5th Canadian Conf. Earthquake Engineering / Ottawa / 1987 / 5ème Conf. Canadienne Génie Sismique

Damage evaluation by static nonlinear analysis

S.K.Mital
Case Western Reserve University, Cleveland, Ohio, USA
N.D.Nathan, D.L.Anderson & S.Cherry
Department of Civil Engineering, University of British Columbia, Vancouver, Canada

ABSTRACT: Building codes provide for quasi-static analysis of structures under seismic lateral forces which assume inelastic action. It is desirable that the completed design should be analyzed to confirm that inelastic energy is absorbed as anticipated, and that the structure is not locally overstrained.

A static analysis procedure is presented, in which the code lateral load is increased incrementally until the deflection reaches a precalculated value, with plastic hinges being iserted as they form. The rotation in the hinges is noted and related to the damage.

1. INTRODUCTION

Building codes typically define the base shear for seismic design on the assumption that a considerable amount of energy will be dissipated inelastically. The National Building Code of Canada, for example, gives the base shear in terms of the factor K, which depends upon the structural form. The value of K is related to the ductility, or the capacity to absorb energy inelastically, associated with that structural form.

This base shear is then distributed vertically and a quasi-static analysis is made, which tacitly allows for the appropriate degree of inelastic action. If an elastic modal analysis is made, the results are scaled down to make the resulting base shear equal to the code value, again allowing for the inelastic action.

Implicit in these procedures is the assumption that energy will be dissipated inelastically in plastic hinges which will be distributed widely across the structural system, and that no particular hinges will be driven beyond their rotational capacity. This important assumption should be verified in structures of any significance by analysis of the mature design.

The object of this paper is to discuss one relatively cheap method of making such an analysis, which will identify the plastic hinges and provide an estimate of the rotational demands made upon them (Mital 1985).

The procedure should also be useful in the study of existing buildings for retrofit problems. In this case the seismic activity, and therefore the probability associated with different levels of rotational damage at points within the structure, can be estimated.

2. BASIS OF THE PROCEDURE

The unfactored seismic base shear is calculated and the lateral load distribution is determined according to the quasi-static code recommendations. The building is then analyzed under the factored gravity loads and the unfactored lateral load, and the tip deflection is noted.

The lateral load is then increased in steps until the tip deflection reaches some precalculated value, with plastic hinges being inserted in the structural model as they form. The rotation in the hinges in the final configuration is noted and related to the damage.

3. DETAILED DESCRIPTION OF THE BASIC PROCEDURE

1) The structure is analyzed elastically for appropriately factored gravity loads. Member forces and deflections are obtained.

2) The "lateral moment capacities" of the members are then determined. These are the actual moment capacities minus the gravity load moments; they are the available moment capacities for resistance of the lateral load moments.

3) The magnitude and distribution of the seismic force is determined according to the relevant building code. A quasi-static analysis is made of the structure under these unfactored loads, using the elastic stiffness matrix from step 1.

4) The lateral moment capacities are then divided by the member end moments from this lateral load analysis.

5) The largest value of this ratio gives the lateral load factor at which the first plastic hinge will form. Deflections and member forces from the lateral load analysis are multiplied by this factor and added to the gravity load analysis.

6) At the location of the first plastic hinge or hinges, an additional node is introduced with the same spatial coordinates, and the same translational degrees of freedom as the original node, but with an independent rotational degree of freedom. (In more sophisticated programs, it may be possible simply to introduce an additional rotational degree of freedom at the same node.)

7) The lateral moment capacity is recomputed, for all member ends which have not yet formed hinges, as the total moment capacity minus the current moments.

8) The stiffness matrix is reformed and the structure analyzed once more under the unfactored lateral loads.

9) Steps 4 to 8 are then repeated for successive hinge formations until either the required tip deflection is reached or the structure forms a mechanism. In each cycle the member forces and the deflections are added to the previous total.

