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SEISMIC EVALUATION OF RC SCHOOL BUILDINGS WITH PUSHOVER ANALYSIS

Fu-Pei Hsiao[1], Yeong-Kae Yeh[2], Shyh-Jiann Hwang[3]

[1] National Center for Research on Earthquake Engineering, Taipei, Taiwan, R.O.C., -mail: fphsiao@ncree.org.tw
[2] Research Fellow, National Center for Research on Earthquake Engineering, Taipei, Taiwan, R.O.C.
[3] Professor, National Taiwan University, Taipei, Taiwan

Abstract

This paper provides a seismic evaluation process for RC school buildings with pushover analysis. It can consider the seismic resistance of the structure is not only controlled by the strength but also by the stiffness. This process introduced the ETABS-Nonlinear to be a tool program for the precisely seismic evaluation. It can be used to get the relative curve of the base shear versus the roof displacement. The relative curve of the base shear versus roof displacement can present the relation of the loading and deflection of the RC structure. Through the verification with the experiment data from in-situ tests done by NCREE in Taiwan, the result from this study can provide a good approximation for RC school buildings. Therefore, the seismic evaluation process which suggested by this paper can provide the engineers a good way to precisely seismic evaluation of RC school buildings.

Keywords: RC building, seismic evaluation, pushover analysis

1. INTRODUCTION

Taiwan is in the region of the circum-pacific seismic zone. Earthquakes are common experiences for people in Taiwan. People are used to earthquakes and even ignore them. In the morning of September 21, 1999, the Chi-Chi earthquake awoke the people of Taiwan with its huge destructiveness. It told us the importance of the seismic capacities of structures. The Chi-Chi Earthquake caused nearly half of the school buildings in the central area of Taiwan to collapse or damage seriously. 656 primary and secondary school buildings were damaged in that earthquake. This disaster told us the seismic capacities of existing school buildings in Taiwan are probably not sufficient. Due to the existence of windowsill in traditional school buildings, the short-column effect caused the weak seismic capacity along the direction of the passage. Serious casualties and losses may result from the collapse of school buildings under strong earthquakes. To avoid casualties in the future earthquakes is the most important job in Taiwan. To retrofit these bad seismic performance school buildings is one solution to reduce the probable casualties. Before the retrofitting the seismic capacity of school buildings should be evaluated with a reliable method.

According the capacity spectrum method proposed by ATC-40 (ATC 1996), the pushover analysis is used to get the nonlinear base-shear to roof-displacement relationship of school buildings which is named as the capacity curve. The seismic capacity of buildings can be specified with the damage peak ground acceleration which can be determined from the pushover curve and the corresponding performance point. The accuracy of the pushover analysis is dependent on the well-defined properties of nonlinear hinges in structure elements. The load-deformation relationships of nonlinear hinges in beams and columns are discussed in this paper.

2. PUSHOVER ANALYSIS

The capacity curve from the pushover analysis is the foundation of the purposed detailed seismic evaluation method in this paper. Many commercial programs like ETABS and SAP2000 can process nonlinear static analyses which are also called pushover analyses. The nonlinear response of a structure is restricted to nonlinear hinges which are assigned on the structural elements of that structure. These nonlinear hinges can be divided to three types, moment hinges, shear hinges and axial hinges. As shown in Figure 1, the mechanic parameters of a nonlinear hinge are constructed from the nonlinear part of the load-deformation curve of a structural member. For moment hinges, load Q is moment M at the location of the moment hinge, and deformation Δ is associate rotation angle θ of the moment hinge. For shear hinges, load Q is lateral force V of the structural member, and deformation Δ is associate lateral displacement δ_v of the structural member. For axial hinges, load Q is axial force P of the structural member, and deformation Δ is associate axial displacement δ_p of the structural member.

In the pushover analysis, the flexural rigidity of reinforced concrete beams is assumed as $0.5E_cI_g$ and the flexural rigidity of reinforced concrete columns is assumed as $0.7E_cI_g$, where E_c is Young's modulus for concrete; and I_g is moment of inertia of gross concrete section. In the following the constructions of nonlinear hinges for beams and columns will be presented.

3. NONLINEAR BEHAVIOR OF BEAMS OR COLUMNS

The deformation of beams or columns in structure frames can be simulated as double curvature deformation of columns as shown in Figure 2. According to the research works of Elwood and Moehle (2005a; 2005b), for the double curvature reinforced concrete columns with light transverse reinforcement, under the axial load P and lateral load V, as the lateral displacement δ reaches yield displacement δ_y, the main reinforcement of columns is yielding, as the lateral displacement δ reaches flexure-shear failure displacement δ_s, large shear cracks will be observed at the regions of plastic hinges and lateral strength will be degraded, and as the lateral displacement δ reaches axial failure displacement δ_a, the columns will lose their axial capacity and their collapse will occur.

