DYNAMIC PUSH-OVER CURVE FOR BUILDING STRUCTURES UNDER 3-DIRECTIONAL EARTHQUAKE INPUT

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SUMMARY

Most of the push-over curves are the static lateral deflections under static lateral loads. They do not reflect the lateral deflections due to actual earthquake excitations. The purpose of this paper is to propose a technique to obtain the dynamic push-over curve for 3D-RC building structures. That is the analytical relationship among peak ground acceleration (PGA), base shear and floor lateral deflection. The site specific acceleration response spectra or the site design acceleration response spectra in EW, NS and UD may input to the structure simultaneously. The output are PGAs, corresponding base shears, member stresses and floor deflections.

Program for nonlinear incremental analysis is developed in this paper. The normalized acceleration spectra in EW, NS and UD directions are input simultaneously as loading to any 3D-RC structure. The PGAs in these directions are increased proportionally from zero up to yield and collapse of the structure. During each loading step, the flexural, shear and axial stiffness of each member, the sidesway of each floor, the damping ratio of the whole structure, the interaction between soil and foundation, the progress failure of local members due to flexural, shear and axial stresses are considered at each incremental loading step. The dynamic response are superposed by SRSS for each mode and for each direction. Finally, the dynamic push-over curve for PGA-base shear-lateral deflection is obtained.

The caculated dynamic push-over curves are compared with 2-story and 7-story RC shaking table specimens tested in UC Berkeley. All the comparisons show reasonable results for the proposed dynamic push-over curves.

INTRODUCTION

Program for nonlinear incremental analysis is developed in this paper. In this analysis, site elastic acceleration response spectra are needed for the caculation. Based on Liu's report [Liu, 1998], parameters of nonlinear incremental spectrum analysis for 3D-RC building frames with shear walls structures are described as follows:

Load combination

The maximum stress of each member is accumulated by the incremental spectrum analysis at each incremental ground acceleration. It is supposed that the maximum stress of each member is appeared at the same time. And, since the earthquake loads are reversed cyclically, the stress combination for major and minor axes of each member section should be considered for both positive and negative earthquake loads.

Dynamic strength of materials

Dynamic compressive concrete strength by Ahmad and Shan [Ahmad and Shah, 1985] is employed as

$$(f_c')_{dynamic} = 1.18 f_c' \tag{1}$$

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stress and strain curve is proposed a moifiied Kent and Park equation [Kent andd Park, 1971] as Figure 1.

Dynamic yield stress of rebars under earthquake loading is considered as Park and Paulay [Park and Paulay, 1987]:

$$(f_y)_{dynamic} = 1.14 f_y \tag{2}$$

Effective dynamic flexural stiffness for columns and beams

Before flexural crack, the flexural stiffness is in elastic, so

$$M_{max} \le M_{cr} \qquad \Delta EI_{tan} = E_c I_{gt} \tag{3}$$

where M_{max} and M_{cr} are maximum and crack moments, ΔEI_{tan} is tangential flexural stiffness, E_c is the modulus of elasticity for concrete and I_{gt} is the gross-transformed moment of inertia.

After flexural crack and flexural ultimate, Equation (4) for secant effective dynamic flexural stiffness by Jerath and Shibani [Jerath and Shibani , 1985] is employed as

$$M_{cr} < M_{max} \le M_u \qquad EI_{dyn} = E_c * \left\{ \left(\frac{\alpha \cdot M_{cr}}{M_{max}} \right)^{\beta} I_{gt} + \left[1 - \left(\frac{\alpha \cdot M_{cr}}{M_{max}} \right)^{\beta} \right] I_{cr} \right\} \tag{4}$$

where M_u is ultimate moment, I_{cr} is the cracked moment of inertia, α and β are coefficients by experimental suggestion. In this paper, it is proposed α be 0.7 and β be 1.0.
Tangential flexural stiffness ΔEI_{tan} is transferred from secant effective dynamic flexural stiffness as: (see Figure 2)

$$(\Delta EI_{tan})_2 = \frac{\Delta M(\Delta a_{23}/\Delta a_{12})}{\frac{M_2 + \Delta M(\Delta a_{23}/a_{12})}{(EI_{dyn})_3} - \frac{M_2}{(EI_{dyn})_2}} \tag{5}$$

where ΔM is the incremental moment of each incremental step, Δa_{12} and Δa_{23} are the prior and later step of incremental acceleration.

After ultimate stage, if the member not collapse by flexural, tangential flexural stiffness $\Delta EI_{tan}$is proposed as $0.1 \cdot E_c I_{cr}$.

