Performance-Based Seismic Design of Reinforced Concrete Building Structures Using Inelastic Displacements Criteria

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ABSTRACT

A performance-based seismic design method for reinforced concrete building structures being developed in Japan is outlined. Technical and scientific background of the performance-based design philosophy as well as recently developed seismic design guidelines are is presented, in which maximum displacement response to design earthquake motion is used as the limit-state design criteria. A method of estimating dynamic response displacement of the structures based on static nonlinear analysis is described. A theoretical estimation of nonlinear dynamic response considering the characteristics of energy input to the system is described in detail, which may be used as the standard method in the new performance-based code. A design philosophy not only satisfying the criteria but also evaluating seismic capacity of the structures is also introduced.

Key words : *performance-based design, earthquake-resistant design, design guidelines, limit state, reinforced concrete building, deformation capacity, inelastic displacement response, earthquake energy input*

1. Introduction

One of the important lessons learned from the damages to buildings caused by the Hyogo-ken-nanbu Earthquake 1995 was that life safety is not enough as the performance objective of seismic design, even after a very rarely severe earthquake. Instead, it was proved from the earthquake damages that whether most of the structures were economically repairable or not will be a fatal performance criteria for quick recovery of the social system. It was also pointed out that accountability on actual structural performance under a severe earthquake had been incomplete in the current building standards, building officials and structural designers. It should have been clearly explained to and understood by the clients that their buildings, even designed by satisfying the latest building standard, might suffer certain damages exceeding the economically repairable limit from the major earthquake.

Frameworks for performance-based seismic design have been drafted worldwide, in which performance objectives are to be clearly prescribed instead of traditional specifications. In the design objectives of the performance-based code, estimated damages shall clearly be related to the expected design motions with adequate reliability.

The Building Standard Law (BSL) of Japan[1] was revised in May 1998 at the first time after about 50 years of its establishment in 1950, into a performance-based style. However, the style, especially on the structural requirements, has not been so much changed in the BSL level. Building Standard Order, other lower level requirements and design guidelines are now under revision in accordance with the performance-

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based design philosophy.

On the other hand, Architectural Institute of Japan published a seismic design guidelines for reinforced concrete building structures, in 1990[2] and the English version in 1994[3] based on the ultimate strength concept and capacity design philosophy. To extend and revise it, the style of the guidelines was completely changed introducing inelastic displacement concept towards a future performance-based code and was published as a draft in 1997.[3] A new committee is active to publish a new guidelines in a perfectly performance-based style within a few years.

This paper outlines the basic ideas on the future performance-based seismic design guidelines for reinforced concrete building structures being developed in Architectural Institute of Japan. Technical and scientific backgrounds of the performance-based design philosophy are presented, in which the maximum displacement response to design earthquake spectra is estimated and used for the verification of the limit-state criteria. A method of estimating dynamic response of the structures based on static nonlinear pushover analysis is described. A philosophy not only satisfying the criteria but also evaluating seismic performance level of the structures is also introduced, which is to be included in the design guidelines. Recently developed design method for reinforced concrete members under moment, shear and axial loads in order to assure quantitative level of deformation capacity is also briefly introduced.

2. Limit states of reinforced concrete building structures

Serviceability and ultimate limit state are adopted as design criteria in the limit state design codes in Europe, which verifies both limit states basically through linear analysis. Actual nonlinear behavior beyond ultimate strength until ultimate deformability, which is expected during strong motion, is not considered explicitly as the criteria.

Most of the structures well designed in accordance with Japanese Code have much more deformation capacity than response displacement calculated under design motions with maximum velocity amplitude of 50 kine(cm/sec.), Which is used as maximum probable earthquake level with return period of several hundreds years, although they will excess serviceability displacement. In other words, Japanese code implicitly requires most of the buildings to remain within much less damage than ultimate deformability under the design motion.

Considering the required levels in the Japanese current building codes, the new performance based seismic design introduces the following three limit states as design criteria:

a) *Serviceability limit state*: functional on use with slight damage for minor-to-moderate earthquake,
b) *Damage control limit state*: economically repairable or restorable with moderate damage for severe earthquake,
c) *Ultimate limit state*: life safe with near collapse damage for ultimate earthquake.

