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고층건물 내진설계기법의 개선

IMPROVED EARTHQUAKE RESISTANT DESIGN OF MULTISTORY BUILDING FRAMES

Dong-Guen Lee† Seok-Yong Lee‡

Summary

An improved procedure for earthquake resistant design of multistory building structures is proposed in this study. The effect of gravity load on seismic response of structures is evaluated through nonlinear dynamic analyses of a single story example structure. The presence of gravity load tends to initiate plastic hinge formation in earlier stage of a strong earthquake. However, the effect of gravity load seems to disapper as ground motion is getting stronger. And one of shortcomings in current earthquake resistant codes is overestimation of gravity load effects when earthquake load is applied at the same time so that it may leads to less inelastic deformation or structural damage in upper stories,and inelastic deformation is increased in lower stories. Based on these observation, an improved procedure for earthquake resistant design is derived by reducing the factor for gravity load and inceasing that for seismic load. Structures designed by the proposed design procedure turned out to have increased safety and stability against strong earthquakes.

Introduction

There are many seismic regulations can be applied for earthquake resistant design of building structures throughout the world. With regard to earthquake response, the common aim of all of those codes is to lead to structures that can resist minor earthquakes undamaged, resist moderate earthquakes without significiant structural damage even though incurring nonstructural damage, and resist severe earthquakes without collapse [1]. For this purpose, every code impose three major requirements as follows:

1. A structure should be able to resist the design seismic loads which are estimated considering seismic hazard, soil properties, fundamental vibration period and effective weight of the structure.
2. Deformation of a structure should be limited within pre-determined criteria.
3. Individual components such as beams, columns, and shear walls should be detailed to provid expected ductility.

† Assistant Professor, Dept. of Civil Engrg., KAIST
‡ Graduate Student, Dept. of Civil Engrg., KAIST

The most important step in the earthquake resistant design is estimation of the base shear force for a building structure. For this purpose, the fundamental vibration period playes an important role to relate the response of a multistory building structure to that of a single degree of freedom(SDF) model using design spectrum. Response spectrum of an earthquake is obtained using SDF models such as the one shown in Fig 1. Property of a SDF model can be defined when mass, damping and stiffness are given. When inelastic response analysis is performed, nonlinear stiffness with a predetermined yield level need be defined. This type of SDF model is not appropriate for seismic response prediction of building structures when gravity load is applied and structural response is inelastic because gravity load leads to initial bending moments in beams and columns before earthquake loads are applied. In such a case, a single story structure can be used as another type of SDF model. When response of model is within elastic limit, both type of SDF mode can be considered identical. However, model of later type will experience inelastic deformation at a much lower level of relative displacement due to initial bending moment caused by gravity load. Main purpose of this study is to examine the effect of gravity load on seismic response of structures and develope an improvement in earthquake resistant design of building structures.

The Effect of Gravity Load

A single story structure shown in Fig. 2 is used to study the effect of gravity load on seismic response of building structures. Columns are assumed to be rigid and plastic hinges are assigned at both ends of the beam. The S00E component of 1940 El Centro earthquake is used as the input ground motion. The computer code STANON[2] is used for inelastic analysis of the structure. Rotation time histories of plastic hinges L and R at both end of the beam are shown Fig. 3-(a). When gravity load is not applied, both of the plastic hinges L and R experience the same moments and rotations. It is clear that the presence of gravity load results in earlier occurance of the plastic hinge at one end of the beam and difference in plastic hinge rotations and moments at both end of the beam. However, we can observe that plastic hinge rotation time histories for plastic hinge rotation are in the same shape with some shift, after 1.9 seconds, for plastic hinges L and R in both model A and B. This observation can lead to an important conclusion such that the effect of gravity load is not significant after some inelastic excursions. Similar conclusion can be drawn from plastic hinge moment time histories that merge to a single line after 1.9 seconds as shown in Fig. 3-(b). When low cycle fatigue damage is calculated for plastic hinges L and R, we can expect almost the same amount of damage whether gravity load are applied or not.

Shortcomings in Current Earthquake Resistant Design Procedure

Current design codes employ reduced design spectrum using a factor such as response modification factor R in ATC3-06 or UBC88 base on the fact that inelastic response spectrum is reduced from elastic one by the maximum ductility of the SDF model. Therefore, design codes have little interest in elastic behavior of multistory buildings which is more complicated than the behavior of a SDF model. Another shortcoming in current earthquake resistant design codes is overestimation of gravity load effects when earthquake load is applied at the same time, while gravity load effect disappears when a structure is subjected to a strong ground motion. In general, contribution of gravity load on design moment is significant in upper stories while major portion of design moment in lower stories is caused by earthquake load in high-rise buildings. Consequently, overestimation of gravity load leads to less inelastic deformation or damage in upper stories and inelastic deformation is increased in lower stories[7]. In cases of major earthquake, we can find many buildings with damage concentrated in lower stories resulting in partial or total collapse.

