



INTRODUCTION OF THE SEISMIC STANDARDS IN JAPAN

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Abstract

In Japan, the first seismic code took place in year 1924 in Urban Building Law (UBL), introducing lateral factor of 0.1. There are many countries which seismic code is in regulation today and there are many structural engineers designing seismic resistant structures. Seismic code of Japan has already proven in many earthquakes, including the Great East-Japan earthquake, that a structure has low possibility of collapse as far as it is built following the code. However, the essential of the code is not known among the structural engineers worldwide.

This paper attempts to introduce the design code of Japan as well as techniques of engineers in practice of moment resisting reinforced concrete designed by route 3 of BSL.

Long-term loads are set based on the weight consisting of dead and live loads as loads to be considered when designing the structure of the building. Separately, short-term loads are set based on short-term parameters such as seismic and wind loads. Earthquakes and typhoons are typical examples of natural disasters in Japan, but structural designs are customarily carried out on the assumption that they do not occur at the same time. In addition, seismic loads are often dominant in many designs. The seismic force acting on buildings is an important factor in structural design not only in Japan but also in countries that have experienced earthquakes. In structural design in Japan, the collapse system of the buildings is calculated using a theoretical and simplified to confirm the safety of the structure. In structural design in the U.S. and elsewhere, the safety of the structure is confirmed by reducing the allowable capacity of the building using the factor and comparing it with the capacity when the first hinge is generated. Both methods are consistent with the purpose of performing structural calculations, but they are often difficult to understand because of the different methods of evaluation. In this paper, we will explain the characteristics of seismic standards and the determination of seismic force in Japan, focusing on calculations such as the lateral load-carrying capacity normally used for buildings of a certain scale or larger. We will also explain the features and introduces the seismic standards in Japan.

Keywords: the determination of seismic force, lateral load-carrying capacity, seismic standards in Japan



1 Introduction

In Japan, the first seismic code took place in year 1924 in Urban Building Law (UBL), introducing lateral factor of 0.1. There are many countries which seismic code is in regulation today and there are many structural engineers designing seismic resistant structures. Seismic code of Japan has already proven in many earthquakes, including the Great East-Japan earthquake, that a structure has low possibility of collapse as far as it is built following the code. However, the essential of the code is not known among the structural engineers worldwide.

This paper attempts to introduce the design code of Japan as well as techniques of engineers in practice of moment resisting reinforced concrete designed by route 3 of BSL.

2 Outline of structural design in Japan

Structural design is ruled by Building Standard Law (BSL) of Japan[1] and practiced by structural engineers who hold national license. BSL regulates design loads (i.e. dead load, live load, snow load, wind load, earthquake load and other relative loads which needs to be considered), method of estimation of each loads, strength of structural materials, method of determination of safety of the structure.

In addition to legal environment that rules the engineers, because severe earthquake occurs anywhere throughout the country of Japan, it is natural to take the seismic effect into consideration in design of architectural components or M & E equipment apart from structure. The motivation for safety is natural among the designers and engineers. No codes or standards of Japan are developed assuming without seismic effect.

Detail of structural design code can be found in "Manual of technical standard for building structure". The publication is updated time to time and it is used as a reference textbook of seminars for licensee which is mandatory to take every 3 years to extend the expiry. Before start of construction work, design document shall be confirmed by an inspection agent appointed by the minister of land, infrastructure, transport and tourism. The agent confirms if the design document is complying with BSL.

BSL requires 2 design phases for seismic design. The 1st phase is to achieve no damage due to moderate earthquake which may occur several times during service period of the building. Stress in all members shall be below allowable stress. Allowable stress is determined based on yield strength of materials, therefore the 1st phase design shall not consider beyond elastic, and it is prohibited to consider the stiffness by crack etc.