It is difficult to define the second criteria, the new criteria, definitely from structural demand, because most of damaged structures could be restored in any way unless it is totally collapsed. Therefore, the criteria must be determined considering expected repair cost as part of the life-cycle cost of the designed structure. Then, the design level could be selected theoretically so that the expected life cycle cost including the repair cost is to be minimized. In case that the initial design level is made higher so that the excess probability of the damage control limit state is made lower, the expected repair cost is lower though the initial cost is higher. If the probability is made higher, then the repair cost is higher though the initial cost is lower. Therefore, the criteria shall ideally be selected in the viewpoint of minimum life-cycle cost.

However, practically the above process is not available because the reliable probability of earthquakes in the future can not be assessed rigorously with our present knowledge. The designer may select the damage control limit state criteria definitely in consideration of: (1) margin to the ultimate limit state, (2) control of damage to non-structural elements, (3) practical and economical restorability, (4) function on use of equipment and installations.

The limit states, especially for the second and third, shall be expressed using inelastic displacement instead of only load or member forces. The requirements adopted in the displacement-based design guidelines of Architectural Institute of Japan[4] is shown in Table 1 as an example of the limit state criteria. In order to adopt these as practical design criteria, non-linear analysis must be popular among designers, which is reliable in accuracy.

Objectives of the performance-based seismic design shall be to verify seismic performance criteria of the designed structure and evaluate

Table 1 Limit states criteria and corresponding earthquake levels adopted in AIJ seismic design guidelines based on inelastic displacement concept

Limit State	Definition	Displacement Criteria	Force Criteria	Design Motion	Commentary
Serviceability LS	Functional on use	Rs≤1/200	Moment≤Yielding Moment Shear≤Cracking Shear	Level 1(moderate) Amax=80~100Gal Vmax=10~15kine Ares=0.2~0.3G	Tr=55yrs. Pr50=60%
Damage Control LS	Economically reparable	Ro≤1/120 ~1/150 (Rs≤1/100)	Moment of Column≤Yielding Moment (Static Load Capacity≥Required)	Level 2 (strong) Amax=300~500Gal Vmax=40~50kine Ares=1G	Tr=225yrs. Pr50=20%
Ultimate LS	Life Safe	Ro≤1/80 (1/100~1/50) Rm≤Ru (1/75~1/40)	Moment of Non-hinge members≤Yielding Moment Shear, Bond, Axial Force≤ Ultimate Strength	Level 3(ultimate) Amax=600~800Gal Vmax=60~90kine Ares=1.5~2G	Tr=1642yrs. Pr50=3%

Rs: interstory displacement in a story

Ro: overall interstory displacement

Rm: member deformation

Ru: ultimate member deformation capacity, expressed in rotation angle(rad.)

Amax, Vmax, Ares: maximum acceleration, velocity and acceleration response of the design motions

Tr: expected return period in years

Pr50: excess probability in 50 years of design service life seismic performance level of the structure for the three design limit states respectively for the three standard levels of the earthquakes as above. These levels, or the corresponding return period, of the earthquakes, by which the structure reach the limit state, can be the performance indices. To interpret the return period into the excess probability, the performance of the designed structure with higher capacity can be expressed. If the performance is evaluated and expressed, the clients will be able to select higher level on the commercial base.

3. Estimation of maximum dynamic shear forces

The standard seismic design of a structure including reinforcing details, in which the formation of a planned yielding mechanism is assured, shall be conducted in advance of this performance based seismic design, which is popular as a "capacity design" procedure. In the mechanism assuring design, flexure yielding of the members planned not to yield (non-hinge members) and shear failure of all members shall be avoided. In order to ensure the mechanism, estimation of actual dynamic shear forces in the column and wall is important. The standard design method is described in the present guide- lines[2,3,4] based on general characteristics dynamic responses.

