17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

Seismic design procedure for building structures based on the equivalent linearization method

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Abstract

This paper discusses the seismic design procedure for building structures based on the equivalent linearization method. The concept of equivalent linearization was adopted in the seismic design processes of Japanese Building Standard Law enacted in 2000 as the “Response and limit capacity calculation”. The properties of design response spectra adopted in the response and limit capacity method are investigated. Estimation method of inelastic response to strong earthquake motion using equivalent linearization approach is discussed. Analysis of an 11-story building frame is shown in order to illustrate the method. Problems concerning the application of equivalent linearization method to seismic design are reviewed. It is considered that the seismic design of buildings should contain the dual processes, i.e., the initial structural design with appropriate design force and the final evaluation of inelastic response of designed structure, including the iterative process, for which the equivalent linearization method will be a convenient and powerful tool.

Keywords: Seismic design, Equivalent linearization method, Seismic design spectra, Design displacement, Tolerable ductility

1. Introduction

As the result of severe building damages experienced in the 1968 Tokachi-oki earthquake and the 1978 Miyagiken-oki earthquake, the new Japanese seismic design method for buildings was enacted in 1981, in which the vibrational properties and inelastic behavior of building structures were taken into consideration.

After the severe damage by the 1995 Hanshin-Awaji earthquake, the concept of performance-based seismic design was introduced into Japanese Building Standard Law in 2000 as the response and limit capacity calculation method, which was based on the equivalent linearization method [1]. Due to a case that happened in 2005 concerning the misuse of the method, the method has not been widely used yet. Recently, the seismic performance evaluation guidelines for reinforced concrete buildings was published from the Architectural Institute of Japan, which utilized the equivalent linearization approach [2].

This paper deals with the application of the equivalent linearization method for the evaluation of inelastic earthquake response of buildings and discusses several problems in the seismic design procedures utilizing equivalent linearization.

2. Design response spectra in the response and limit capacity calculation method

The response and limit capacity calculation method (referred to as RLCC method hereafter) introduced in the Japanese Building Standard Law in 2000 gives the procedure for judging the seismic safety of designed buildings by modelling the building into the equivalent 1-degree-of-freedom system and evaluating the maximum response against design earthquake using equivalent linearization method. Two levels of design earthquakes are considered in the codes, i.e., medium-scale earthquake to check the behavior within the allowable stress range (design limit state) and large-scale earthquake to check the inelastic response behavior after yielding (performance limit state). The intensity of the medium-scale earthquake is 1/5 of the large-scale earthquake. In this paper, only the case for large-scale earthquake is treated.

Design earthquake force for the large-scale earthquake is given by the design response spectra considering the effect of surface ground as shown in the following. The force distribution and the area seismicity are not discussed here, though they are specified in the Japanese Building Standard Law [1].

The design response acceleration spectra of the large-scale earthquake for three types of surface soil are obtained by multiplying the response spectrum at the engineering base-rock by the ground amplification factors, as shown from eq. (1) to eq. (4).

Engineering Base-Rock

$$S_a(m/s^2) = \begin{cases} 3.2 + 30T & (T < 0.16s) \\ 8 & (0.16 \le T < 0.64s) \\ 5.12/T & (T \ge 0.64s) \end{cases} \quad (1)$$

Soil Type 1, 2, 3

$$S_a(m/s^2) = \begin{cases} 4.8 + 45T & (T < 0.16s) \\ 12 & (0.16 \le T < T_S) \\ A_S/T & (T \ge T_S) \end{cases} \quad (2)$$

$$A_S = \begin{cases} 1.35 \times 5.12 = 6.912 & (Type\ 1,\ Hard\ soil) \\ 2.025 \times 5.12 = 10.368 & (Type\ 2,\ Medium\ soil) \\ 2.7 \times 5.12 = 13.824 & (Type\ 3,\ Soft\ soil) \end{cases} \quad (3)$$

$$T_S(\text{sec}) = \frac{A_S}{12} = \begin{cases} 0.576 & (Type\ 1) \\ 0.864 & (Type\ 2) \\ 1.152 & (Type\ 3) \end{cases} \quad (4)$$

where T= natural period, S_a= acceleration response spectrum, T_s=boundary period depending on the soil type

The ground amplification factor in the range of constant acceleration $T<T_s$ is 1.5 for all soil types. In the constant velocity range of $T \ge T_s$, the amplification factor is 1.35 for soil type 1, 2.025 for soil type 2, and 2.7 for soil type 3. The period T_s in eq. (4) is the boundary period separating the constant acceleration range and the constant velocity range. Fig.1 shows the acceleration response spectra for the surface soil type 1 to 3, together with that of the base rock. It is to be noted that the response spectra for T<0.16s will not be treated in the following discussion due to the practical point of view.

The displacement response spectra shown below are also important for seismic calculation.

$$S_d(m) = S_a(T/2\pi)^2 = \begin{cases} 0.1216T^2 + 1.14T^3 & (T < 0.16s) \\ 0.304T^2 & (0.16 \le T < T_S) \\ B_S T & (T \ge T_S) \end{cases} \quad (5)$$

$$B_S = \begin{cases} 0.1751 & (Type\,1) \\ 0.2626 & (Type\,2) \\ 0.3502 & (Type\,3) \end{cases} \quad (6) \qquad d_S(m) = 12\left(\frac{T_S}{2\pi}\right)^2 = \begin{cases} 0.1008 & (Type\,1) \\ 0.2269 & (Type\,2) \\ 0.4034 & (Type\,3) \end{cases} \quad (7)$$

where S_d= displacement response spectra, d_s= border displacement separating constant acceleration range and constant velocity range.