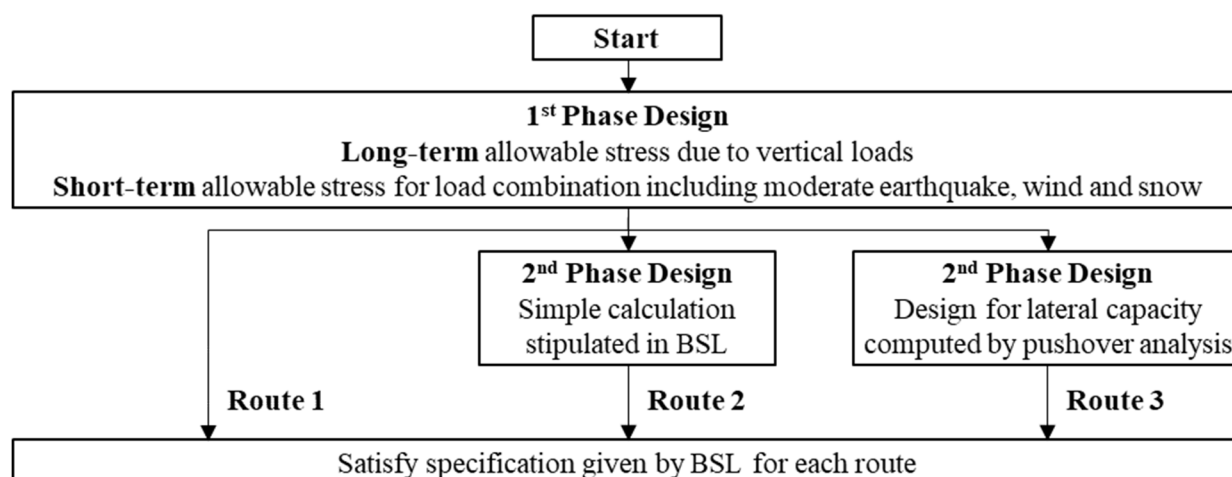


Fig. 1 Flow chart of structural design by BSL

There are 3 routes of design procedure regulated for seismic design. Route 1, 2 and 3 (See Fig. 1). Engineer shall take one of them as far as the project matches the requirement of the procedure, which includes



size, height and structural material of the building. Route 1 has detail specification all determined in BSL, but without 2nd phase design. Route 2 requires some details of construction but simplified calculation can be used for 2nd phase. And route 3 has high flexibility of design by assessing the ultimate lateral shear with pushover analysis in 2nd phase. If the building doesn't match the requirement of all routes, it is considered as a special building. It requires permission by the minister, which consist of detailed study to fulfill a committee organized by the authority for each project.

The 2nd phase is to confirm not to collapse due to severe earthquake which may occur only once during service period of the building. The "ultimate lateral seismic capacity Q_u " needs to exceed "specified ultimate lateral shear Q_{un} ". Analysis to compute Q_u shall be pushover analysis considering plastic hinges. Pushover analysis is done by load-increment-method and no plastic hinge is allowed in columns even if Q_{un} is achieved.

Naturally, strength of structure consists not only of material and dimensions of members but also the system of the structure, or geometry of the structure. For building with vertical and horizontal frame, strength and stiffness of building differs depending on structural system. The stress and stiffness of the structure needs to be assessed by the analysis.

It is common to categorize the seismic design to two types, "ductile design" and "strong design". The strong design achieves required strength with less plastic hinge compared to ductile design, such as shear wall or vertical brace inserted structure. Ductile design requires many plastic hinges to achieve the specified ultimate lateral shear. Because many plastic hinges are needed to achieve the ultimate lateral shear specified by BSL. The deformation becomes large and the joints are required flexible enough to rotate without losing high stress occurred. Therefore, BSL requires more strict detail to ductile structure. And BSL requires higher lateral seismic shear if brittle member is used.

3 1st Phase design

3.1 Load combination

Design stress shall be computed from conditions shown in Table 1. Permanent stress shall be verified against long-term allowable stress whereas temporary stress shall be verified against short-term allowable stresses.

Table 1 Load Combination

Stress type	Load Condition	Ordinary district	Heavy snow district	Note
Permanent	Normal	G+P	G+P	
	Snow		G+P+0.7S	
Temporary	Snow	G+P+S	G+P+S	When considering the overturning of a building and pull-out of columns, stress by P shall be reduced.
	Wind	G+P+W	G+P+W	
			G+P+0.35S+W	
Earthquake	G+P+K	G+P+0.35S+K		

In this table, G, P, S, W, and K is ;

G : stress due to dead loads

P : stress due to live load

S : stress due to snow load

W : stress due to wind load

K : stress due to earthquake load

3.2 Seismic force

Lateral seismic shear of i th story Q_i is shown in Eq. (1), with weight of building above i th story, and lateral story shear factor C_i of i th floor is, shown in Eq. (2),

$$Q_i = C_i W_i \quad (1)$$

$$C_i = Z R_t A_i C_0 \quad (2)$$



3.2.1 Seismic zoning factor Z

Z represents the relative factor of expected earthquake ground motions based on earthquake records, and is specified for each region in range from 1.0 to 0.7.

