Structural performance evaluation of historical landmark (Yokufu-en) built in the Taisho period

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Abstract

The main building of Yokufu-en has been designated as a "Selected Historical Building of Tokyo Metropolitan Government," built after the 1923 Great Kanto Earthquake in the Taisho period. This building was designed by Yoshikazu Uchida, a famous university professor and his assistant. As a result of the study, the building has enough strength of structure, and the concrete shows sufficient compressive strength. However, as the 3rd story has a high eccentric factor it is necessary to be reinforced for earthquake resistance. The reinforcement plan was designed for obtaining a prescribed seismic capacity performance in Japan.
Keywords: seismic evaluation, historical-landmark, structural performance evaluation, Yoshikazu Uchida.

1 Introduction

The main building of Yokufu-en (the main building of current Yokufukai, a social welfare corporation) reported in this paper was a hospital which was built to protect people who were affected by the 1923 Great Kanto Earthquake. It was designed by Yoshikazu Uchida and his assistant Tatuto Toki in 1925 and completed in 1926 [1]. It is a valuable building designated as a "Selected Historical Building of Tokyo Metropolitan Government" in 2001. The building is currently used as offices. Structural and seismic evaluations were conducted to check its seismic capacity. This paper reports the details of the technical data used when the building was designed including the Urban Building Law, and the

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structural and seismic evaluation results of the building for the structural performance of this historical landmark designed and constructed in the Taisho period.

2 Outline of the building

Location	: Tokyo
Application	: Offices (original design: hospital rooms, offices)
Designed	: 1925 (Completed: 1926)
Total floor area	: 1,708m^2
No. of story	: 3 above ground, 1 below ground, 3-story PH
Structural classification	: Reinforced concrete structure
Structural type	: Rigid-frame structure with seismic resisting walls
Foundation type	: Spread foundation
Design	: Yoshikazu Uchida, Tatuto Toki
Construction	: Obayashi Corporation, Nakamura Koumusyo

Elevators were installed in this building in 1995, and part of the exterior wall was repaired in 2005. The building has not been affected by severe natural disasters since its completion. Figure 1 shows a representative elevation and Figure 2 shows a plan of the representative floor. Its tiled exterior walls have been repaired, and part of the surface of the interior wall has been repaired and repainted. Though exposed reinforcing steel was found under part of the eave during appearance and crack check, no obvious cracks, rust on reinforcing steel, or rust fluid were found.

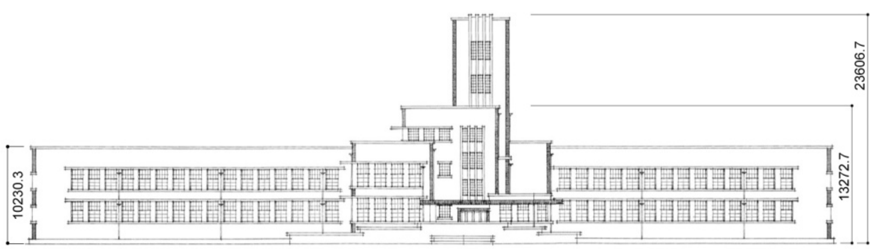


Figure 1: Elevation of the north side.

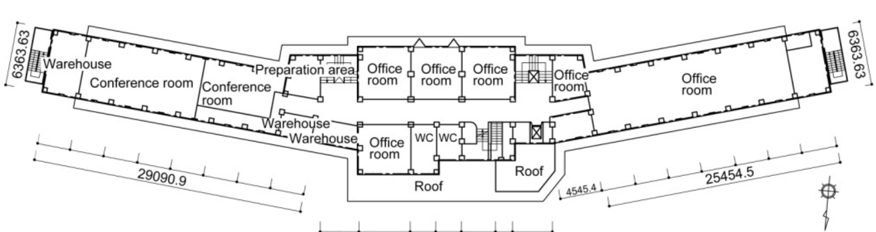


Figure 2: Plan of the representative floor.

