Seismic retrofit of reinforced concrete buildings in Japan using external precast, prestressed concrete frames

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- Two types of seismic retrofits have been developed in Japan for use on existing reinforced concrete buildings.
- Both methods involve attaching external precast, prestressed concrete frames to the buildings.
- This paper describes the retrofit methods and examines two buildings retrofitted before the Tohoku earthquake of March 11, 2011. Both performed as designed.

Seismic design methods in Japan have progressed significantly because of the lessons learned from experience. Existing buildings found not to conform to current codes are required to be retrofitted.

Seismic-resistant design methods in Japan date back to 1924, just after the Great Kanto earthquake of 1923. Buildings were designed to resist a horizontal force equal to the building weight multiplied by a seismic coefficient of 0.1. This was the first seismic building code in the world. When the *Building Standard Law of Japan*[1] was enacted in 1950, the seismic coefficient was increased to 0.2 to ensure consistency with the doubled allowable stresses in concrete and reinforcing bars. As a result, the required strength of buildings remained the same. The 1968 Tokachi-oki earthquake and the 1978 Miyagi-oki earthquake caused significant and unexpected damage, especially by brittle shear failure in low- and midrise reinforced concrete buildings. Japanese engineers learned much from these earthquakes, and the mitigation of earthquake damage became urgent. This encouraged a variety of research. As a result, the *Standard for Revised Earthquake Resistant Design*[2] enacted in 1981 adopted a ductility design method in addition to a conventional strength design method.

Methods to evaluate the seismic safety of existing reinforced concrete buildings became important, as did procedures to retrofit them and reduce the loss of lives. After the 1995 Kobe earthquake, the *Law for Promotion of Seismic Retrofit of Buildings*[3] was enacted in 1997. This law requires existing buildings to be evaluated and retrofitted to conform to the current standard. The seismic evaluation and retrofit of reinforced concrete buildings in practice are based on the *Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings*[4] and *Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings*.[5]

Classification of retrofit methods

A variety of seismic retrofitting methods in Japan have been developed. These methods can be roughly classified into three groups by their design objective (**Fig. 1**). The first group increases the lateral load–carrying capacity by installing or attaching frames or walls. Although adding a concrete wall will increase the shear strength of a building, it also adds weight. Thus the capacity of the foundation must be verified. This retrofit method may involve interior or exterior reinforcement. Interior reinforcement displaces the occupants during construction. Exterior reinforcement allows the occupants to use the building without interruption while the retrofit is in progress and maintains the function of the interior. A concrete outer frame may be either cast-in-place reinforced concrete or precast, prestressed concrete.

The second type of retrofit method increases the ductility of existing columns or beams, for example, by wrapping with carbon-fiber-reinforced polymer (CFRP). A CFRP retrofit requires few workers, but fire prevention measures are required. A seismic slit mitigates brittle failure of short columns but reduces the lateral load capacity of the building.

The third type of retrofit is mitigation of seismic response, for example, by installing seismic isolators or damping devices. A seismic isolator lengthens the period of a building and lessens the earthquake energy input. A seismic damping device absorbs earthquake energy and enhances seismic performance.

Reinforced concrete buildings retrofitted by external precast, prestressed concrete frames

Several kinds of precast, prestressed concrete seismic retrofit methods have been developed and adopted for many buildings in Japan. The Tohoku earthquake of March 11, 2011, a 9.0 on the moment magnitude (Mw) scale, including the maximum Japan Meteorological Agency (JMA) seismic intensity of 7 (XI–XII on the Modified Mercalli Intensity [MMI] scale) at Kurihara, Miyagi prefecture, shook the Tohoku and Kanto areas. After the earthquake, 19 buildings retrofitted with a precast, prestressed concrete outer frame and 40 buildings retrofitted with a parallel unit frame were evaluated. The seismic intensity experienced by these buildings ranged from 4 to 6 upper on the JMA scale (corresponding approximately to V to XI on the MMI scale) (**Fig. 2**). Cracks were not observed in the precast concrete frames or their connections.[7] Visual examination conducted by teams of engineers following the earthquake indicated that cracks over 0.2 mm (0.008 in.) were not present. For reference, the Japan Building Disaster Prevention Association (JBDPA) classifies earthquake damage as shown in **Table 1.**[8] The results verified that the buildings retrofitted with precast concrete frames that were designed to meet the performance of the structure to the assumed earthquake forces performed satisfactorily.

Standard for seismic evaluation of existing reinforced concrete buildings in Japan

The seismic evaluation standard provides three levels of calculation procedures, from simple to sophisticated. The first-level screening procedure is valid for strength evaluation in buildings with many walls and can be used for approximate evaluation. The second level screening procedure is valid for buildings likely to have column failures. Most buildings are evaluated by this procedure. The third-level screening procedure is valid for buildings likely to have beam failure and bearing wall rotation. This procedure requires a frame analysis, which involves a nonlinear analysis and an earthquake response analysis. This paper includes an example of a second-level screening.

Figure 3 shows the flowchart of the evaluation of an existing building.

Step 1 establishes the seismic demand index of structure I_{S0}[4–6] defined by Eq. (1).

$$I_{S0} = E_s ZGU \tag{1}$$

where

E_S = basic seismic demand index of structure

= 0.6 for the second-level screening procedure

Z = zone index

G = ground index

U = usage index

Typically, I_{S0} is 0.6 when Z, G, and U equal 1. Its value rises to 0.7, 0.8, or more according to the priority of the building. A higher priority is assigned to facilities such as schools, hospitals, firehouses, and government offices, which must function just after an earthquake. Such des-

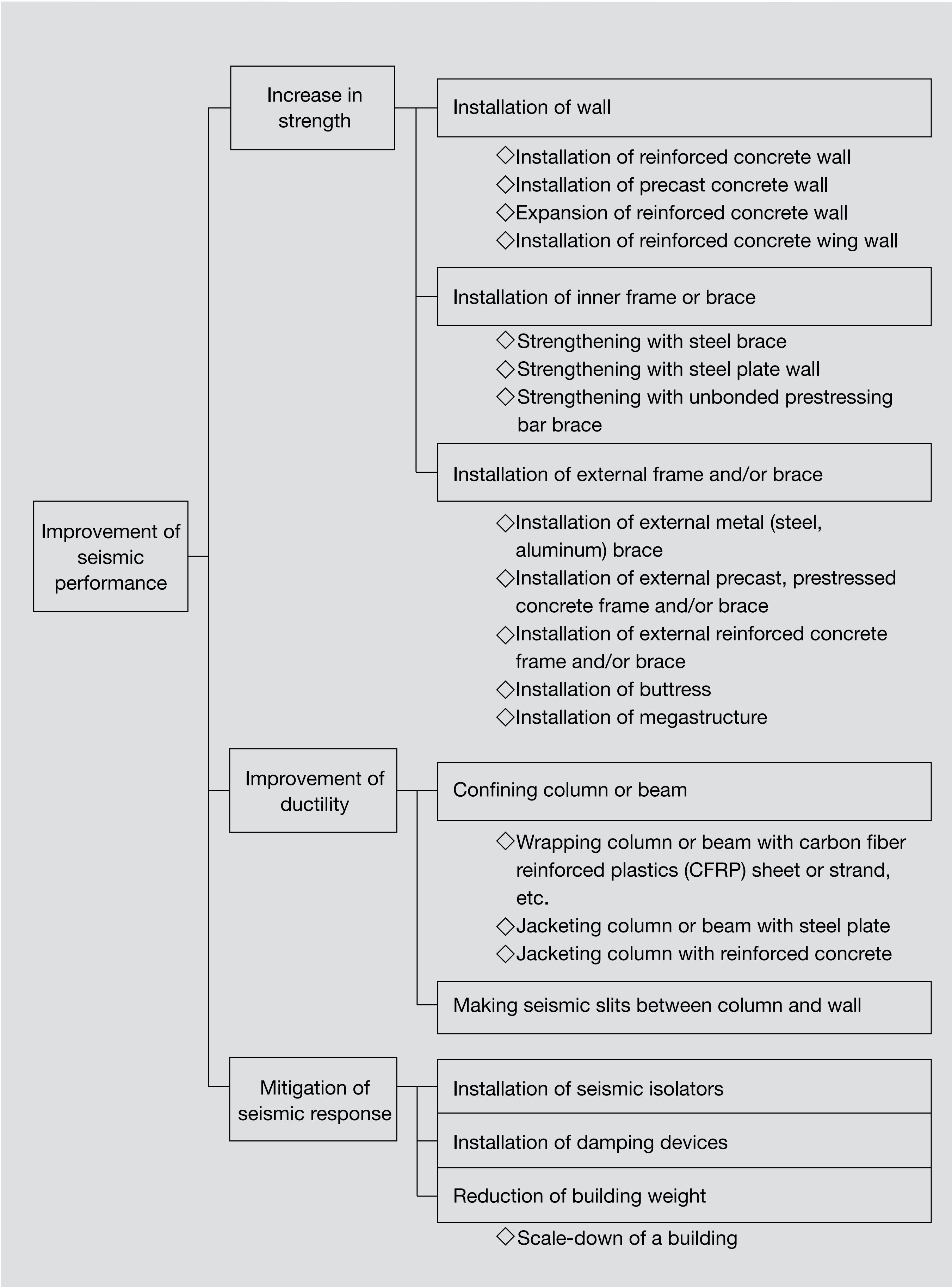


Figure 1. Classification of seismic retrofit methods in Japan.