3.2.2 Vibration characteristic factor R_t

R_t is amplification of building motion depending on soil category in terms of fundamental natural period of building. The formula is shown in Eq. (3) and the graph in Fig. 2.

$$R_t = \begin{cases} 1.0 & \text{for } T \leq T_c \\ 1.0 - 0.2 \left(\frac{T}{T_c} - 1 \right)^2 & \text{for } T_c < T \leq 2T_c \\ \min\left(1.6 \frac{T_c}{T}, 0.25\right) & \text{for } 2T_c < T \end{cases} \quad (3)$$

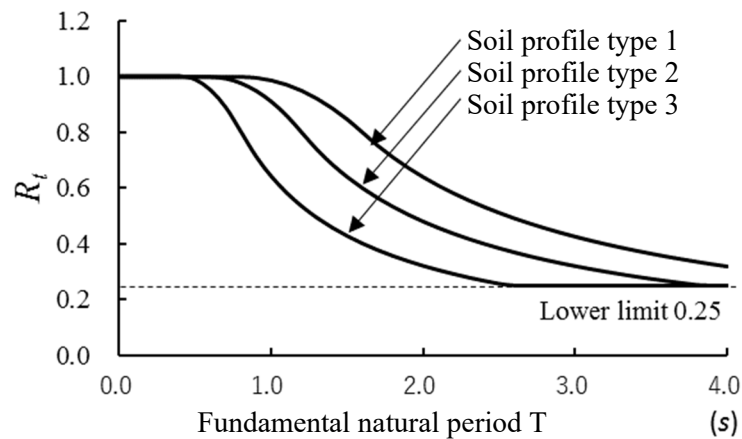


Fig. 2 Vibration characteristic factor

Fundamental natural period of building T can be estimated by Eq. (4), where h is the height of the building above ground, α is a ratio of total height of stories which are made of timber or steel divided by h . The natural period of ground T_c can be chosen by Table 2, based on the type of soil below the foundation or bearing layer for pile foundation. The soil profile type can be classified by special research.

$$T = h (0.02 + 0.01\alpha) \quad (4)$$

Table 2 Site classification

	Description	T_c (sec.)
Soil profile type 1	Majority is composed of rock beds, hard sand gravel and other strata before dilapidation, or those deemed to have similar ground period based on soil investigation or research.	0.4
Soil profile type 2	Other from soil profile type 1 and 3 ground	0.6
Soil profile type 3	Soil, mud and the like, mostly composed of alluvial layers for 30 meters or more, those within 30 years after the reclamation of swamp, mud sea, etc., or those deemed to have similar ground period based on soil investigation or research.	0.8

3.2.3 Vertical distribution factor A_i

The vertical distribution of story shear is determined by A_i with following formula.



$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \cdot \frac{2T}{1+3T} \quad (5)$$

α_i is ratio of i th story mass divided by total mass. Fig. 3 shows plots based on Eq.(5).

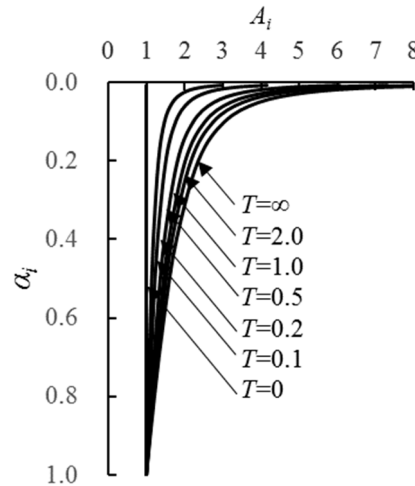


Fig. 3 A_i distribution

3.2.4 Standard shear factor C_0

C_0 in the allowable stress design is determined as 0.2 or more.

3.2.5 Comparison of formulas for seismic action with Japan and the United States

Table 3 shows the factors used in formulas for seismic force in Japan and its relevant factors in US.