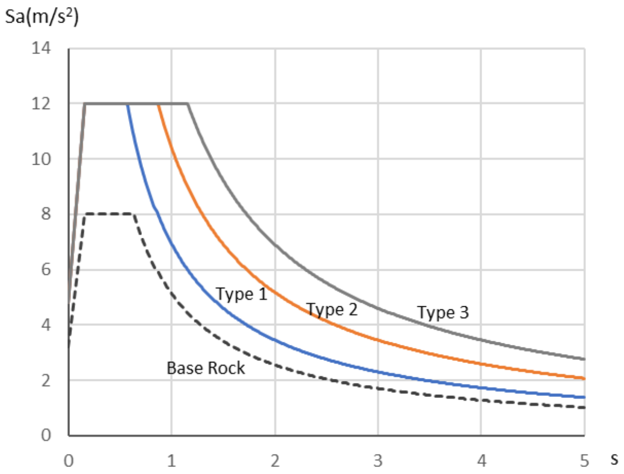


Fig. 1 Acceleration response spectra

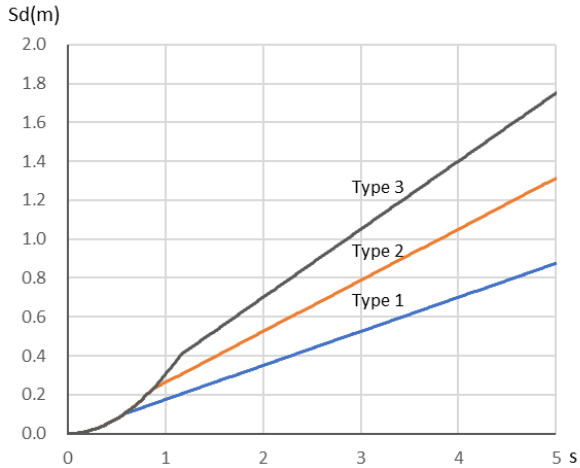


Fig. 2 Displacement response spectra

The displacement response spectra are shown in Fig.2, in which the response displacement is proportional to the building period T in the long period range and differs according to the soil type, while proportional to the square of T in the short period range.

In the seismic design methods, displacement is often expressed in terms of displacement angle. By assuming the following relations about the dynamic properties of buildings, the displacement response spectra are converted to the displacement angle spectra.

$$T = (0.02 \sim 0.03)H \quad (8) \qquad H_e = 0.7H \quad (9) \qquad \gamma = S_d / H_e \quad (10)$$

where H= total building height (m), H_e= effective height of building, γ= displacement angle

The displacement angle corresponding to the displacement response spectrum for the soil type 2 is expressed as follows.

$$\gamma = \begin{cases} 1.737\times10^{-4}H \quad (H < 43.2m) \sim 3.909\times10^{-4}H \quad (H < 28.8m) \\ 7.514\times10^{-3} \quad (H \geq 43.2m) \sim 11.27\times10^{-3} \quad (H \geq 28.8m) = \left(\frac{1}{133} \sim \frac{1}{89}\right) \end{cases} \quad (11)$$

The displacement angle takes a value irrespective of the height in the long period range under the above assumptions. It is noted that eq. (8) adopted in the Japanese seismic codes is based on the data for the small amplitude and further consideration on the coefficient in eq. (8) is needed for the larger amplitude expected in large-scale earthquakes when using it for the response estimation, especially in RC structures.

The response spectra can be expressed in the form of the S_a-S_d spectra, namely, the acceleration-displacement response spectra as shown in Fig. 3. The period T is expressed by the group of straight lines in Fig. 3. The design response acceleration-displacement spectra show the constant value in the short period and the inverse relation in the long period as seen in eq. (14). It is noted that eq. (14) includes A_s^2 in the coefficient, which means that the S_a corresponding to the same S_d is different largely according to the soil type.

$$S_a = \frac{A_S}{T} \quad (12)$$

$$S_d = \frac{S_a}{\omega^2} = \frac{T^2}{4\pi^2} S_a \quad (13)$$

$$S_a = \frac{A_S^2}{4\pi^2}\frac{1}{S_d} \quad (14)$$

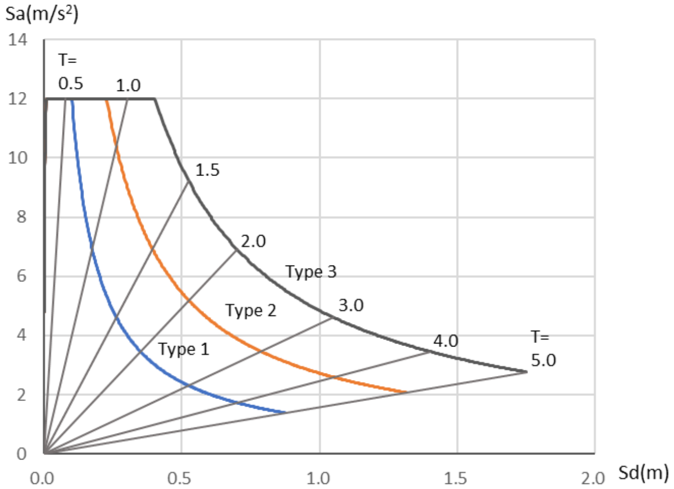


Fig. 3 Acceleration-displacement spectra

3. Response estimation using equivalent linearization method

The estimation methods for the maximum response by use of the equivalent linearization have long been studied worldwide [3][4][5]. In the RLCC method, the building structure is modelled with the 1-degree-of-freedom (1-DOF) system having the inelastic force-displacement relation as shown in Fig. 4, which is derived from eq. (15) and eq. (16) using the data from static inelastic push-over analysis of the structure against appropriate lateral force distribution schematically shown in Fig. 5.

