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NONLINEAR SEISMIC ANALYSIS OF SETBACK REINFORCED CONCRETE FRAMES

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SUMMARY

This paper presents a numerical method for the seismic analysis of reinforced concrete frames having setback. Earthquake ground motion is considered in the form of acceleration time history. The governing dynamic equations of motion being nonlinear have been solved using the stiffness matrix method together with the Wilson-θ method. In establishing the structure stiffness matrix the P-Δ effect has been also considered. Two hysteresis models were chosen to represent the force—deformation relationship of reinforced concrete members. Different cases of symmetric and nonsymmetric frames were solved in order to study the effect of different parameters on the response.

INTRODUCTION

During earthquakes structures may behave inelastically according to the properties of the structure and severity of the earthquake. Even moderate earthquakes may be expected to produce inelastic deformations in typical buildings. Therefore, a knowledge of the expected inelastic deformations during an earthquake excitation is essential for the proper design of buildings. A considerable amount of work has been carried out by several investigators to estimate the nonlinear behaviour of buildings during strong earthquakes. Several models have been proposed to describe the nonlinear behaviour of material under the effect of cyclic loading (Refs. 1-4). Generally, these models have been used to study the behaviour of uniform structures. Research work on the nonlinear seismic analysis of irregular reinforced concrete structures is scarce. Berg(Ref.5) made an exploratory study of the effect of setbacks in multistory buildings by analyzing a vertical shear-deflection stepped cantilever beam. Jhaveri (Ref.6) made an extensive study of the elastic behaviour of structures with setbacks. Multistory steel buildings having setback have been also studied using a bilinear model (Refs. 7,8). In 1974 Varma (Ref.9) has performed elastic and elasto-plastic analyses for steel frame with setback. In 1983, an eight—story reinforced concrete frame with setback has been studied using the Q-Hyst model(Ref.10).

The objectives of this paper are as follows:(i) to present a numerical procedure for the nonlinear seismic analysis of reinforced concrete frames with setbacks using a time—history analysis and (ii) to study the effect of different parameters on the response of such structures.

BEHAVIOUR OF RENFORCED CONCRETE ELEMENTS

Basic Concepts The use of ultimate strength design method has many advantages, amongst which is the capability to assess the ductility of the structure in the post-elastic range. The stress-strain relationships used in this paper are an elasto-plastic curve for steel and that recommended by CEP-FIP for concrete (Ref.11).

Member Idealization To establish properties of reinforced concrete member, a member is assumed to consist of a flexible elastic line element, two nonlinear rotational springs at each end of the elastic element, and two rigid zones outside of the rotational springs as shown in Fig.1. Details for properties of these elements are given in (Ref.12). For the flexural element a trilinear primary moment-rotation curve has been used and is based on moments and rotations at tensile cracking of concrete, yielding of tensile steel and crushing of concrete at compression fibers. Deterioration of bond between concrete and steel within a joint core is simulated by the rotation of joint spring according to (Ref.13). To represent the force-deformation relation under stress reversals two hysteresis models are used. The first is the well known Takeda's model(Ref.4) while the second is a simplified version of it. The latter has a bilinear primary curve and is used to simulate the member—end rotation due to bond slip within a joint core.

METHOD OF ANALYSIS

The governing dynamic equations of motion for a multidegree-of-freedom system can be put in the following incremental form:

$$[M]\{\Delta\ddot{u}_i\} + [C]\{\Delta\dot{u}_i\} + [K]\{\Delta u_i\} = -[M]\{\Delta\ddot{y}_{si}\} \qquad (1)$$

in which [M] , [C] and [K] are, respectively, the mass, damping and stiffness matrices. $\{\Delta u_i\}$, $\{\Delta\dot{u}_i\}$, $\{\Delta\ddot{u}_i\}$ and $\{\Delta\ddot{y}_{si}\}$ are vectors of incremental relative displacement, velocity, acceleration and ground acceleration. The system properties are determined using the stiffness matrix method (Ref.12). The effect of gravity loads (P—Δ) effect has been considered in establishing the structure stiffness matrix.

The solution of the nonlinear dynamic equations of motion is obtained using the step-by-step linear acceleration method with extended time step known as the Wilson—θ method. Such method is unconditionally stable (Ref.12).

This method of solution is implemented into a computer program to be run on an IBM computer. The program calculates the maximum displacements, velocities and accelerations at floor levels in addition to the maximum base shear and overturning moments. Besides, maximum response values are obtained for each frame member in terms of moments, rotations and ductility of flexural elements and joint springs (Ref. 12).

NUMERICAL RESULTS

Cases Considered The class of reinforced concrete frames considered in this paper are of the type shown in Fig.2. These represent frames with symmetric and nonsymmetric setback. The effect of different parameters on the response of such frames has been studied. These parameters include the setback level ratio "L_s", the variation of beam properties, earthquake intensity and the type of nonlinear model used. This leads to studying four groups of frames including a total of seventeen cases. The first group includes seven frames with different "L_s" values. The second group contains seven frames having variable beam properties. Frames in these two groups were subjected to the N-S component of the 1940 El-Centro earthquake. The third group consists of two frames subjected to 50% of El-Centro earthquake. The fourth group has only one frame that has been studied in (Ref.10) using the Q-Hyst nonlinear model. Details of properties for considered frames (e.g., geometry, material properties, concrete dimensions, reinforcement and primary moment—rotation relations) may be found in (Ref.12). A large amount of results were obtained. However, only samples of these results will be presented hereinafter.