3 Structural performance evaluation of concrete and reinforcing steels

3.1 Physical test of concrete

The physical test of concrete was conducted using core samples. Each of the samples (12 in total) was collected from the inside of each room. Table 1 shows where the samples were collected and the results of the compressive strength test. The concrete strength test was conducted in accordance with JIS A 1107 (Japanese Industrial Standard). The corrected compressive strength of the samples is obtained by obtaining the correction factor from the ratio between the height and the average diameter, dividing the largest compressive force by the area, and multiplying the value by the correction factor. The average value of all data is 29.9 N/mm^2, and that for above ground is 30.7 N/mm^2. The standard deviations are shown in Table 1. Though the values differ slightly, the strength would be sufficient for Japanese standard for seismic evaluation.

Table 1: Results of concrete compressive strength test.

Code	Floor	Portion	Room name	Corrected compressive strength (N/mm^2)	Standard deviation (N/mm^2)
B1-1	1st basement level	Wall	Dry area	30.1	4.4
B1-2	1st basement level	Wall	Elevator machine room	29.5	
B1-3	1st basement level	Wall	Staircase	22.2	
1-1	1st floor	Wall	Warehouse	24.5	4.6
1-2	1st floor	Wall	Staircase	32.1	
1-3	1st floor	Wall	Conference room	32.8	
2-1	2nd floor	Wall	Warehouse	41.9	8.1
2-2	2nd floor	Wall	Warehouse	32.4	
2-3	2nd floor	Wall	Corridor	25.7	
3-1	3rd floor	Wall	Stairs	31.2	1.9
3-2	3rd floor	Wall	Book room	28.0	
3-3	3rd floor	Wall	Book room	27.9	

3.2 Reinforcing steel strength test

One reinforcing steel sample was collected from the interior wall of the warehouse on the 3rd floor. Table 2 shows the results of the tensile strength test. It also shows the comparison with the provisions of the Urban Building Law revised in 1924 and the current JIS (Japanese Industrial Standard). This building satisfies the standard for the reinforcing steel under the Urban Building Law when it was designed. While the tensile strength is slightly lower than the standard value specified under the current JIS, the yield point and the elongation are SR235 (JIS) level.

Table 2: Results of reinforcing steel tensile strength test.

Standard	Diameter	Yield point (N/mm^2)	Tensile strength (N/mm^2)	Elongation (%)
Measured value	10φ (9.64 mm)	318	367	37
Urban Building Law	10φ	No specification.	353	20 or more
JIS standard SR235	10φ	235 or more	380 to 520	20 or more

4 Consideration of the Urban Building Law when the building was designed and the concrete mix

A bill of architecture was created from the end of the Meiji period to the beginning of the Taisho period. It was finally promulgated as the Urban Building Law in April 1919, and took effect in December 1920 [2]. Yoshikazu Uchida, the designer of the main building of Yokufukai, made a great contribution to the draft of the law. Following the 1923 Great Kanto Earthquake, the provisions of the law relating to structures were revised and the seismic coefficient method was introduced in 1924. It is estimated that the structural design for the main building of Yokufukai designed in 1925 incorporated the contents. The outline of the structural provisions under the Urban Building Law when the building was designed is described below. The Urban Building Law is referred to as “Urban Law” and the Building Standards Law is referred to as “Standards Law” for short. Both SI units and original units are used in comparison. Those for the reinforcing steels are shown in Section 3.2.

4.1 Design live load

Table 3 shows comparison of design live load. When the long-term stress is examined, the live load according to the present usage is lower than the design live load defined when the building was designed.

Table 3: Comparison of design live load.

Law	Applications	S For slabs	B For beams	R For rigid-frames		E For earthquakes
				Girders	Column	
Urban Law	Offices, hospital rooms	370 3628	333 3266	296 2903	259 2540	— (no defined)
Standards Law	Offices	300 2900	300 2900	180 1800		80 800

Upper: (kgf/m^2); Lower: (N/m^2).

