Damage Level Evaluation of Existing R/C Buildings under an Earthquake

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Kazushi SHIMAZAKI, born in 1955, received master's and doctorate degrees from the Tokyo Institute of Technology. Member of ACI, AIJ, and JCI. Research interests include earthquake resistant resign of buildings, especially the design method oriented to the displacement response, and structural behavior of reinforced concrete members.

Summary

This paper proposes a method to estimate the maximum displacement response of existing R/C buildings in an earthquake using the response spectrum. After a parametric study on the non-linear displacement response of SDOF models with tri-linear hysteresis rules, a method to estimate the maximum displacement response is proposed using dimensionless parameters.

A tri-linear SDOF model for an existing building can be evaluated as follows. The elastic stiffness for existing buildings can be roughly estimated from the dimensions of columns and walls. The first break point and the second break point of a tri-linear SDOF system model for a reinforced concrete building can be estimated using the cumulative strength index and ductility index.

Finally, a simple method to estimate the maximum displacement response for evaluating the damage level of existing R/C buildings is proposed, and it showed satisfactory results.

Keywords: Reinforced Concrete Building, Earthquake Resistance, Damage Level, Displacement Response, Response Spectrum

1. Introduction

The structural performance must be clarified in earthquake-resistant design, when the emphasis is on the performance. It has been shown that displacement response is a simple index of performance. Many works to estimate the displacement response have been carried out, and were summarized in references [1,2]. The authors [3,4] have examined design methods based on response displacement, and investigated the condition of equal displacement response in elasto-plasticity displacement response of the one-mass system using the bi-linear hysteresis model.

In those studies, the elastic response spectrum was smoothed for each ground motion used in the calculation. The structures were idealized as SDOF systems having bi-linear hysteresis. Many earthquake response analyses were conducted for systems using dimensionless parameters *TR* and *SR*, where *TR* is the ratio of the initial period in the system to the boundary period T_1 between the constant acceleration spectrum region and the constant velocity spectrum region, and *SR* is the ratio of the shear strength of the system to the elastic resistant response.

The calculated displacement response was normalized as *DR* divided by the smoothed elastic displacement response spectral value with the same initial period and damping factor. The results are plotted in a figure as the relation of *SR* and *TR*, and can be summarized as follows.

1) $SR+TR>1$: The displacement response does not exceed the elastic displacement response regardless of the type of hysteresis model and strength.

2) $SR+TR<1$: *DR* is proportional to *SR*, and increases hyperbolically. The magnitude is affected by the characteristics of the hysteresis model (difference of energy absorbing ability).

This relation was established by experimental research [5].

Tri-linear type hysteresis models are usually used for reinforced concrete structure members with break points of concrete cracking and main rebars yielding. The stiffness of a reinforced concrete structure degrades with 1) cracking of members, 2) yielding of members, and 3) the mechanism.

For the SDOF model of a reinforced concrete structure, the model of tri-linear hysteresis with two break points of the first stiffness degrading point and close to the mechanism point is used.

To extend the results obtained using bi-linear hysteresis models to the system with tri-linear hysteresis models, nonlinear earthquake response calculations of 12,600 systems were carried out for 20 ground motions [6]. The Takeda tri-linear type hysteresis model [7] shown in Figure 1 was used. Parameters were energy absorption ability, break points of models, yielding stiffness degrading, and type of ground motion as shown in Table 1.

TABLE 1 Parameters used in analyses

Ratio of break point in force, *FR*	$\frac{\text{Cracking force, } F_c}{\text{Yielding force, } F_y}$	0.2–0.8 (0.1)	7 types
Ratio of stiffness, *KR*	$\frac{\text{Yielding stiffness, } K_y}{\text{Elastic stiffness, } K_o}$	0.2–0.6 (0.1)	5 types
Ratio of strength, *SR*	$\frac{\text{Yielding strength}}{\text{5\% damping elastic resistant response}}$	0.1–1 (0.1)	10 types
Ratio of period, *TR*	$\frac{\text{Initial period, } T_o}{\text{Boundary period shown in fig. 1, } T_1}$	1/3, 2/3, 1, 2, 3, and 5 sec	6 types
Unloading parameter shown in Fig. 3, β		0–1 (0.2)	6 types
Ground motions		—	20 types
Ratio of displacement, *DR*	$\frac{\text{Maximum displacement response}}{\text{Smoothed elastic displacement response spectral value with 5\% damping factor}}$	—	—

(*DR* is not a parameter of the analysis, but will be used to evaluate the results.)