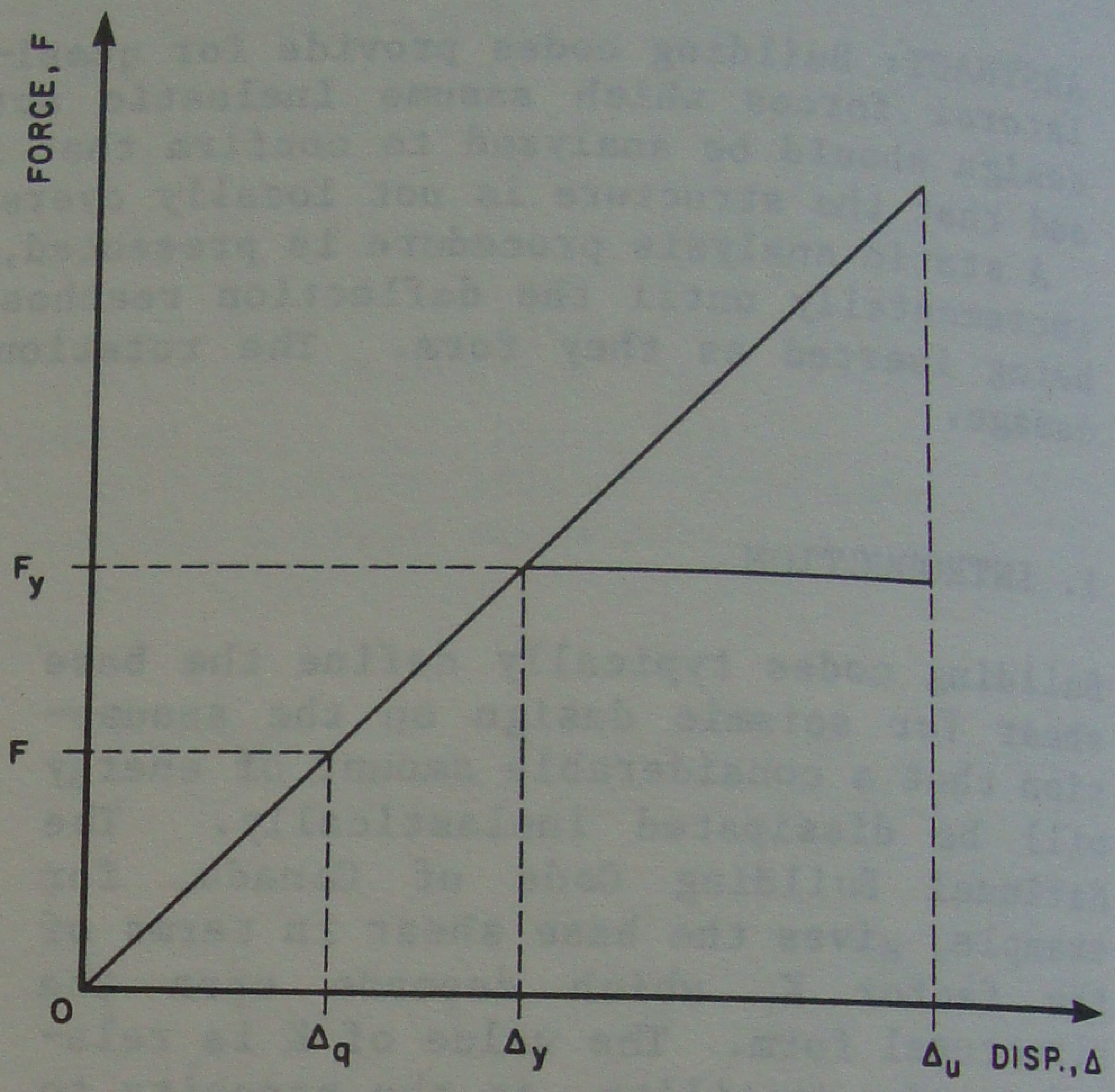
10) If a collapse mechanism is attained, the two nodes representing the last hinge are connected by a fictitious member of very small flexural stiffness, in order to maintain the stability of the computational algorithm. The remaining deflection to reach the required tip value is divided by the tip deflection of this final model under the unfactored lateral loads. The results from the final analysis are multiplied by this ratio and added to the total.

11) The hinge rotations are then calculated as the difference between the rotation of the two coincident nodes representing the hinge.

In unsymmetric structures, the procedure should be repeated with the lateral forces acting in the opposite direction.

4. CALCULATION OF THE REQUIRED ULTIMATE TIP DEFLECTION

The basic assumption in the seismic design of structures is that, whatever the degree of inelastic action, the ultimate tip displacement is essentially the same as that of a structure with equal initial stiffness, which remains elastic throughout the earthquake. This is illustrated in Fig. 1.



$$\Delta_y = \alpha\, \Delta_q$$
$$\Delta_p = \mu\, \alpha\, \Delta_q$$

Figure 1. Deflection Relations.

The ultimate tip deflection is thus given by

$$\Delta_u = \Delta_g + \alpha\, \mu\, \Delta_q \qquad (1)$$

where

Δ_g = the deflection under appropriately factored gravity loads
Δ_q = the deflection under unfactored earthquake lateral loads
α = the load factor
μ = the ductility.

For most codes of practice,

$$\alpha = \frac{\psi\, \alpha_Q}{\phi} \qquad (2)$$

where

ψ = the combination factor, if applicable

α_Q = the load factor for seismic loads

ϕ = the capacity reduction factor, or the combined effect of material resistance factors.

In the Canadian National Building Code,

ψ =
- 1 for the seismic load acting alone
- 0.7 for seismic load acting in combination with live load, or with loads resulting from volume changes or differential settlements
- 0.6 for seismic load acting in combination with loads from both the above sources.

α_Q = 1.5

CSA standard CAN3-A23.3-M84 for concrete structures provides for material resistance factors. The nominal resistance may be taken as 1.20 times the factored resistance, so that ϕ in eq. (2) may be taken as 1/1.20.