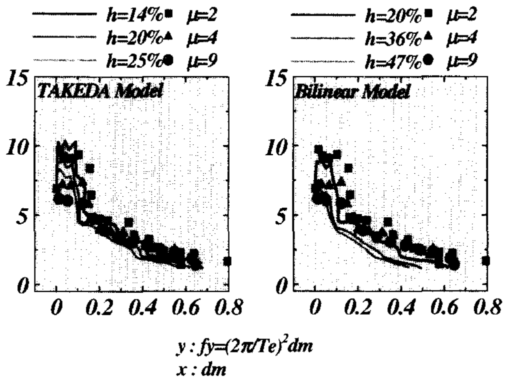
The estimated shear capacity from the design equation has redundancy to the static shear. However, the dynamic wall shear responses could be higher due to the higher mode effects. Dynamic magnification of the responses of maximum shear forces into the wall can roughly be estimated theoretically as the modal sum of static maximum and higher mode responses.[5,6] Because the decomposed higher modes generally correspond to the magnification estimated as elastic modes, the maximum responses are formulated using earthquake spectra and the inelastic static shear.

4. Estimation of non-linear displacement response

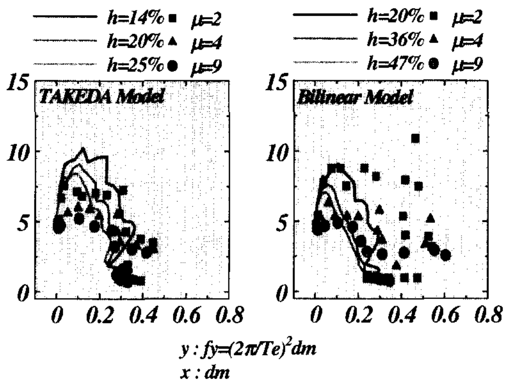
4.1 Equivalent linealization of nonlinear system

In the displacement-based design, which defines the limit states as the design criteria using the inelastic displacement, the maximum displacement responses of the inelastic system should be correlated theoretically in general to the characteristics of the input earthquake motion. If the design earthquake is given as spectra for linear system, the estimation of nonlinear response from the spectra is nothing but a kind of equivalent linearization.

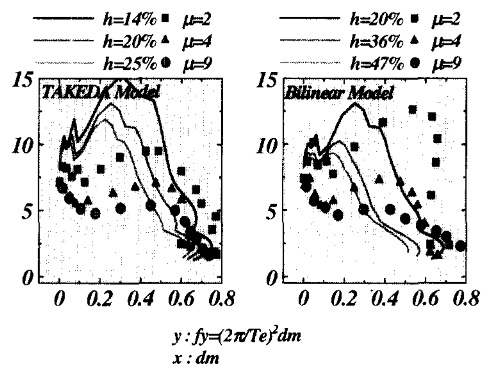
Based on pushover analysis and response spectra, a simple linearization using Sa-Sd spectra, namely acceleration-displacement spectra, often called as capacity design spectra, became popular in Japan and US. The accuracy of the capacity spectra estimation method is verified for the responses of the nonlinear systems with hysteresis models of Takeda-model and Bilinear model as shown in Fig. 1. A fair correlation I observed only in the case of artificial design motion used in Building Center of Japan amplified to 75 kine as shown in Fig. 1(a), in which stationary response is dominant. In the other cases under recorded motions, the estimation based on capacity spectra is much worse, as shown in Fig. 1(b) for Tohoku University



(a) Artificial design motion by Building Center of Japan (Vmax=75 kine)



(b) 1978 Miyagiken Oki, Tohoku University (NS, x2.0)



(c) 1995 Hyogoken-Nanbu Earthquake, Fukiai Staion (N30W, x1.0)

Fig. 1 Estimation of nonlinear response from capacity spectra

record (NS) amplified by two times during 1978 Miyagi-ken-oki Earthquake and Fig. 1(c) for Fukiai Station record (N30W) during 1995 Hyogoken-Nanbu Earthquake.