Proposed Earthquake Resistant Design Procedure

In the design codes such as ACI, UBC or AISC, gravity load and earthquake load are combined using load factors. In general, when ultimate strength design (or plastic design) procedure is applied, a structure is designed for the load combinations as follows;

$$1.7(D+L) \tag{1}$$

$$1.3(D+L+E) \tag{2}$$

where D, L and E are dead load, live load and earthquake load respectively. These load combinations are used to provide sufficient strength and stiffness to a structure that is subjected to gravity load which last for a long time or gravity load and seismic load that is applied with a limited short duration. Considering the effect of gravity load on the inelatic response of a structure that is subjected to a strong earthquake, it is proposed in this study to replace the second load combination by the following one;

$$0.5D + 0.5L + 1.7E \tag{3}$$

The main purpose of this load factor modification is to reduce the effect of gravity load in the design and provide increased resistance against seismic load. In addition to such modification in load factors, the design moment for the bottom of the columns in the first story is increased by 30 % as a special consideration.

Example Structures

One bay ten story multistory moment resisting steel frame shown in Fig. 4 was selected as an example structures. The bay width is taken to be 24 feets in both directions. A linear variation of stiffness with height was assumed for columns and girders. Elastic properties are the same for both structures designed according to the conventional design and the proposed design. However, design yield moment of each girder is determined according to each load case. Based on the strong column - weak girder concept, inelastic deformations are limited to beams and column bases.

Fundamental period of vibration of example structure is tuned to 1.0 second by adjusting the modulus of elasticity. Estimated gravity load for typical floor is dead load of 100 psf and live load of 60 psf for each floor and dead load of 80 psf and live load of 40 psf for the roof. The effective weight is estimated to be 60 kips per floor. Design seismic load is estimated based on the provisions in ATC3-06. The seismic coefficients A_a and A_v are assumed to be 0.4 and the site coefficient S is assumed to be 1.2. Response modefication factors 4 and 8 were used to assess differences of gravity load effect in design load.

Applicability of the proposed design procedure is verified by comparing seismic responses of building frames designed according to the conventional code type procedure and the proposed design procedure. In the following presentation of analysis results each frame is identified with two parameter code in sequence. The first character implies the applied design procedure and the second one indicates the response modification factor. Therefore, structure C4 and C8 represent example structure designed using the conventional load combinations in eqs. (1) and (2) while structure P4 and P8 represent example structure designed using the proposed load combinations in eqs. (1) and (3). Design moment envelopes for four example structures are shown in Fig. 5. In general, the design moments based on the proposed design are reduced in upper stories and increased in lower stories compared to the design moments based on the conventional design procedure.

Estimation of the Inelastic Response of Multistory Frames

As the results of nonlinear dynamic analysis of building structures, the rotation time histories of each plastic hinge that forms at both ends of girders are evaluated from computer code STANON[2]. The plastic hinge rotation time histories were used to estimate more detailed information on the response of multistory buildings.

The conventional procedure for assessing the seismic performance of structures is to evaluate the maximum deformation demands or maximum ductility requirements for the complete structure or for individual critical components. Because of the cyclic nature of seismic response, the amount and severity of inelastic deformation can not be represented by the maximum ductility requirements. Therefore it is more appropriate to consider the cumulative effect of all inelastic excursions rather than the maximum excursion alone. For steel frame structures, simple cumulative damage models can be used. These models which are only approximate, utilize the Coffin-Manson relationship and Miner's rule of linear damage accumulation to assess component performance. Using these two relationships a model for cumulative damage after n cycles of different plastic deformation range $\Delta\delta_{pi}$ can be given as

$$D = \sum_{i=1}^{n} \frac{1}{N_{fi}} = C \sum_{i=1}^{n} (\Delta\delta_{pi})^c \qquad (4)$$

where, D represents damage level ($D \geq 1$ means failure), n is the number of damage cycles, $\Delta\delta_{pi}$ is plastic deformaion range of cycle i, N_{fi} is the number of cycles of constant plastic deformation range $\Delta\delta_{pi}$ to cause failure and C and c are structural performance parameters. It should be noted that $\Delta\delta_{pi}$ means not the maximum plastic deformation but the one corresponding to the full cycle of yield excursions.