The National Building Code of Canada provides for the ductility to be demanded of the selected structural form through the factor K which appears in the base shear calculation. This factor appears to be related to the assumed available ductility approximately by the relationship

$$K\,\mu = 2.8$$

so that

$$\mu = 2.8/K \qquad (3)$$

Thus a concrete ductile frame structure in which the K-factor is 0.7, and the lateral deflection essentially arises from lateral forces alone (so that ψ=1), would lead to:

eq.(2): α = (1)(1.5)(1.2) = 1.8
eq.(3): μ = 2.8/0.7 = 4.0
eq.(1): $\Delta_u = \Delta_g + 7.2\,\Delta_q$

5. RELATIONSHIP BETWEEN HINGE ROTATION AND CURVATURE DUCTILITY DEMAND

To derive a relationship between the rotation that occurs in the hinge and the curvature, one needs to know the length of the hinge region. Studies (Mander 1984) suggest that, for solid reinforced concrete members, this length may vary from 0.35 to 0.65 times the overall depth. 0.5 has been recommended as an average value.

The curvature subsequent to yield, ϕ_p, is given by

$$\phi_p = \theta_p/.5\,t \qquad (4)$$

where

θ_p = rotation in hinge
= difference between rotation of coincident nodes at hinge.

t = overall depth of member.

The yield curvature is given by

$$\phi_y = M_y/EI_y \qquad (5)$$

where

M_y = yield moment, which may be taken as the moment to yield the reinforcement

EI_y = secant modulus of rigidity of member cross-section at yield.

EI_y may be taken as the cracked value.

Eq. (6) may be used in place of eq. (5) for greater accuracy:

$$\phi_y = \frac{\varepsilon_y}{d-c_y} \qquad (6)$$

where

ε_y = yield strain of steel

d = effective depth of member

c_y = neutral axis depth at yield of steel.

For the steel ratios likely to be used in seismic design ($\rho-\rho' < .015$) the value of c_y is given very closely by the linear design value, so that

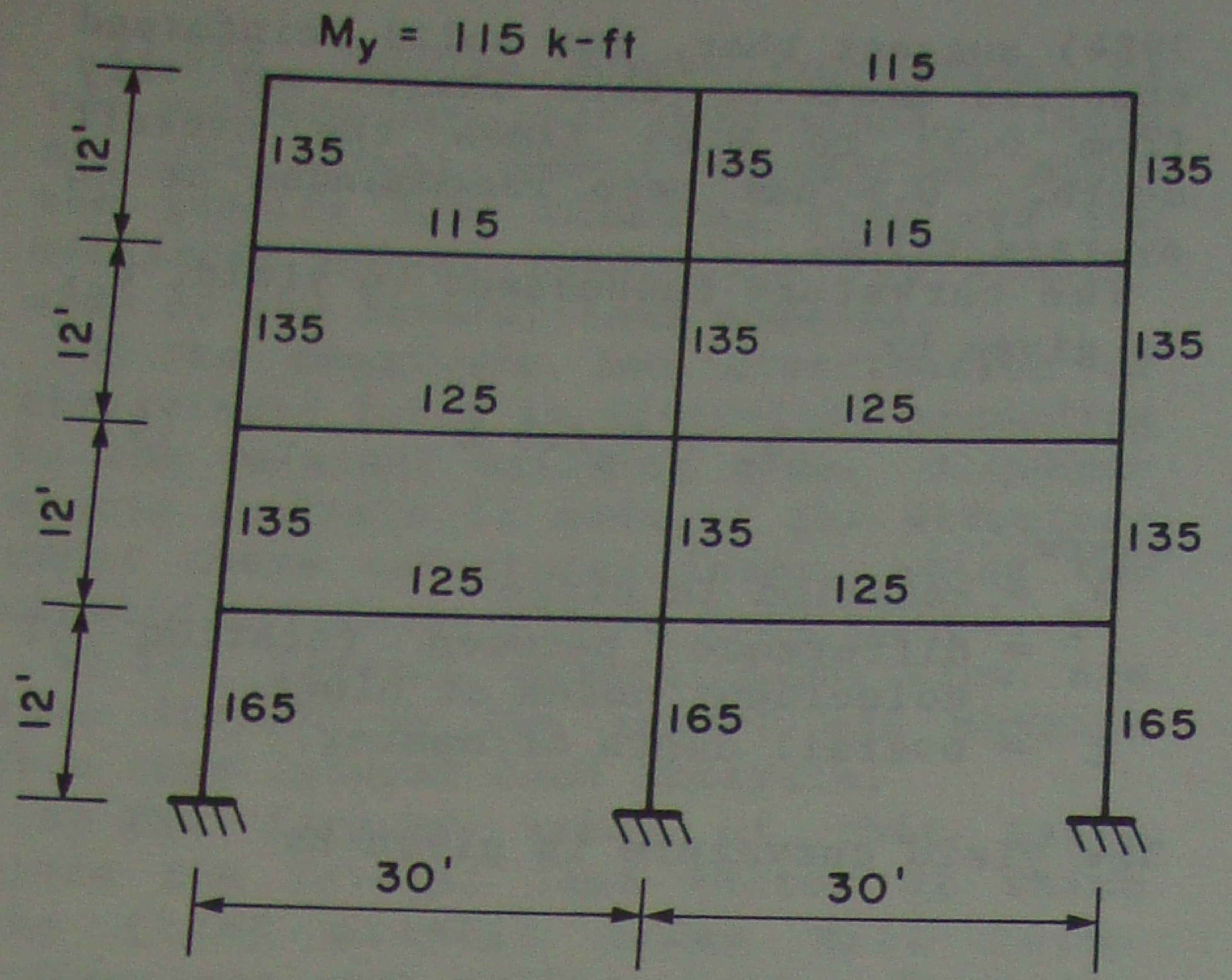
$$\phi_y = \frac{f_y}{E_s d\left(1-n\rho+\sqrt{(n\rho)^2 + 2n\rho}\right)} \qquad (7)$$

