottom	2-D25 and 2-D16	2-D25	2-D25
2, 3	400x900 4 Ls-75x75x12	Top	4-D25	2-D25	2-D25
		Bottom	2-D25	2-D25	2-D25
		Stirrup	φ13 @300		
Foundation	450x1500	Top	5-D22	5-D22	5-D22
		Bottom	5-D22	5-D22	5-D22
		Stirrup	φ13 @300		

Table A1.3.2: Beam reinforcement part 2 (all dimensions in mm)

Story	Axis B and D, between Axes 1 through 9, Axis C between Axes 2 through 7			
	Width x Depth, Steel angles		At the ends	Mid-span
R	400x800 4 Ls-65x65x6	Top	4-D22	2-D22
		Bottom	2-D22	2-D22
6, 7, 8, 9		Top	4-D25	2-D25
		Bottom	2-D25	2-D25
5	400x800 4 Ls-75x75x6	Top	4-D25	2-D25
		Bottom	2-D25	2-D25
4	400x900 4 Ls-75x75x6	Top	4-D25	2-D25
		Bottom	2-D25	2-D25
2, 3	400x1100 4 Ls-75x75x9	Top	4-D25	2-D25
		Bottom	2-D25	2-D25
Foundation	450x1500	Top	6-D22	5-D22
		Bottom	6-D22	5-D22
		Stirrups	φ13 @ 300	

Table A1.3.3: Beam reinforcement part 3 (all dimensions in mm)

Story	Axes 1 through 9, between Axes A and B and Axes D and E				
	Width x Depth, Steel angles		Axes A and D	Mid-span	Axes B and D
3	450x1000 4 Ls-75x75x9	Top	4-D25	2-D25	4-D25 and 2-D22
		Bottom	2-D25	2-D25 and 2-D16	2-D25 and 2-D16
		Stirrups	φ13 @ 300	φ 9 @ 300	φ13 @ 300
2	450x1100 4 Ls-75x75x9	Top	4-D25	2-D25	4-D25
		Bottom	2-D25	2-D25	2-D25
		Stirrups	φ13 @ 300	φ 9 @ 300	φ13 @ 300
Foundation	450x1500	Top	6-D22	5-D22	6-D22
		Bottom	6-D22	5-D22	6-D22
		Stirrups		φ13 @300	

Table A1.3.4: Beam reinforcement part 4 (all dimensions in mm)

Story	Width x Depth, Steel angles	Axes A and E, between Axes 1 and 9		
			At the ends	Mid-span
3	450x1000 4 Ls-75x75x6	Top	4-D22	2-D22
		Bottom	2-D22	2-D22
		Stirrups	φ9 @ 300	
2	450x1100 4 Ls-75x75x9	Top	4-D25	2-D25
		Bottom	2-D25	2-D25
		Stirrups	φ9 @ 300	
Foundation	450x1500	Top	4-D22	4-D22
		Bottom	4-D22	4-D22
		Stirrups	φ13 @ 300	