4.2 Concrete mix

It is estimated that concrete which “passed the provisions for the Portland cement test method set forth in the announcement No. 485 of the Agriculture and

Commerce Ministry" was used from the description of the enforcement regulations for the Urban Law, and that the mix was "cement 1: sand 2: gravel or crushed stone: 4" by volume ratio. However, the water-cement ratio was not set forth in the Enforcement regulations for the Urban Law, and therefore, the Architectural Institute of Japan issued the "Concrete and reinforcing steel concrete standard specification" in 1929 and the "Standard for Structural Calculation of Reinforced Concrete Structures" (RC standard) [3] in 1933 in the form of rules and guidelines. However, the relationship between concrete strength and water-cement ratio has already been known academically, Journal of Architecture and Building Science reported that the water volume greatly affects concrete properties [4].

The concrete for the building above ground was placed between March and May. The average temperature in Tokyo in those days was 6 deg C to 20 deg C and the average humidity was 60 to 80%, which were relatively favourable for concrete placement and curing. In addition, the involvement of top-class Japanese engineers at the time also contributed to the strength to increase. The long-term strength of ordinary concrete using normal Portland cement tends to increase in proportion to the 28-day strength under climate conditions close to those mentioned above [5, 6], which could have also improved the strength. Table 4 shows the concrete mix and the allowable stress set forth under the Urban Law in those days.

Table 4: Stress intensity of concrete set forth under the Urban Building Law.

Type	Resistant pressure	Resistant tension	Resistant shear force
Concrete (Material ratio = 1:2:4)	45 4.4	4.5 0.44	4.5 0.44
Concrete (Material ratio = 1:3:6)	30 2.9	3.0 0.29	3.0 0.29

The material ratio is the ratio among cement: sand: gravel (crushed stone). Upper: (kgf/cm^2); Lower: (N/mm^2).

5 Results of neutralization test

5.1 Results of neutralization test

The locations of the neutralization test were the same as those for the concrete compressive strength test. The test results are shown in Table 5. The warehouse on the 1st floor (1-1) had the largest depth of 61 mm, but it is approximately 30 to 40 mm as a whole. The entire average is 32.5 mm, and the average value above ground is 34.5 mm. Based on this, it is estimated that the neutralization nearly progresses to the position of the reinforcing steel.

Table 5: Results of neutralization test.

Code	Average neutralization depth (mm)	Standard deviation (mm)	Code	Average neutralization depth(mm)	Standard deviation (mm)
B1-1	34.1	5.9	2-1	29.6	6.2
B1-2	15.4	6.8	2-2	29.3	3.0
B1-3	30.3	10.6	2-3	28.4	2.2
1-1	54.6	4.9	3-1	32.1	2.3
1-2	23.6	2.6	3-2	31.7	4.4
1-3	39.1	0.9	3-3	41.7	4.9

5.2 Neutralization curve created from the estimated water-cement ratio

The water-cement ratio estimated from the references in 4.2 and the background of the designer is approximately 0.6. Figure 3 shows the neutralization curve calculated from the Kishitani formula [7] based on the value and the measured values. The neutralization ratio was set at 1.0. Though there are large variations, the rate of neutralization calculated from the estimated water-cement ratio and the average value show a similar response. As reference data, data of two school buildings which were completed in the same period shown in the reference [8] were plotted. (K: Completed in 1925; W: Completed in 1932)

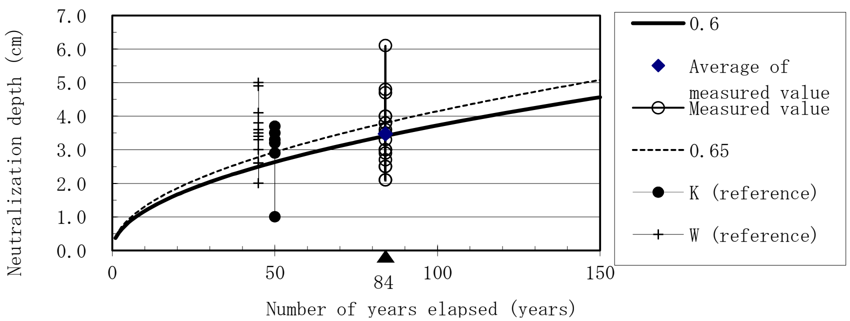
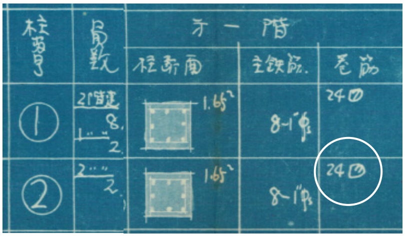


Figure 3: Comparison of measured and calculated neutralization depth.