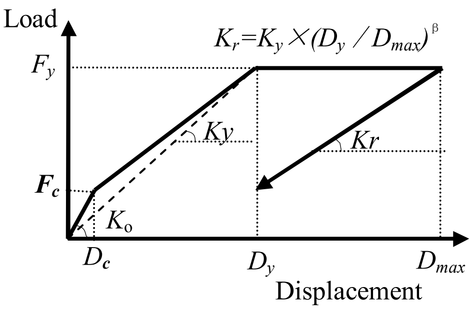


Figure 1: Trilinear Model

The results were classified by using the dimensionless parameters based on properties of the system and the frequency characteristics of the ground motion, and the relation of the dimensionless parameters, strength ratio *SR* and displacement ratio *DR* was plotted. This relation was fitted by a hyperbolic curve for each plotted figure, and hence the following equation to estimate displacement response was obtained using the parameters shown in Table 1.

$\sqrt{FR+KR} > 1$

$$DR = \frac{1}{\dfrac{1}{SR^{3(0.9-0.2\beta)TR}}}$$

$\sqrt{FR+KR} < 1$

$$TR \le 1 \qquad DR = \frac{3.5 - 2TR - (2.5 - 2TR)\sqrt{FR+KR}}{SR^{1/\{3(0.9-0.2\beta)TR\}}}$$

$$TR > 1 \qquad DR = \frac{1.5 - 0.5\sqrt{FR+KR}}{SR^{1/\{3(0.9-0.2\beta)TR\}}} \tag{1}$$

This estimation showed satisfactory results and was on the safe side[6].

2. SDOF Trilinear Model

2.1 Elastic Story Stiffness

The elastic stiffness for existing buildings can be roughly estimated by the dimensions of columns and walls. According to the D-value method [8], horizontal stiffness (Q/δ) of a column is given by following equations:

$$\frac{Q}{\delta} = \frac{12EI}{h^3} \times a \tag{2}$$

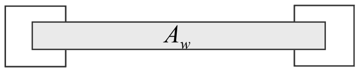
where, E: Young's modulus, I: geometrical moment of inertia, h: member height, and the value of a takes the following values depending on the condition of the column base:

general floor $\bar{k} = \dfrac{k_1 + k_2 + k_3 + k_4}{2k_c}$ $a = \dfrac{\bar{k}}{2 + \bar{k}}$

fixed base $\bar{k} = \dfrac{k_1 + k_2}{2k_c}$ $a = \dfrac{0.5 + \bar{k}}{2 + \bar{k}}$

where, k_c: column stiffness ratio (= column relative stiffness/standard relative stiffness)

k_1, k_2, k_3, k_4: stiffness ratio of each beam

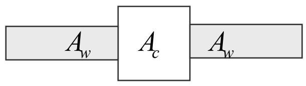


For a standard existing RC building, the column cross section is assumed to be 600 mm×600 mm, and Young's modulus 2.3×10^4N/mm^2 for a concrete compressive strength of 20 N/mm^2. The story height is set at 3 m. In the common story, taking the stiffness ratio of beams and columns as equal values, coefficient a becomes 0.5. At the base floor, the coefficient is set at 1.25 times that of the common story. Then, the elastic horizontal stiffness of a column becomes:

$$\frac{Q}{\delta} = \frac{12EI}{h^3} \times a = \frac{12\times23000\times D^2/12\times bD}{3000^3} = \frac{12\times23000\times30000\times A_C}{3000^3} \approx 0.15A_C \tag{3}$$

To derive the elastic stiffness of a column with spandrel walls, using an extra coefficient as a function of spandrel wall cross section A_w, column cross section A_C, and correction factor α, we obtain:

$$\frac{Q}{\delta} = 0.15\,(1+\alpha\frac{A_W}{A_C})\,A_C \tag{4}$$



where, α: correction factor (= 4–6).