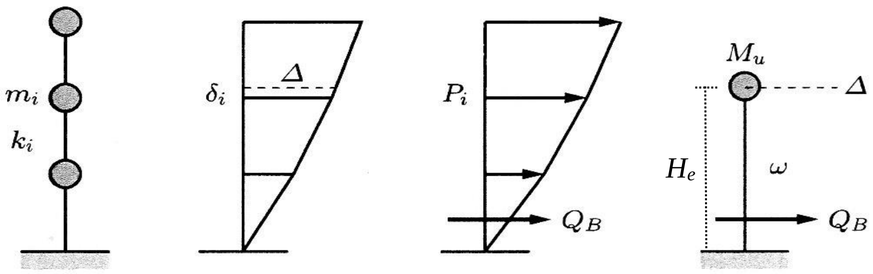


Fig. 4 Equivalent 1-DOF system

$$\Delta = \frac{\sum_{i=1}^{N} m_i \delta_i^2}{\sum_{i=1}^{N} m_i \delta_i} = \quad (15) \qquad Q_B = \sum_{i=1}^{N} P_i \quad (16)$$

$$M_u = \frac{\left(\sum_{i=1}^{N} m_i \delta_i\right)^2}{\sum_{i=1}^{N} m_i \delta_i^2} = \quad (17) \qquad H_e = \frac{\sum_{i=1}^{N} m_i \delta_i H_i}{\sum_{i=1}^{N} m_i \delta_i} \quad (18)$$

where Δ= representative displacement, δ_i= displacement of the i-th story relative to the base, m_i= mass of the i-th story, P_i= force applied at the i-th story, Q_B= base shear, M_u= effective mass,

H_e=equivalent height, H_i=height of the i-th story from the base.

The restoring force characteristics of the equivalent 1-DOF system are expressed by the relation between the representative displacement Δ and the base shear Q_B as shown in Fig. 6. The points A, B and C correspond to the damage limit, safety limit and ultimate limit defined in the RLCC method. It is assumed that the points B and C are the same in this study. If the base shear expressed by the equivalent acceleration Q_B/M_u is denoted by S_a and the representative displacement Δ by S_d, we can draw the inelastic force-displacement relation of the equivalent 1-DOF system on the same graph as the acceleration-displacement response spectra, which is called the "capacity curve". The equivalent restoring force characteristic is modelled by the elasto-plastic relation by the line OYC in Fig.6, or the line OYC in Fig.7 where S_y is equal to Q_y/M_u. The straight-line OY corresponds to the yield-point period of the equivalent 1-DOF system. It is to be noted that in RC structures the difference between the initial period T_i and the period at yield point T_y is large, as illustrated in Fig.7, though only the yield-point period T_y is of main concern in the following.

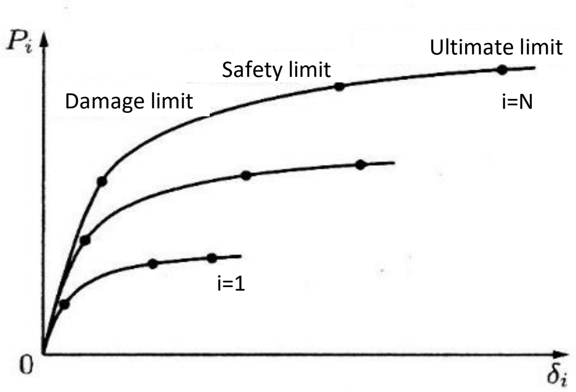


Fig. 5 Force-displacement relation at each story

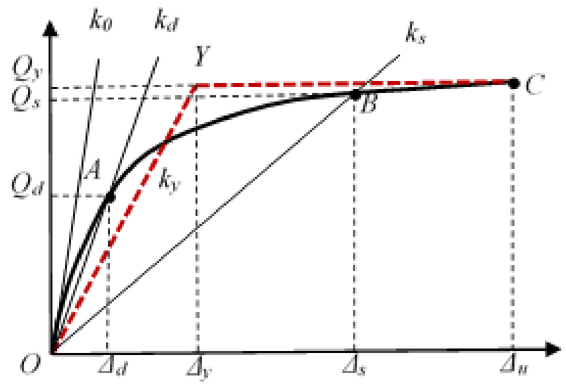
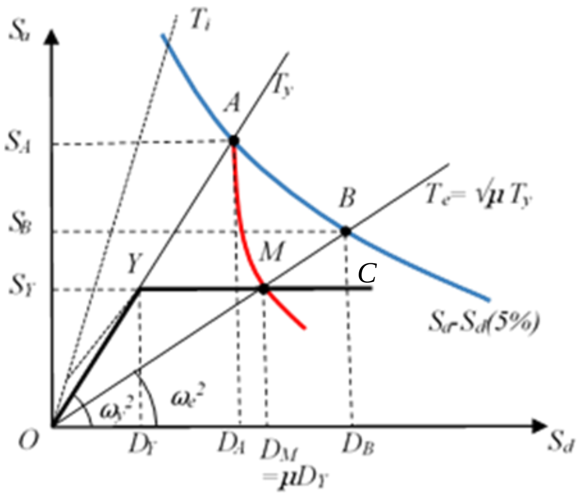


Fig. 6 Force-displacement relation of equivalent 1-DOF system

The response values defined by the design response spectra are plotted on the straight line corresponding to the period T in the plane of acceleration-displacement spectra in Fig.7, in which the elastic response of the system having the period T_y is expressed by the point A (D_A, S_A) on the line OYA.



$$S_Y = Q_y / M_u$$
$$D_y = \Delta_y$$
$$\mu = D_M / D_Y$$
$$S_A / D_A = S_Y / D_Y = \omega_y^2 = (2\pi / T_y)^2$$
$$S_B / D_B = S_Y / D_M = \omega_e^2 = \omega_y^2 / \mu$$
$$S_A / D_A = \mu S_Y / D_M = \mu S_B / D_B$$