Table 3 Comparisons of various factors for seismic effects

factors	BSL of Japan	IBC & ASCE 7
Seismic zoning factor	Z	S_s, S_1 Maps
Site factor	R_t (Three curves)	F_a, F_v
Structural design factor	D_s	$1/R$
Design response spectrum	$2.5 R_t$	As same as ISO3010
Seismic shear distribution	A_i	Seismic force with inverted triangle-parabolic distribution

3.3 Analysis of the 1st phase design

The stress of structural member shall be determined by linear elastic analysis and analysis model must be same for vertical load and for lateral load. Shear walls are modeled by line element, plane element, or complexed models by occasion. The analysis should include stiffness, considering shear deformation of wall girders and short columns. For analysis of short-term stress may consider degraded stiffness in elastic range that does not result in a plastic state for the corresponding stress and deformation that occur in each member.

Seismic force shall be applied in each direction X and Y of the building. If the horizontal stiffness of the structural member has irregularity on plan, 3D model analysis is needed to compute the distribution of horizontal resisting element and the stress in the floor slab shall be reflected in the design.

In-plane stiffness of the floor slab is usually assumed as rigid body. In case of floor opening or etc. the in-plane deformation of floor shall be considered.

3.4 Modeling of foundation



The condition at support of analysis model such as rotational, vertical shall be determined considering horizontal stiffness of the ground, foundation shape or connection of the pile. The condition of supports give effect to the result of the analysis. Usually, ground beams are modeled, and the ground is assumed rigid. Ground beam and bottom of column is moment connected and pin-supported on ground.

3.5 Verification of section

Columns, beams, bearing walls, and foundations are generally considered as the main structure. The structure shall be verified based on analysis, including members, their anchor length and development length of reinforcement, at the column face of both ends of beams and at the top and bottom of columns. All design stress is without factor except the short-term design shear force Q_D is calculated by Eq. (6).

$$Q_D = Q_L + nQ_E \quad (6)$$

where: Q_L Long-term design shear force
 Q_E Shear force due to seismic force
 Increment factor $n \geq 1.5$

In 1st phase design, secondary beams, floor slabs, and studs shall be secondary members and not required to design against stress other than permanent stress, except transfer of the seismic force to the other side of the slab is critical, the shear capacity shall be verified in same direction with the shear wall.

3.6 Simplified calculation method of beam reinforcement

If section area of reinforcement a_t is less than or equal to equivalent compression block by stress, the required section area of reinforcement can be estimated using a simple equation. The relation between stress of reinforcement σ_{st} and bending moment M can be described as Eq. (7), using T as tensile force in reinforcement,

$$M = Tj = a_t \sigma_{st} \quad (7)$$

Therefore, required section area a_{req} is determined by the following formula where the allowable tensile stress of reinforcement is f_t ,

$$a_{req} \geq M / f_t j \quad (8)$$

The RC Standard published by AIJ (Architectural Institute of Japan) allows the effective depth j to be $(7/8)d$ in Eq.(8). Then tensile reinforcement ratio p_t can be obtained by Eq.(9) beam width b and depth d ,

$$p_t = a_t / bd \quad (9)$$

p_t shall be less than reinforcement ratio p_{tb} , which gives reinforcement amount equivalent to compression stress. When the bar ratio p_t exceeds p_{tb} , increase the reinforcement in tension. The ratio of reinforcement section area at compression side a_c against area in tension is denoted with γ ,

$$\gamma = a_c / a_t \quad (10)$$

3.7 Minimum requirement of RC structure

Minimum requirements are regulated in terms of durability, serviceability, workability for concrete casting, prevent neutralization and fire resistance. Column, beam, and shear wall have minimum size, minimum amount of reinforcement (both longitudinal and transverse), minimum spacing and clearance between bars, Minimum coverage of reinforcing bar is specified. The standard requires to add 10 mm to the minimum coverage by law to avoid unexpected short of coverage. The bent angle and bending radius of reinforcement is specified by location, by types(plain or deformed) and by diameters. Development length is specified by bar type, strength of concrete, combination with hook and location. Not only lap splice but gas pressure welded splice and mechanical splices are commonly used.



4 2nd Phase design

4.1 Story drift

Story drift for verification shall be computed for moderate earthquake. The story drift angle shall not exceed 1/200 of story height. Story drift angle shall be computed by the story drift divided by the story height.