The flexure-shear failure displacement δ_s can be calculated as (Elwood and Moehle 2005a)

$$\frac{\delta_s}{L} = \frac{3}{100} + 4\rho'' - \frac{1}{40}\frac{\upsilon}{\sqrt{f_c^{'}}} - \frac{1}{40}\frac{P}{A_g f_c^{'}} \geq \frac{1}{100} \tag{1}$$

where L is the length of the column; ρ'' is the transverse reinforcement ratio as A_{st}/bs; A_{st} is the area of the transverse reinforcement; b is the column section width; s is the spacing of the transverse reinforcement; υ is the maximum nominal shear stress in MPa as V/bd; d is the depth to centerline of tension reinforcement; $f_c^{'}$ is the concrete compressive strength in MPa; and A_g is the gross cross-sectional area of the column.

The axial failure displacement δ_a can be calculated as (Elwood and Moehle 2005b)

$$\frac{\delta_a}{L} = \frac{4}{100}\frac{1+(\tan\theta)^2}{\tan\theta + P\dfrac{s}{A_{st} f_{yt} d_c \tan\theta}} \tag{2}$$

where θ is the angle from horizontal of critical shear-failure plane, and can be assumed to be 65 degrees; f_{yt} is the yield strength of the transverse reinforcement; and d_c is the depth of the column core from centerline to centerline of the ties.

Some researchers (Priestley et al. 1994; Sezen and Moehle 2004) have shown that shear strength of columns reduces with increasing lateral displacement ductility demand. So the shear strength degradation curve restricts the development of the lateral strength of columns. According to the difference between the flexural strength and the shear strength, the failure modes of columns can be divided to three kinds, flexure-shear, shear, and flexure failures.

Flexure-shear failure mode

As shown in Figure 3, as the shear strength V_n is larger than the flexural strength V_m, the column deforms with stiffness k to its flexural strength V_m and the main reinforcement yields. Assuming no strain hardening, the lateral strength keeps constant to the lateral displacement δ_s, the lateral force reaches the degraded shear strength, and the flexure-shear failure occurs. After that, the lateral strength decays to the lateral displacement δ_a, the lateral strength approaches to zero, the column loses its axial capacity, and the axial failure occurs. The flexure-shear failure displacement δ_s and axial failure displacement δ_a can be calculated as equations (1) and (2). The lateral stiffness k of a double curvature column can be calculated as

$$k = 12(EI)_c / L^3 \tag{3}$$

where $(EI)_c$ is the flexural rigidity of the column.

According to the suggestion of Sezen and Moehle (2004), the shear strength can be calculated as

$$V_n = \frac{A_{st} f_{st} d}{s} + \left(\frac{0.5\sqrt{f_c^{'}}}{a/d} \sqrt{1 + \frac{P}{0.5\sqrt{f_c^{'}} A_g}} \right) 0.8 A_g \tag{4}$$

where a is the shear span as $L/2$ for a double curvature column and has a range $2 \le a/d \le 4$; and $f_c^{'}$ is the concrete compressive strength in MPa. The flexural strength of a double curvature column can be calculated as

$$V_m = 2M_n / L \tag{5}$$

where M_n is the nominal moment strength of a reinforced concrete column (ACI 2005).

Shear failure mode

As shown in Figure 4, as the shear strength V_n is smaller than the flexural strength V_m, the column deforms with stiffness k to its shear strength V_n and the shear failure occurs. After that, the lateral strength decays to the lateral displacement δ_a, the lateral strength approaches to zero, the column loses its axial capacity, and the axial failure occurs. The axial failure displacement δ_a can be calculated as equation (2) but is restricted to the value $0.04L$ for the brittle shear failure mode.

Flexure failure mode

As shown in Figure 5, as the degraded shear strength is always larger than the flexural strength V_m, the column deforms with stiffness k to its flexural strength V_m and the main reinforcement yields. Assuming no strain hardening, the lateral strength keeps constant to a very large lateral displacement until the concrete core crush or the main reinforcement breaks. This column never loses its axial capacity.