Fig. 7 Estimation of inelastic response by capacity curve and demand curve

The RLCC method adopts the equivalent linearization method in order to estimate the maximum inelastic response, which assumes that "the inelastic response is approximately equal to the equivalent linear response of the system having reduced stiffness and increased damping corresponding to the inelastic response level" [3]. In Fig.7, the inelastic response level is expressed by the ductility factor μ which is the ratio of the maximum inelastic displacement to the yield displacement. The stiffness of the system at yield point corresponds to the gradient of the straight line OYA, which is equal to the square of the natural circular frequency at yield point ω_y. The reduced stiffness of the system corresponds to the gradient of line OMB. The equivalent period of the reduced stiffness system T_e and the equivalent damping factor h_e for the ductility factor μ are expressed as follows. (It is noted that more detailed evaluation of h_e for the whole structure considering the equivalent ductility of each constituent member is given in the RLCC method [1].)

$$T_y = \frac{2\pi}{\omega_y} = 2\pi\sqrt{\frac{M_u}{k_y}} \quad (19) \qquad T_e = \frac{2\pi}{\omega_e} = 2\pi\sqrt{\frac{M_u}{k_y/\mu}} = \sqrt{\mu}T_y \quad (20)$$

$$h_e = 0.25\left(1 - \frac{1}{\sqrt{\mu}}\right) + 0.05 \quad (21)$$

The reduction of response by equivalent damping h_e is expressed by the following relation if the response spectra of 5% damping is used as the standard, which can also be expressed by ductility factor μ.

$$F_h = \frac{1.5}{1+10h_e} = \frac{1.5\sqrt{\mu}}{4\sqrt{\mu}-2.5} \quad (22)$$

Estimated inelastic response corresponding to the ductility factor μ is given as follows, which corresponds to the point M (D_M, S_Y) on the straight-line OMB.

$$S_Y = S_B \times F_h = S_a(\sqrt{\mu}T_y, 5\%) \times F_h(\mu) \quad (23)$$

$$D_M = D_B \times F_h = S_d(\sqrt{\mu}T_y, 5\%) \times F_h(\mu) \quad (24)$$

The value S_Y corresponds to the required yield force of the elasto-plastic system for the specified ductility factor of μ. By increasing the ductility factor μ from 1, we can draw the curve of the series of response point M, conceptually illustrated by the red line in Fig. 7, which is called the “demand curve”. If the elasto-plastic capacity curve OYC is uniquely determined from the force-displacement relation of the equivalent 1-DOF system, the maximum response point M is determined by the cross point of the capacity curve and the demand curve, which is equivalent to solving eq. (23) concerning μ for the given values of S_Y and T_y. The corresponding displacement D_M is obtained from the determined μ by eq. (24).

Estimation of maximum response against the design response given in eq. (2) can be done as shown below depending on two parameters, the yield-point period T_y and the yield acceleration S_Y of the system. The design response spectra consist of the constant acceleration range and the velocity constant range, and are defined by the boundary period T_s separating both the ranges given in eq. (4) and the coefficient A_s given in eq. (3). Note that the range of T<0.16sec in Eq. 2 is not considered here.

Let the boundary ductility factor μ_0 and the corresponding boundary yield acceleration S_{Y0} be defined as follows in case of $T_y \leq T_s$, where T_s is the boundary period for the design spectra.

$$\sqrt{\mu_0} = \frac{T_s}{T_y} \geq 1 \quad (25) \qquad S_{Y0} = S(T_y, T_s) = 12 \times \frac{1.5\sqrt{\mu_0}}{4\sqrt{\mu_0}-2.5} = \frac{18T_s}{4T_s - 2.5T_y} \leq 12 \quad (26)$$

Case 1: $T_y \leq T_s$ and $S_Y \geq S_{Y0}$

If $T_y \leq T_s$ and $S_Y \geq S_{Y0}$, the response ductility factor μ is obtained as follows.

$$S_Y = 12 \times \frac{1.5\sqrt{\mu}}{4\sqrt{\mu}-2.5} \quad (27) \qquad \therefore \quad \mu = \mu_1 = \left(\frac{2.5S_Y}{4S_Y - 18}\right)^2 \quad (28)$$

Case 2: $T_y \leq T_s$ and $S_Y < S_{Y0}$

If $T_y \leq T_s$ and $S_Y < S_{Y0}$, the response ductility factor μ is estimated as follows.

$$S_Y = S_B F_h = \frac{A_S}{\sqrt{\mu} T_y} \times \frac{1.5\sqrt{\mu}}{4\sqrt{\mu} - 2.5} \quad (29) \qquad \therefore \quad \mu = \mu_2 = \left(\frac{5}{8} + \frac{3}{8} \cdot \frac{A_S}{S_Y T_y} \right)^2 \quad (30)$$

Case 3: $T_y > T_s$ and $S_Y < S_A(T_y)$

If $T_y > T_s$, the response ductility factor μ is estimated by the same equation as Case 2, i.e., eq. (30).

For all three cases, the maximum displacement D_M is obtained as follows.

$$D_M = \mu \delta_y = \mu S_Y \left(\frac{T_y}{2\pi} \right)^2 \quad (31)$$

Example:

Thee example calculations are shown for the design spectrum for soil type 2 (Medium soil) given in Eq.2 (A_s=10.368, T_s=0.864sec).

Example 1: T_y=0.3s、S_Y=8m/s^2 (Case 1, $S_Y > S_{Y0}$=5.75m/s^2)

$$\mu = \mu_1 = 2.041$$

$$D_M = \mu \delta_y = \mu S_Y \left(\frac{T_y}{2\pi} \right) = 2.041 \times 8 \times \left(\frac{0.3}{2 \times 3.1416} \right)^2 = 0.037(m)$$

Example 2: T_y=0.6s、S_Y=6m/s^2 (Case 2, $S_Y < S_{y0}$=7.95m/s^2)

$$\mu = \mu_2 = 2.907$$

$$D_M = \mu \delta_y = \mu S_Y \left(\frac{T_y}{2\pi} \right)^2 = 2.907 \times 6 \times \left(\frac{0.6}{2 \times 3.1416} \right)^2 = 0.16(m)$$

Example 3: T_y=1.2s、S_Y=4m/s^2 (Case 3, $S_Y < S_A$=8.64m/s^2)

$$\mu = \mu_2 = 2.059$$

$$D_M = \mu \delta_y = \mu S_Y \left(\frac{T_y}{2\pi} \right)^2 = 2.059 \times 4 \times \left(\frac{1.2}{2 \times 3.1416} \right)^2 = 0.30(m)$$

The results of Example 1, 2 and 3 are shown in Fig. 8, 9 and 10, respectively.