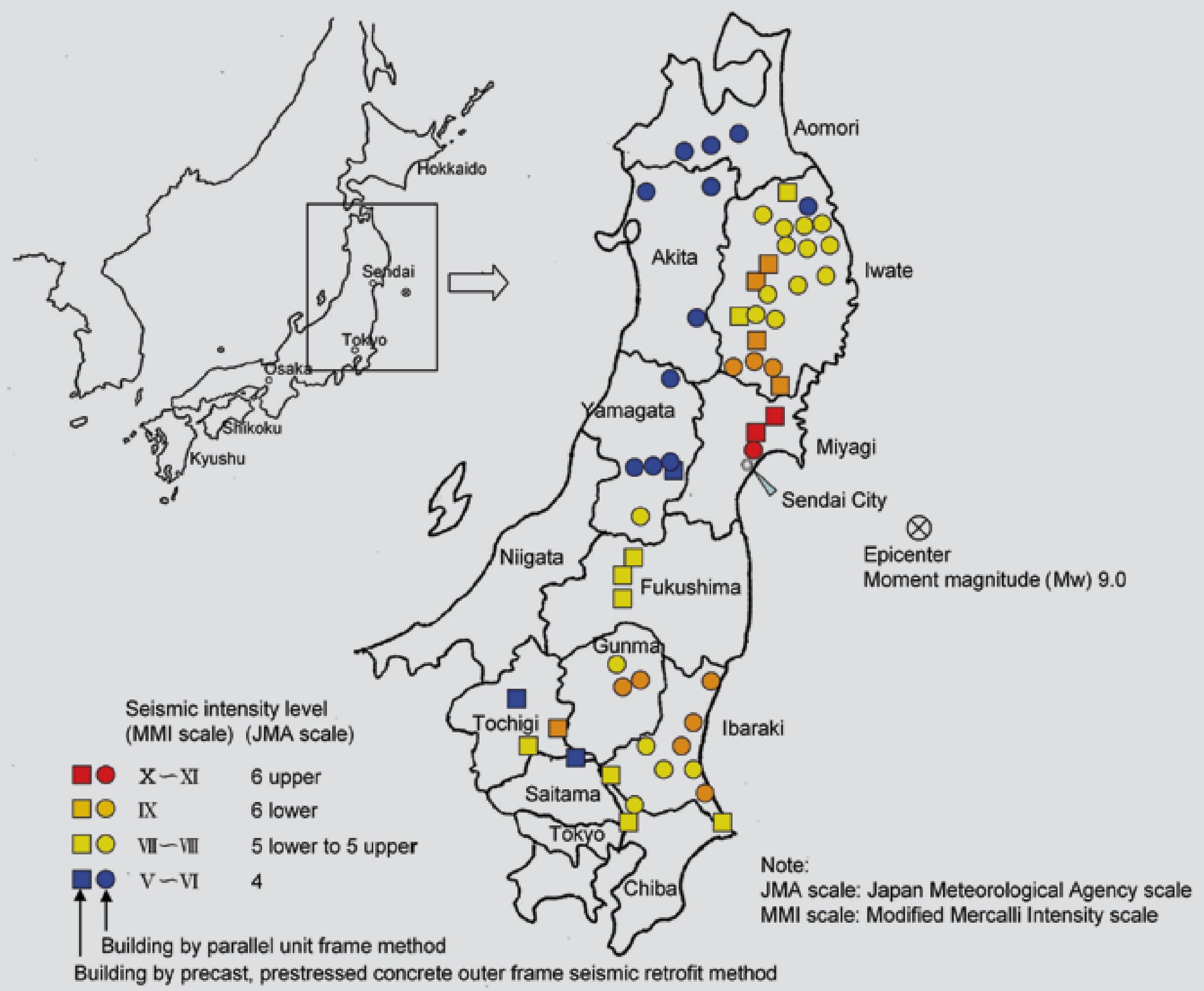


Figure 2. Map of Tohoku and Kanto areas of buildings retrofitted by the parallel unit frame method and by the outer frame seismic retrofit method. Note: Strict conversion from the seismic intensity of the Japan Meteorological Agency (JMA) scale to the Modified Mercalli Intensity (MMI) scale is difficult because their scales are classified based on human perception. The present JMA scale uses the measured value of 4313 seismic intensity meters (at the time of August 2011), which were installed all over Japan starting in 1996. The contrast of the JMA scale and the MMI scale here is based on the authors' decision due to the description of each damage level. Also, the moment magnitude scale is in common use worldwide for large earthquakes instead of the JMA scale or the Richter scale because of magnitude saturation. That is, the Richter scale reaches a ceiling at approximately 6.5 to 7.0.

Table 1. Classification of damage by an earthquake

Damage level of column and bearing wall		Description of damage
I	Negligible	Invisible crack without looking closely, crack width is 0.2 mm or less
II	Almost negligible	Crack visible to the naked eye, crack width ranges from approximately 0.2 mm to 1 mm
III	Slightly damaged	Comparatively large crack but little spalling of concrete, crack width ranges from approximately 1 mm to 2 mm
IV	Half damaged	Many larger cracks of more than 2 mm occur, spalling of concrete is heavy and many reinforcing bars are exposed
V	Badly damaged	Reinforcing bars are buckled, concrete inside reinforcing cage has fallen apart, and vertical deformations of columns and walls are seen; distinct feature is that subsidence and inclination occurred and/or reinforcing bars were broken

Note: 1 mm = 0.0394 in.

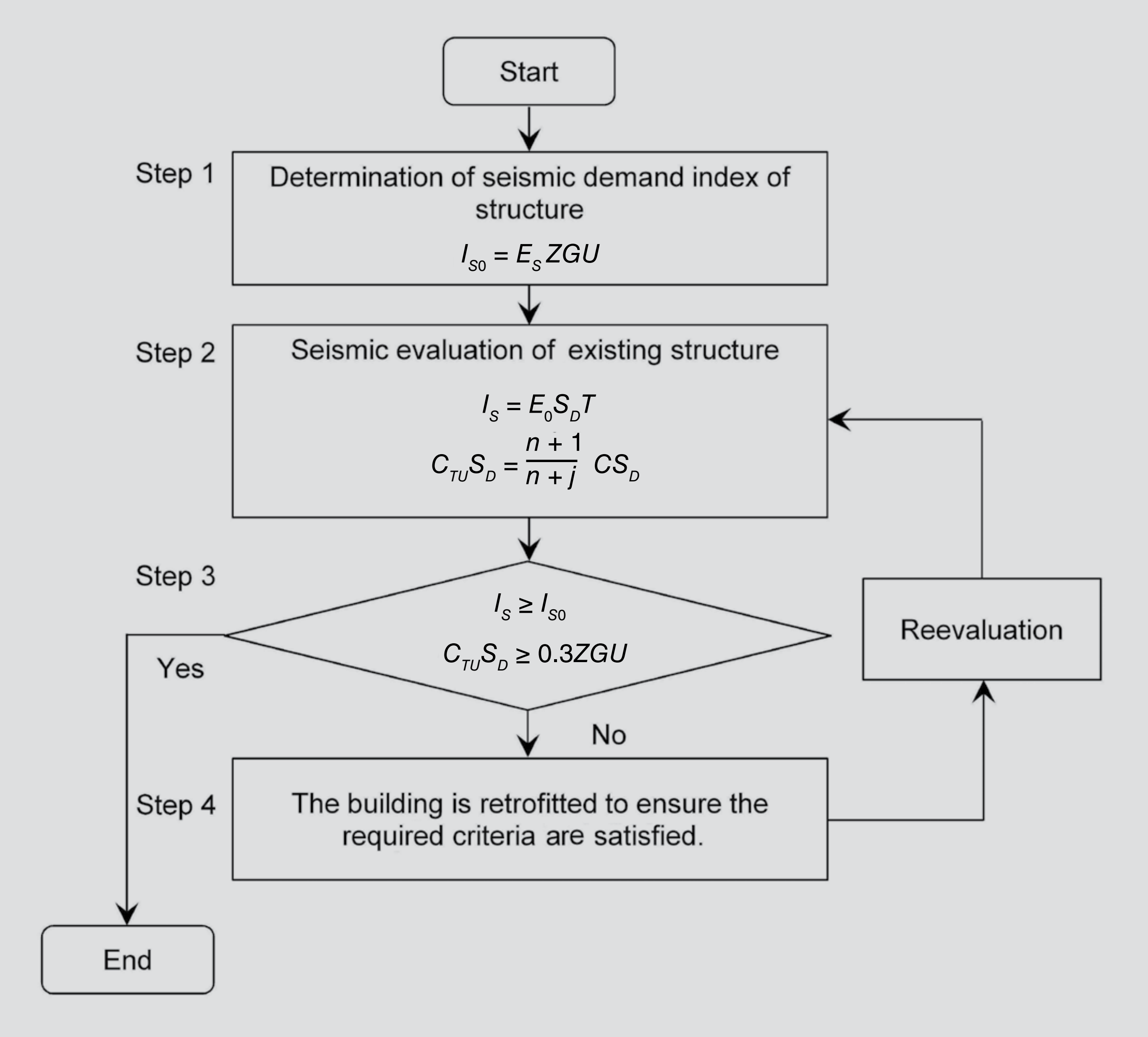


Figure 3. Flowchart of seismic evaluation of the existing building. Note: C = strength index; C_{TU} =cumulative strength index at ultimate deformation of a building; E_0 = basic seismic index of structure; E_s = basic seismic demand index of structure; G = ground index; I_S = seismic index of structure; I_{S0} = seismic demand index of structure; j = jth-story level of an n-story building; n = total number of stories of a building; S_D = irregularity index; T = time index; U = usage index; Z =zone index.

ignations are often determined by municipal governments and may differ from place to place.

Step 2 calculates the seismic index of structure I_S by Eq. (2) at each story in each principal direction of the building before retrofit.[4–6]

$$I_S = E_0 S_D T \tag{2}$$

where

E_0 = basic seismic index of structure

S_D = irregularity index (0.4 to 1.0)

T = time index

The basic seismic index of structure E_0 is the product of the strength index C, ductility index F, and story-shear modification factor $\frac{n+1}{n+j}$.

n = total number of stories of a building

j = jth story of an n-story building

When E_0 is considered to be ductility-dominant, then E_0 is defined by Eq. (3).[4–6]

$$E_0 = \frac{n+1}{n+j}\sqrt{E_1^2 + E_2^2 + E_3^2} \quad (3)$$

where

$E_1 = C_1 F_1$

$E_2 = C_2 F_2$

$E_3 = C_3 F_3$

C_1 = strength index of the first group (with small F)

C_2 = strength index of the second group (with medium F)

C_3 = strength index of the third group (with large F)

F_1 = ductility index of the first group

F_2 = ductility index of the second group

F_3 = ductility index of the third group

For the calculation of the basic seismic index of structure E_0, vertical members are classified by the ductility indices F into three groups in order of the smallest values of the ductility indices to the largest.

Ductility index F ranges from 1.0 (mostly brittle, with interstory drift angle 1/250 radian) to 3.2 (mostly ductile, with interstory ssdrift angle 1/30 radian).

When the basic seismic index of structure E_0 is considered to be strength-dominant, then E_0 is defined by Eq. (4).[4–6]

$$E_0 = \frac{n+1}{n+j}\left(C_1 + \sum_j \alpha_j C_j\right)F_1 \quad (4)$$

where

α_j = effective strength factor in the jth group elements at ultimate deformation R_1 corresponding to the first group elements (ductility index F_1)

C_j = strength index of the jth group (j = 2, 3)

F_1 usually ranges from 0.8 to about 1.5; 0.8, 1.0, 1.27, and 1.5 correspond to 1/500, 1/250, 1/150, and 1/125 radian, respectively, of interstory drift.