6 Results of seismic evaluation

6.1 Reinforcing steels

The intervals of bar arrangement were surveyed using the electromagnetic radar system. The bar arrangement was surveyed using 22 columns on the 1st and the 2nd floors. As a result, the authors confirmed that the number of main reinforcement of columns corresponded to the drawing created during the design. The intervals of measurement of the shear reinforcement were 191.2 mm on average, and the largest displacement was within 20 mm. The intervals used for the calculation were set at 200 mm. Figure 4 to Figure 6 show the drawings used during design. The cross-section chart (Figure 4) only shows the number of windings of shear reinforcement. The detailed structural drawing (Figure 5) shows the number of windings and the intervals. In the drawing, the top and the bottom of the columns are indicated with the number of windings as @0.40 (Shaku) (approx. 121 mm), and the center of the column as @0.60 (Shaku) (approx. 182 mm). However, there was no clear agreement between the cross-section chart (Figure 4) and the number in the detailed structural drawing (Figure 5). For this reason, results of the radar survey were used for column shear reinforcement.



* The drawing of $\boxed{/}$ in the spiral reinforcement (circled section) indicates the processing of reinforcing steel.

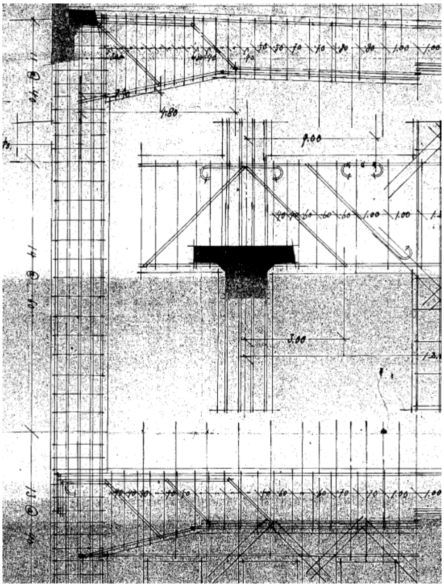


Figure 4: Cross-section chart. Figure 5: Detailed structural drawing.

For hook shape at the edge of the column shear reinforcement, the figure shows a 135-deg hook (Figure 4). Estimating from the background of architectural engineers, the authors determined that the building was constructed in accordance with the drawing and that the 135-deg hook was used. Several years later, Article 36 of the “Reinforcing steel concrete standard specification” [3] in 1933 specified that the folding of the terminal of joining reinforcement (shear reinforcement) should be 135 deg or more.

Since wall bar arrangement is not found in any drawing, it was determined to be 300 mm pitches with a staggered bar arrangement from the results of actual measurement. The diameter of the wall reinforcement was determined to be 9φ.

For fixing of beam main reinforcement, the detailed beam main reinforcement structure drawing of Figure 6 shows that the top reinforcement exceeds the center of the column and that the bottom reinforcement is folded about 30 cm near the center of the column. Since the smallest column width is approx. 50 cm

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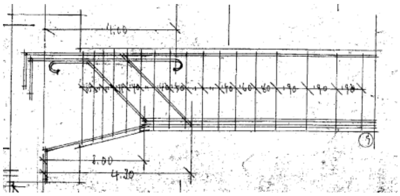

Figure 6: Detailed structural drawing of beam main reinforcement.

and the main reinforcement diameter is 3/4 inch (approx. 19 mm), the anchorage length is 29d or more from the column surface.

Table 6 shows comparison between the structural provisions set forth in the then Urban Building Law (after revised i n 1924), the description in design drawings (structural drawings), and the provisions set forth under the present Building Standards Law. The drawings satisfy the structural provisions after revised in 1924.