Effective dynamic shear rigidity for columns and beams

Before diagonal crack, tangential shear rigidity ΔGA_{tan} is taken as $E_c A_g / 2.4$; after diagonal cracks, the secant dynamic shear rigidity by Yang [Yang, 1992] is employed and transferred into tangential stiffness. (see Figure 3)

$$GA_{dyn} = C_s G_c A_g \tag{6}$$

where G_c is the modulus of shear rigidity; A_g is the cross section area; C_s is the reduced coefficient. [Liu, 1998]

Strength and stiffness for RC shear walls

The empirical equations of Liu's paper [Liu,1998] are used. For example,
if the wall is subjected a compression axial force $P \ge 0$, the crack shear force of concrete V_c is

$$V_c = 0.166\left(1+\frac{P}{13.72A_g}\right)\sqrt{f_c^{'}}\left(W-W_0\right)\cdot T \le \frac{M_{cr}}{H_s}$$

if the wall is subiected a tensile axial force $P<0$, the crack shear force of concrete V_c will be as

$$V_c = 0.166\left(1+\frac{P}{3.43A_g}\right)\sqrt{f_c^{'}}\left(W-W_0\right)\cdot T \le \frac{M_{cr}}{H_s}$$

The shear force taken by rebars V_s is :

with boundary columns: $V_s = 0.67A_v f_{vy} + 0.33A_{b-boop} f_{by} + 0.33A_h f_{hy}$

without boundary columns: $V_s = 0.18(W/H)^{1.1} f_v f_{vy} + 0.33A_{b-boop} f_{by} + 0.33A_h f_{hy}$

The ultimate shear capacity of RC walls is $V_u = V_c + V_s$.
Where W and W_0 are widths of wall and opening, H is height of wall, T is thickness of wall, A_v, A_h and A_{b-hoop} are areas of vertical, horizontal rebars in wall and stirrup of boundary columns rebars, f_{vy} and f_{hy} are yield stress of vertial and horizontal rebars.
The tangential flexural and shear stiffness of RC walls ΔEI_{tan} and ΔGA_{tan} are as:

For elastic stage, $\Delta EI_{tan} = E_c I_{gt}$; $\Delta GA_{tan} = G_c A_{gt}$
For crack stage, $\Delta EI_{tan} = 0.1\cdot E_c I_{gt}$; $\Delta GA_{tan} = 0.1\cdot G_c A_{gt}$
For yield stage, $\Delta EI_{tan} = 0.01\cdot E_c I_{gt}$; $\Delta GA_{tan} = 0.01\cdot G_c A_{gt}$
For ultimate stage, $\Delta EI_{tan} = 0.001\cdot E_c I_{gt}$; $\Delta GA_{tan} = 0.001 G_c A_{gt}$

P - Δ effect of floor sideways

P - Δ effect due to vertical loads are considered as equivalent lateral forces acting on each floor.

Rigid zone of columns and beams

The bending moment at beam-column joints is reduced to the bending moment at outer face of rigid zone. Using this reduced bending moment, the effective flexural stiffness is caculated at every losd step as Figure 4.

Damping ratio of whole structures

If the PGA is less than 40% of collapse ground acceleration (CGA), damping ratio is 3%. For PGA between 40% to 70% of CGA, damping ratio is 6%. For PGA larger than 70% of CGA, damping ratio is 10%.

Incremental ground acceleration at each step

For 3% damping ratio, incremental ground acceleration is 0.06g. For 6% damping ratio, incremental ground acceleration is 0.04g. For 10% damping ratio, incremental ground acceleration is 0.02g.

Others

The modification of stiffness for brick wall, tie-beams and footings are also considered. [Liu, 1998]

EXAMPLES

In order to verify the accuracy of the dynamic analytical results, two shaking table specimens [Oliva, 1980], [Bertero, Aktan, Charney and Sause, 1984] are checked. The first specimen is one-third scale 2-story reinforced concrete frame and the second is one-fifth scale 7-story frame with RC shear walls subjected to earthquake load.

One third scale 2-story reinforced concrete frame specimen test

The test model was a one-third scale 2-story reinforced concrete frame, rectangular in floor plan, and had floor slabs cast integrally with beams spanning in longitudinal and transverse directions between four supporting columns which were also of rectangular cross section as Figure 5, with inelastic biaxial motion included through earthquake excitation on a shaking table. General frame properties are listed in reference 7 [Oliva, 1980].