Figure 4 shows the relation of ductility index F and strength index C in Eq. (4). Strength index summation is the strength index C_1 of the first group plus the sum of strength indices C_2 and C_3 multiplied by effective strength factors α_2 and α_3, respectively, at the ultimate deformation of the first group (ductility index F_1).

The strength index C in the second-level procedure is calculated by Eq. (5):

$$C = \frac{Q_u}{\Sigma W} \quad (5)$$

where

Q_u = ultimate lateral load–carrying capacity of the vertical members in the story concerned

ΣW = weight of the building including the live load for seismic calculation supported by the story concerned

Also, step 2 calculates the product of the ultimate cumulative strength index C_{TU} and the irregularity index S_D by Eq. (6) to avoid irreparable damage and unacceptable residual deformation during a major earthquake.

$$C_{TU}S_D = \frac{n+1}{n+j}CS_D \quad (6)$$

The cumulative strength index at ultimate deformation of a building C_{TU} is the product of the story-shear modification factor $\frac{n+1}{n+j}$ and the strength index C.

Step 3 uses Eq. (7) to compare the seismic demand index of structure I_{S0} with the seismic index of structure I_s to identify the structural safety in an earthquake.[4–6]

$$I_S \ge I_{S0} \quad (7)$$

$C_{TU}S_D$ must meet the minimum requirement of Eq. (8).[4–6]

$$C_{TU}S_D \ge 0.3ZGU \quad (8)$$

If I_{S0} is greater than I_S and/or $C_{TU}S_D$ is less than 0.3ZGU, the building must be retrofitted.

Precast, prestressed concrete outer-frame seismic retrofit method

Description

The precast, prestressed concrete outer-frame seismic retrofit increases a building's lateral load-carrying capacity by attaching a precast, prestressed concrete frame to the outside of a reinforced concrete building (**Fig. 5**).

The exterior frame is built with precast concrete columns and beams on the existing foundation or on a newly installed cast-in-place concrete foundation that is integrated

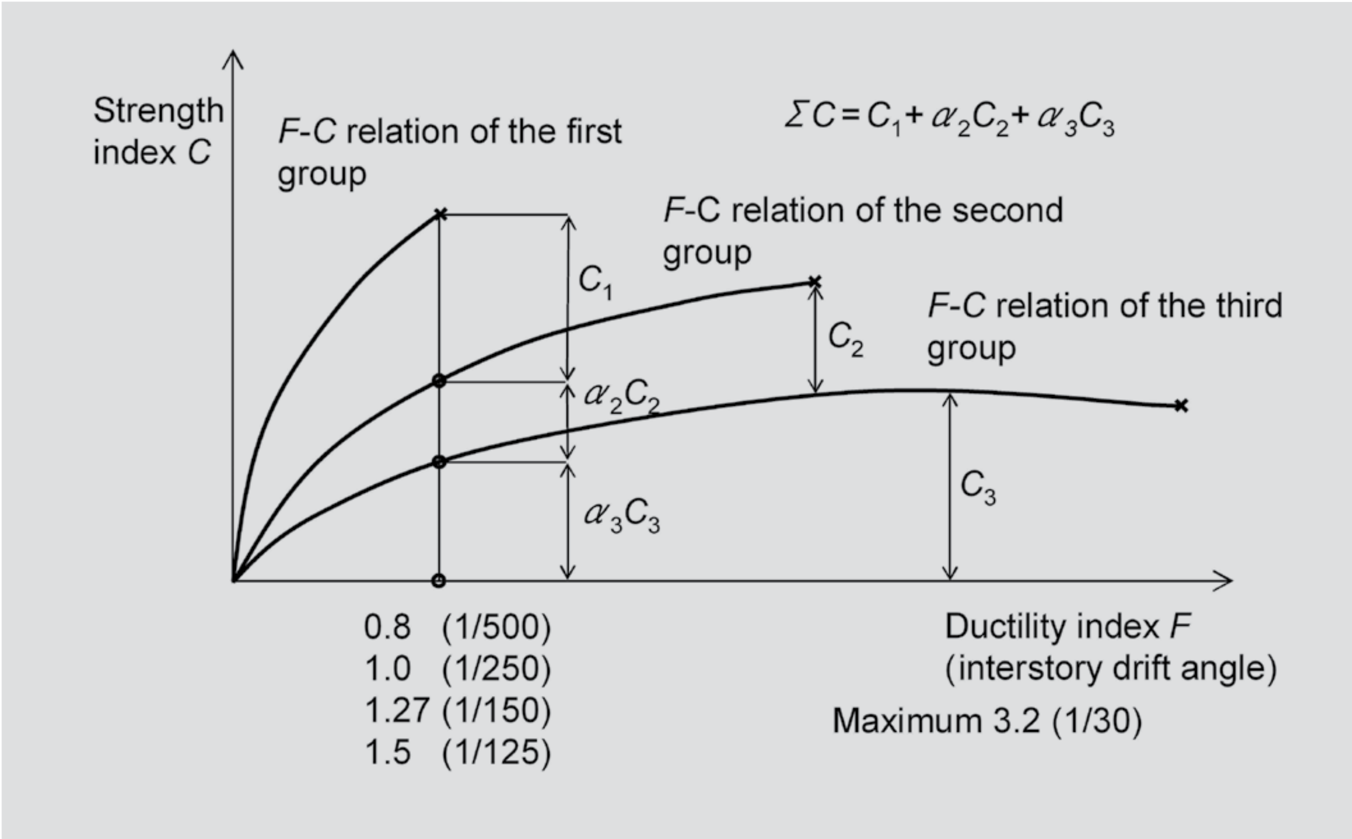


Figure 4. Relationship between the strength index C and ductility index F at strength-dominant structure. Note: The number in parentheses is interstory drift angle. Ductility indices 0.8, 1.0, 1.27, and 1.5 (corresponding to interstory drift angles 1/500, 1/250, 1/150 and 1/125, respectively). C_1 = strength indices of the first group; C_2 = strength indices of the second group; C_3 = strength indices of the third group, α_2 = effective strength factors in the second group at ultimate deformation corresponding to the first group (ductility index F_1); α_3 = effective strength factors in the third group at ultimate deformation corresponding to the first group (ductility index F_1).

with the existing foundation. Splice sleeve connectors comprise the column splices and column-to-foundation joints; the beam-column joints are posttensioned (**Fig. 6**).

The shear force is transmitted from the building to the exterior frame through prestressing steel bar or a cast-in-place concrete floor slab between the frame and the building.

Shear transfer by prestressing bars The transmission of shear in this method is by friction (friction coefficient $\mu = 0.7$) between the existing beam and the exterior frame (the left side of **Fig. 7**). This method can be used when space is limited. It requires drilling holes into the beam for the prestressing bars. The lateral load–carrying capacity of this method is limited because of the shared existing foundation.

When the overturning moment due to lateral force causes uplift of the end column of the exterior frame, the weight of the foundation plus the friction resistance of the piles must exceed the pull-out force. However, the axial forces of the building columns may also be included in calculating the resistance to uplift.

Shear transfer by the floor slab In this method, the transmission of shear force from the building to the exterior frame is achieved by the cast-in-place concrete slab between them, the bolts anchored in the building, and reinforcing bars embedded in the exterior frame (the right side of Fig. 7). This method can be used in a building with a balcony.

For the moment due to the eccentricity between the exterior frame and the building during an earthquake, the orthogonal beams and anchored bolts at the far ends of the frame react in axial tension and compression (**Fig. 8**).

Scope

The precast, prestressed concrete outer-frame seismic retrofit method is applicable to reinforced concrete buildings and steel-frame reinforced concrete buildings up to 14 stories high. Between 1999 and 2012, 493 projects, including school buildings, apartments, city halls, and hospitals, were retrofitted by this method. **Figure 9** shows a 14-story apartment building retrofitted in 2010. The failure mode of the frame is basically column yielding, and both the columns and the beams of the frame should have flexural yielding to avoid brittle failure. Only the end columns of the frame can allow beam yielding by limiting the clear

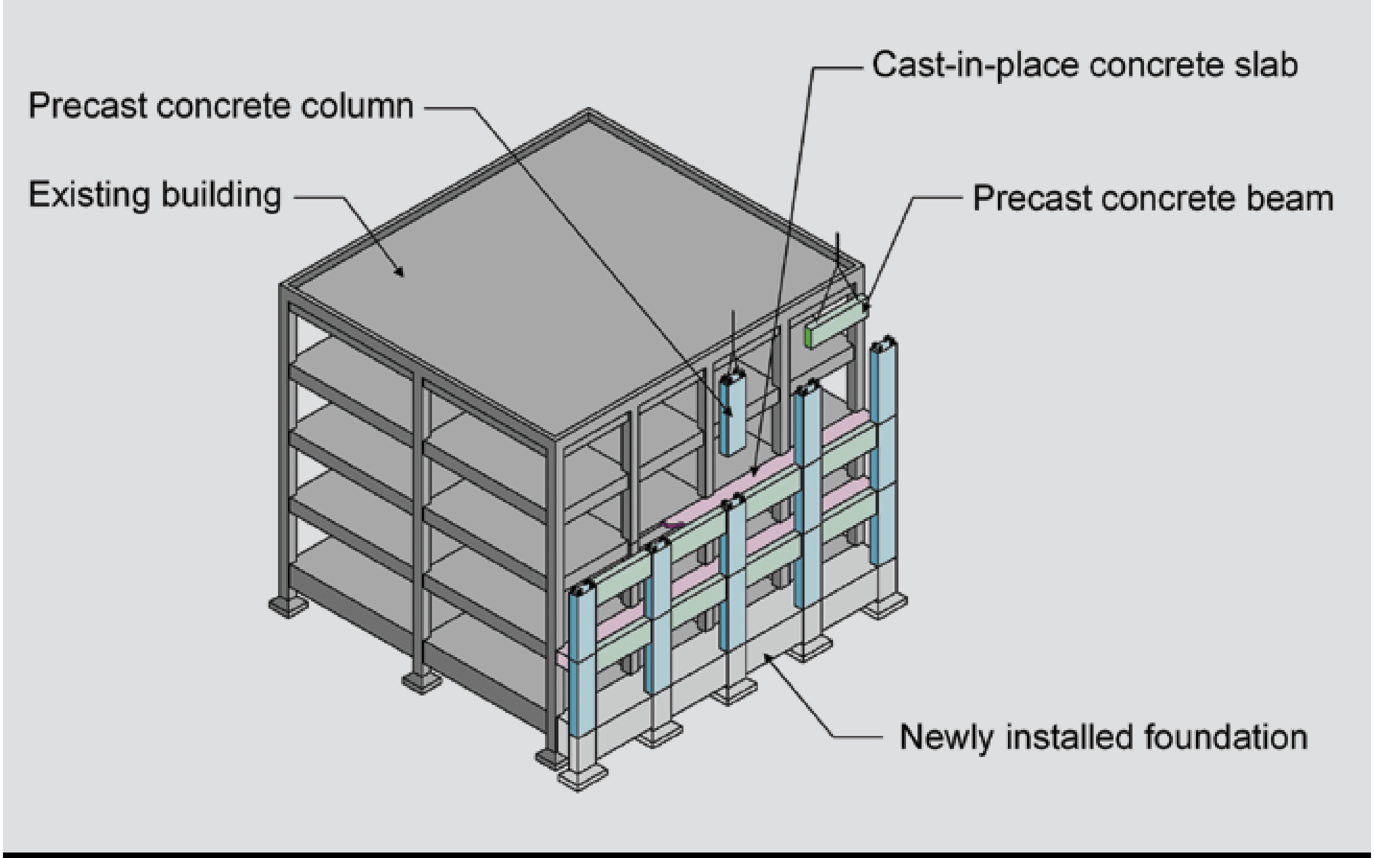


Figure 5. Outline of outer-frame seismic retrofit method.