4.2 Estimation of nonlinear displacement based on instantaneous energy concept

The maximum displacement response of a single-degree-of-freedom system can theoretically be estimated under strong earthquake motion based on the concept of instantaneous energy.[7] In the estimation, the instantaneous energy input was defined as the work done to the system averaged during one-fourth of the equivalent period of vibration :*Te/4*. The instantaneous energy input, which was calculated for the inelastic systems with a hysteresis rule of reinforced concrete structure (Takeda model), was verified to correspond to those of the equivalent linear systems with the period corresponding to the secant stiffness to the maximum displacement. Therefore, the inelastic displacement response could be estimated as an elongation of equivalent period of vibration at the point where the instantaneous energy input spectra of the equivalent linear system and the hysteretic energy absorption capacity of the system are equal. Note that the equivalent linear spectra shall be made by modifying the equivalent viscous damping coefficient in accordance with the equivalent period for which the hysteretic damping is to be determined from the characteristics of the system.

The equation of motion for a single-degree-of-freedom system is integrated with respect time from 0 to t, by multiplying incremental time *dt*, to obtain energy based Eqns. (2) and (3).

$$\ddot{x} + 2hw\dot{x} + w^2 x = -\ddot{x}_0 \tag{1}$$

$$\int_0^t \ddot{x}\dot{x}d\tau + 2hw\int_0^t \dot{x}^2 d\tau + w^2\int_0^t x\dot{x}_0 d\tau = -\int_0^t \dot{x}\ddot{x}_0 d\tau \tag{2}$$

$$E_K(t) + E_H(t) + E_S(t) = E_T(t) \tag{3}$$

The instantaneous energy input is defined here as the work done to the system averaged during the finite time length of $\triangle t$ as $\triangle E_T$ of Eqn. (4) or $\triangle V_E$ of Eqn. (5) in terms of velocity.

$$\triangle E_T = E_t(t) - E_T(t-\triangle t) = -\int_{t-\triangle t}^{t} \dot{x}(\tau)\ddot{x}_0(\tau)d\tau \tag{4}$$

$$\triangle V_E = \sqrt{2 \times \triangle E_T / m} \tag{5}$$

$$\triangle V_{EA} = \sqrt{2 \times (\triangle E_S + \triangle E_H)/m} \tag{6}$$

$$\triangle V_{Eo} = \frac{\mu}{2} P_y d_y (\pi \cdot h_{eq} + 1) \tag{7}$$

The instantaneous energy input was calculated by numerical time-history analysis for nonlinear single-degree-of-freedom systems with Takeda hysteresis model representing reinforced concrete structure. The finite time length Δt was defined as one fourth of an equivalent fundamental period *Te*, which was supposed to represent the maximum cyclic amplitude of vibration. In this definition, the kinetic energy must be removed for the calculation of the input energy as in Eqn. (6). The hysteretic energy absorption capacity of the system and the equivalent period of vibration is defined as the secant stiffness to the maximum displacement as shown in Fig. 2. The maximum displacement of the nonlinear system can be estimated from the instantaneous energy input spectra of the equivalent linear system and the energy absorption capacity as an elongation of the period. The hysteretic energy shaded in Fig. 2 can be formulated for Takeda model as Eqn. (7) with ductility factor μ equivalent hysteretic damping coefficient h_{eq} in stationary cycle. In various cases, an example of which is shown in Fig. 3, the estimation based on the balance of input energy and energy dissipation capacity as the crossing point of two lines generally gives a fair estimation of calculated nonlinear responses.[7]