where $n = E_s/E_c$, the modular ratio.

ρ = reinforcement ratio

The curvature ductility is given by

$$\mu_k = 1 - \phi_p/\phi_y$$



E = 3760 ksi

Floor weight is 100 kips at all levels

Gravity load on all beams is 1.1k/ft

BEAMS	SIZE
First and second story	18" × 18"
Third and fourth story	15" × 18"
COLUMNS	20" × 20"

Figure 2. Test Structure No. 1.

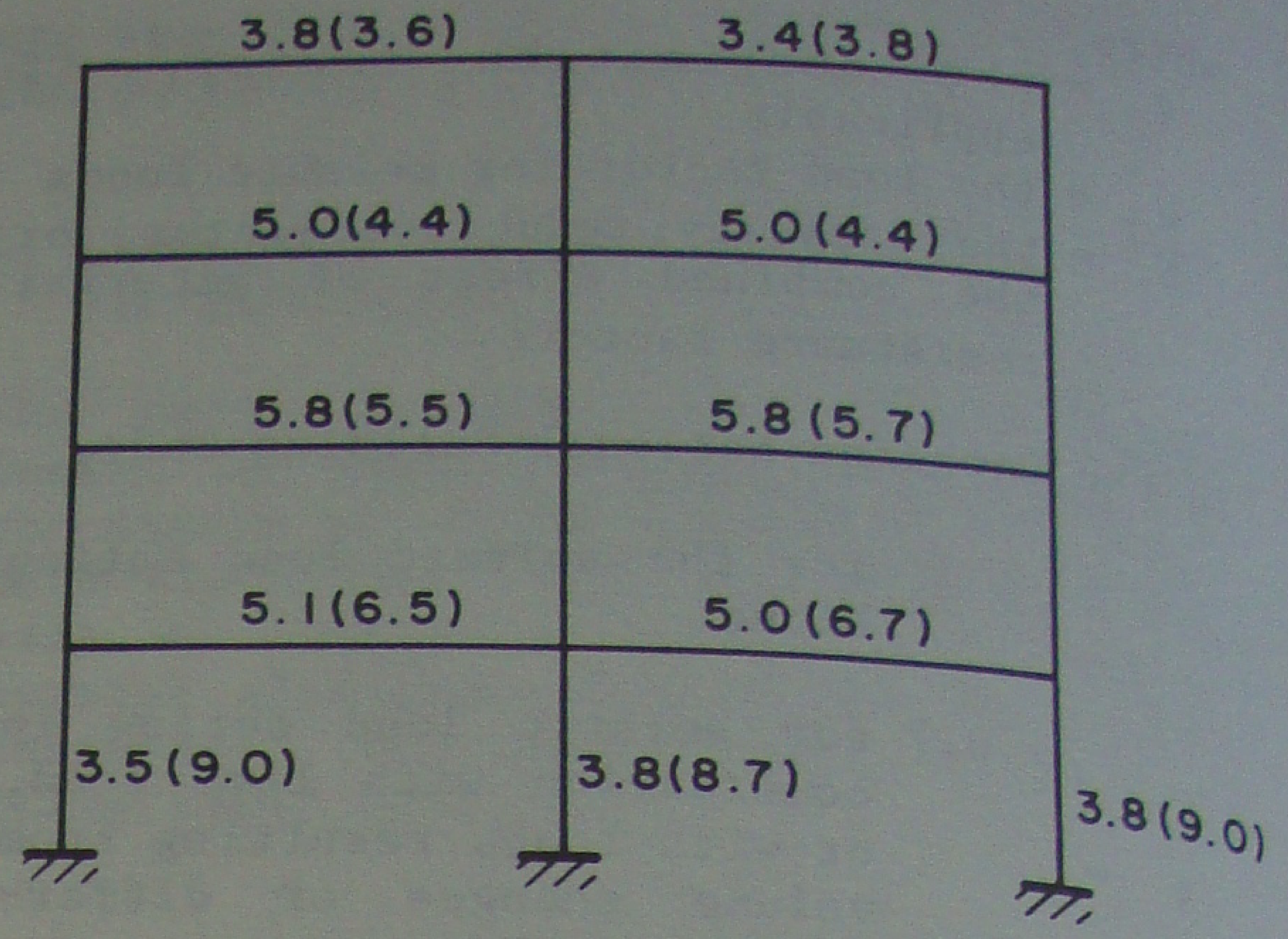


Figure 4. Static damage evaluation curvature ductilities (sensitivity indices).