Figure 6 and Figure 7 show the comparisons of the load-deflection envelopes. For the base shear-lateral deflection curve of the figure, The analytical curve is almost an external envelope of the experimental curve. The $P-\Delta$ curve is an important foundation of aseismic assessment. For PGA-base shear curve of the figure, it shows the softening of the structure when PGA is large. If the specimen responses elastically at 0.4 g, base shear will be 148kN, as point $y2'$ in Figure 6. Just because the ductility, the base shear at 0.4g is down to 113kN, as point $y2$ in Figure 6. This curve is an index of ductility reduction of base shear.

One fifth scale 7-story frame with RC shear walls specimen test

The model was designed to comply with the similitude requirements for a direct reduced-scale model of the full-scale model.[Harris, Bertero and Clough, 1981]. The model was rectangular in floor plan, and had twelve coulmns, one boundary wall located at frame B of x axis and four walls without boundary columns located at Frame 1 and 4 of y axis. Its plan and elevation are show in Figure 8.

Figure 9 also shows the comparisons of the load-deflection envelopes. PGA-base shear curve is almost linear before yield of the structure. This shows the ductility is not good for frame structure with RC walls. The analytical curves is in good match with the experimental curves.

Others

Four actural buildings seriously damaged after 1986 and 1990 earthquakes in Taiwan are also used to check vulnerability assessment. The failure modes are predicted resonably. [Sheu and Chang, 1993]

CONCLUSIONS

1. The push-over curves using dynamic nonlinear incremental analysis is developed in this paper and has a reasonable accuracy when compared with experimental models and actual building structures.

2. The base shear-deflection curve and PGA-base shear curve are good index for ductility and vulnerability assessment.

3. Damping ratio increases from 3 to 10% and incremental acceleration decreases from 0.06 to 0.02g with increase of PGA.

4. Dynamic flexural stiffness EI_{dyn} of beam and column may be used for dynamic analysis.

5. Dynamic compressive stress of concrete is higher 18% than static compressive stress of concrete; Dynamic yield stress is higher 14% than static yield stress of rebars.

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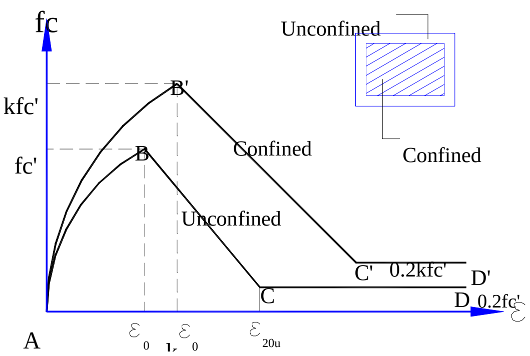