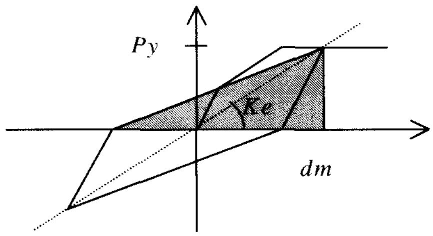


Fig. 2 Hysteretic energy dissipation capacity during Te/4

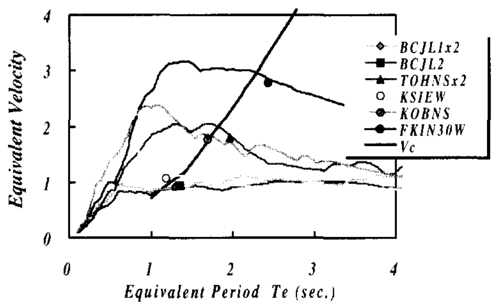


Fig. 3 Estimation of nonlinear response based on instantaneous energy

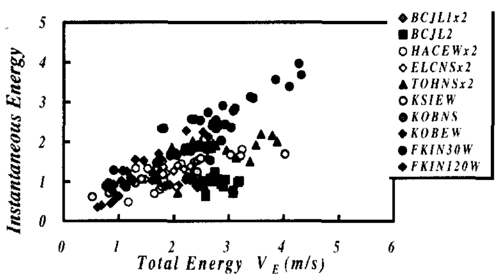


Fig. 4 Relation between total energy vs. instantaneous energy to nonlinear systems

As is expected from above formula, the instantaneous energy spectra expressed in terms of velocity almost correspond to the pseudo-velocity response spectra, although strictly they differ depending on the damping ratio. Approximately pseudo-velocity response spectra can be used instead of energy spectra. The ratios of the instantaneous energy to the total input energy to the structure are apparently different in the cases of responses to far-field earthquakes and near-field earthquakes, although general characteristics have been found using the duration of the earthquake. Theoretical reduction factors of the instantaneous energy spectra due to the damping ratio are also formulated[7] using the equivalent duration of earthquake as:

$$D_h(h,t1,T) = \frac{S_v(h)}{S_v(h=0)} = \sqrt{\frac{1-e^{-4\pi h(t1/T)}}{4\pi h(t1/T)}} \qquad (8)$$

where, T: fundamental period of the system, h: damping coefficient, $S_v(h)$: pseudo-velocity response spectra with damping coefficient of h, $t1$:one-fouth of the duration of the earthquake $t0$, which is defined as the time from 10% to 90% of total frequency ensemble work, i.e., time-integral of square of acceleration. Therefore, the displacement response of an inelastic system can be estimated consistently and theoretically, if the total design energy input to the structure and the duration of earthquake are given, which may essentially be related to the following two representative parameters: the magnitude of and the distance to the source of an earthquake.

4.3 Estimation of displacement response considering duration of earthquake

Above method gives fair estimates generally for the longer period range. However, in the short period range, the error is not negligibly small. A sophisticated method is recently proposed based on the characteristics of the energy input to the system,[8] in which the relationships among the total energy, the instantaneous energy during *Te/4*, *Te/2* and *Te*, velocity spectra and pseudovelocity spectra are clearly related. Although the ratio of the instantaneous energy to the total energy has variety as shown in Fig. 5 the instantaneous energy input to the system could clearly be related to the total energy if it is divided by the duration of earthquake $t0$ as shown in Fig. 5. From this relation, the instantaneous energy could be approxi- mated as two times larger as the time- averaged energy. in terms of velocity, in other words, by four times in terms of energy. Based on the general characteristics time-history of energy input, shown as in Fig. 6 for recorded motions, can be idealizes as shown in Fig. 7. Then, the following approximate equation can be derived for the estimation of equivalent period considering smaller ductility in negative direction hafe-cycle before.[8]

$$T_{eq} = \sqrt{\mu^2 \frac{T^4}{t_0^{\,2}} + \mu T^2} - \mu \frac{T^2}{t_0} \qquad (9)$$

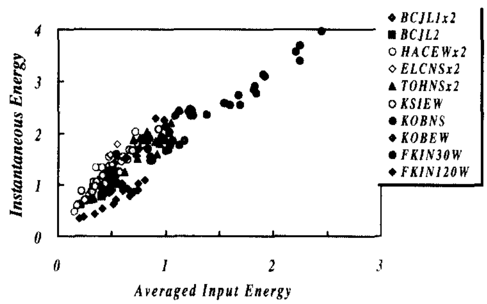


Fig. 5 Relation between time-averaged energy vs. instantaneous energy to nonlinear systems