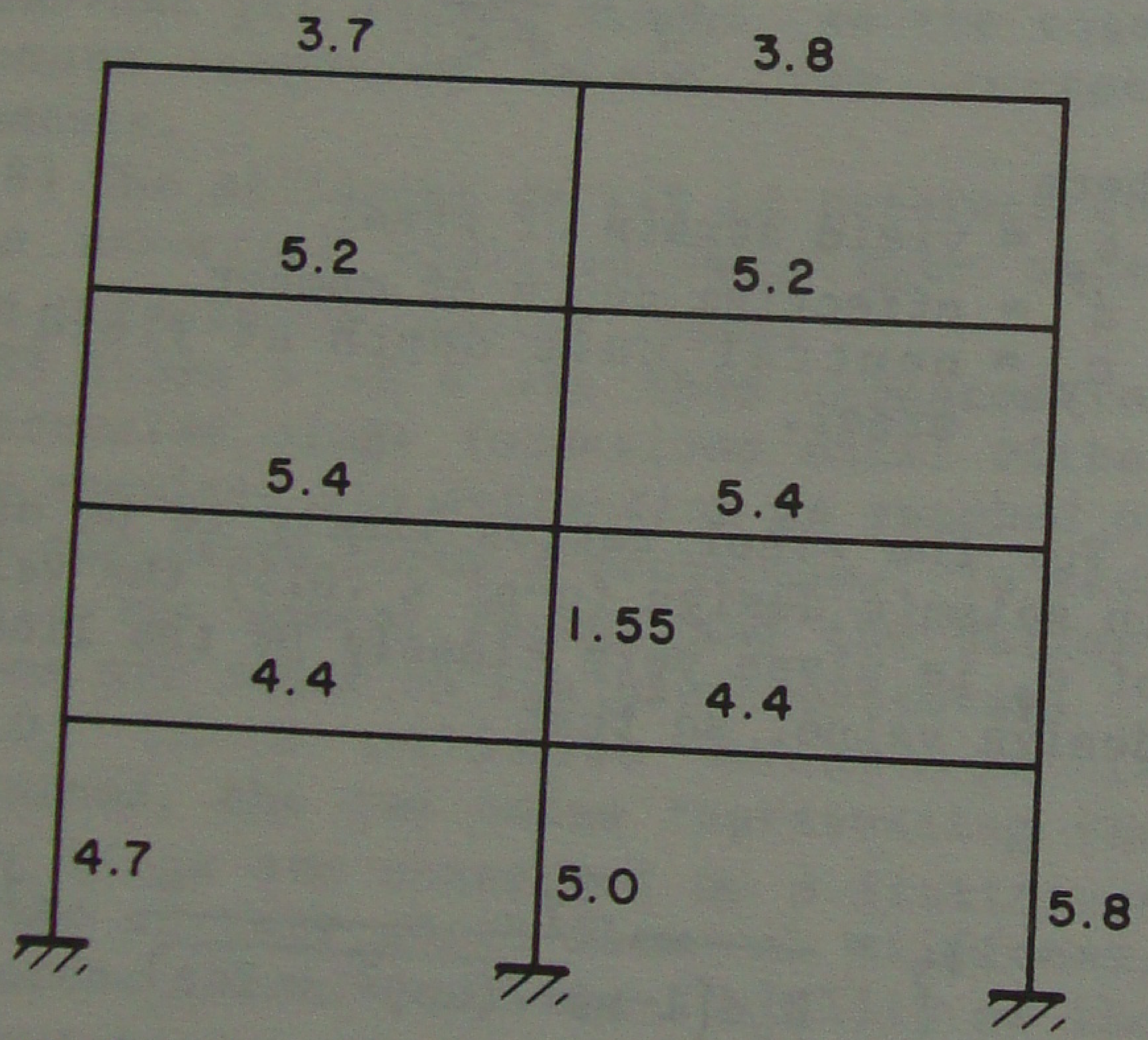