FIGURE 1. STRESS-STRAIN CURVE

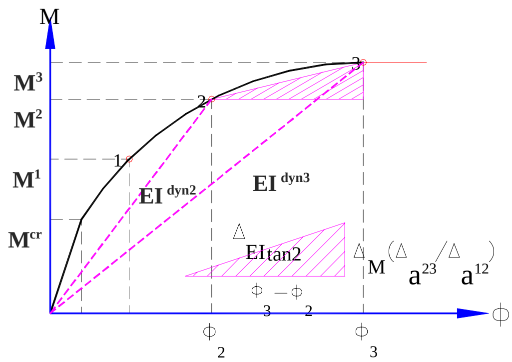

Figure 2. Intimation of incremental flexural stiffness

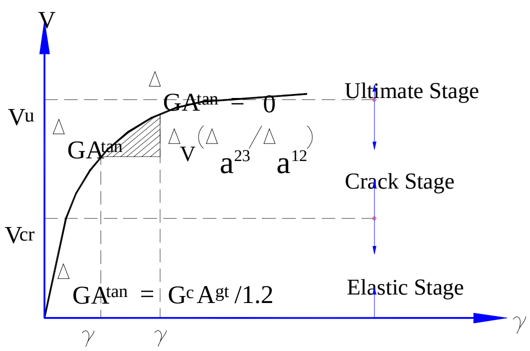

Figure 3. Intimation of incremental shear stiffness

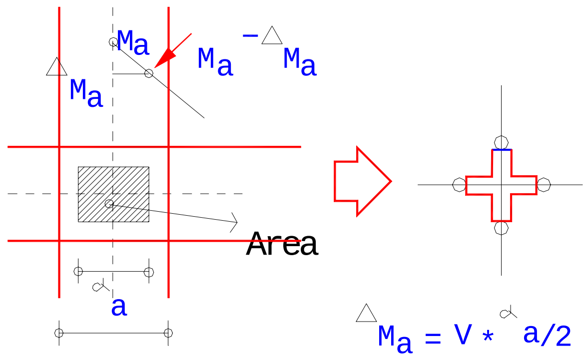

FIGURE 4. INTIMATION OF RIGID ZONE

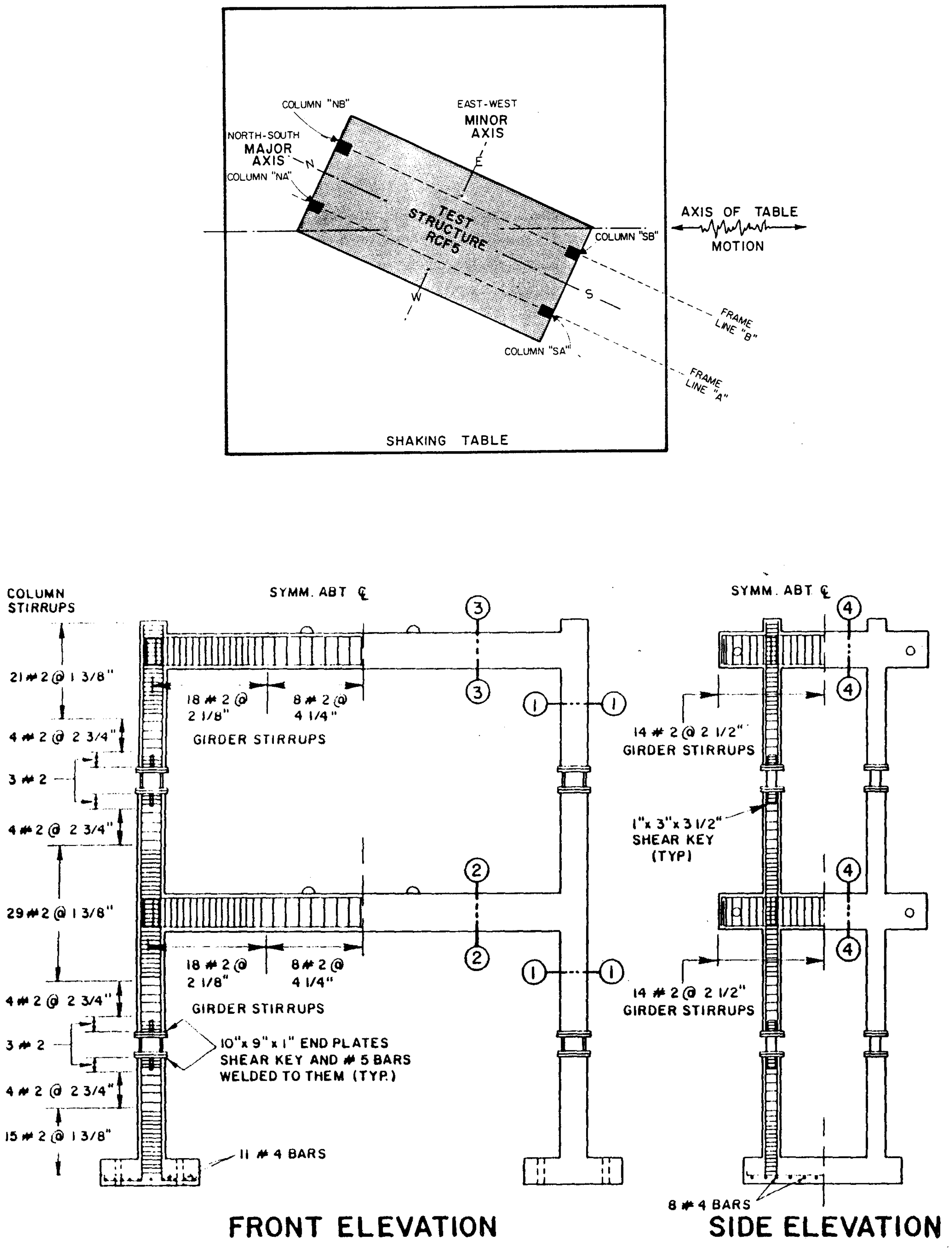

Figure 5. Plan and elevation of 1/3 sacle 2-story frame [Oliva, 1980]