Fig. 11 shows the normalized response estimation of Figs. 8, 9 and 10 with respect to the elastic response for $T=T_Y$, i.e., the relation between D_M/D_A and S_Y/S_A, where (D_A, S_A) is the elastic response and (D_M, S_Y) is the inelastic response. The normalized relation between elastic and inelastic response under the assumption of usual energy conservation rule is given by eq. (34), which is also shown in Fig. 11, It is noted that the energy conservation rule seems to correspond to the average of the Case 2 and the displacement conservation rule is almost similar with the Case 3.

$$\frac{D_M}{D_A} = \frac{\mu}{\sqrt{2\mu - 1}} \quad (32) \qquad \frac{S_Y}{S_A} = \frac{1}{\sqrt{2\mu - 1}} \quad (33) \qquad \frac{S_Y}{S_A} = \frac{D_M}{D_A} - \sqrt{\left(\frac{D_M}{D_A} \right)^2 - 1} \quad (34)$$

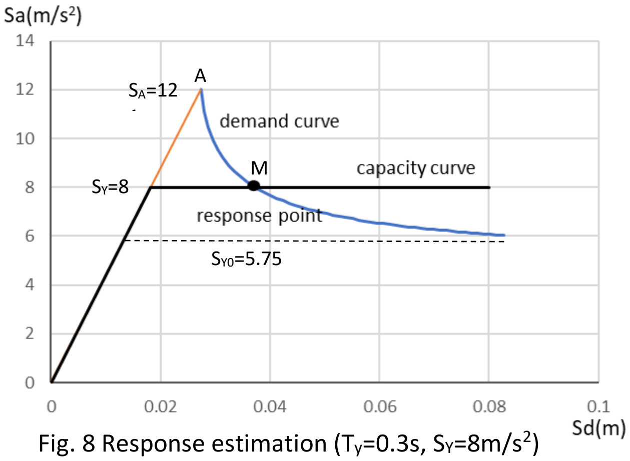


Fig. 8 Response estimation (T_y=0.3s, S_Y=8m/s^2)

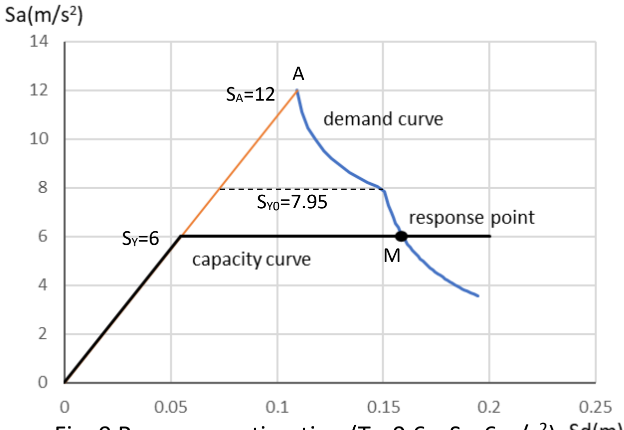


Fig. 9 Response estimation (T_y=0.6s, S_Y=6m/s^2)

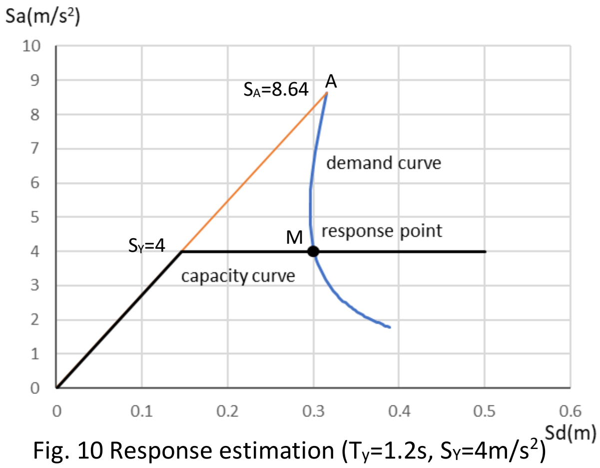


Fig. 10 Response estimation (T_y=1.2s, S_Y=4m/s^2)

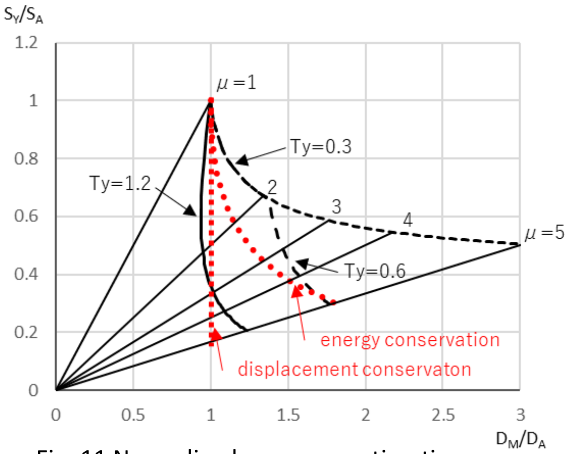


Fig. 11 Normalized response estimation

4. A case study of response estimation for an 11story SRC frame structure

Estimation of earthquake response by equivalent linearization method is made using an example plane frame shown in Fig.12. The frame is taken from an existing 11-story steel-reinforced concrete building, which is located at Sendai and was constructed in 1977 [6]. This building experienced both the 1978 Miyagiken-oki earthquake and the 2011 East Japan earthquake. The damage from the two earthquakes was rather minor and has been repaired to allow for further use.