Table 6: Comparison between the structural provisions and the drawings.

	Drawing (comparison with the Urban Law)	Urban Law (after revised in 1924)	Standards Law (current rules)
Fixing of reinforcing steel	Beam top: Fixed exceeding the column center Beam bottom: Fixed at the column center.	Fastening of the terminal to other structures or fixing by folding the terminal.	Designation of the terminal position to start folding for fixing Terminal of round bar, with hook
Joint length of main reinforcement	Beams satisfy the standards from the structural drawing (Diameter of beam main reinforcement: 19φ) Columns were checked from the photo taken during construction.	25D or more of the main reinforcement diameter	Tensile reinforcement of the beam for fixing the column: 40D or more Location with small tension: 25D or more
Beam shear reinforcement	The largest interval is 1 Shaku (303 mm) with the beam center depth of 500 mm, which satisfies the Urban Law.	Placement of binding reinforcement Intervals of binding reinforcement: 2/3D or below	Beam: Stirrup reinforcement 3/4D or below
Beam reinforcing steel	Double reinforcement	Placement of double reinforcement	Placement of double reinforcement
Column structure	8 to 16 columns	4 or more main reinforcement	4 or more main reinforcement
	The reinforcing steel area of all the columns covers 1/80 or more of the column area. (Column main reinforcement diameter: 25φ)	1/80 (1.25%) or more of the concrete cross-section area	0.8% or more of the concrete cross-section
	Indication of the number of windings (Average of measured value: 191.2 mm)	Intervals of binding reinforcement are 1 Shaku (303 mm) or below and less than 15 times the main reinforcement diameter	10 cm for column top/base (15 cm for others) and less than 15 times the thin main reinforcement diameter Hoop reinforcement ratio: 0.2% or more
	All the columns satisfy the standards.	Column small diameter: 1/15 or more of the major spans	Column small diameter: 1/15 or more of the major spans

6.2 Results of seismic evaluation

Based on the results of the field survey and the material test, the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (The Japan Building Disaster Prevention Association) were used to create seismic evaluation and reinforcement plan. From the average value above ground obtained from the results of the concrete compressive strength test (see 3.1), the concrete strength for calculation was set to Fc28. For reinforcing steels, though the tensile strength is slightly lower than the JIS standard, the yield point and the elongation are satisfied. Therefore, they were set to SR235.

The entire building was evaluated as a six-layer building because the proportion of the 4th or higher floors, which are the penthouse, is large when compared with the 3rd floor in terms of both the area and the weight. The building weight by unit area is larger than that of ordinary buildings (Table 7), which is caused by relatively small spans, a high proportion of exterior walls due to the wing-shaped form, the exterior walls finished with brick-like scratch tiles, and other factors.

Table 7: Area and weight of each floor.

Floor	Floor area of each floor Af (m^2)	Total floor area Σ Af (m^2)	Total weight Σ W (kN)	Unit weight w_2 (kN/m^2)	Ai
6 (PH3)	26.5	26.5	467	17.7	3.16
5 (PH2)	26.5	52.9	1016	19.2	2.45
4 (PH1)	26.5	79.4	1623	20.5	2.14
3	114.4	193.8	3460	17.9	1.75
2	739.5	933.3	13145	14.1	1.25
1	785.4	1718.7	24692	14.4	1.00

PH: Penthouse.

Figure 7 to Figure 9 show framing plans and framing elevations of representative sections. Most of the wing-shaped sections (AX1 - AX8, CX1 - CX8) have a rigid-frame structure, but seismic resisting walls are provided to the terminal in both X and Y directions. The core area at the center of the building has frames with seismic resisting walls around the corridor and room boundaries.