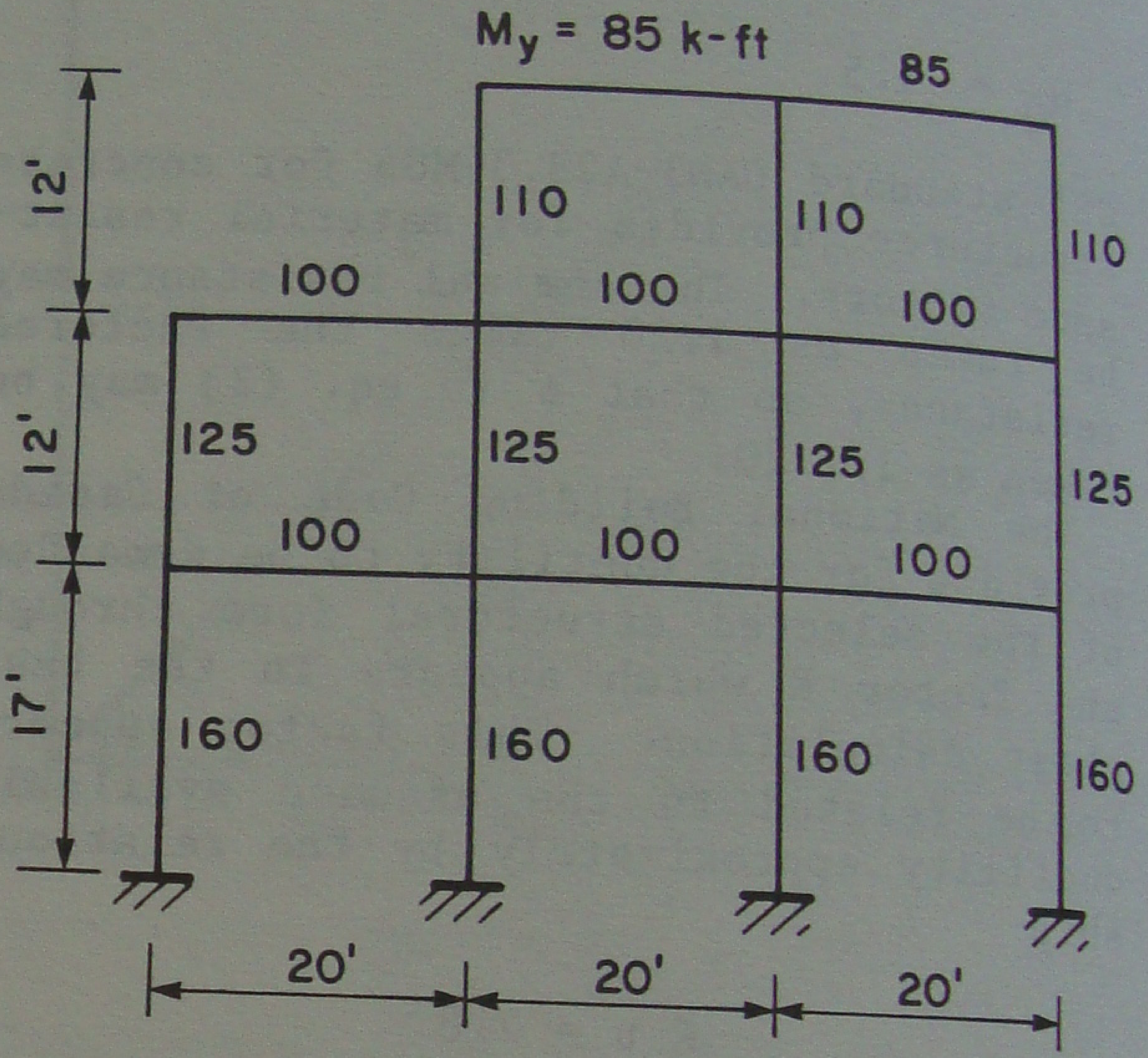