4.2 Horizontal and vertical irregularity

Stiffness eccentricity ratio R_{ei} of i th story shall be computed and limited as shown in Eq. (11). Where, e_i is the eccentricity of the center of stiffness from the center of gravity, and r_{ei} is the elastic radius.

$$R_{ei} = e_i / r_{ei} \leq 0.15 \quad (11)$$

Lateral stiffness ratio R_{si} of i th story shall be computed and limited as shown in Eq. (12), where r_{si} is the stiffness of i th story given as inverse of story drift and \bar{r}_s is the average of r_{si} among all stories except basement.

$$R_{si} = r_{si} / \bar{r}_s \geq 0.6 \quad (12)$$

4.3 Ultimate lateral capacity

Ultimate lateral capacity Q_{un} is defined as shown in EQ. (13).

$$Q_{un} = D_s F_e F_s Z R_t A_i C_0 W_i \quad (13)$$

4.3.1 Structural characteristic factor D_s

The procedure of setting the structural characteristic factor D_s is shown in case of RC structure in this section. In the process of computing D_s , it requires stress value when collapse mechanism. The method to compute the stress is not regulated in detail, but it is common to run pushover analysis only to determine D_s value, nowadays. For deriving D_s value, member class shall be set based on Table 4.

Table 4 Classification of structural member(RC column and beam)

Type of failure	Condition				Beam	Class
	Minimum value of h_0 / D	Maximum value of σ_0 / F_c	Maximum value of p_t	Maximum value of τ_u / F_c		
Other than shear failure, bond-rapture, compression failure or any failure which cause instant loss of resistant	2.5	0.35	0.8	0.100	0.15	FA
	2.0	0.45	1.0	0.125	0.20	FB
	N/A	0.55	N/A	0.150	N/A	FC
If not fall to FA, FB nor FC						FD

Note:

h_0 : clear height of column

D : width of column

σ_0 : axial stress when the story reach to mechanism

F_c : specified strength of concrete

p_t : ratio of section area of longitudinal bars in tension divided by section area of the column

τ_u : shear stress when the story reach to mechanism



If column and beam become different class at a joint, the lowest class should be taken. Then the class by member group is set by Table 5. D_s is chosen considering the share of shear wall as shown in Table 6.

Table 5 Member group class per story

Class of member group at the story	Ratio of sum of capacity of FA divided by total capacity of column	Ratio of sum of capacity of FC divided by total capacity of column
A	50% or greater	20% or less
B	N/A	Less than 50%
C	N/A	50% or greater
D	If any of the column is FD.	

Table 6 Structural characteristic factor D_s

			Class of column and beam group			
			A	B	C	D
Class of shear wall member group	A	$0 < \beta_u \leq 0.3$	0.30	0.35	0.40	0.45
		$0.3 < \beta_u \leq 0.7$	0.35	0.40	0.45	0.50
		$0.7 < \beta_u$	0.40	0.45	0.45	0.55
	B	$0 < \beta_u \leq 0.3$	0.35	0.35	0.40	0.45
		$0.3 < \beta_u \leq 0.7$	0.40	0.40	0.45	0.50
		$0.7 < \beta_u$	0.45	0.45	0.50	0.55
	C	$0 < \beta_u \leq 0.3$	0.35	0.35	0.40	0.45
		$0.3 < \beta_u \leq 0.7$	0.40	0.45	0.45	0.50
		$0.7 < \beta_u$	0.50	0.50	0.50	0.55
	D	$0 < \beta_u \leq 0.3$	0.40	0.40	0.45	0.45
		$0.3 < \beta_u \leq 0.7$	0.45	0.50	0.50	0.50
		$0.7 < \beta_u$	0.55	0.55	0.55	0.55

Note: β_u : Total of lateral capacity of shear wall divided by ultimate lateral capacity.

4.3.2 Shape factor $F_e F_s$

Shape factor of i th story F_{ei} and F_{Si} is set per story based on R_{ei} and R_{Si} as shown in Table 7 and Table 8.

Table 7 Shape factor F_{ei} to add eccentricity effect

R_{ei}	F_{ei}
$R_{ei} \leq 0.15$	1.0
$0.15 < R_{ei} \leq 0.3$	Linear interpolation between 1.0 and 1.5
$0.3 \leq R_{ei}$	1.5

Table 8 Shape factor F_{Si} to add irregularity effect of stiffness

R_{Si}	F_{Si}
$R_{Si} \geq 0.6$	1.0
$0.6 > R_{Si}$	$2.0 - R_{Si} / 0.6$

5 Example of design by BSL

5.1 Outline of the building



Building for case study is set as below, considering usage and structural type commonly built in many countries. With frames shown in Fig. 4, analyzed[2] using including linear analysis and pushover analysis.