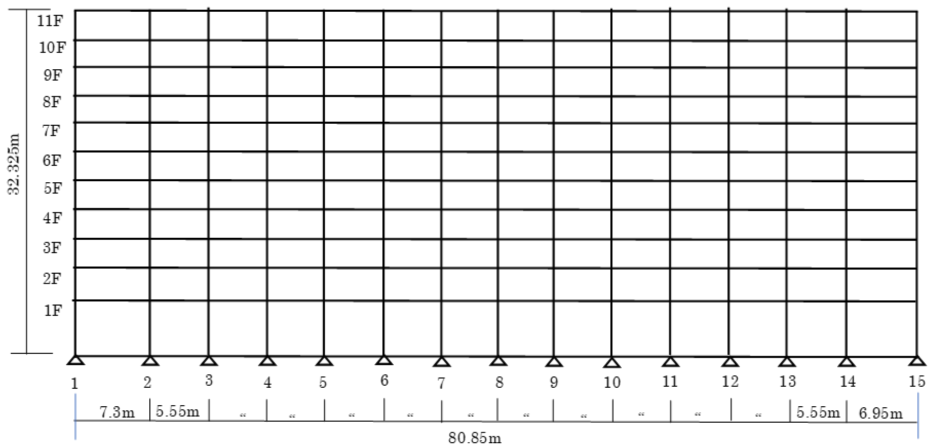


Fig. 12 Elevation of the model frame

Table. 1

i	h_i (m)	w_i (kN)
11	2.700	3238
10	2.650	3153
9	2.650	3190
8	2.675	3150
7	2.700	3247
6	2.700	3114
5	2.700	3298
4	2.725	3191
3	2.750	3403
2	3.025	3384
1	5.050	4476

The building has two longitudinal frames, one, a pure open frame and the other, an open frame with many side walls. The pure open frame was adopted for the analysis in this study. The story height for structural analysis and the floor weight are shown in Table. 1. The floor weight of the example is assumed to be 1/3 of the original one. The initial 1^{st} mode period of the frame in the elastic range is 0.67sec.

The static push-over analysis was made against the shear force distribution of Ai type currently used for seismic design in Japan [1], for which the computer program STERA-3D developed by one of the authors was used [7]. Fig. 13 shows the story displacement – story shear relation obtained by static analysis up to the top displacement angle of 1/25. From this result, the equivalent 1- DOF tri-linear model for the maximum response estimation is constructed as shown in Fig. 14 by the method given in the reference [6].

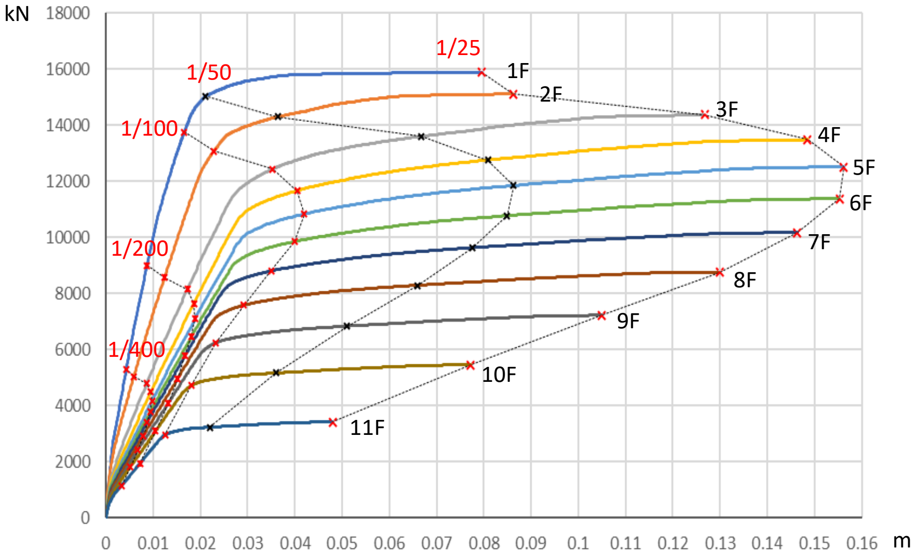


Fig. 13 Story displacement-story shear relation

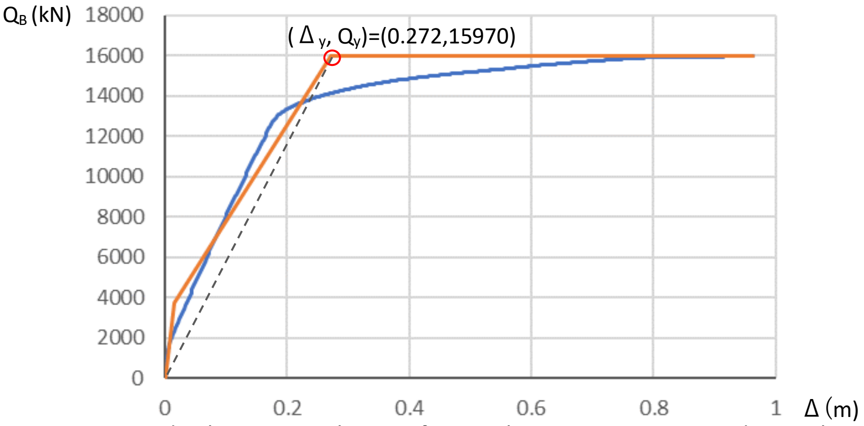


Fig. 14 Force-displacement relation of equivalent 1 DOF system and its tri-linear model model

The relation between the representative displacement Δ and the base shear Q_B was obtained using eq. (15) and eq. (16). Fig.14 shows the Δ and the base shear Q_B, together with the tri-linear model assuming that the stiffness ratio after yielding is 0, i.e., elasto-plastic [6]. The base shear coefficient at the yield point is 0.43. The effective mass M_u at yielding is calculated as 2724 kNs^2/m by eq. (17) using the displacement shape at the yield point. The yield point period is 1.35sec. The ratio of the initial period and the yield point period is 0.49, which means the ratio of the initial stiffness and the yield point stiffness is about 4 to 1.