In this study only a relative assessment of damage is attempted by assigning a constant value of 1.0 to the coefficient C and a constant value of 1.5 to the exponent c[5]. With these values and plastic hinge rotation for the selected deformation quantity, the accumulated low cycle fatigue damage after n cycles of different plastic ranges is given as

$$D^* = \sum_{i=1}^{n} \left(\frac{\Delta\theta_{pi}}{\theta_y}\right)^{1.5} \qquad (5)$$

where, D^* represents cumulative damage index, n is the number of plastic deformation cycles, $\Delta\theta_{pi}$ means the plastic rotation range of cycle i and θ_y is the yield rotation of a girder. For this cumulative damage model, the rain flow cycle counting method is used in order to convert the irregular time history of story deformation into a many closed cycles as possible.

Improvement in Seismic Response

The S00E component of 1940 El Centro earthquake ground acceleration and the N21E component of 1952 Taft earthquake ground acceleration scaled to have the peak ground acceleration(PGA) of 0.4g and 0.6g were used as the input ground motions. According to the provisions in ATC3-06, 0.4g represents the design earthquake intensity. Since it is equal to the effective peak acceleration which is usually less than the PGA of major earthquakes, 0.6g was introduced to represent the intensity of major, severe earthquakes.

Seismic response of building structures are studied in terms of maximum plastic hinge rotation and distribution of cumulative low cycle fatigue damages. Maximum plastic hinge rotation for beams in each story and column base in the first story for the conventional design is presented in Fig. 6. In general, plastic hinge rotations in lower stories of building structures are increased remarkably. This concentration of inelastic deformations in lower stories is due to overestimation of gravity load in determination of design moment based on the conventional design procedure. Since the contribution of gravity load in design moment for C8 is larger than the one for C4, the increase of inelastic deformations in lower stories for C8 is more severe than for C4. For Taft N21E ground motion maximum plastic hinge rotations in medium to top floors are somewhat increased as a result of higher modes contributions on dynamic response. In Fig. 7 cumulative damage responses for the conventional design are presented. Overall trends are the same with the one for maximum plastic hinge rotation case, more severe concentration of cumulative damages in bottom story can be observed. Therefore, when building strcutures are subjected to earthquakes, inelastic deformation demand can not be evaluated properly based on maximum plastic excursion alone and the cumulative effect of all inelastic excurions must be considered appropriately. For the conventional design, the increase of inelastic deformations in lower stories is considerable against design earthquakes. And as the intensity of earthquake excitation is getting stronger, the concentration of plastic hinge rotations in lower stories increases more significantly in the conventional design. Therefore, in the conventional design, increased inelastic deformation in lower stories may lead to partial or total collapse of building structure against major earthquakes.

Based on the effect of gravity load during strong earthquakes, it is desirable to reduce the contribution of gravity load in design and provide increased resistance against seismic load in order to reduce increased inelastic deformations in lower stories. It can be expected from the envelopes of design moment that in the proposed design case, the more parts of inelastic deformations in lower stories can be shared with upper stories. A comparison of the maximum plastic hinge rotation and distribution of cumulative damage for beams in each story and column base in the first story for the conventional and the proposed designs is presented in Fig. 8 and 9. The sudden increase of inelastic deformation demands in lower stories are reduced remarkably. In the proposed design, inelastic deformations in lower stories are reduced significantly and those in medium to upper stories are increased somewhat against design earthquakes. And in the proposed design, the sudden increase in plastic hinge rotations in lower stories does not occur when structures are subjected to major earthquakes. As a result, it can be said that building structures designed by the proposed design procedure turned out to have increased safety and stability against strong earthquake motions.

Conclusions

In this paper, an improved earthquake resistant design procedure is proposed based on the effect of gravity load on seismic responses of multistory building structures. Applicability of the proposed design procedure is verified through nonlinear dynamic analyses of example structures. Based on the results of this study, the following conclusions were reached:

1. The presence of gravity load tends to initiate plastic hinge formation in earlier stage of strong earthquake. However, the effect of gravity load seems to disapper as ground motion is getting stronger.

2. The contribution of gravity load on design moment causes increased inelastic deformation in lower

stories of multistory building structures.

3. Building structures designed by the proposed design procedure turned out to have increased safety and stability against strong earthquake motions compared to those structures designed by the conventional code procedure.

4. Extensive future study on the effect of gravity load to the seismic response of various type of building structures can lead to an improvement in earthquake resistant design of multistory building structures.

