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BEHAVIOR OF HIGH-STRENGTH CONCRETE STRUCTURES UNDER SEISMIC LOADS

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ABSTRACT

The purpose of this study was to investigate the applicability of high-strength concrete to complete structures subjected to cyclic loads as in the case of seismic excitation. The nonlinear dynamic behavior of frames made of normal strength and high-strength concrete and subjected to combined horizontal and vertical earthquake motions was examined. In order to study the nonlinear dynamic response, a fiber model using the incremental stiffness approach was developed. In this model the stress and strain are monitored through time at each fiber of several cross sections along each member. The tangent moduli for steel and concrete resulting from the assumed nonlinear stress-strain relationships are then used to assemble a tangent stiffness matrix for the structure at each step. Several models for the concrete and the steel were first reviewed and compared. The effect of these models on the curvature relationship for a cross section was investigated and results were compared to experimental data. The process was repeated for simple members under static cyclic loading. Finally the fiber model was used to study the nonlinear dynamic response of frames to earthquake base motion. A one-bay three-story building frame subjected to the El Centro earthquake record was used for this purpose.

KEYWORDS

High-strength concrete; Fiber model; Nonlinear dynamic response;
Seismic loads; Cyclic loads; El Centro earthquake; Incremental stiffness approach;
Constant average acceleration; Hysteretic behavior;
Tangent modulus

INTRODUCTION

For the purpose of this work reinforced concrete can be divided into two categories : Normal Strength Concrete (NSC), with a compressive strength of up to 6000 psi (ACI-363, 1984) and High-Strength Concrete (HSC) with a strength higher than 6000 psi. NSC and HSC have not only different values of the compressive strength but also different mechanical behavior as shown in laboratory tests (Carrasquillo *et al.*, 1981). Most of the work done to date on the seismic behavior of reinforced concrete structures has been concerned with Normal Strength Concrete. There is a need to explore further the applicability of high strength concrete in seismic areas assessing its potential advantages and disadvantages over NSC.

MATERIAL MODELS

When reinforced and prestressed concrete structural members are subjected to their ultimate load carrying capacity, part of the concrete in the compression zone has stresses which are beyond the peak and in the descending portion of the stress-strain curve. Thus a rational prediction of the behavior of structural members near failure requires an accurate knowledge of the complete stress-strain curve. The stress-strain curves of concrete under cyclic loading are normally enveloped by the stress-strain curve under monotonic loading. The stress-strain relationship for monotonic and cyclic loading recommended by Mander *et al.*, (1988) for the case of NSC was modified in this work to represent the hysteretic behavior of high-strength concrete (Armendariz, 1995). The stress-strain relationship for monotonic loading recommended by Mander *et al.*, (1988) is given by the following expression:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \tag{1}$$

where

f_c = Compressive strength at a given strain ε_c

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \tag{2}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + R\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \tag{3}$$

$$r = \frac{E_c}{E_c - E_{sec}} \tag{4}$$

For normal strength concrete:

$$E_c = 57000\sqrt{f'_{co}} \qquad (f'_{co} \leq 6000 psi) \tag{5}$$

and for high-strength concrete ACI-363 (1984):

$$E_c = (40000\sqrt{f'_{co}} + 1000000) \qquad (3ksi \leq f'_{co} \leq 12ksi) \tag{6}$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \tag{7}$$

$$f'_{cc} = f'_{co} + k_1 f'_l \tag{8}$$

where

$f'_{cc}, \varepsilon_{cc}$ = stress and strain at peak for confined concrete,

$f'_{co}, \varepsilon_{co}$ = stress and strain at peak for unconfined concrete,

E_c = initial tangent modulus of elasticity of concrete,

f'_l = lateral confining pressure on concrete,

k_1 = concrete strength enhancement coefficient due to lateral confining pressure,

R = factor used in calculating strain, ε_{cc}, $R = 5$ is recommended by Mander *et al.*, (1988)

ε_{co} can be computed using the following equations recommended by Setunge *et al.*, (1993)

$$\frac{E_c \varepsilon_{co}}{f'_{co}} = \frac{4.26}{\sqrt[4]{f'_{co}}} \quad \text{Crushed aggregate} \tag{9}$$

$$\frac{E_c \varepsilon_{co}}{f'_{co}} = \frac{3.78}{\sqrt[4]{f'_{co}}} \quad \text{Gravel aggregate} \tag{10}$$

Compression unloading

In order to compute the compressive unloading branch of the curve the plastic strain ε_{pl} must be obtained. The procedure to compute the plastic strain is:

$$\varepsilon_a = a\sqrt{\varepsilon_{un}\varepsilon_{cc}} \tag{11}$$

where

$$a = \frac{\varepsilon_{cc}}{(\varepsilon_{cc} + \varepsilon_{un})} \tag{12}$$

or

$$a = \frac{0.09\varepsilon_{un}}{\varepsilon_{cc}} \tag{13}$$

ε_{cc} = strain corresponding to the peak stress f'_{cc}

$(\varepsilon_{un}, f_{un})$ = strain and stress corresponding to a reversal point on unloading