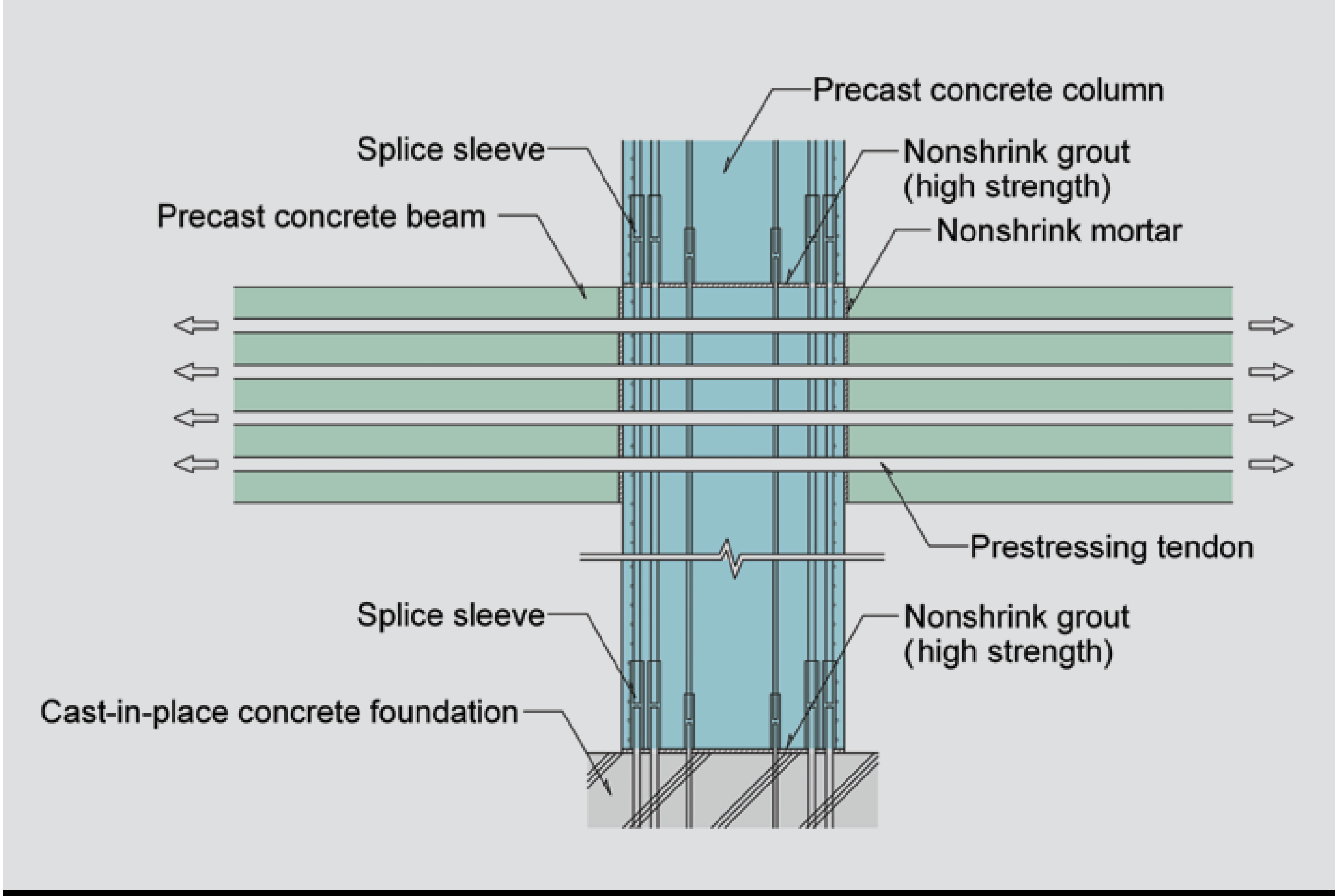


Figure 6. Connection of precast concrete elements.

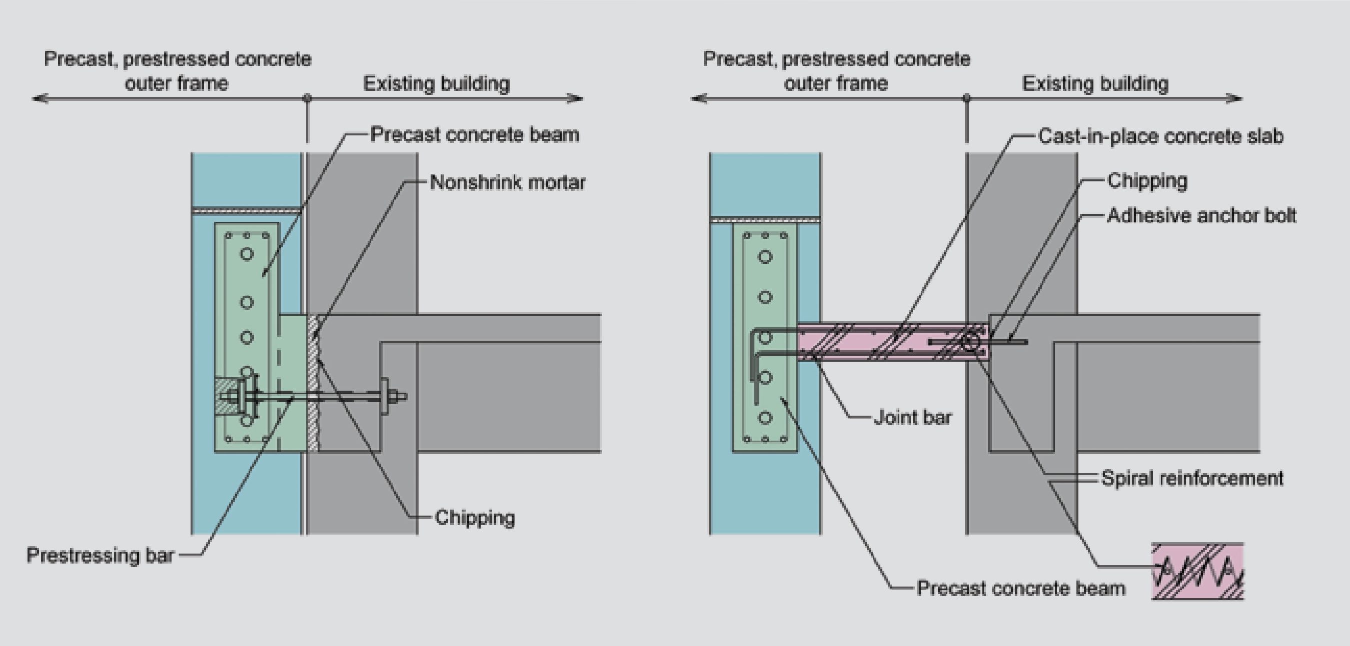


Figure 7. Two methods for shear transfer from the building to the exterior frame. The first is by friction of the prestressing steel bar at the attached connection. The second is by a cast-in-place concrete slab with anchor bolts for the floor slab.

span-to-depth ratio to a maximum of 8 to prevent large deformation after beam yielding.

The concrete strength of the existing building needs to be greater than 18,000 kPa (2610 psi), or, for an attached connection type, greater than 13,500 kPa (1960 psi). For components that are cast-in-place concrete, the concrete strength should be greater than 18,000 kPa in the existing building.

The minimum required concrete strength of reinforced concrete structures in Japan is 18,000 kPa (2610 psi) in the *Standard for Structural Calculation of Reinforced Concrete Structure Based on Allowable Stress Concept*,[9] revised in 1999. However, it had been 13,500 kPa (1960 psi) in the *Standard for Structural Calculation of Reinforced Concrete Structure* of 1982.[10] Only a few reinforced concrete buildings having concrete strengths above 13,500 kPa (1960 psi) were built before 1999 in Japan. This outer-