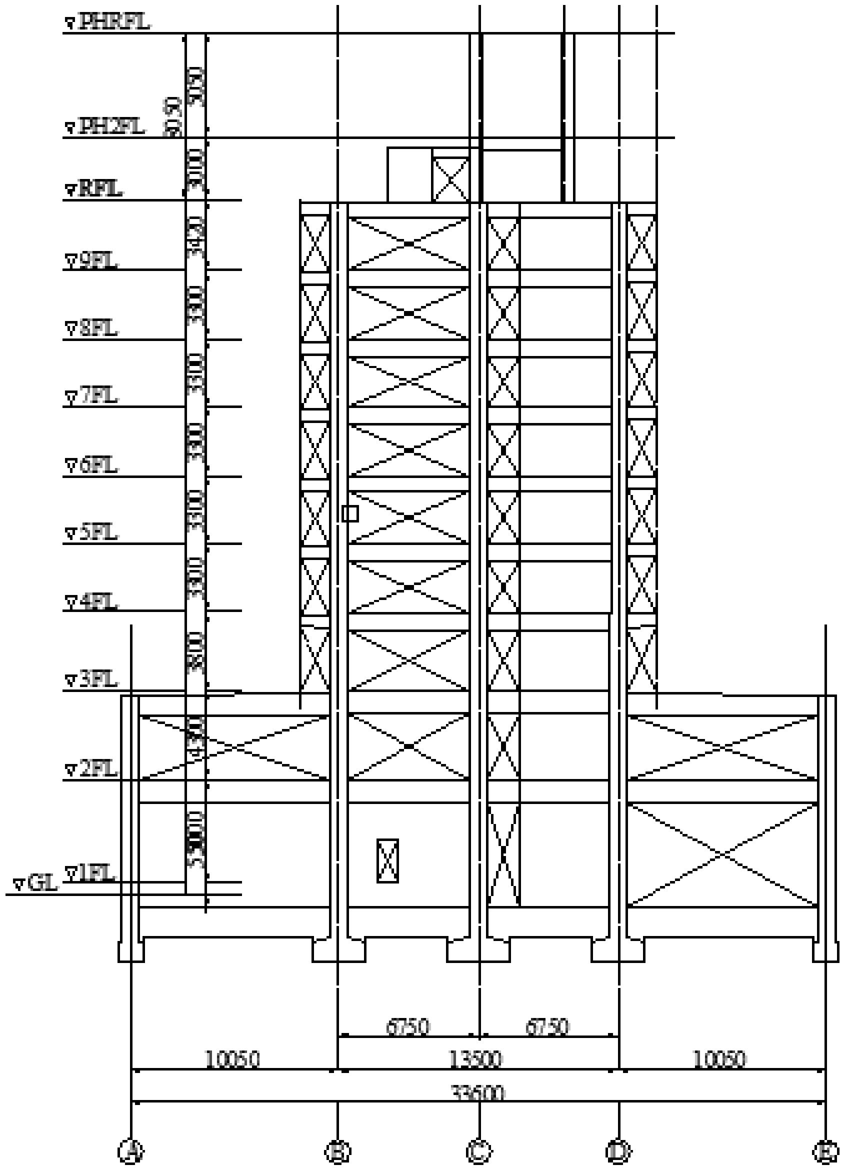


Figure 23: Elevation of Axis 4 (all dimensions in mm)

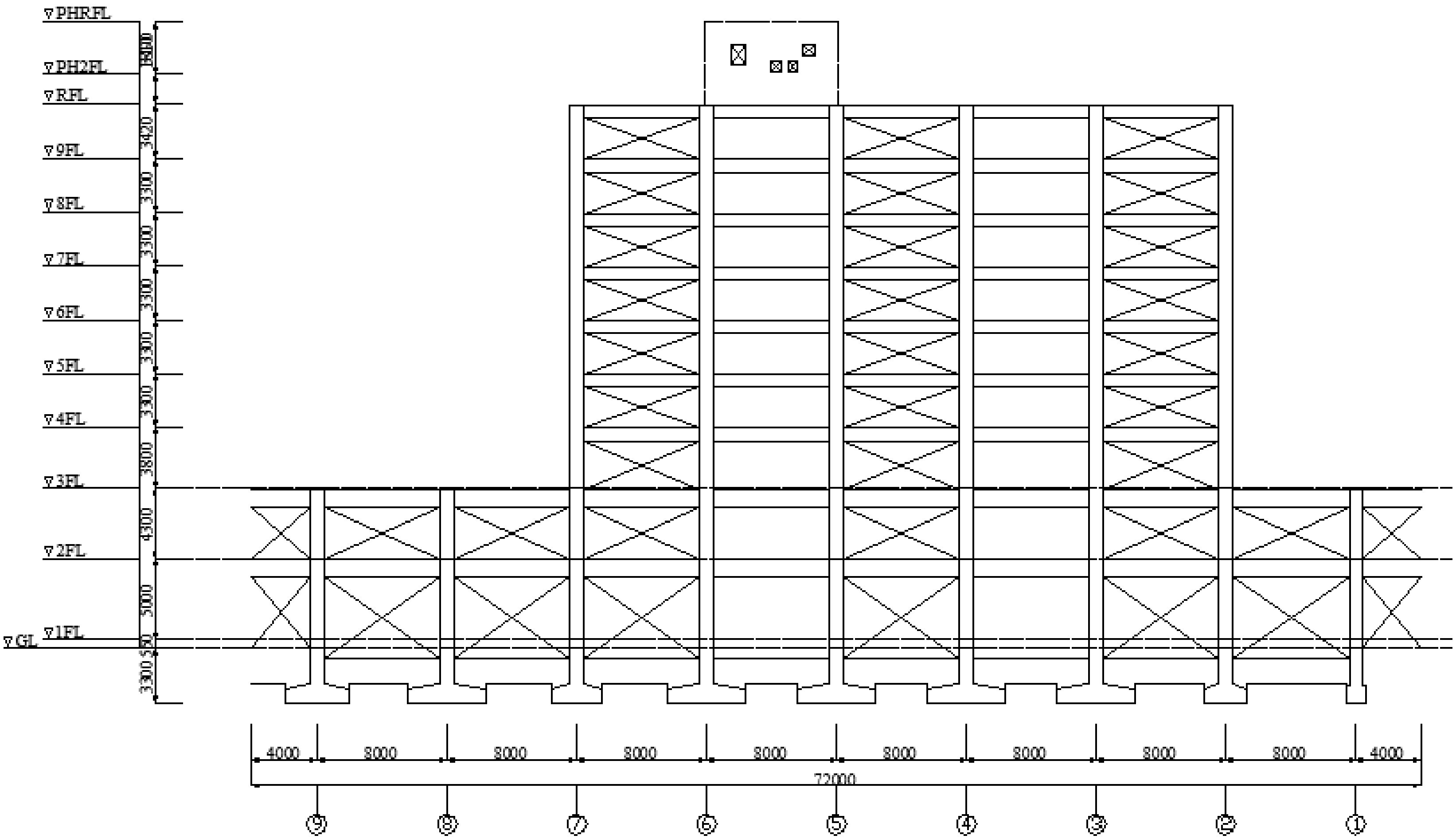


Figure 24: Elevation of Axis D (all dimensions in mm)