The elastic stiffness of a column with retaining wall is calculated by equation (3) using the clear span h_0 instead of h.

For earthquake resisting walls, the formula for calculating the elastic stiffness uses a simplification of the D-value method. The D value of walls is,

$$D_W = \frac{Q_W}{\delta_W} \times \frac{h^2}{12EK_0} = n\,D_C\,\frac{A_W}{A_C} \tag{5}$$

where, D_c is the D value of columns.

$$D_C = \frac{Q_C}{\delta_C} \times \frac{h^2}{12EK_0} \approx 0.15A_C \times \frac{h^2}{12EK_0} \tag{6}$$

From equations (5) and (6), and assuming n=3.3 as common value, the following equation is obtained to estimate the elastic stiffness of walls using wall cross section A_W:

$$\frac{Q_W}{\delta_W} = 3.3\times0.15A_C\,\frac{A_W}{A_C} \approx 0.5A_W \tag{7}$$

The story initial stiffness K_0 (N/mm) becomes the substituted value of equations (3), (4), and (7) for each member.

2.2 Trilinear Hysteresis Model of Story Shear and Story Drift

The first break point and the second break point of a tri-linear model in the story deflection and story shear force relation for a reinforced concrete building can be estimated using cumulative strength index C and ductility index F defined in "Standard for seismic capacity assessment of existing reinforced concrete buildings"[9].

C is an index of story lateral strength expressed in terms of story shear coefficient, and F is an index of story ductility, calculated from the ultimate deformation capacity normalized by the story drift angle R=1/250 when a typical-sized column is assumed to fail in shear. F is assumed to be 1.27–3.2 for bending failure columns, 1.0 (R=1/250) for shear failure columns and walls, and 0.8 (R=1/500) for extremely brittle short columns. Total story shear strength can be obtained from the sum of member strength multiplied by strength contribution coefficient at each deflection[9].

It is assumed that at the deformation of R=1/500, extremely brittle short columns demonstrate 100% of ultimate strength, and that shear failure columns and walls demonstrate 70%, and bending failure columns 50% as shown in Figure 2. The story shear strength for R=1/500 can then be calculated by:

$$Q_{1/500} = \tau_{SC} A_{SC} + 0.7 \times (\tau_{W1} A_{W1} + \tau_{W2} A_{W2} + \tau_{W3} A_{W3}) + 0.5 \times \tau_C A_C \tag{8}$$

where, A_{SC} : section of extremely brittle short columns, A_C : section of bending failure columns A_{W1} : section of wall with boundary columns, A_{W2} : section of wall with single boundary column, A_{W3} : section of wall without boundary column, and ultimate unit shear strengths are:

$\tau_{W1} = 3 \quad \tau_{W2} = 2 \quad \tau_{W3} = 1 \quad \tau_C = 1 \quad \tau_{SC} = 1.5$ [N/mm^2]

At the deformation of R=1/250, the shear failure columns and walls demonstrate 100% of ultimate strength, and bending failure columns demonstrate 70% as shown in Figure 2. Extremely brittle short columns are assumed to keep their strength. The story shear strength for R=1/250 can then be calculated by:

$$Q_{1/250} = \tau_{SC} A_{SC} + (\tau_{W1} A_{W1} + \tau_{W2} A_{W2} + \tau_{W3} A_{W3}) + 0.7 \times \tau_C A_C \tag{9}$$

In the deformation of R=1/100, the bending failure columns demonstrate 100% of ultimate strength, and the others lose their strength. The story shear strength for R=1/100 can then be calculated by:

$$Q_{1/100} = \tau_C A_C \tag{10}$$

The trilinear hysteresis model is set using the story initial stiffness K_0 and the story shear strength for R=1/250 and 1/500. The story shear strength at R=1/250 is made to be the yield point shear capacity, and the strength is assumed to keep in large deformation. The first break point is set at the cross point of the elastic stiffness line and the line which connects R=1/500 and R=1/250 shear capacity as shown in Figure 3. When the first break point strength exceeds 1/3 of the yield point shear capacity, it is set to 1/3 of the yield point shear capacity.