- Location: capital city area in Japan (Seismic zone factor: $Z=1.0$)
- Site: Soil profile type 2
- Occupancy: Office, with o
- Structural system: Reinforced concrete 5-story buildings with moment frame systems
- Configuration: symmetry in two directions
- Number and length of spans: 4 spans of 7meters in X direction and 2 spans of 8meters in Y direction
- Height: 5.00 m for each stories, Total $h = 25$ m (fundamental natural period $T = 0.5$ sec), so $R_t=1.0$
- Concrete strength: Fc 30 for 1F - 3F, Fc 27 for 3F - 5F, Fc24 5F - RF
(Number after “Fc” shows compressive strength of cylinder specimen.)
- Reinforcement grade: SD295 for D10 - D16, SD345 for D19 - D25, SD390 for D29
(Number after “SD” shows yield strength of steel.)

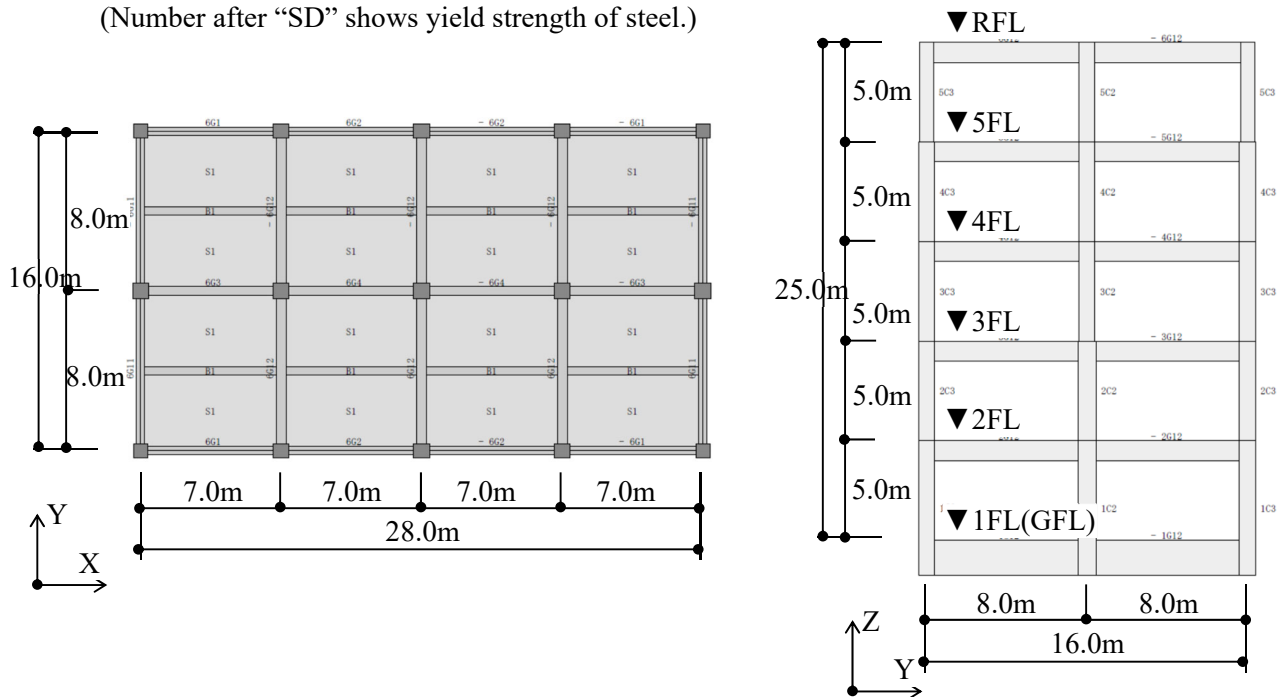


Fig. 4 Framing plan and elevation

5.2 Outline of Analysis

5.2.1 Story shear

Calculation of story shear is shown in Table 9. A_i factor is almost 1.919 at 5th story which means that the story shear factor is amplified to double of base shear at 5th story. It is common to review the accuracy of input with dividing the floor weight w_i by floor area S_i (w_i/S_i). Usually, the value falls to 10 to 15 kN/m².