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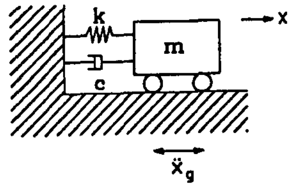
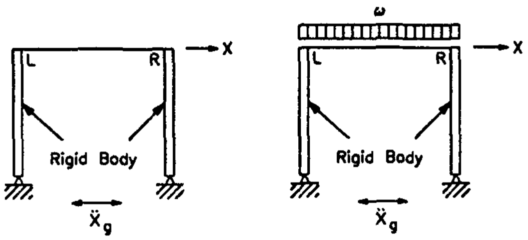
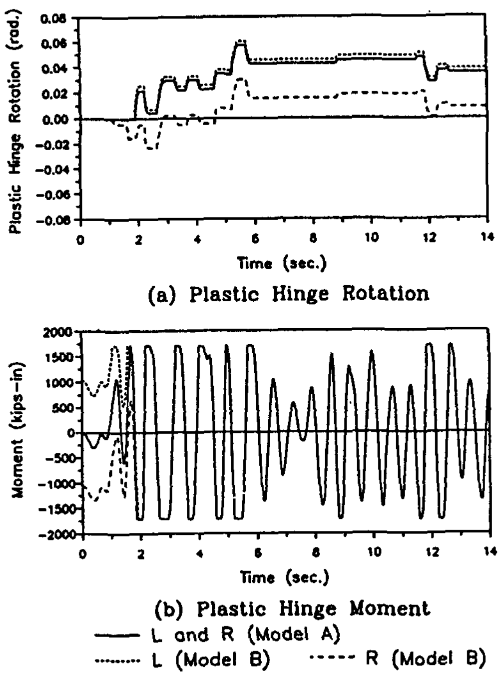


Fig. 1. SDF Model



(a) Model A (b) Model B

Fig. 2. Single Story Frame Models



(a) Plastic Hinge Rotation

(b) Plastic Hinge Moment

—— L and R (Model A)
.... L (Model B) ----- R (Model B)

Fig. 3 Plastic Hinge Response Time Histories

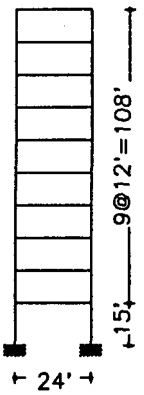
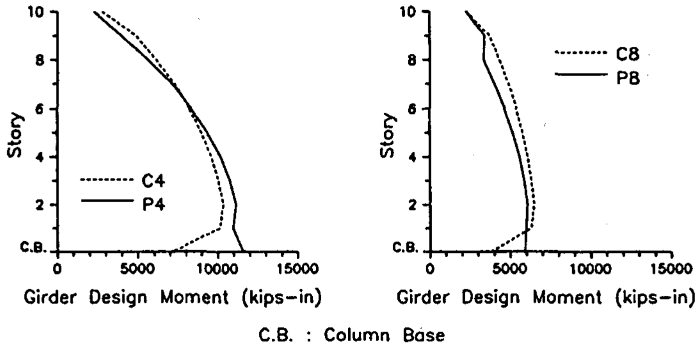
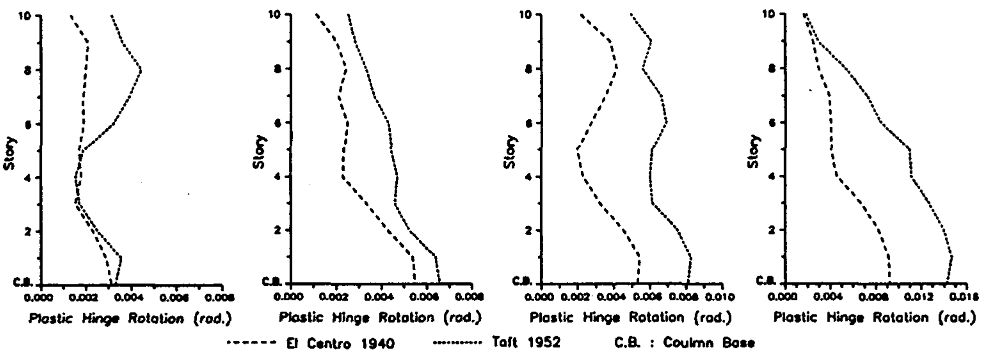