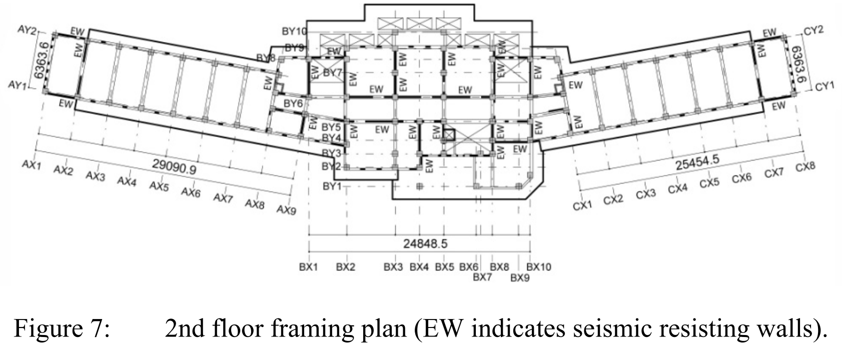

Figure 7: 2nd floor framing plan (EW indicates seismic resisting walls).

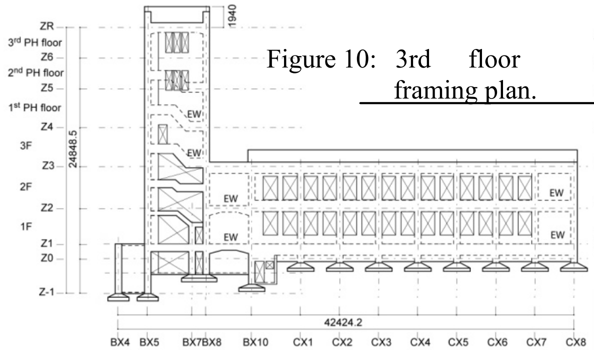

Figure 10: 3rd floor framing plan.

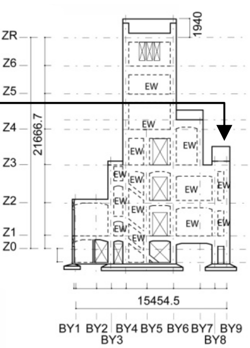

Figure 8: BY4-CY1 framing elevation.

Figure 9: BX5 framing elevation.

The aging index was set at 0.927 based on the results of the field survey. No structural cracks were found during the appearance check. From the progress of neutralization, the neutralization range was set to 1/3 or more of the total members. Though no rust fluid or expansion cracks were found in the corrosion of the reinforcing steels, they were set below 1/9 of the members taking neutralization into consideration.

The seismic evaluation results show that all the floors except the 3rd satisfy the Is value: 0.6 or more and $C_{TU} \bullet S_D$: 0.3 or more (Table 8). Spans are small, and the entire building is relatively rigid. Many vertical members have the F-value of 1.0, indicating that the building is resistant to earthquakes with its strong structure. Since the design seismic intensity when the building was designed was 0.1 common to every floor, and was not specified for the penthouse, which was set according to necessity, higher floors see more difficulty compared with the

Table 8: Seismic evaluation results (present condition).

	Floor	C	F	E_0	S_D	I_S	C_{TU}, S_D	Evaluation
X direction	6 (PH3)	6.03	1.00	1.61	0.95	1.42	1.53	OK
	5 (PH2)	5.10	1.00	1.79	0.95	1.57	1.70	OK
	4 (PH1)	2.33	1.00	0.95	0.95	0.83	0.90	OK
	3	1.59	1.00	0.81	0.63	0.47	0.51	NG
	2	2.41	1.00	1.83	0.95	1.61	1.74	OK
	1	1.58	1.00	1.58	0.95	1.39	1.50	OK
	Floor	C	F	E_0	S_D	I_S	C_{TU}, S_D	Evaluation
Y direction	6 (PH3)	5.18	1.00	1.38	0.95	1.22	1.31	OK
	5 (PH2)	5.56	1.00	1.95	0.95	1.72	1.85	OK
	4 (PH1)	3.24	1.00	1.32	0.95	1.16	1.25	OK
	3	1.76	1.00	0.89	0.69	0.57	0.62	NG
	2	1.81	0.80	1.10	0.95	0.96	1.30	OK
	1	1.19	0.80	0.95	0.95	0.83	1.12	OK

The codes in the table are based on the references [9].
Aging index T: 0.927, Criteria: Iso = 0.60 < Is, $C_{TU} \bullet S_D$ > 0.30.

present seismic evaluation standard. In spite of this, the 3rd floor and the penthouse are strong due to their small area and many walls. However, the position of the penthouse (4th to 6th floors) has a planar eccentricity against the plane arrangement of the 3rd floor. Therefore, the Is value fell below the standard value (= 0.6) due to the eccentricity ratio of the 3rd floor of 0.359 in the X direction; 0.241 in the Y direction.