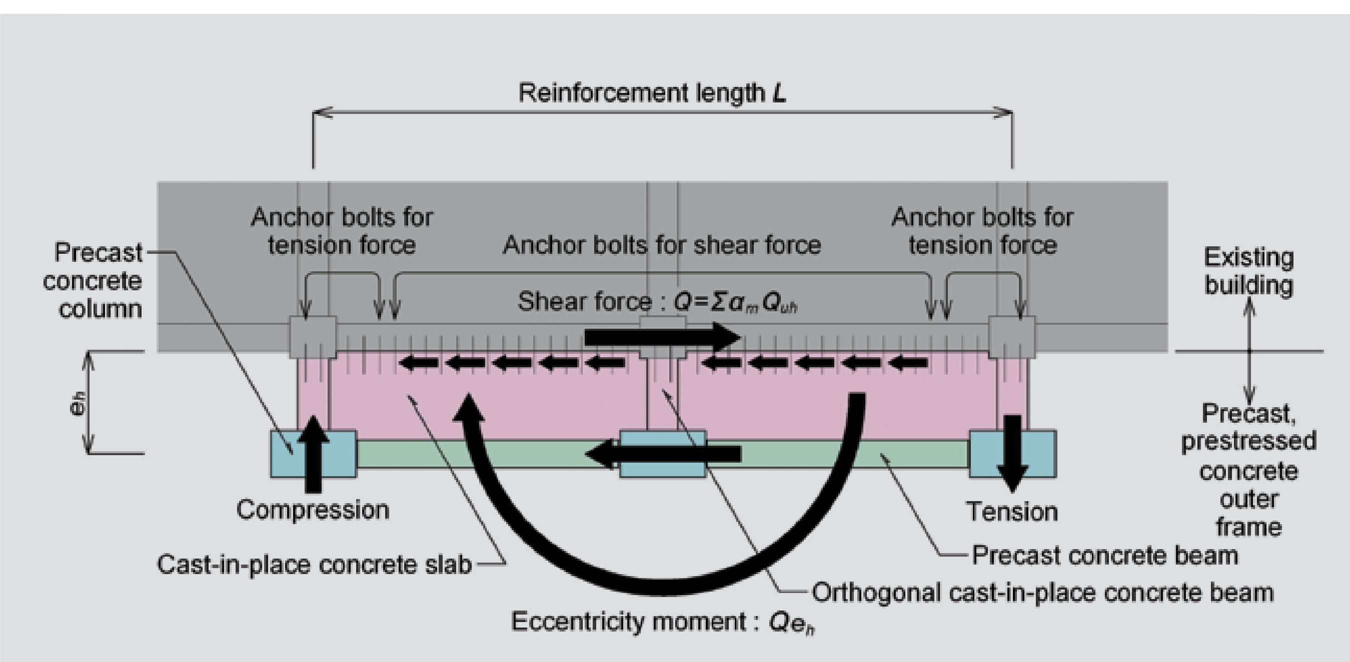


Figure 8. Shear transfer mechanism from building to exterior frame at floor slab. Note: e_h = distance between building and exterior frame; L = length between orthogonal beams of both ends; Q = shear force; Q_{uh} = ultimate lateral load–carrying capacity of precast concrete columns of the next floor below; α_m = effective strength factor in precast concrete columns of the understory.

Figure 9. Fourteen-story reinforced concrete apartment retrofitted by exterior frame method. Constructed in 1970 and retrofitted in 2010.

frame seismic retrofit method was confirmed by tests[11] to be applicable to the existing buildings of concrete strength 13,500 kPa (1960 psi).

Cost

The cost for the precast, prestressed concrete frame is approximately $20,000 to $25,000 per bay, depending on project size and site conditions. This cost includes erection, assembly, and posttensioning.

Miyagi Prefecture High School

The Miyagi Prefecture High School is a four-story reinforced concrete building 156 m (512 ft) in the longitudinal direction and 10 m (32.8 ft) in the transverse direction for a total floor space of 6457 m^2 (69,670 ft^2).[12] The building was completed in 1969 and retrofitted in 2005. **Figure 10** shows its second-floor plan and elevation with the planned exterior-frame retrofit.

Evaluation of the building

The building was evaluated by the second screening method and by the strength-dominant basic index of structure E_0 in Eq. (4). The seismic demand index of structure I_{S0} was set by Eq. (1).

$I_{S0} = E_{SZGU} = 0.7$

Table 2 shows the results of the building evaluation before the retrofit. The following paragraphs explain the calculation procedure.

Step 1 obtained the irregularity index S_D of 0.950 from the evaluation list[4] regarding the plane shape, the section, and the eccentricity ratio of the building.

Step 2 obtained the time index T of 0.992 from the evaluation list[4] of the cracks, deformations, deterioration, etc., of slabs, beams, columns, and walls for each floor of the building.

Step 3 selected the ductility index F of 0.8 (interstory drift angle 1/500 radian) because the columns restrained by spandrel walls in the longitudinal direction were extremely brittle, that is, the ratio of the clear height h_0 to the depth D was less than 2.

The following calculations from step 4 to step 9 refer to the third floor in the longitudinal direction.

Step 4 calculated the story weight w_i of 19,136 kN (4302 kip) and the weight of the upper stories Σw_i of 38,581 kN (8673 kip).

Step 5 calculated the story-shear modification factor $(n + 1)/(n + j)$ of 0.714, where the total number of stories

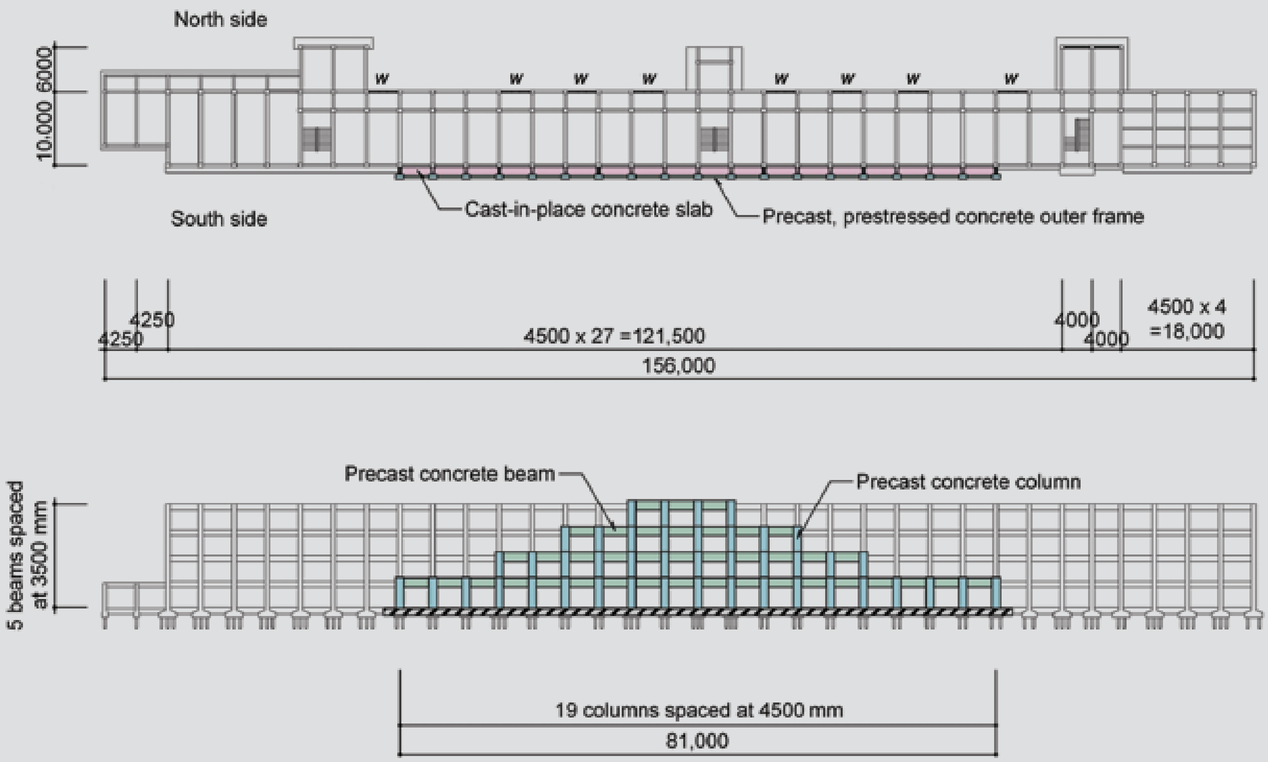


Figure 10. Junior high school building retrofitted by exterior frame method and reinforced concrete shear walls. Plan of the second floor and elevation view. Note: All measurements are in millimeters. *W* = installed reinforced shear wall. 1 mm = 0.0394 in.

Table 2. Results of seismic evaluation before retrofit

Direction and story		Story weight w_i, kN	Weight of upper stories $\sum w_i$, kN	Story-shear modification factor $\frac{n+1}{n+j}$	Accumulated strength index $\sum C$	Ductility index F	Basic seismic index of structure E_0	Irregularity index S_D	Time index T	Seismic index of structure I_S	$C_{TU}S_D \geq 0.3$	Seismic demand index of structure I_{S0}	Evaluation $I_S \geq I_{S0}$
Longitudinal direction	4	19,446	19,446	0.625	1.030	0.8	0.515	0.950	0.992	0.485	0.612	0.7	Unsatisfactory
	3	19,136	38,581	0.714	0.693	0.8	0.396	0.950	0.992	0.373	0.470	0.7	Unsatisfactory
	2	19,842	58,423	0.833	0.535	0.8	0.357	0.950	0.992	0.336	0.423	0.7	Unsatisfactory
	1	21,724	80,147	1.000	0.430	0.8	0.344	0.950	0.992	0.324	0.409	0.7	Unsatisfactory
Transverse direction	4	19,446	19,446	0.625	2.235	1.0	1.397	0.950	0.992	1.316	1.327	0.7	Satisfactory
	3	19,136	38,581	0.714	1.419	1.0	1.013	0.950	0.992	0.955	0.963	0.7	Satisfactory
	2	19,842	58,423	0.833	0.944	1.0	0.786	0.950	0.992	0.741	0.747	0.7	Satisfactory
	1	21,724	80,147	1.000	0.852	1.0	0.852	0.950	0.992	0.803	0.809	0.7	Satisfactory

Note: C_{TU} = ultimate cumulative strength index. 1 kN = 0.225 kip.

of the building n was 4 and the jth story level of the n-story building was 3.

Step 6 calculated the accumulated strength index $\Sigma C = C_1 + \Sigma\alpha_j C_j$, where j = 2,3, and ultimate deformation R_1 = 1/500 radian in this case. The result was 0.693, but the calculation process is abbreviated here.

Step 7 calculated the basic seismic index of structure E_0 by Eq. (4).

$$E_0 = \frac{n+1}{n+j}\left(C_1 + \sum_j \alpha_j C_j\right)F_1 \tag{9}$$

$$= (0.714)(0.693)(0.8) = 0.396$$

Step 8 calculated the seismic index of structure I_S by Eq. (2).

$I_S = E_0 S_D T$ = (0.396)(0.950)(0.992) = 0.373

Step 9 calculated the product $C_{TU}S_D$ of the cumulative strength index C_{TU} and the irregularity index S_D by Eq. (6) and confirmed that $C_{TU}S_D$ was more than or equal to 0.3 (in this case, zone index Z, ground index G, and usage index U were set to 1).

$$C_{TU}S_D = \frac{n+1}{n+j}CS_D = (0.714)(0.693)(0.950) = 0.470$$

Step 10 compared the seismic index of structure I_S and the seismic demand index of structure I_{S0}. If the seismic index of structure I_S is greater than or equal to I_{S0} and $C_{TU}S_D$ is greater than or equal to 0.3, the seismic evaluation of the building is satisfactory (S).