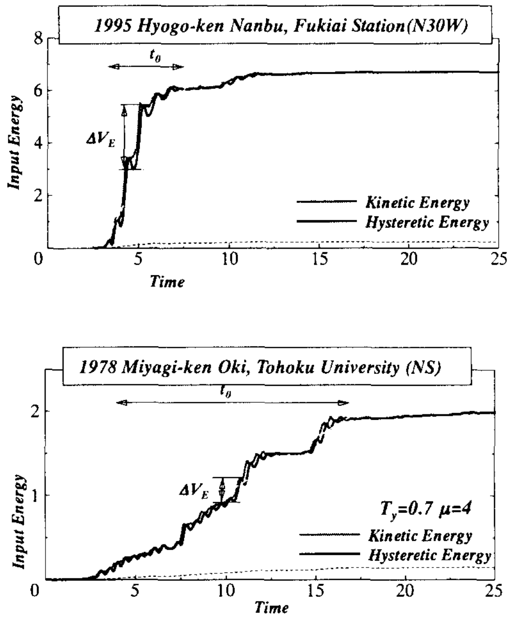


Fig. 6 Time-history of energy input of recorded earthquake

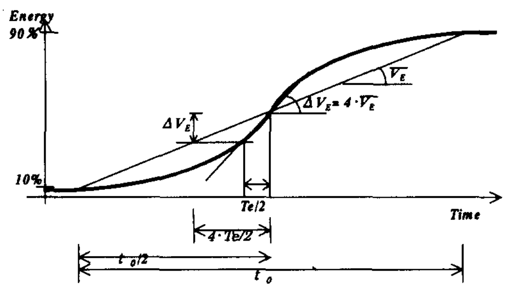


Fig. 7 Idealized time history of energy input from duration of an earthquake

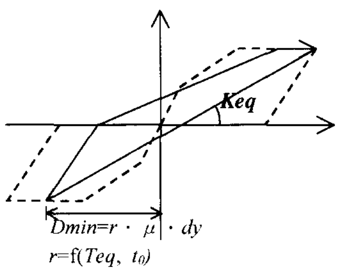


Fig. 8 Equivalent linear system considering earthquake duration and system ductility

The method can consider the case in which maximum displacement occurs in one direction after small ductility of former opposite direction. Relatively better correlation with calculated nonlinear responses can be observed in Fig. 9(b) from above formula than in the case by the simple estimation in Fig. 9(a) based on the equivalent system shown in Fig. 2.

4.4 Estimation of MDF response based on SDF response

Maximum response displacement of the structure as a multi-degree-of-freedom (MDF) system can be estimated with reduced equivalent single-degree-of-freedom (SDF) system as follows.

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+\{f\}=-[M]\{e\}\ddot{x}_0 \tag{10}$$

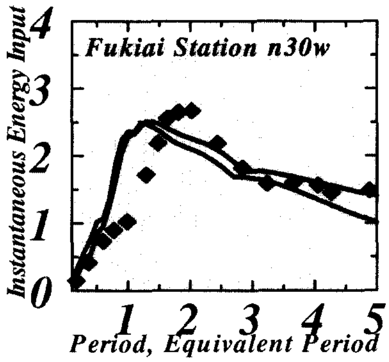
where, $[M]$: mass matrix, $\{\ddot{x}\},\{\dot{x}\}$: relative acceleration and velocity vectors, $[C]$: damping matrix, $\{f\}$: restoring force vector, $\{e\}$: unit matrix. The above equation of motion for a multi-degree-of-freedom system can be reduced to the following equation of motion for an equivalent SDF system by assuming a dominant basic mode of $\{x\}=\{u\}\beta x_e$ and $\{f\}=[m]\{u\}\beta f_e$,

$$\ddot{x}_e + c\dot{x}_e + f_e = -\ddot{x}_0$$

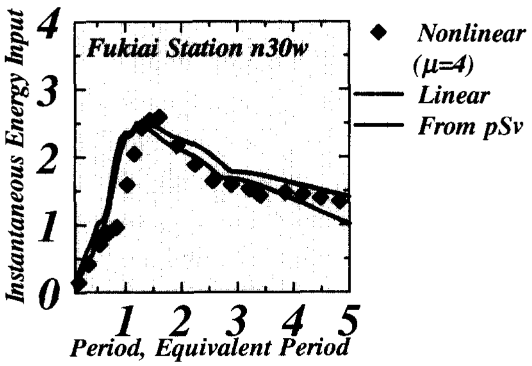
$$\beta=\frac{\{u\}^T[M]\{e\}}{\{u\}^T[M]\{u\}}; \tag{11}$$

where, x_e : equivalent displacement and f_e : equivalent force, and β : participation factor.