Figure 3. Average of time-step analysis (curvature ductilities).



Roof weight 85 kips

Floor weights 100 kips

E = 3600 ksi

Gravity load on all beams is 1.0 k/ft

BEAMS	17.7" × 19.7" (450 × 500)
COLUMNS	19.7" × 19.7" (500 × 500)

Figure 5. Test Structure No. 2.

6. SENSITIVITY

It is useful to determine the sensitivity of the ductility demands to the ultimate tip deflection, which is to say, to the magnitude of the earthquake. For this purpose, the ultimate tip deflection is increased by 10% of the calculated ultimate value, and the increase in the curvature ductilities is calculated.

A sensitivity index is then defined as 10 times this change in the curvature ductility demand. A high value suggests that the calculated ductility demand for that member is less reliable.

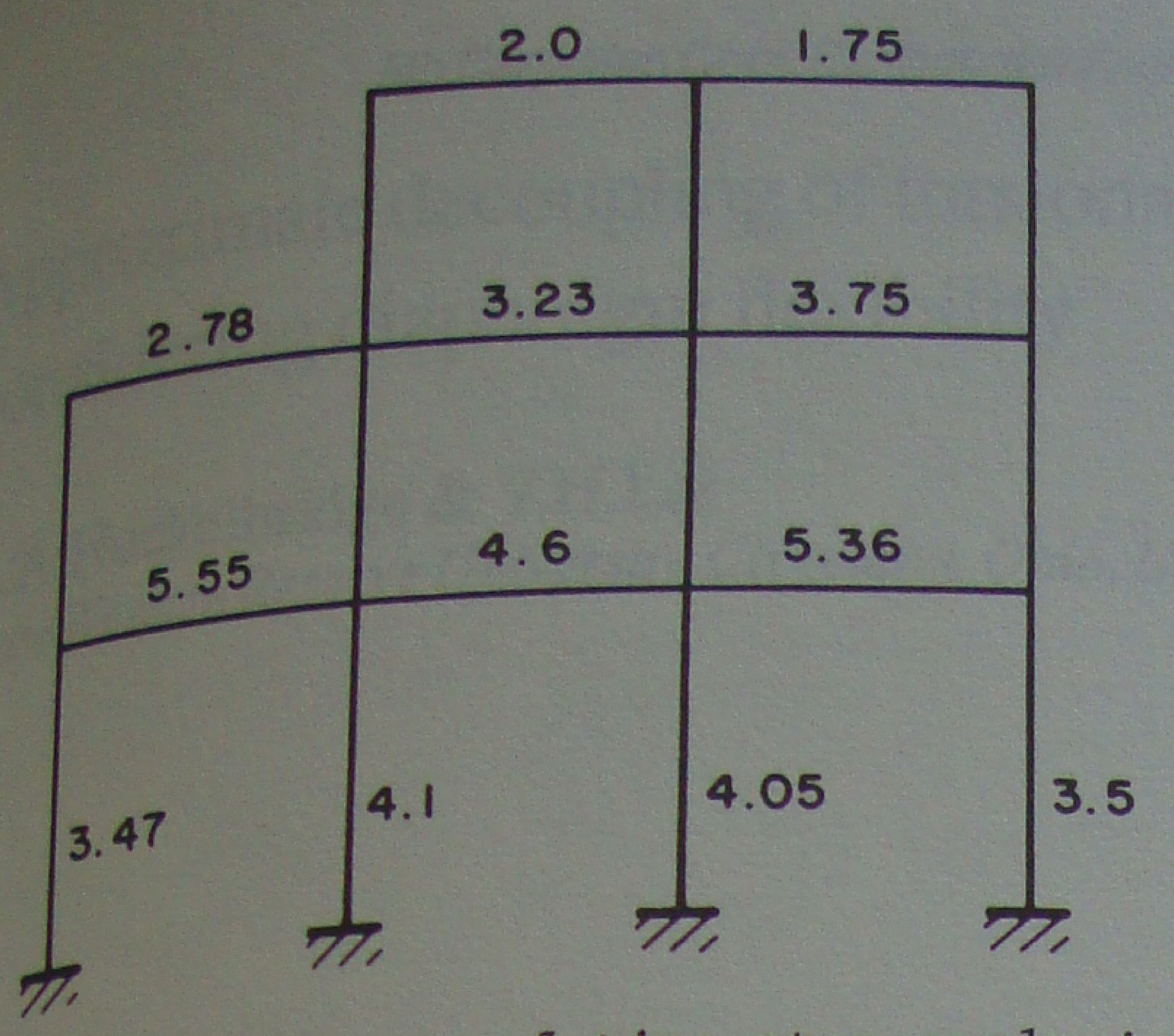


Figure 6. Average of time-step analysis (curvature ductilities).