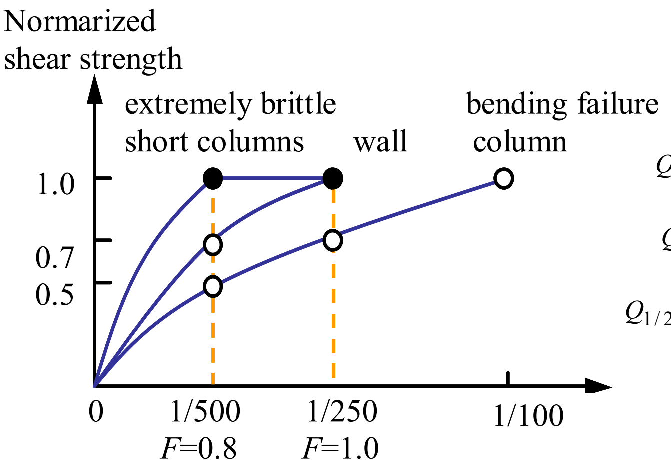


Figure 2: Rough Estimation of shear capacity for vertical members[9]

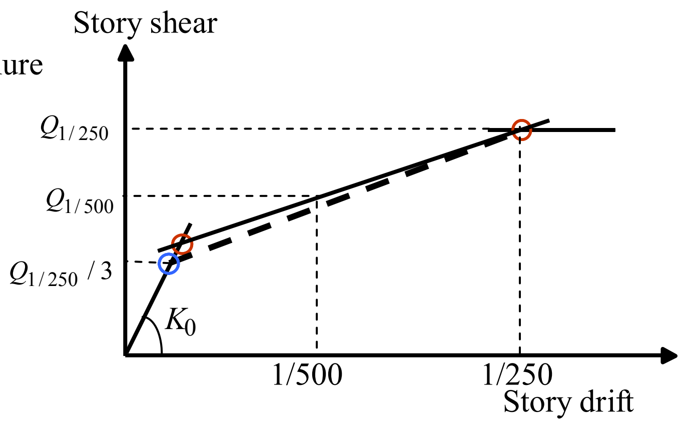


Figure 3: Story shear – story drift relation

2.3 Equivalent SDOF Model

In the substitution to the equivalent SDOF system, the relationship between RF deformation and 1F shear force is required. Assuming an inverse triangular deformation distribution, the acceleration distribution also becomes an inverse triangular shape. This defines the shear force distribution. By event-to-event follow-up procedures, the relationship between RF deformation and 1F shear force is obtained from the deformation–shear force relation of each floor. In converting the hysteresis model of this whole building into the trilinear type, either floor reaching the first break point first is made to be the first break point, and either floor reaching the yield point first is made to be the yield point.

A tri-linear SDOF model can be evaluated for an existing building using this relation, the effective mass, and the participation factor. Then, using equation (1) and the design response spectrum, the maximum displacement response can be estimated for the fixed base system.

3. Soil–Building Interaction

3.1 Interaction Spring

The soil–building interaction effect is estimated by the simple equivalent linear sway-and-rocking spring model proposed previously [10,11]. The sway spring constant *Ks* is set to be the rigidity of ground only considering the embedding effect. The rocking spring constant *Kr* consists of ground rigidity and pile stiffness. In the longitudinal direction of the building, there is assumed to be no rocking deformation.

Sway spring: $$K_s = {}_F K_s \left(1 + \frac{H_0}{r_s}\right) \tag{11}$$

where, ${}_F K_S = \frac{8}{2-\nu} \times \frac{\gamma}{g} \times V_{eq}{}^2 \times r_s$: sway spring of ground

H_0: embedding depth, r_s: equivalent radius of the rectangle foundation for sway, ν: Poisson's ratio, γ: unit soil weight, g: acceleration of gravity, V_{eq}: equivalent shear wave velocity

Rocking spring $$K_R = {}_F K_R \left(1 + \frac{2.5H_0}{r_R}\right) + {}_P K_R \tag{12}$$

where, ${}_F K_R = \frac{8}{3(1-\nu)} \times \frac{\gamma}{g} \times V_{eq}{}^2 \times r_R{}^3$: rocking spring of ground, ${}_P K_R = \frac{9000 N_B{}^{1/3}}{V_{eq}} \times {}_F K_R$: rocking spring of pile [10], r_R: equivalent radius of the rectangle foundation for rocking, N_B: building story number