Table 9 Calculation of story shear

Story	w_i (kN)	Σw_i (kN)	α_i	A_i	C_i	Q_i (kN)	w_i/S_i (kN/m ²)
5	5164.9	5164.9	0.165	1.919	0.383	1982.5	11.53
4	6018.3	11183.2	0.357	1.526	0.305	3413.8	13.43
3	6437.7	17620.9	0.563	1.307	0.261	4608.1	14.37
2	6816.8	24437.6	0.782	1.139	0.227	5570.2	15.22
1	6808.1	31245.8	1.000	1.000	0.200	6249.2	15.20



5.2.2 Section of structural members

Beam schedule is shown in Fig. 5. Beam depth is concluded to 1,000mm with 450 to 750mm in width. Column size is 900mm square at lower stories and 800mm at higher stories.

2FL Beam	END	MID
B x D	600 x 1000	
TOP	6 - D29	5 - D29
BOTTOM	5 - D29	5 - D29
STIRRUP	□ - D13@150	
WEB	4 - D10	

Fig. 5 Beam schedule (2FL Y-direction)

Column schedule is shown in Fig. 6.

1FL Column		
	Dx x D y	800 x 800
Main bar	18 - D29	
Hoop ²	□ - D13@100	

Fig. 6 Column schedule (2FL)

5.2.3 Eccentricity and story drift

Story drift is shown in Table 10. The limit in BSL is 1/200, so, all floors for both direction is passing.

Table 10 Story drift (Elastic analysis)

Story	Hight (mm)	Story drift (mm)	Story drift angle $1 / r_s$ (rad)	Lateral stiffness ratio R_s	Story drift (mm)	Story drift angle $1 / r_s$ (rad)	Lateral stiffness ratio R_s
		X (+ direction)			Y (+ direction)		
5	5000	3.073	1/1627	1.404	3.673	1/1361	1.306
4	5000	4.492	1/1112	0.962	5.013	1/997	0.957
3	5000	4.945	1/1011	0.875	5.547	1/901	0.865
2	5000	5.411	1/924	0.799	5.819	1/859	0.825
1	5000	4.513	1/1107	0.958	4.594	1/1088	1.045



F_s or F_e are factors that become larger than 1.0 if the story has irregularity as shown in Table 11. This building found to be well-formed and well-balanced stiffness among plan and upper and lower floors, because F_s and F_e are all 1.0.

Table 11 Eccentricity (X+ and Y+)

Floor	center of gravity (m)		center of stiffness (m)		Elastic radius (m)		Eccentricity (m)		Eccentricity ratio		Shape factor	
	g_x	g_y	l_x	l_y	r_{ex}	r_{ey}	e_y	e_x	R_{ex}	R_{ey}	F_{ex}	F_{ey}
5	14.000	8.148	14.004	8.000	10.763	11.763	0.148	0.004	0.014	0.000	1.000	1.000
4	13.980	8.102	13.999	8.000	11.016	11.641	0.102	0.019	0.009	0.002	1.000	1.000
3	13.975	8.086	13.999	8.003	10.789	11.437	0.083	0.024	0.008	0.002	1.000	1.000
2	13.973	8.078	13.999	8.000	10.675	11.072	0.078	0.026	0.007	0.002	1.000	1.000
1	13.971	8.073	14.000	8.001	11.213	11.318	0.072	0.028	0.006	0.002	1.000	1.000

5.2.4 Pushover analysis

Pushover analysis is applied as shown in Fig. 7.