Estimation of the maximum response by the equivalent linearization method was done against the artificial wave BCJ-L2 [8]. The maximum acceleration of the wave is 3.56m/s^2 and the duration is 120sec. Response spectra are shown in Fig.16 The acceleration response in the short period range for 5% damping is about 10m/s^2, which corresponds to the design force level in the current Japanese codes enforced since 1981.

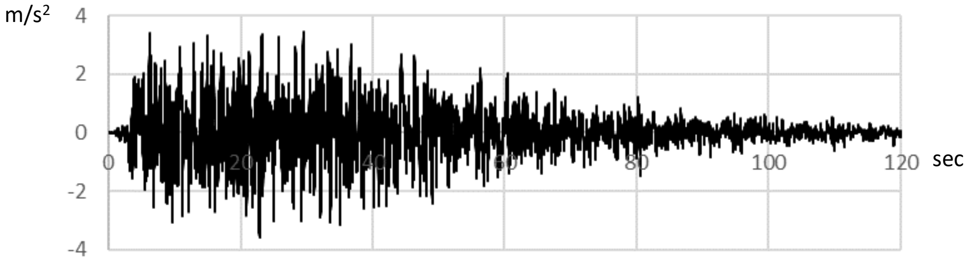


Fig. 15 Acceleration wave of BCJ-L2

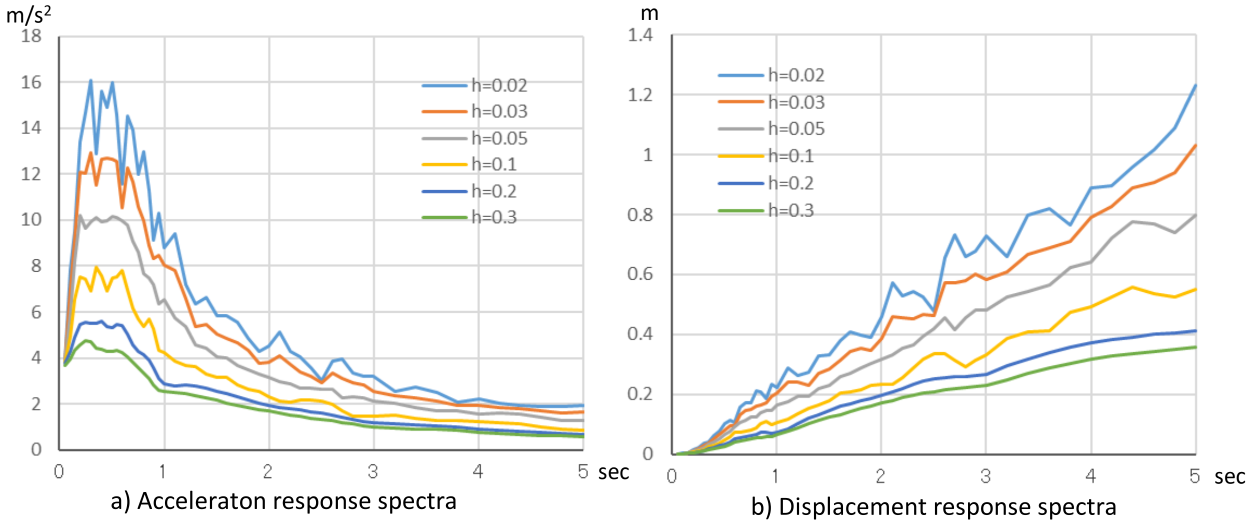


Fig. 16 Response spectra of BCJ-L2

Estimation of the maximum response of the equivalent 1-DOF system for the BCJ wave multiplied by 2.0 was done using the equivalent linearization method. The acceleration-displacement response spectrum (Sa-Sd spectrum) for 2 times the BCJ-L2 wave is shown in Fig. 17.

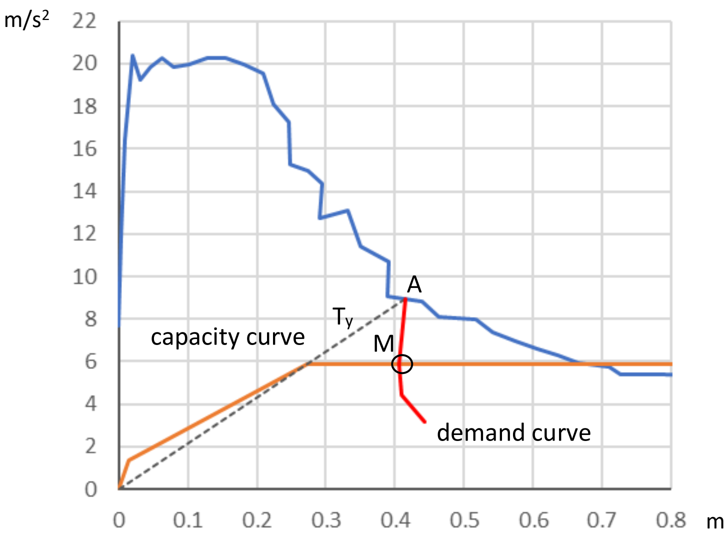


Fig. 17 Response etimation by capacity curve and demand curve

The force-displacement (Q_B-Δ) relation of the equivalent 1-DOF system, namely the capacity curve, is also shown in Fig. 17 by expressing Q_B/M_u as S_a and Δ as S_d. The yield acceleration $S_y=Q_y/M_u$ is 5.76m/s^2 and the yield displacement Δ_y is 0.272m.