Step 11 was a comprehensive evaluation. The seismic indices of structure I_S in the transverse direction were calculated to be from 0.741 to 1.316. As these values exceed the seismic demand index of structure I_{S0}, a retrofit was not deemed necessary. The indices I_{S0} in the longitudinal direction were calculated as 0.324 to 0.485. These values were less than the seismic demand indices of structure I_{S0}. Therefore retrofitting was required.

Adoption of exterior frame retrofit method

For the retrofit, construction had to be completed without interrupting school sessions, which continued during summer vacation. Considering these requirements, the exterior frame method of the floor slab type was adopted for the south side longitudinal direction. For the north side longitudinal frame of the building, the lateral load–carrying capacity was increased with newly installed cast-in-place concrete shear walls. Brittle failure of the short columns was prevented by providing seismic slits between the column and wall. **Figure 11** shows the cross section of the precast concrete beams and columns.

The exterior frame was installed at a distance of 1.6 m (5.25 ft) from the existing building to avoid interference with the balconies and existing foundation. The reinforced concrete slab was set under the balcony. The ductility index F was improved from 0.8 (interstory drift angle 1/500 radian) to 1.0 (interstory drift angle 1/250 radian) by making seismic slits between the columns and spandrel walls and by installing shear walls between the extremely brittle columns. The lateral load–carrying capacity of the columns was designed to be 1800 kN (405 kip) at the fourth floor, 2000 kN (450 kip) at the third floor, 2000 kN (450 kip) at the second floor, and 1500 kN (337 kip) at the first floor to prevent column yielding failure for the ductility index F of 1.0. **Table 3** listed the results of the seismic evaluation after the retrofit.

Construction

The seismic retrofit consisted of installing the four-story precast, prestressed concrete frame for 39 bays on the south side in the longitudinal direction, 23 reinforced concrete shear walls, and 96 seismic slits between the columns and spandrel walls, and closing 9 openings on the north side.

Construction lasted eight months, from the end of March until early December. Before construction, the dimensions of the column bays and the floor heights of the existing building were measured and checked against the drawing. The precast concrete elements were assembled for every floor. The precast concrete columns and beams were connected with posttensioning tendons, and splice sleeves between the foundation and columns were filled with high-strength nonshrink grout. The column-to-foundation joints were grouted after posttensioning. The floor height was too great, that is, 81 m (266 ft) to the first floor and 49.5 m (162 ft) to the second floor, and so, to cope with the deformation of the frame by prestressing, a countermeasure was performed by using large sleeves and adjusting them to the position of the column beforehand. The results were within the tolerance of 5 mm (0.2 in.).

Parallel unit frame method

Description

The parallel unit frame method increases a building's lateral load–carrying capacity by means of an exterior precast, prestressed concrete frame. The capacity of the precast concrete rigid frame and the diagonal tension ties within each bay of the frame correspond to the lateral force. **Figure 12** shows the detail of the parallel unit frame. Splice sleeves comprise the column splices and the beam-column joints diagonally connected with tension ties. The ends of

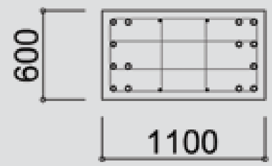
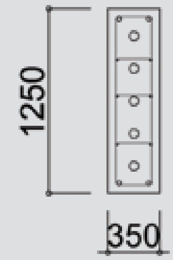
	Precast concrete column	Precast concrete beam
Cross section		
Width × depth	600 x 1100	350 x 1250
Main reinforcement	14 no.10 + 4 no.6	Top and bottom reinforcement: 2 no.7
Shear reinforcement	⊞ no.4 at 100	□ no.4 at 100
Prestressing tendon		5c-7-SWPR7B-12.7 mm
Note		Web reinforcement: 6 no.3

Figure 11. Cross section of precast concrete elements of exterior frame. The reinforcing bars of the column are the same in both directions. That is, the seven top and bottom reinforcing bars are valid for the moment of longitudinal and transversal direction, respectively. Note: All measurements are in millimeters. no. 3 = 10M; no. 4 = 13M; no. 6 = 19M; no. 7 = 22M; no. 10 = 32M; 1 mm = 0.0394 in.

the tension ties are embedded in the beam-column joint together with a ring-shaped steel plate and reinforcing bars beforehand, and the tension ties themselves are connected with couplers during erection and are posttensioned.

The construction procedure is essentially the same as for the precast, prestressed concrete outer-frame seismic retrofit method except for the diagonal tension ties. **Figure 13** shows the three cases of integration of the parallel unit frame to the existing building. The method is classified by whether a building has a balcony and whether a new or expanded foundation is necessary.

Scope

This method is applicable to an existing reinforced

Table 3. Results of seismic evaluation after retrofit

Direction and story		Story weight w_i, kN	Weight of upper stories $\sum w_i$, kN	Story-shear modification factor $\frac{n+1}{n+j}$	Accumulated strength index $\sum C$	Ductility index F	Basic seismic index of structure E_0	Irregularity index S_D	Time index T	Seismic index of structure I_S	$C_{TU}S_D \geq 0.3$	Seismic demand index of structure I_{S0}	Evaluation $I_S \geq I_{S0}$
Longitudinal direction	4	20,489	20,489	0.625	1.328	1.0	0.830	0.950	0.992	0.782	0.789	0.7	Satisfactory
	3	20,882	41,310	0.714	1.064	1.0	0.760	0.950	0.992	0.716	0.722	0.7	Satisfactory
	2	22,466	63,776	0.833	0.924	1.0	0.770	0.950	0.992	0.726	0.731	0.7	Satisfactory
	1	25,611	89,388	1.000	0.750	1.0	0.750	0.950	0.992	0.707	0.713	0.7	Satisfactory

Note: C_{TU} = ultimate cumulative strength index. 1 kN = 0.225 kip.

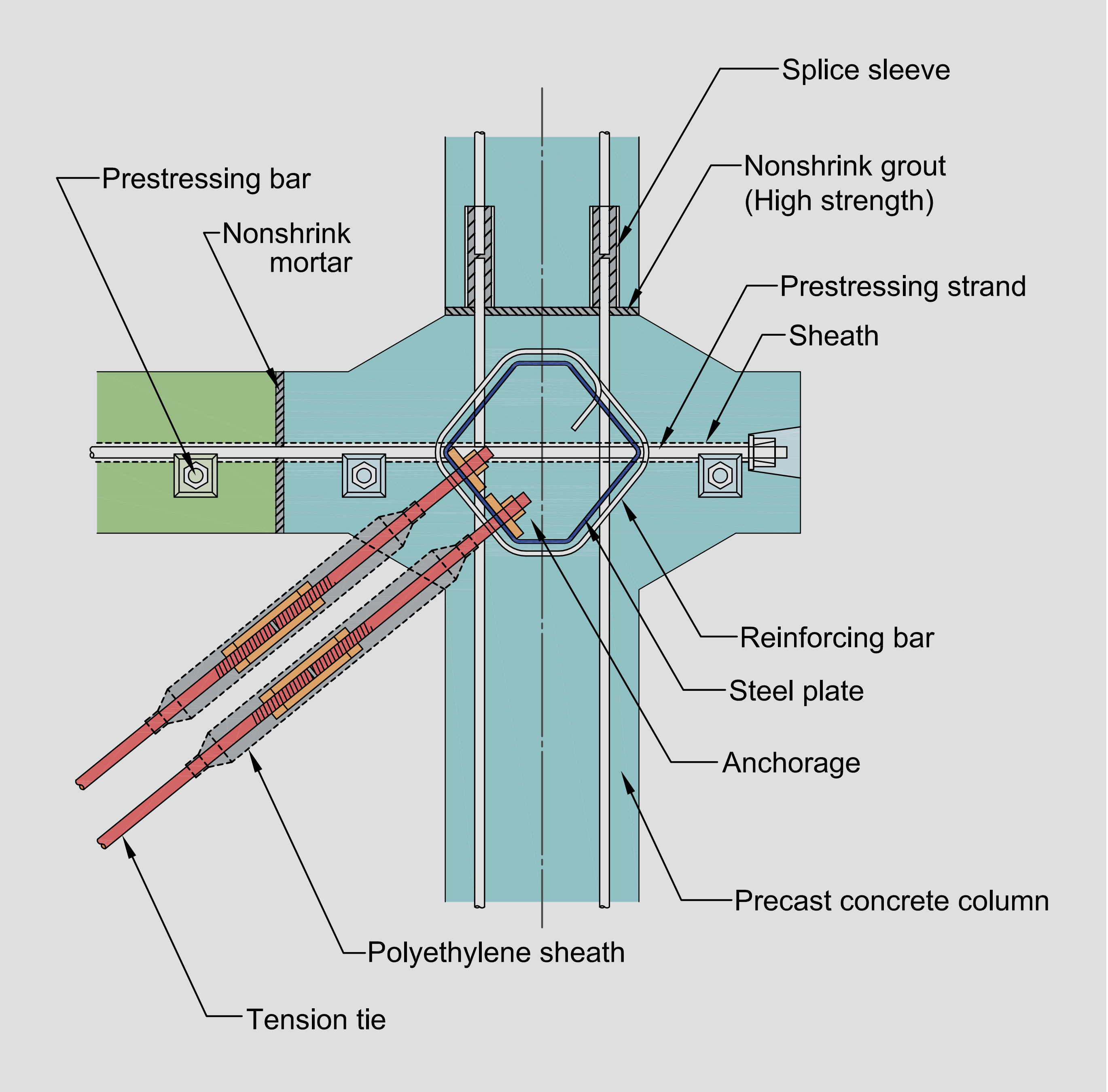


Figure 12. Connection details of parallel unit frame.

concrete building that has concrete strength of more than 13,500 kPa (1960 psi). Retrofitting of buildings with this weak concrete strength had been verified by tests.[13]

The maximum number of floors of the retrofitted buildings is 12. From 2005 to 2012, 220 projects, including school buildings, apartments, city halls, offices, and hospitals, were retrofitted using this method.

Cost

The construction cost of the parallel unit frame method is approximately $20,000 to $25,000 per bay, comparable to that for the outer-frame method.

Municipal Junior High School in Miyagi prefecture

This school is a four-story reinforced concrete building.[7] It was completed in 1974 and retrofitted in 2010. **Figure 14** shows its plan and elevation.