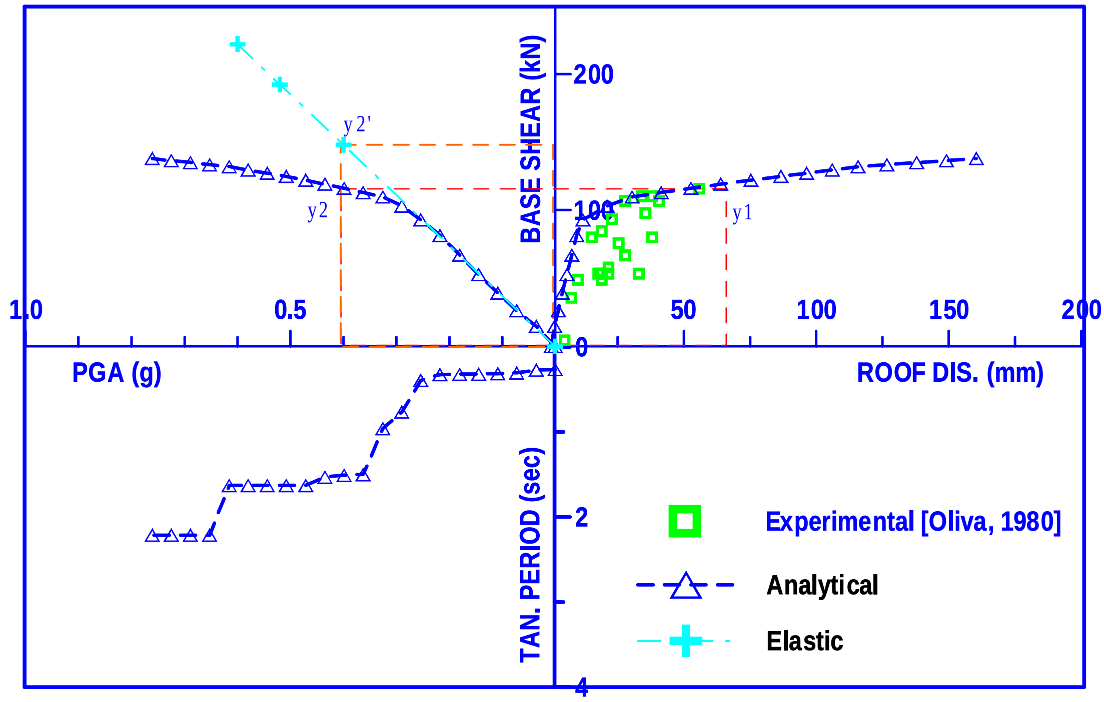

Figure 6. Vulnerability Diagram In Major Direction

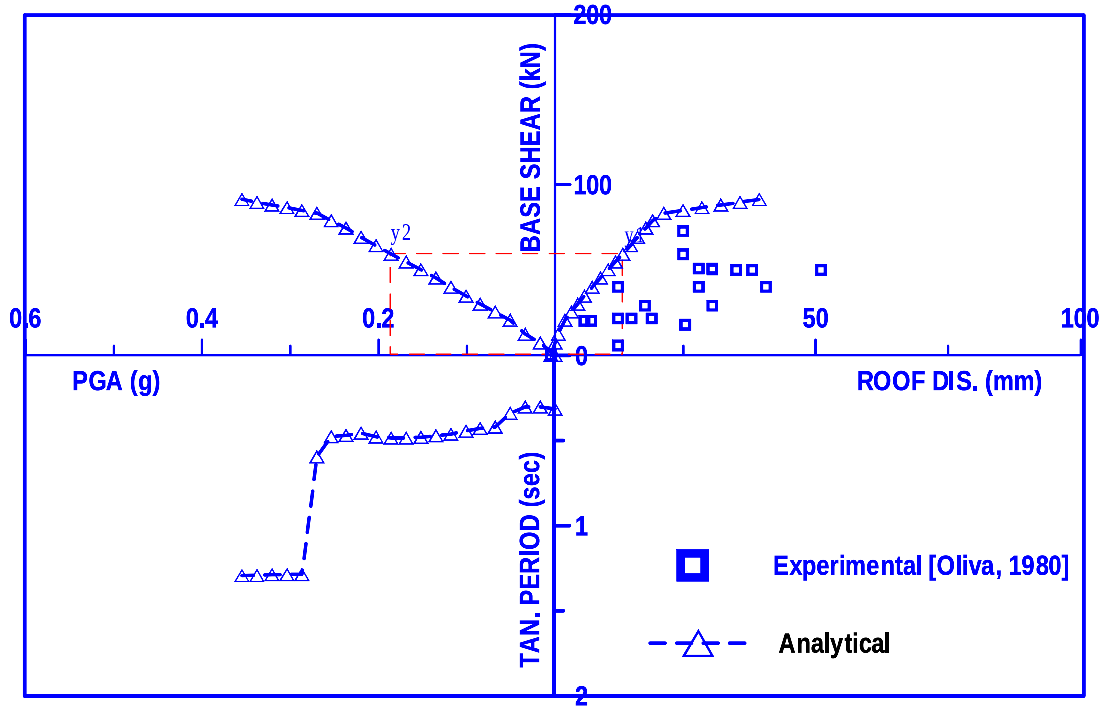

Figure 7. Vulnerability diagram in minor direction