The period T_y at the yield point is 1.35sec, and the corresponding 5% damping elastic response point is expressed by the point A in Fig.17. Using the eqs. (22), (23), (24) and changing the ductility factor μ successively, the demand curve is obtained as shown in Fig. 17. The crossing point of the capacity curve and the demand curve corresponds to the estimated maximum response point M. In this calculation, the estimated maximum displacement Δ_{max} of the equivalent 1-DOF system is 0.406m and the response ductility factor is 1.49. The maximum displacement angle Δ_{max}/H_e is 0.0172 (1/58).

The estimated response obtained by the equivalent linearization method described above is compared with the result from the inelastic frame time-domain response analysis using the computer program, STERA-3D [7]. In the frame dynamic analysis, the initial damping matrix proportional to the stiffness matrix is considered with the 1st mode damping factor of 5%.

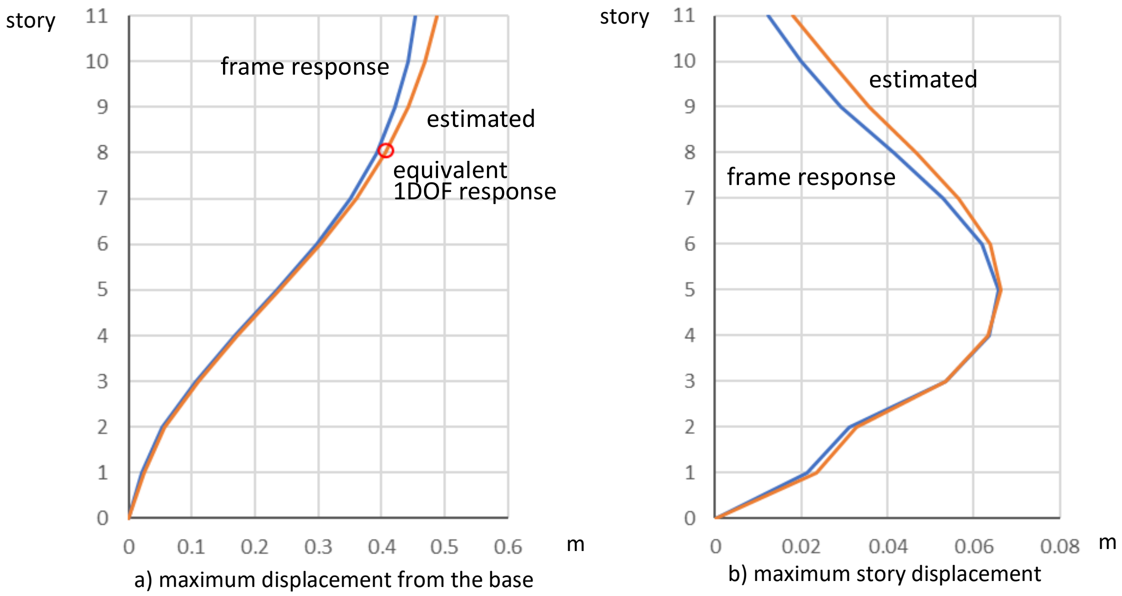


Fig. 18 Comparison of estimated response and frame response

The estimated 1-DOF response is combined with the displacement shape at the yield point obtained from the static push-over analysis. It is assumed that the estimated 1-DOF response corresponds to the response of the 8th story, considering that the equivalent height ratio H_e/H of 0.72 is close to 8/11=0.73. The maximum displacement from the base for frame response in Fig.18 a) is the sum of the maximum story displacement from the base without considering the time-lag. The maximum story displacement for estimated response in Fig.18 b) is determined by the difference of the displacement shape using static push-over analysis. The displacement response behavior from the frame dynamic analysis is compared with the estimated response by equivalent linearization method in Fig. 18, which shows good agreement in this case.

5. Performance-based seismic design procedure

The process for seismic design of buildings against severe earthquakes should contain the dual procedures of the initial structural design and final evaluation of inelastic response of the designed structure, including iterative processes.

In the initial process of structural design, the design seismic force is determined from the design response spectra and several assumptions about the dynamic properties of buildings, usually the period, inferred from the height, and the structural type of the building. Also, the level of tolerable inelastic deformation, commonly expressed by the ductility factor, should be assumed for large earthquakes, which depends on the structural type and characteristics of the building structure. Reduction of elastic response force is considered depending on the tolerable ductility. It is noted that in the most commonly used Japanese seismic design method (Horizontal load-carrying capacity calculation), the reduction of design force is specified by the structural characteristic factor Ds depending on the type of frames and members, which takes, for example, a value between 0.3 to 0.55 for RC structures. The usage of eqs. (22) to (24) would be one of the alternative methods to determine the reduction factor in the performance-based design, in which case the due judgement of the structural designer is required as to the allowable inelastic deformation.

In the final process of performance-based seismic design, evaluation of inelastic response behavior of the designed structure is considered necessary. Estimation of inelastic response can be made in several ways. The evaluation procedure of the maximum response by equivalent linearization method is adopted in the RLCC method for building structures under 60m, which conforms to the performance-based seismic design, though this method takes the form of comparing the required lateral strength corresponding to the tolerable inelastic displacement with the actual lateral strength. An alternative method would be to compare directly the tolerable inelastic displacement with the estimated displacement response. Dynamic time-domain response analysis against artificial ground motions generated based on the design response spectra would be another applicable method.

6. Conclusions

The seismic design of buildings for the next generation should use the performance-based design. Evaluation of inelastic earthquake response of the designed structure in the final process of seismic design is necessary to check the performance of the building when subjected to large-scale earthquakes. The equivalent linearization method will be an effective tool for estimating the inelastic earthquake response behavior of building structures in a simple manner.

7. Acknowledgements

Authors would like to express their deep thanks to the Committee on the Evaluation of Earthquake Resistant Capacity of Existing Middle-Rise Reinforced Concrete Buildings (Head: Prof. Eiichi Inai, Yamaguchi University) in the Architectural Institute of Japan for allowing us the use of their research data in this study.

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