Fig. 4. Example Structure



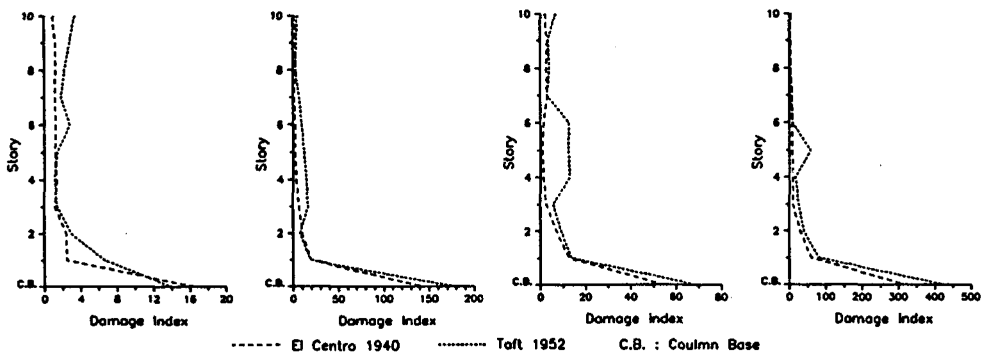
(a) Frame with R=4 (b) Frame with R=8

Fig. 5 Comparison of Girder Design Moment



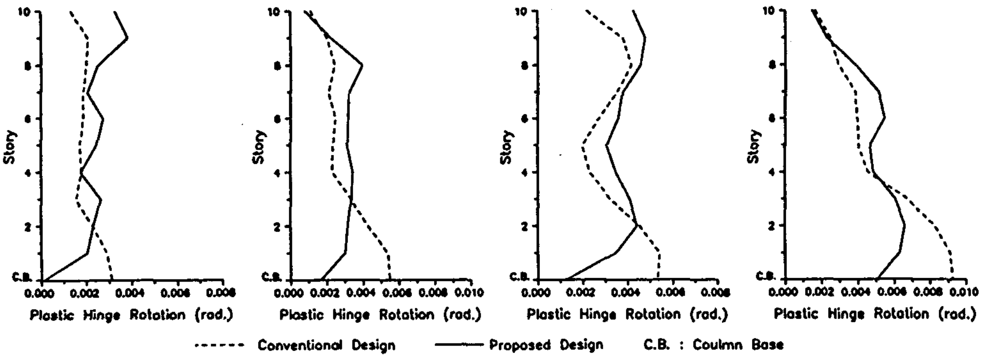
(a) R=4, PGA=0.4g (b) R=8, PGA=0.4g (c) R=4, PGA=0.6g (d) R=8, PGA=0.6g

Fig. 6 Plastic Hinge Rotation Envelopes for the Conventional Design



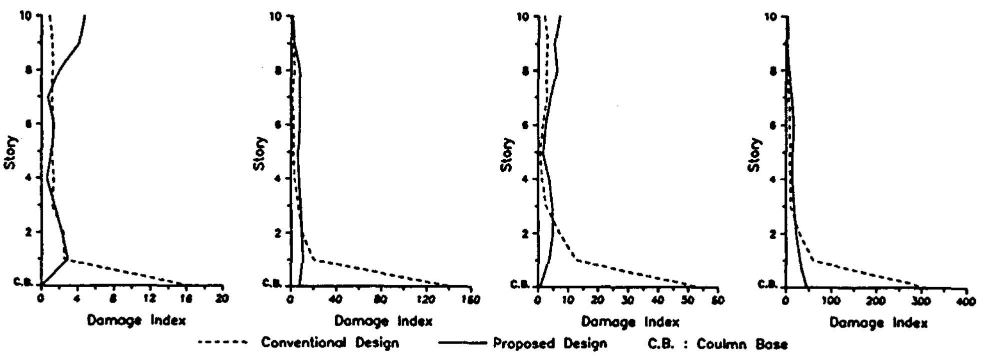
(a) R=4, PGA=0.4g (b) R=8, PGA=0.4g (c) R=4, PGA=0.6g (d) R=8, PGA=0.6g

Fig. 7 Damage Index Envelopes for the Conventional Design



(a) R=4, PGA=0.4g (b) R=8, PGA=0.4g (c) R=4, PGA=0.6g (d) R=8, PGA=0.6g

Fig. 8 Comparison of Plastic Hinge Rotation Envelopes (El Centro 1940)



(a) R=4, PGA=0.4g (b) R=8, PGA=0.4g (c) R=4, PGA=0.6g (d) R=8, PGA=0.6g

Fig. 9 Comparison of Damage Index Envelopes (El Centro 1940)