The equivalent viscous damping of the ground spring can be calculated from the real part and imaginary part of the complex rigidity spring. Here, it is assumed to be the value based on the following formula simplified using the dimensionless frequency [11]:

$$ {}_s h_s = (21.7a_s - 0.1a_s{}^2 + 0.483a_s^3) \times 0.01 \tag{13}$$

$$ {}_s h_R = (-2.18a_R + 8.45a_R^2 + 0.307a_R^3) \times 0.01 \tag{14}$$

where, $a_S = \omega \times \frac{r_s}{V_{eq}}$: sway dimensionless frequency, $a_R = \omega \times \frac{r_R}{V_{eq}}$: rocking dimensionless frequency, ω : circular frequency of S-R system.

3.2 Estimation Method

To estimate the displacement response from *DR* using equation (1), each dimensionless parameter of *FR*, *KR*, *β*, *SR* and *TR* should be determined. The parameters of *SR* and *TR* are determined from the response spectrum and function of the initial period of the S-R system, and *SR* is also a function of damping. The following method is proposed in this study.

1) The elastic displacement response of the superstructure is calculated from the displacement

response spectra using the equivalent period and damping as the S-R system.

2) The nonlinear displacement response of the superstructure is estimated by equation (1) with the revised damping factor. Equation (1) is based on the response spectrum with 5% damping for the elastic response; in this case, however, the elastic response is estimated with the equivalent damping of the S-R system and its value is usually larger than 5%.

3) Total displacement is calculated as the sum of the superstructure displacement response and S-R components.

4. Verification

4.1 Model Buildings and Ground Motions

The ground motions used are three simulated earthquake ground motions at a soft (V_{eq}=100 m/s), moderate (V_{eq}=200 m/s), and hard (V_{eq}=300 m/s) site having the response spectrum shown in Figure 6 for a moderate earthquake.

The model building is the ridge direction of the 3- and 7-story reinforced concrete building shown in Figure 5. Each column has a spandrel wall and its size is defined as a ratio to the column width. The natural periods of the model structure obtained from the 3-D analysis and the proposed model are shown in Figure 6. The values agree closely, so the proposed model gives the correct initial stiffness. For the 3-story building on the soft site, the period becomes 1.5 times of the base fixed model.

The load-deflection relation of the 7-story building set by the method proposed in this paper and calculated from the 3-D static incremental analysis result is compared in Figure 7 for cases of β=0, 1, and 3. Both relations show good agreement.