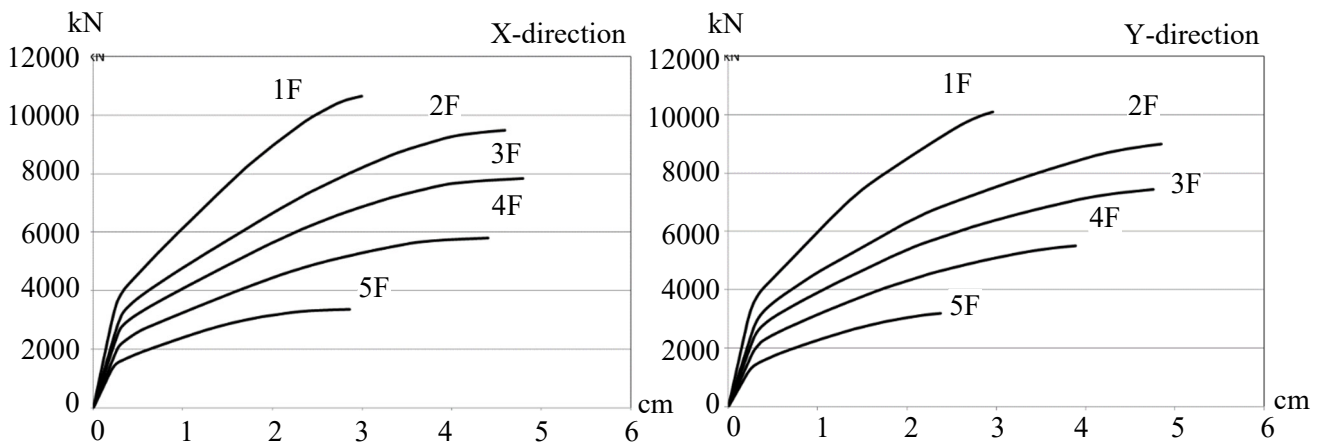


Fig. 7 Force displacement curve of pushover analysis

5.2.5 Ultimate lateral seismic capacity

Verification of ultimate lateral seismic capacity is shown in Table 12. Structural characteristic factor D_s is determined by pushover analysis. The D_s became 0.3 for all stories and both directions. The analysis was stopped before reaching to full mechanism, because the maximum story drift angle reached to 1/100.

Table 12 Verification of ultimate lateral seismic capacity

Story	D_s	F_e	F_s	Q_{un} (kN)	Q_u (kN)	Q_u/Q_{un}	Result	D_s	F_e	F_s	Q_{un} (kN)	Q_u (kN)	Q_u/Q_{un}	Result
5	0.30	1.0	1.0	2974	3369	1.13	OK	0.30	1.0	1.0	2974	3200	1.07	OK
4	0.30	1.0	1.0	5121	5802	1.13	OK	0.30	1.0	1.0	5121	5510	1.07	OK
3	0.30	1.0	1.0	6912	7832	1.13	OK	0.30	1.0	1.0	6912	7438	1.07	OK
2	0.30	1.0	1.0	8355	9468	1.13	OK	0.30	1.0	1.0	8355	8992	1.07	OK
1	0.30	1.0	1.0	9374	10623	1.13	OK	0.30	1.0	1.0	9374	10088	1.07	OK



5.3 Comparison of analysis

Verification of ultimate lateral seismic capacity is shown in Table 13. It shows the story shear in terms of C_i and story drift in terms of $1 / r_s$. The deflection is amplified larger than increase of seismic shear when non-linear analysis is done. For example, the seismic shear at 5th story for Y direction, it increased 1.6 times ($0.615 / 0.383$), but the story drift increased 6.3 times ($(1/217) / (1/1361)$).

From this table, it seems impossible to predict the result of pushover analysis from elastic analysis. In this study, the structure is RC structure with moment frame system without horizontal nor vertical irregularity. From actual experience, the range of variation will be wider if the material of structure, structural system, geometry of structure is different.

Therefore, the pushover analysis is considered as the only the reliable method other than dynamic analysis to estimate the seismic capacity of structure.

Table 13 Comparison of analysis between 1st and 2nd phase design

Story	X (+ direction)				Y (+ direction)			
	1st phase		2nd phase		1st phase		2nd phase	
	C_i	$1 / r_s$ (rad)	C_i	$1 / r_s$ (rad)	C_i	$1 / r_s$ (rad)	C_i	$1 / r_s$ (rad)
5	0.383	1/1627	0.649	1/172	0.383	1/1361	0.615	1/217
4	0.305	1/1112	0.517	1/114	0.305	1/997	0.490	1/128
3	0.261	1/1011	0.442	1/102	0.261	1/901	0.419	1/102
2	0.227	1/924	0.385	1/109	0.227	1/859	0.364	1/100
1	0.200	1/1107	0.339	1/167	0.200	1/1088	0.321	1/167

6 Example of design

The seismic design code of Japan is introduced and shown that the pushover analysis is important to be carried out to estimate the lateral seismic capacity of the building.

7 Acknowledgement

This paper is based on discussion at a subcommittee of seismic resilience under CIB Committee in Architectural Institute of Japan (AIJ)[3].

8 References

- [1] Building Standard Law of Japan, <http://law.e-gov.go.jp/htmldata/S25/S25HO201.html>
- [2] Super Build SS3 from Union System Inc.
- [3] Subcommittee of seismic resilience under CIB Committee in Architectural Institute of Japan (AIJ) <https://www.aij.or.jp/x4xx-12/x402/x404.html>