First Group Results Figs.3 and 4 show that no abrupt changes in the displacements occur at the setback level. The tower portion exhibits larger displacement as "L_s" decreases compared with the uniform frame (L_s=1.0). The maximum interstory drift occurs at the intermediate floors for L_s 0.375 as shown in Figs. 5 and 6. Base shear increases and base moment decreases with decreasing values of L_s, Fig.7. At upper floors the ductility demand for beams increases as L_s decreases, Fig.8. As L_s decreases ductility demand for external columns may exhibit larger values, Figs.9 and 10.

Second Group Results The results for this group are compared with the corresponding cases in the first group. As for displacements the change in beam properties leads to smaller displacments, Fig.11 which is contrary to uniform frames. Similar trend is observed for interstory drift, Fig.12, and for beam ductility demand, Fig.13. However, columns ductility demand increases for setback frame with varying beam properties, Fig.14.

Third Group Results The results for this group are compared with those of the first group. Displacements and column ductility demand are shown in Figs. 15 and 16, respectively. It can be easily noticed that the change in response is not linear.

Fourth Group Results It has been found that Takeda model yields lower results that that using Q-Hyst model by 10-14% for ductility demand and displacements, respectively.

CONCLUSIONS

Seismic analysis of frames with setback based on the presented numerical method has showed that such frames should be treated with great care. Parameters that affect the response of such frames include the setback ratio, the beam properties, earthquake intensity and the nonlinear model used in the analysis.

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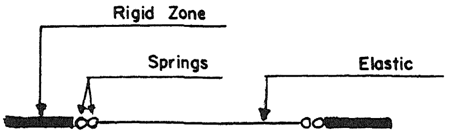

Fig. 1 Idealized Member.

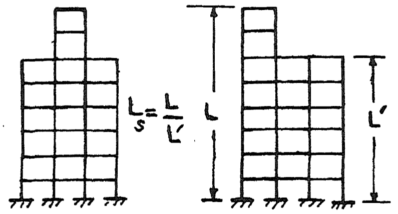

Fig. 2 Class of Frames Considered

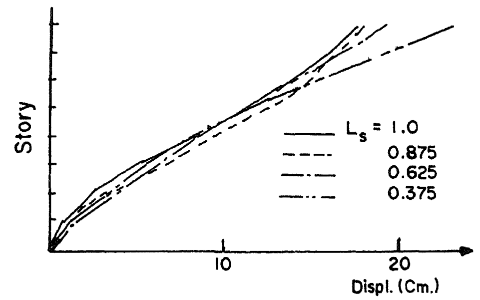

Fig. 3 Story Displacement for Symmetric Frames.

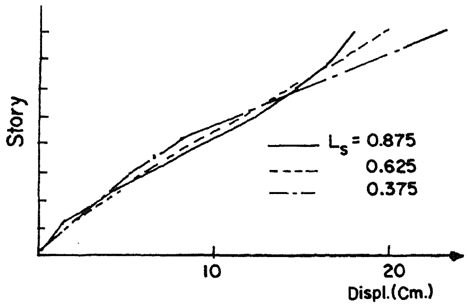

Fig. 4 Story Displacement for Unsymmetric Frames.

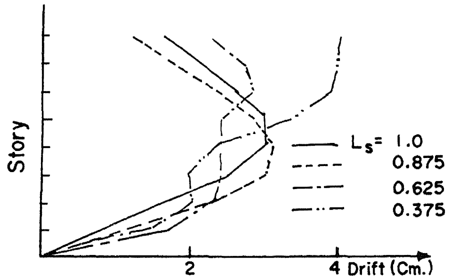

Fig. 5 Interstory Drift for Symmetric Frames.

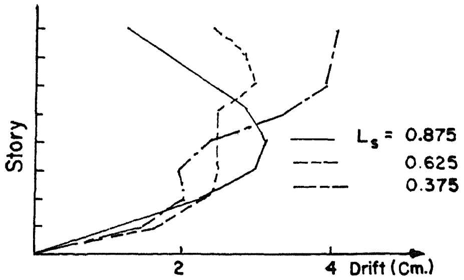

Fig. 6 Interstory Drift for Unsymmetric Frames.