Evaluation of the building

The evaluation of the building used the second screening method by the strength-dominant basic seismic index of

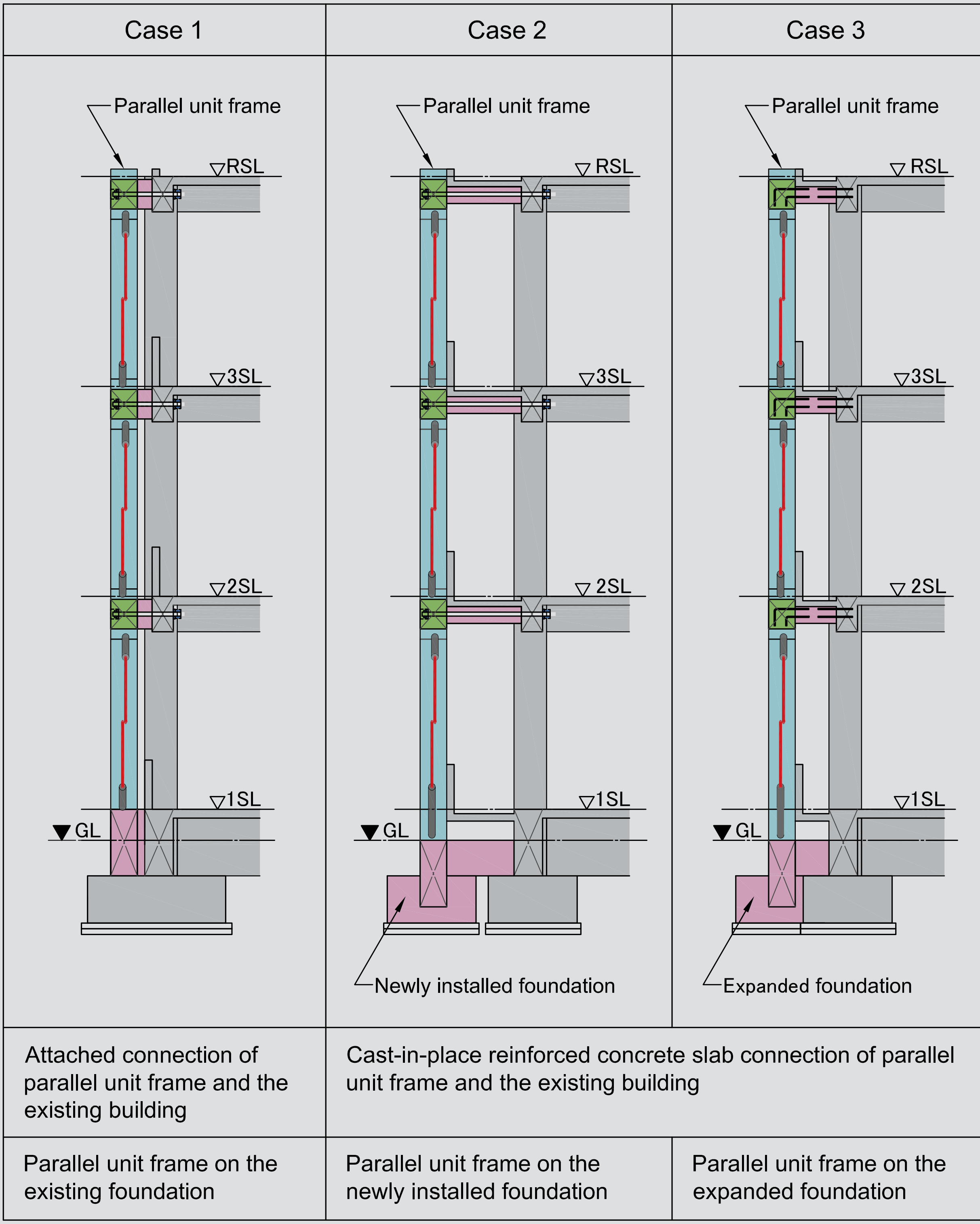


Figure 13. Three ways to connect parallel unit frame and building. Case 1 can be used for a building without overhang. Case 2 can be used for a building with large overhang. Case 3 can be used for a building with a small overhang.

structure E_0 (Eq. [4]). The seismic demand index of structure I_{S0} was determined by Eq. (1).

$$I_{S0} = E_S ZGU = 0.7$$

Table 4 shows the results of the evaluation of this building before the retrofit. The seismic indices I_S of the structural elements for all stories were calculated as 0.383 to 0.673. These values were less than the seismic demand index

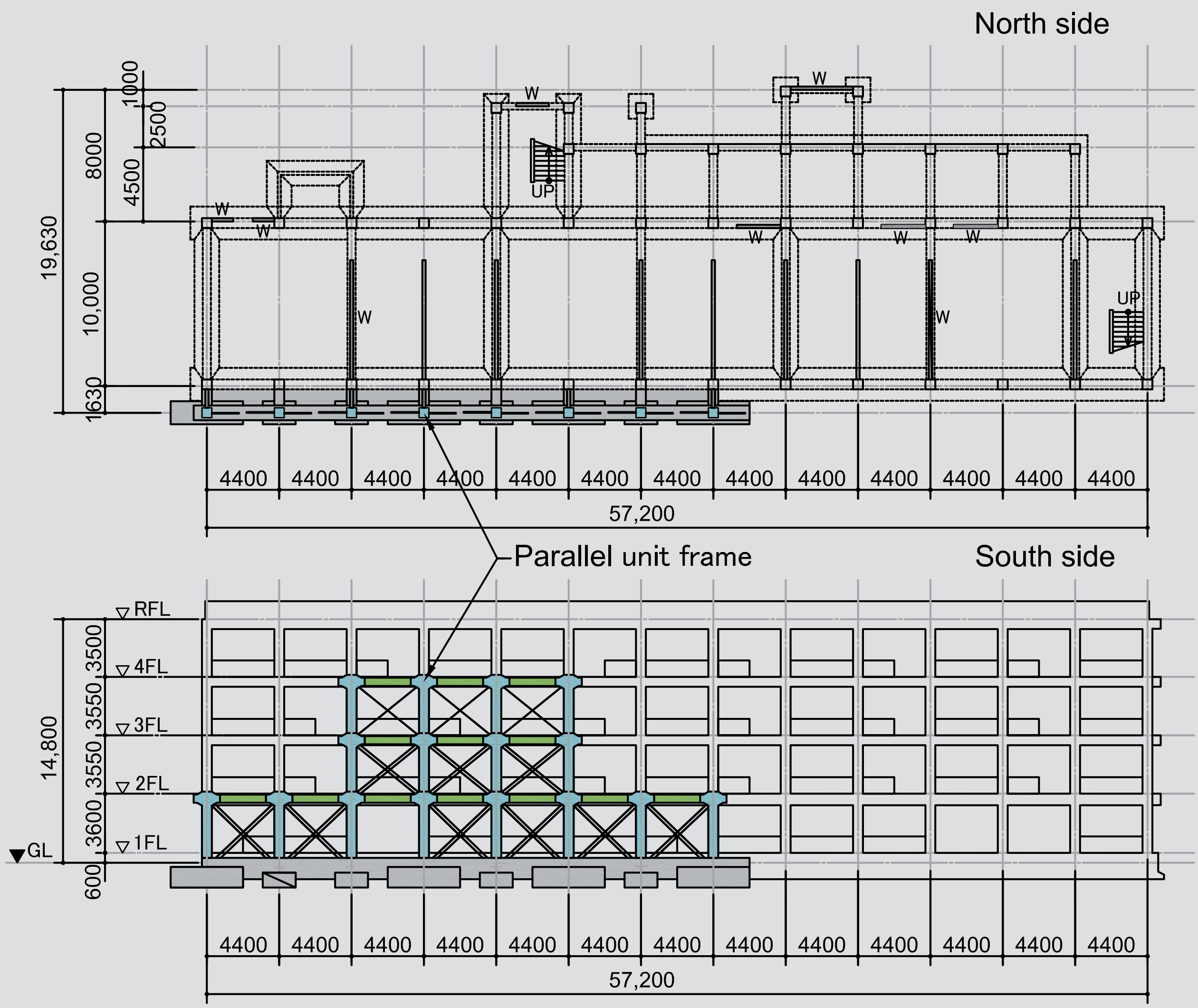


Figure 14. Junior high school building retrofitted by parallel unit frame method. Plan of the first floor and elevation. Note: All measurements are in millimeters. *W* = installed reinforced shear wall. 1 mm = 0.0394 in.

of structure I_{S0} of 0.7. Thus, the results of the evaluation required a retrofit of the south and north sides in the longitudinal direction.

Adoption of the parallel unit method

The parallel unit method was well suited for the retrofit of this building because of the following reasons:

- It could be completed during summer vacation.
- Not much interior work was needed.
- Ventilation and lighting after the retrofit were almost the same as before because of the fine diagonal tension ties.
- The room layout remained the same.

The building was retrofitted using the ductility index F equal to 1 (interstory drift angle is 1/250 radian). The south side frame of the longitudinal direction was retrofitted with a parallel unit frame attached to the edge of the balcony, and the north side of the longitudinal direction, with its extremely brittle columns, was strengthened by cast-in-place reinforced concrete shear walls in the bays. After the ductility index F was set, the deformation of the frame and the elongation and/or the stress of a tension tie were calculated. The tension tie for each floor was selected from among prestressing bars 32 mm (1.2 in.), 36 mm (1.4 in.), and 40 mm (1.6 in.) so that the total of the above stress, posttensioning stress (less than one half of the yield strength), and safety margin was within the elastic stress. The eccentricity of the retrofitted building was calculated to determine whether torsion analysis was necessary. However, the effect of torsion was ultimately ignored because the eccentricity was less than 0.15.

Table 4. Results of seismic evaluation before retrofit

Direction and story		Story weight w_i, kN	Weight of upper stories Σw_i, kN	Story-shear modification factor $\frac{n+1}{n+j}$	Accumulated strength index ΣC	Ductility index F	Basic seismic index of structure E_0	Irregularity index S_D	Time index T	Seismic index of structure I_S	$C_{TU}S_D \geq 0.3$	Seismic demand index of structure I_{S0}	Evaluation $I_S \geq I_{S0}$
Longitudinal direction	4	8884	8994	0.625	1.206	1.0	0.754	0.903	0.989	0.673	0.681	0.7	Unsatisfactory
	3	9833	18,727	0.714	0.661	1.0	0.472	0.903	0.989	0.422	0.426	0.7	Unsatisfactory
	2	9981	28,708	0.833	0.515	1.0	0.429	0.903	0.989	0.383	0.387	0.7	Unsatisfactory
	1	11,374	40,082	1.000	0.559	1.0	0.559	0.903	0.989	0.499	0.504	0.7	Unsatisfactory

Note: C_{TU} = ultimate cumulative strength index. 1 kN = 0.225 kip.