It is known from various response analyses of frame models that the respesentative displacement can be estimated from frame structure, the assumed mode shape, reduced system generally. However, in the especially of the force vector, is very sensitive to the displacement response in well inelastic range and the effect of higher modes of response should be considered carefully.[9] The higher mode response, which affects displacement distribution, is affected by the phase characteristics of the earthquake. Therefore, further study is needed for the estimation of the MDF displacement response.



(a) simple method (Fig. 2)



(b) sophisticated method (Fig. 7)

Fig. 9 Verification of sophisticated linear system for estimation of nonlinear responses

5. Seismic design guidelines for performance evaluation

5.1 Displacement-based design procedure

Dynamic response of the structure as multi-degree-of-freedom system may be evaluated based on the response of a reduced SDF system. A standard procedure based on the pushover analysis and the equivalent SDF system is summarized as follows:

a) Nonlinear static analysis is conducted to obtain lateral load-displacement relations of the MDF structure.
b) The load-displacement relations are reduced into those of the equivalent SDF system assuming a constant basic mode shape.
c) Linear response spectra of the input instantaneous energy are defined for the levels of the design earthquake motions with corresponding damping ratios.
d) On the load-displacement relations of the SDF system, response displacement is determined as elongated equivalent period, Teq.
e) Member deformations or strains can be calculated from pushover analysis at the corresponding displacement determined by SDF response.
f) Serviceability, damage control or ultimate limit state criteria defined and selected for each member using deformation are verified with appropriate degrees of reliability. The ultimate deformation capacity of the members is given in the recently published AIJ design guidelines.[4]
g) Member forces, especially shear in the wall, are magnified superposing the static force and higher mode forces determined

from acceleration spectra.

h) Shear capacity is verified to be larger than the maximum response at the ultimate limit with appropriate degrees of reliability. A method of calculating the ultimate shear capacity including deterioration due to deformation is given in AIJ design guidelines.[3]

5.2 Evaluation of ultimate deformation capacity

AIJ Design guidelines[2,3] gives complete design equations not only for the calculation of cracking, yielding and ultimate strengths, but also for the calculation of ultimate deformation capacity. The equation based on truss and strut model, in which effective concrete strength is assumed to be lower based on the panel tests. The equations have been verified through wide variety of test data.

5.3 Evaluation of seismic performance

To estimate the cost for the recovery, not only the repair cost but also th equivalent cost due to the loss of functions and their influence, e.g., inconvenience of the sufferer, must be accounted. Therefore, most of the structures are expected to survive to maintain basic function on use, from the social point of view, even after the ultimate earthquake. However, in design of one building, the structural designer does not necessarily give redundant performance level. The performance redundancy is being given arbitrarily by the variety in practice.

The design philosophy will be effective in which actual performance level of designed structures is quantitatively evaluated and expressed. The designer and the users may select upper performance level. The requirement based on it is called performance-evaluating code, which is different from just performance-based code or requirement, which requires only the verification of the performance criteria of the code. The performance-evaluating code includes the method of evaluating synthetic seismic performance index, which expresses the potential performance level related to expected damages.

6. Conclusions

A displacement-based seismic design procedure for reinforced concrete building structures was presented, which was based on the static pushover analysis and the response of the reduced equivalent single-degree-of-freedom system. A method of estimating the inelastic SDF displacement responses was applied to the design procedure, which was theoretically based on the equilibrium of an instantaneous energy input to the system. A consistent formula for the inelastic response is possible if the total design energy input to the structure and the duration of earthquake or pseudo-velocity spectra are given. A sophisticated method considering the duration of the earthquake is also outlined. The applicability of the procedure is verified through nonlinear dynamic analysis. Based on the recent research, an idea on the future performance-evaluating design guidelines is presented.

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