4. NONLINEAR HINGES FOR BEAMS OR COLUMNS

Because the position of the inflection point on column or beam varied with the applied loading, the failure mode can not be realized before the pushover analysis. Therefore, it can be set moment hinges in the each ends of column or beam to present the flexure-shear failure mode or flexure failure mode in the pushover analysis. And it also can be set shear hinge in the middle of column or beam to present the shear failure mode in the pushover analysis. The engineers can define the hinge properties by research papers or experimental data with their experience. This paper depends on the previous lateral loading-displacement curves of column and beam, and suggests the parameters of moment hinges and shear hinges which can be the reference for the engineers.

Nonlinear moment hinge

At the both ends of a beam or column, nonlinear moment hinges are assigned to represent the flexure-shear or flexure failure mode. According to the lateral force-displacement relationship of the element as shown in Figure 3, the parameters of the nonlinear moment hinge are shown in Figure 6 and table 1. The moment SF is the nominal moment strength M_n and the rotation SF is 1. The parameters are

$$c = \frac{\delta_s - V_m/k}{L} \tag{6}$$

$$d = \frac{\delta_a}{L} \tag{7}$$

Nonlinear shear hinge

At the center of a beam or column, a nonlinear shear hinge is assigned to represent the shear failure mode. According to the lateral force-displacement relationship of the element as shown in Figure 5, the parameters of the nonlinear shear hinge are shown in Figure 7 and table 2. The shear SF is the nominal shear strength V_n and the displacement SF is L. The parameter is

$$d = \frac{\delta_a}{L} \tag{8}$$

5. COMPARISON OF ANALYTICAL AND TEST RESULTS

The research team composed of crews of NCREE, the department of construction engineering, National Taiwan University of Science and Technology (NTUST) and National Yunlin University of Science and Technology (NYUST), and the department of civil engineering, National Taiwan University (NTU) used an old school building of Kouhu elementary school, Yunlin, which is about to be demolished as the subject of a pushover test, as shown in Figure 8. The specimen was tested by static lateral load and pushed to totally collapse. The test results can be used to verify the seismic analysis model. The numerical model of ETABS was established as Figure 9, and the plastic hinges were set in the columns.

Figure 10 show the comparison of analytical and experimental pushover curves of in-situ test. The comparison shows that the analytical model presents well prediction before the pushover curve is decurved. In the future research, the negative slope of pushover cure will be improved by modify with the plastic hinge properties.

6. CONCLUSIONS

The detailed seismic evaluation method proposed in this paper can reasonably provide a measure to determine the seismic capacity of buildings. Through the verification with the experiment data from in-situ tests done by NCREE in Taiwan, the result from this study can provide a good approximation for RC school buildings. Therefore, the seismic evaluation process which suggested by this paper can provide the engineers a good way to precisely seismic evaluation of RC school buildings.

7. REFERENCES

ATC (1996), *Seismic Evaluation and Retrofit of Concrete Buildings*, ATC-40 Report, Applied Technology Council, Redwood City, California, USA.

Elwood, K. J., and Moehle, J. P. (2005a), "Drift Capacity of Reinforced Concrete Columns with Light Transverse Reinforcement," *Earthquake Spectra*, Vol. 21, No. 1, pp. 71-89.

Elwood, K. J., and Moehle, J. P. (2005b), "Axial Capacity Model for Shear-Damaged Columns," ACI Structural Journal, Vol. 102, No. 4, pp. 578-587.

Table 1. Parameters of the nonlinear moment hinge (M3 type)

Points	Moment/SF	Rotation/SF
A	0	0
B	1.0	0.0
C	1.0	c
D	0.0	d
E	0.0	d

Table 2. Parameters of the nonlinear shear hinge (V2 type)