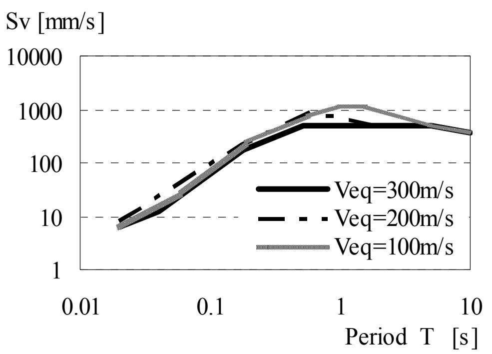


Figure 4: Sv of Ground motions

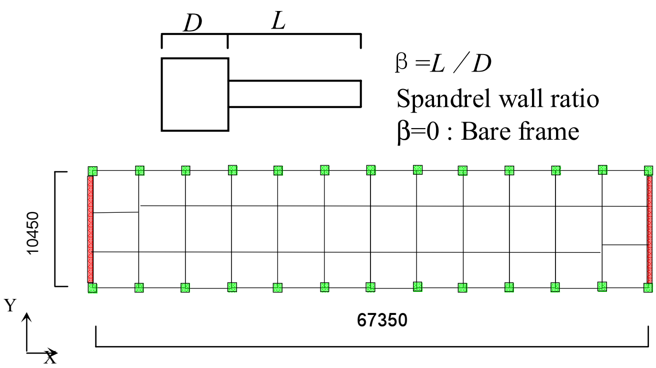


Figure 5: Model building

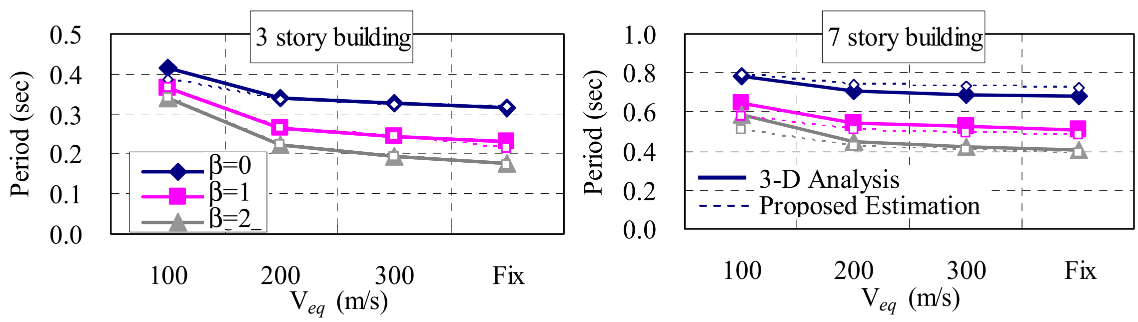


Figure 6: Natural period of the model building with S-R effects

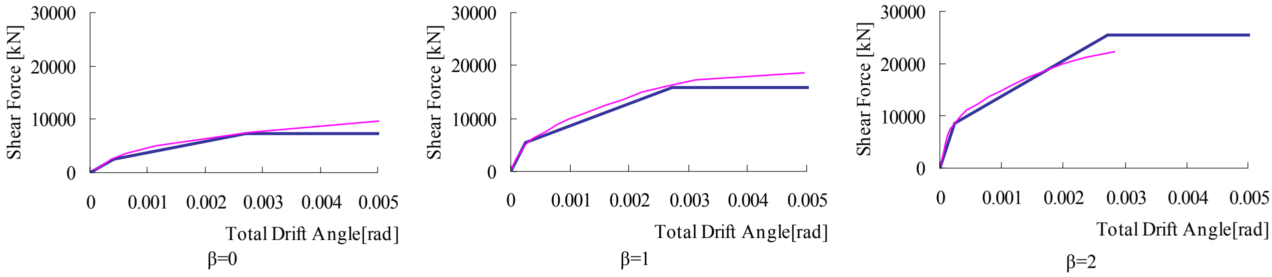


Figure 7: Comparison of load-deflection relations

4.2 Displacement Response

The estimated maximum nonlinear displacement responses are compared with the calculated values using the 3-D dynamic response analysis in Figure 8. The figure shows the results for 3- and 7-story buildings with the parameter of spandrel wall ratio of 0, 1, and 2. The horizontal axis shows the difference of site stiffness. At each point, different response spectrum shape and different ground motion shown in Figure 6 were used. The thick solid lines show calculated values and the thick dotted lines show estimated values.

For the 7-story building, the estimated values are always larger than the calculated values and on the safe side. For the 3-story building of the S-R system, the estimated values are lower than the calculated values at the soft site, especially for the model of β=0 which has low shear strength