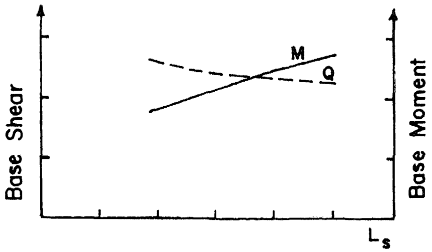

Fig. 7 Variation of Base Shear and Moment with L_s

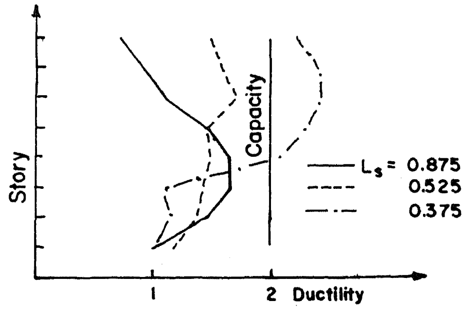

Fig. 8 Beams Ductility Demand for Unsymmetric Frames.

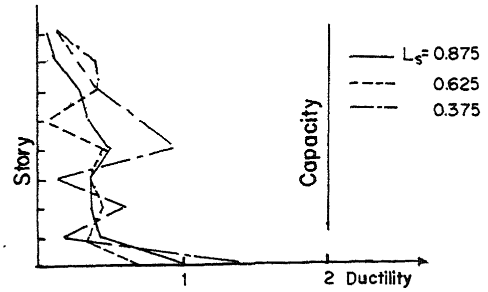

Fig. 9 Columns Ductility Demand for Symmetric Frames.

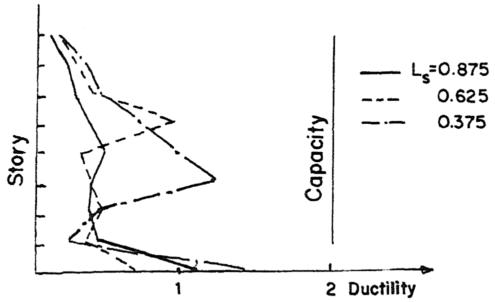

Fig. 10 Columns Ductility Demand for Unsymmetric Frames.

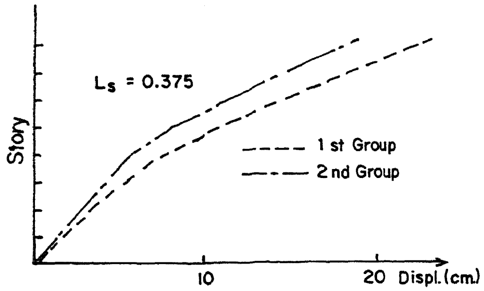

Fig. 11 Effect of Beams on Displacements.

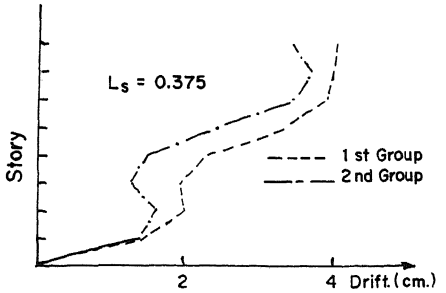

Fig. 12 Effect of Beams on Drift

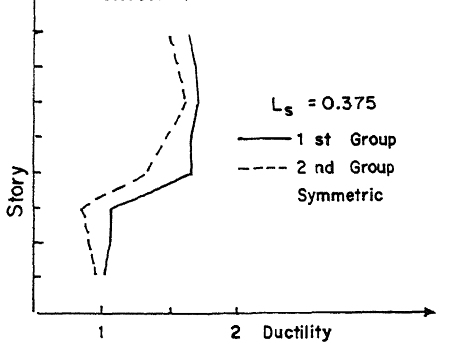

Fig. 13 Effect of Beams on Ductility of Beams.

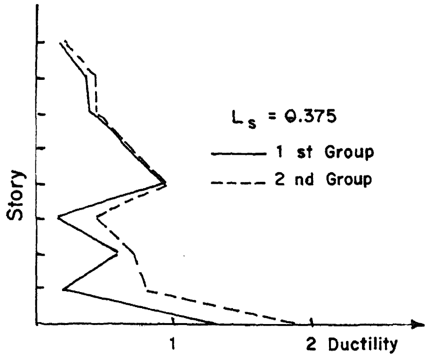

Fig. 14 Effect of Beams on Ductility of Columns.

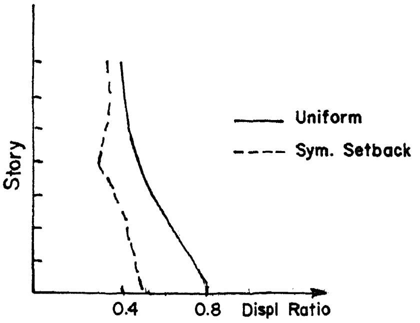

Fig. 15 Effect of Earthquake Intensity on Displacements

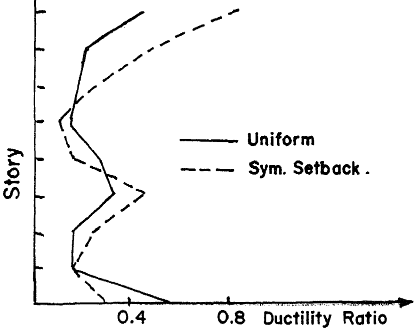

Fig. 16 Effect of Earthquake Intensity on Columns Ductility.