Points	Shear/SF	Disp./SF
A	0	0
B	1.0	0.0
C	1.0	0.0
D	0.0	d
E	0.0	d

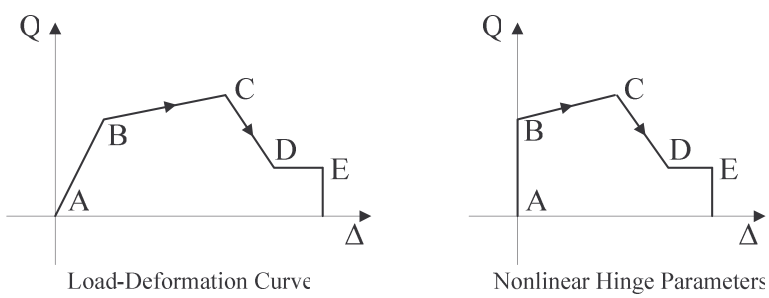

Figure 1. Nonlinear hinge parameters

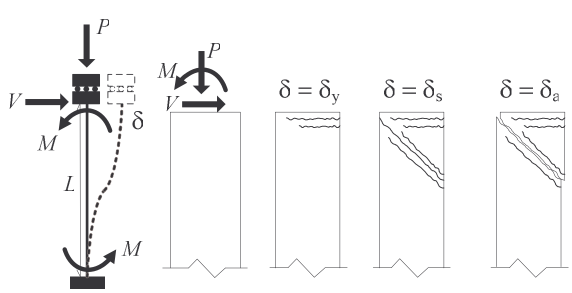

Figure 2. Double curvature deformation of columns

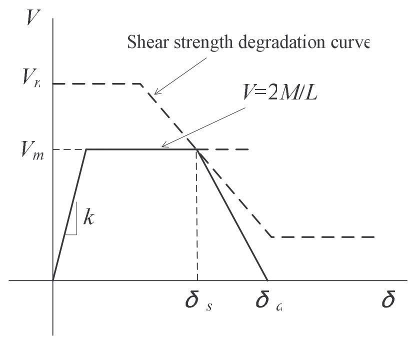

Figure 3. Force-displacement relationship of flexure-shear failure mode

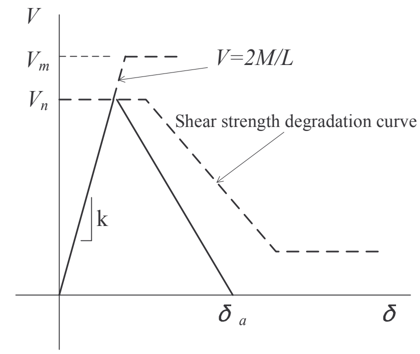

Figure 4. Force-displacement relationship of shear failure mode

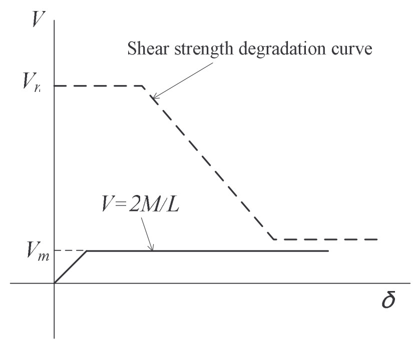

Figure 5. Force-displacement relationship of flexure failure mode

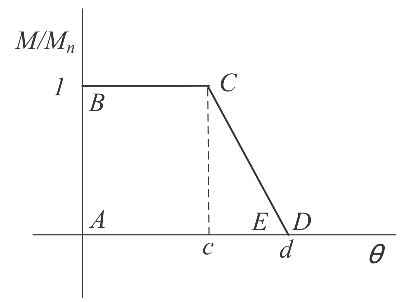

Figure 6. Moment-rotation relationship of nonlinear moment hinge

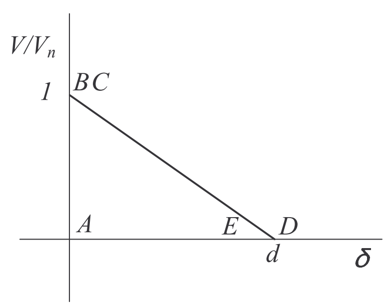

Figure 7. Shear-displacement relationship of nonlinear shear hinge

Figure 8. Pushover test in Kouhu elementary school

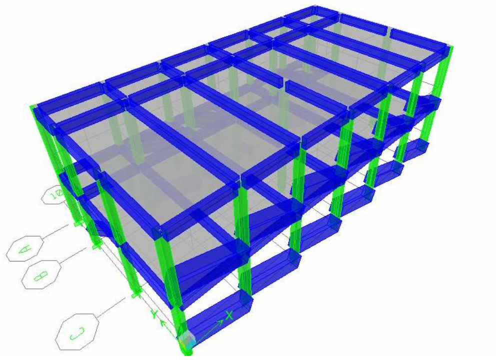

Figure 9. Numerical model of ETABS for the specimen in Kouhu elementary school

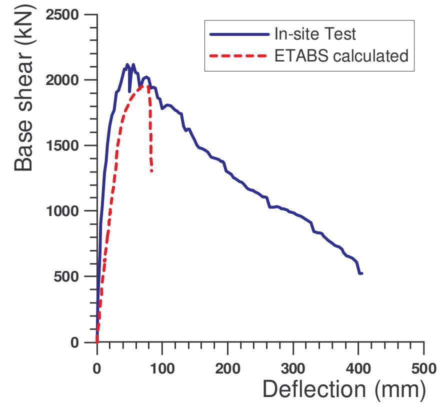

Figure 10. Comparison of analytical and experimental pushover Curve of in-sited test