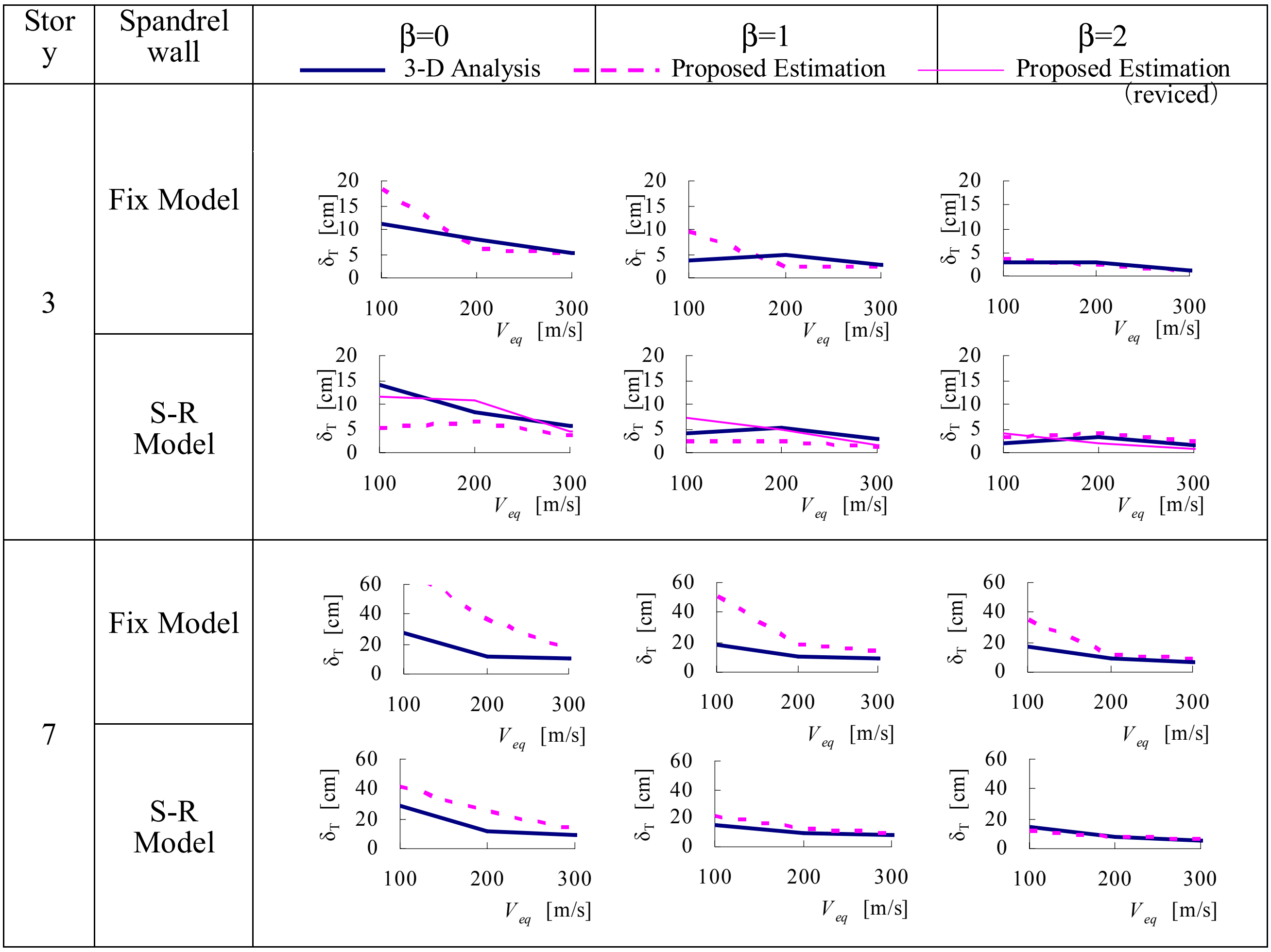


Figure 8: Maximum displacement response

capacity. The effective period of the S-R system at the soft site is 1.5 times of the base fixed system, and the equivalent damping factor becomes large, so the acceleration response reduces. Accordingly, the value of *SR* becomes large, and *DR* becomes small. For the S-R system, the second mode motion behaves like the base fixed system especially for the acceleration response. So, the response shear force does not reduce so much, and *SR* will not become a large value. In this situation, it would appear to be better to use the initial period and damping factor of the base fixed system for calculating *SR*. The thin solid line for the 3-story S-R model in Figure 8 shows the values estimated using *SR* defined according to such concept. The results show good agreement.

By using this estimated displacement response, the damage level of the building can be evaluated.

5. Conclusion

This paper presented a simple method to estimate the maximum displacement response for evaluating the damage level of existing reinforced concrete buildings. The main conclusions obtained were as follows.

1. The estimated maximum displacement response obtained by the proposed method showed good agreement with the value obtained from 3-D dynamic response analysis and was on the safe side.
2. For the low shear capacity building at the soft site, the displacement response showed good agreement when using the initial period and damping factor of the base fixed system for calculating *SR*.

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