7 Seismic reinforcement plan and seismic capacity

From the results described in Section 6, the reinforcement plan was made to increase the bearing force in the X direction on the 3rd floor, and solve the eccentricity ratio caused by the penthouse in the Y direction. To solve the eccentricity ratio, the seismic reinforcement plan took into account that the building is a historical landmark, and increased concrete of the interior walls to increase the strength and reduce the eccentricity ratio, not providing slits to the walls so that the appearance of the building would not be affected. Figure 10 shows the details of the reinforcement plan.

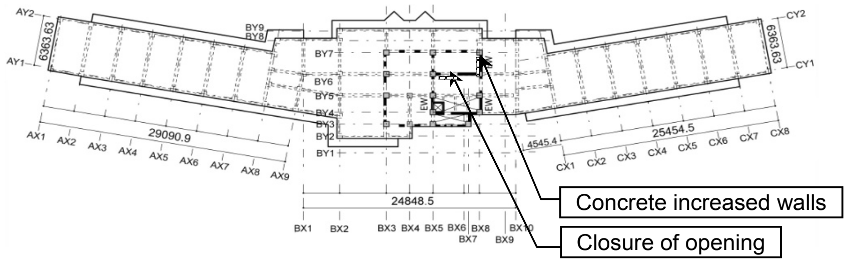

Figure 10: Seismic reinforcement location chart (3rd floor framing plan).

Table 9: Seismic evaluation results (after reinforcement).

	Floor	C	F	E_0	S_D	I_S	C_{TU}, S_D	Evaluation
X direction	6 (PH3)	6.03	1.00	1.61	0.95	1.42	1.53	OK
	5 (PH2)	5.10	1.00	1.79	0.95	1.57	1.70	OK
	4 (PH1)	2.33	1.00	0.95	0.95	0.83	0.90	OK
	3	2.18	1.00	1.11	0.63	0.65	0.70	OK
	2	2.41	1.00	1.82	0.95	1.61	1.73	OK
	1	1.58	1.00	1.58	0.95	1.39	1.50	OK
	Floor	C	F	E_0	S_D	I_S	C_{TU}, S_D	Evaluation
Y direction	6 (PH3)	5.18	1.00	1.38	0.95	1.22	1.31	OK
	5 (PH2)	5.56	1.00	1.95	0.95	1.72	1.85	OK
	4 (PH1)	3.24	1.00	1.32	0.95	1.16	1.25	OK
	3	1.77	1.00	0.90	0.78	0.65	0.70	OK
	2	1.81	0.80	1.10	0.95	0.96	1.30	OK
	1	1.19	0.80	0.94	0.95	0.83	1.12	OK

The codes, the aging indices, and the criteria in the table are the same as those of Table 8.

The plan increased the bearing force in the X direction and improved mainly the S_D index in the Y direction. Consequently, Is and $C_{TU}\cdot S_D$ satisfied the criteria values on all the floors. Table 9 shows the calculation results after the reinforcement plan was implemented.

8 Conclusion

This paper reported the seismic capacity of a building for disaster reconstruction which was designed and constructed soon after the 1923 Great Kanto Earthquake at the end of Taisho period as an example.

Yoshikazu Uchida, a concrete architectural engineer, made a great contribution to this building. The building shows his design purpose for constructing a building which is highly rigid and can endure a great earthquake in response to the damages caused the 1923 Great Kanto Earthquake. Since the concrete is strong, it is estimated that the building was constructed carefully. From the drawings used during the design and photos during construction, the authors learned exemplary bar arrangement methods before the standards including the Architectural Institute of Japan were created.

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