Table 5. Results of seismic evaluation after retrofit

Direction and story		Story weight w_i, kN	Weight of upper stories Σw_i, kN	Story-shear modification factor $\frac{n+1}{n+j}$	Accumulated strength index ΣC	Ductility index F	Basic seismic index of structure E_0	Irregularity index S_D	Time index T	Seismic index of structure I_S	$C_{TU}S_D \geq 0.3$	Seismic demand index of structure I_{S0}	Evaluation $I_S \geq I_{S0}$
Longitudinal direction	4	9924	9928	0.625	1.724	1.0	1.077	0.903	0.989	0.962	0.973	0.7	Satisfactory
	3	10,213	20,141	0.714	0.162	1.0	0.829	0.903	0.989	0.741	0.749	0.7	Satisfactory
	2	10,462	30,603	0.833	0.951	1.0	0.793	0.903	0.989	0.708	0.717	0.7	Satisfactory
	1	11,986	45,589	1.000	0.950	1.0	0.950	0.903	0.989	0.847	0.858	0.7	Satisfactory

Note: C_{TU} = ultimate cumulative strength index. 1 kN = 0.225 kip.

The arrangement of the parallel unit frame was three bays on the second and third floors and seven bays on the first floor. For the entrance to the first floor, a unit frame without a tension tie was used. The distance between the parallel unit frame columns and the existing building columns was 1.63 m (5.35 ft) to avoid adding to the forces on the existing foundation. The cast-in-place concrete slab for shear transfer was installed below the balcony and connected with prestressing bars. **Table 5** shows the results of the evaluation after the retrofit.

Construction

The parallel unit frame had 13 bays for 3 floors. The work schedule of the retrofit was more than three months, from June to September. However, the installation of the parallel unit frame took only two months. The assembly of the parallel unit frame was completed on each floor. The construction process was as follows:

1. Erection of the precast concrete columns.
2. Setting of the precast concrete beams.
3. Filling the joints with nonshrink mortar for horizontal members and high-strength nonshrink grout for vertical members.
4. Posttensioning the beam-column joint.

5. Arranging and posttensioning the diagonal tension ties.

6. Installation of the cast-in-place reinforced concrete slab.

7. Integration between the parallel unit frame and the existing structure.

Figures 15 shows the construction procedure of the parallel unit frame.

Conclusion

The Tohoku earthquake of March 11, 2011, which was 9.0 on the moment magnitude scale, heavily shook the Tohoku and Kanto areas. In these areas, 59 reinforced concrete buildings were retrofitted by two companies using external precast, prestressed concrete frames. All of the retrofitted buildings were investigated after the earthquake. However, no damage was observed and the buildings were found to be structurally sound.

Evaluating torsion due to eccentricity

Eccentricity is defined as the ratio of the distance between the center of gravity and the center of stiffness to the torsional resistance of the building. The seismic evaluation standard[4] provides that a building with eccentricity less than or equal to 0.15 does not require verification of torsional resistance. When the eccentricity exceeds 0.15, the seismic index of structure I_S (Eq. [2]) decreases, and the greater the difference between I_{S0} and I_S, the greater the required lateral load–carrying capacity. Therefore, the seismic evaluation standard[4] allows a building with eccentricity less than 0.15 to be analyzed neglecting the torsion and drift angle.

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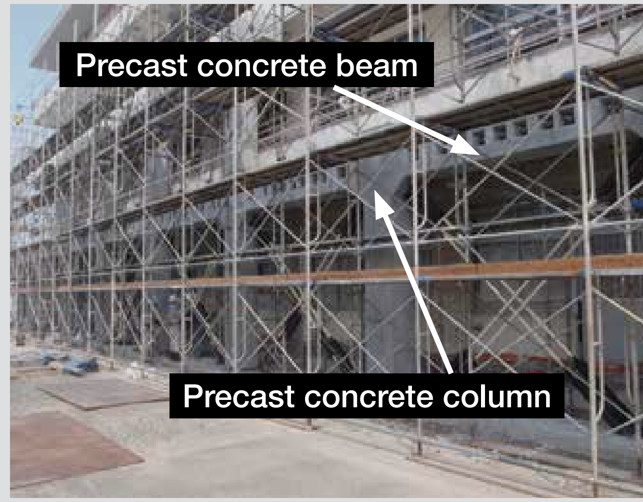
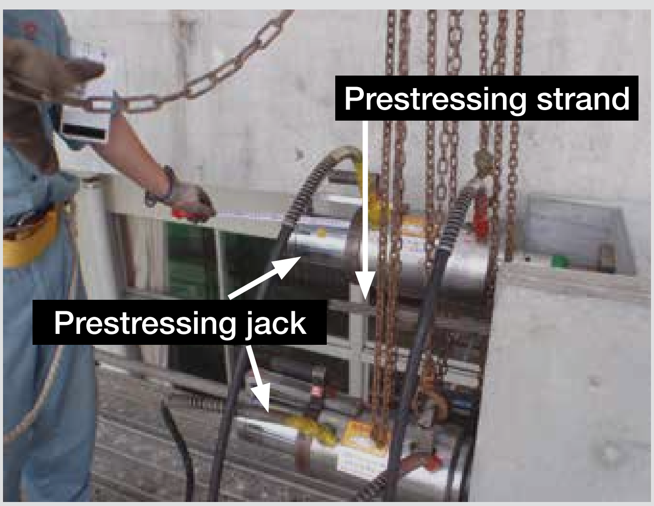
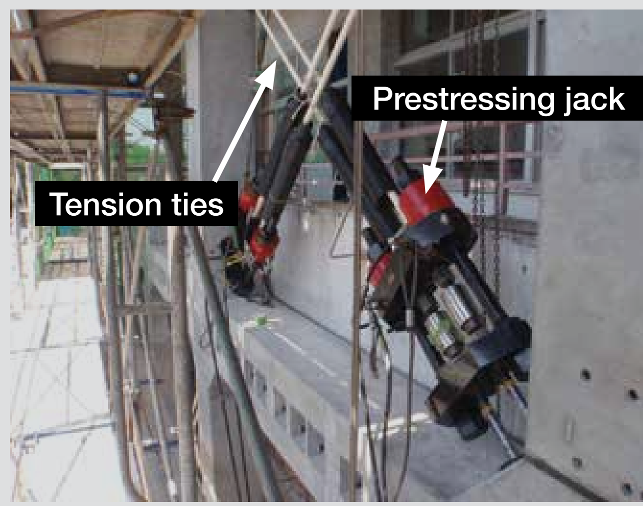
Excavation for newly installed foundation



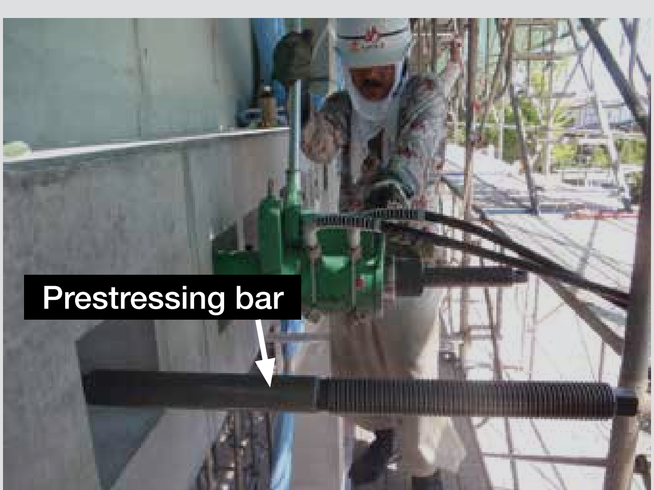
Assembly of precast concrete columns and beams



Posttensioning beam–column joints



Tensioning diagonal tension ties: 2 jacks were used for crossed tension ties to avoid biased stress to the precast concrete frame



Connecting the parallel unit frame and the building with prestressing steel tendon

View after retrofit

Figure 15. Retrofit work procedure of the parallel unit frame method of the school building.

Notation

C = strength index

C_1 = strength index of first group (with small F)

C_2 = strength index of second group (with medium F)

C_3 = strength index of third group (with large F)

C_i = strength index of ith group

C_j = strength index of jth group

C_{TU} = ultimate cumulative strength index

D = depth

e_h = eccentric distance between building and exterior frame

E_0 = basic seismic index of structure

E_1 = product of strength index C_1 and ductility index F_1 of first group

E_2 = product of strength index C_2 and ductility index F_2 of second group

E_3 = product of strength index C_3 and ductility index F_3 of third group

E_S = basic seismic demand index of structure

F = ductility index

F_1 = ductility index of first group

F_2 = ductility index of second group

F_3 = ductility index of third group

F_i = ductility index of ith group

G = ground index

h_0 = clear height

I_S = seismic index of structure

I_{S0} = seismic demand index of structure

j = jth story level of an n-story building

L = length between orthogonal beams of both ends

n = total number of stories of a building

$\frac{n+1}{n+j}$ = story-shear modification factor

Q = shear force

Q_u = ultimate lateral load–carrying capacity of vertical members in the story concerned

Q_{uh} = ultimate lateral load–carrying capacity of precast concrete columns of next floor below

R_1 = interstory drift angle at ultimate deformation corresponding to first group

S_D = irregularity index (0.4 to 1.0)

T = time index

U = usage index

w_i = calculated story weight

W = installed reinforced shear wall

Z = zone index

α_2 = effective strength factors in second group at ultimate deformation corresponding to first group (ductility index F_1)

α_3 = effective strength factors in third group at ultimate deformation corresponding to first group (ductility index F_1)

α_j = effective strength factor in jth group elements at ultimate deformation R_1 corresponding to first group elements (ductility index F_1)

α_m = effective strength factor of precast concrete columns of next floor below

μ = friction coefficient

ΣC = accumulated strength index

Σw_i = weight of upper stories

ΣW = weight of building including live load for seismic calculation supported by story concerned

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Abstract

Two external types of precast, prestressed concrete seismic retrofit methods were applied in two school buildings in Miyagi prefecture, which was strongly affected by the Tohoku earthquake of March 11, 2011. Following the earthquake, inspection showed no damage other than small cracks in the retrofitted buildings. The paper describes how the calculations were performed and the basic construction procedures.

Keywords

Earthquake, retrofit, seismic, standard.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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