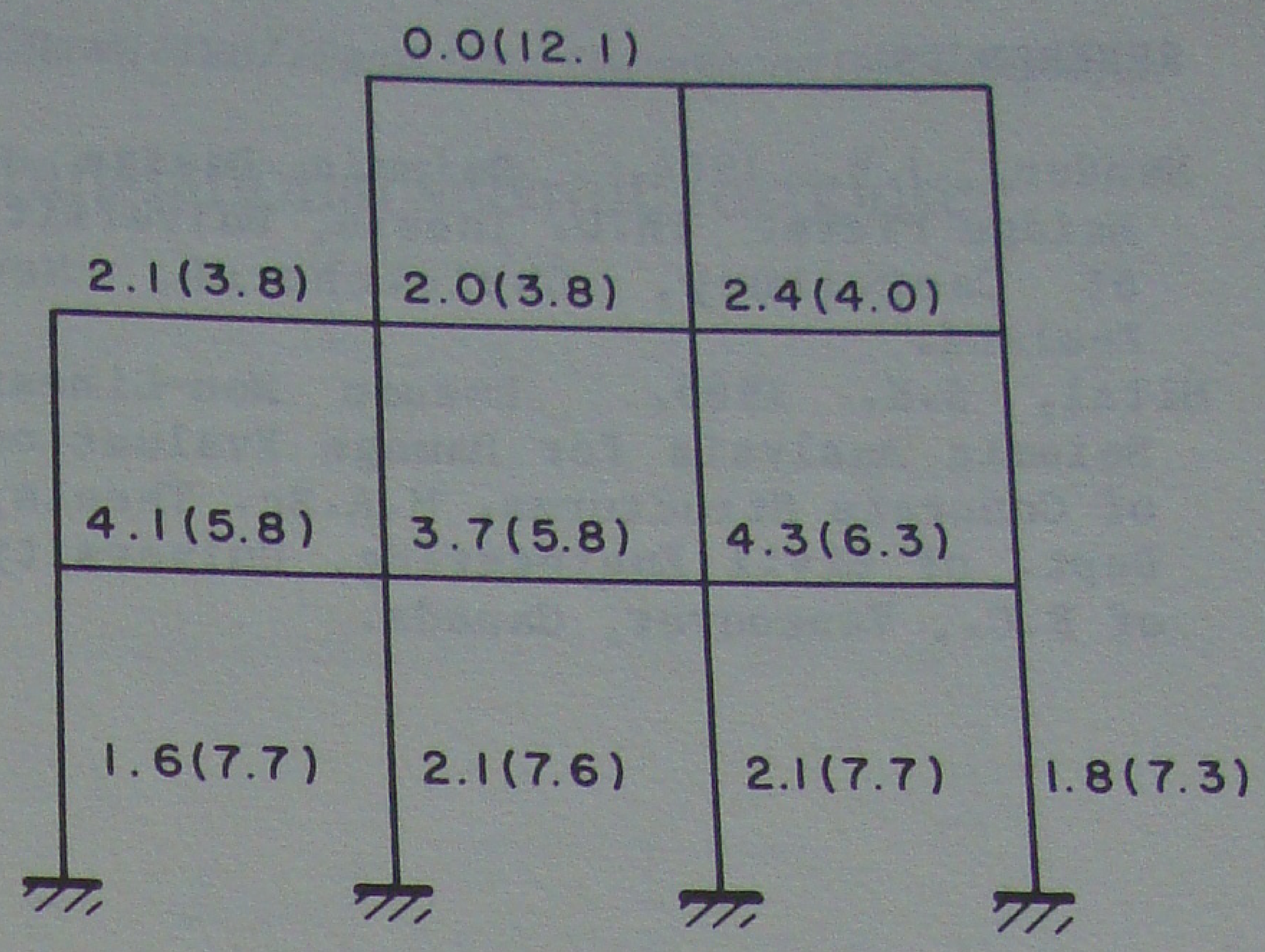


Figure 7. Static damage evaluation curvature ductilities (sensitivity indices).

7. EXAMPLES

Two of the test structures analyzed are shown in Figs. 2 and 5. In each case the analysis was based on cracked sections, whose moment of inertia was taken as half the gross moment of inertia of the concrete sections for the columns, and one third of the gross moment of inertia for beams. With the safety parameters in use at the time the work was done, the tip deflection was 8.3 times the deflection under unfactored seismic forces.

The structures were analysed using the nonlinear time-step program DRAIN-2D to determine the "correct" results. Four earthquake records were used: El Centro 1940 NS, El Centro 1940 EW, Taft 1952 N21E, and Taft 1952 S69W; these ground motions were scaled to give peak accelerations equal to the value believed to correspond to the quasi-static analysis of the National Building Code of Canada. A ten-second segment of each record, which resulted in the maximum structural response, was used (generally the first 10 seconds). The results from these four records were averaged and taken to represent the "true" results.

The curvature ductilities were obtained by assuming that the maximum values of the plastic hinge rotations were distributed over hinge lengths of 0.5 times member depth. The values for all members which formed plastic hinges are shown in Figs. 3 and 6.

Figures 4 and 7 show the results obtained by the static nonlinear analysis. The sensitivities are shown in parenthesis after each value.

It will be seen that the comparisons are quite good for the regular symmetric structure of Fig. 2. Where the errors are greater, the sensitivities are also greater, warning the designer that these areas are somewhat open to question.

For the structure of Fig. 5, which is unsymmetric as well as having weaker columns in the ground floor, the results are less promising. The differences in ductility demands as indicated by the static analysis and by DRAIN-2D are large, but it can be said that the static procedure gives an indication of the damage pattern.

8. CONCLUSIONS

The static nonlinear procedure for damage evaluation that was examined shows promise for regular structures. It is rather suspect in the case of unsymmetric structures or when there is a weak storey. It is intended to make further studies of large structures to investigate these points.

As indicated by the sensitivity indices, the results are often quite strongly dependent on the selected tip deflection, and this point, too, merits further study.

9. ACKNOWLEDGEMENTS

This work was funded by the National Research Council of Canada.

REFERENCES

Mander, J.B. 1984. Seismic Design of Bridge Piers. Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.

Mital, S.K. 1985. Pseudo Non-Linear Seismic Analysis for Damage Evaluation of Concrete Structures. M.A.Sc. Thesis, Dept. of Civil Engineering, University of B.C., Vancouver, Canada.