In order to use equation (11) take the greater of equation (12) or (13).
The plastic strain is then given by

$$\varepsilon_{pl} = \varepsilon_{un} - \frac{(\varepsilon_{un} + \varepsilon_a)f_{un}}{(f_{un} + E_c\varepsilon_a)}$$

$$f_c = f_{un} - \frac{f_{un}xr}{r - 1 + x^r} \tag{14}$$

in which

$$r = \frac{E_u}{E_u - E_{sec}} \tag{15}$$

$$E_{sec} = \frac{f_{un}}{\varepsilon_{un} - \varepsilon_{pl}} \; ; \quad E_u = b\,c\,E_c \; ; \quad b = \frac{f_{un}}{f'_{cc}} \geq 1 \; ; \quad c = \left(\frac{\varepsilon_{cc}}{\varepsilon_{un}}\right)^{0.5} \leq 1 \tag{16}$$

$$x = \frac{\varepsilon_c - \varepsilon_{un}}{\varepsilon_{pl} - \varepsilon_{un}} \tag{17}$$

Tensile unloading

The effect of preloading in compression on the tension strength of concrete for HSC is not available. However, the results obtained by Morita and Kaku (1975) were used in this project. On unloading from the compressive branch, the tension strength becomes:

$$f_t = f'_t\left(1 - \frac{\varepsilon_{pl}}{\varepsilon_{cc}}\right) \tag{18}$$

if $\varepsilon_{pl} > \varepsilon_{cc}$ then $f_t = 0$. The stress-strain relationship is

$$f_t = E_t(\varepsilon_c - \varepsilon_{pl}) \tag{19}$$

where

$$E_t = \frac{f_t}{\varepsilon_t} \tag{20}$$

$$\varepsilon_t = \frac{f'_t}{E_c} \tag{21}$$

When the tensile strain is exceeded, cracks open and the tensile strength of concrete for all subsequent loadings is assumed to be zero.

Reloading branches

A linear stress-strain relation is assumed between $(\varepsilon_{ro}, f_{ro})$ and $(\varepsilon_{un}, f_{new})$.

$(\varepsilon_{ro}, f_{ro})$ = strain and stress corresponding to a reloading point

$(\varepsilon_{un}, f_{new})$ = strain and stress corresponding to the transition point

$$f_c = f_{ro} + E_r(\varepsilon_c - \varepsilon_{ro}) \tag{22}$$

The stress f_{new} is given by

$$f_{new} = 0.92 f_{un} + 0.08 f_{ro} \tag{23}$$

where

$$E_r = \frac{f_{ro} - f_{new}}{\varepsilon_{ro} - \varepsilon_{un}} \tag{24}$$

A third degree polynomial is used as a transition curve between the linear stress-strain relationship and the monotonic stress-strain curve. The stress-strain relationship is given by

$$f_c = A_2 + B_2 X + C_2 X^2 + D_2 X^3 \tag{25}$$

where

$$X = (\varepsilon_c - \varepsilon_{re}) \tag{26}$$

$$A_2 = f_{re} \tag{27}$$

$$B_2 = E_{re} \tag{28}$$

$$C_2 = \left[f_{new} - (f_{re} + E_{re} X_o) - D_2 X_o^3\right]\frac{1}{X_o^2} \tag{29}$$

$$D_2 = \frac{(E_r + E_{re})X_o - 2(f_{new} - f_{re})}{X_o^3} \tag{30}$$

The common return strain ε_{re} is assumed to be

$$\varepsilon_{re} = \varepsilon_{un} + \frac{f_{un} - f_{new}}{E_r\left(\frac{f'_{cc}}{f'_{co}}\right)} \tag{31}$$

$$X_o = \varepsilon_{un} - \varepsilon_{re} \tag{32}$$

E_{re} is the common return point tangent modulus. $(\varepsilon_{re}, f_{re})$ = strain and stress corresponding to a common return point. Figure 1 shows the stress-strain curve including unloading and reloading branches.

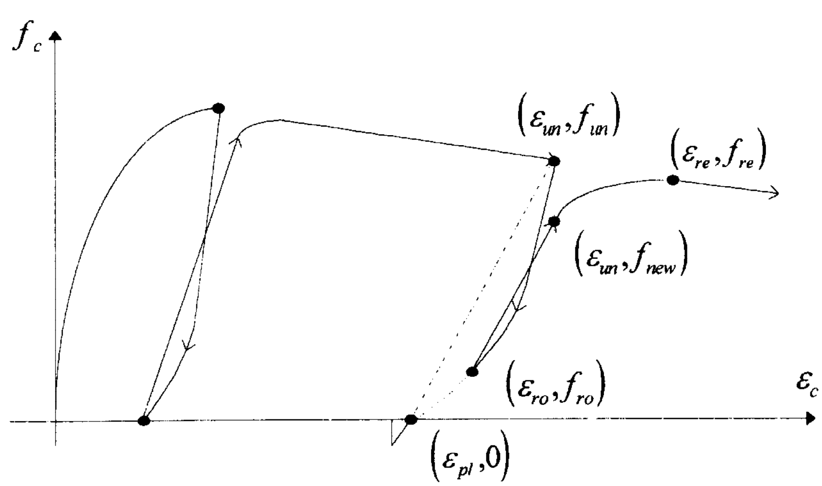

Fig. 1. Stress-strain curve for concrete

The stress-strain curve for the reinforcing steel is also important to predict the behavior of reinforced concrete structures. Experimental work has shown that the behavior of reinforcing steel under cyclic loading is highly nonlinear. Based on a number of parametric studies comparing the results for various steel models (Armendariz, 1995) a modified bilinear stress-strain curve was used in this work to represent the hysteretic behavior of the reinforcing steel.

MEMBER LEVEL STUDIES

The fiber model along with the incremental stiffness formulation was used to study the behavior of NSC and HSC beams (Armendariz, 1995). The experimental data by Brown (1970), Popov *et al.*, (1972), and Darwin *et al.*, (1988) were used to evaluate the model. The first set of data correspond to NSC and the other to HSC. For the fiber model analysis the member was divided into nineteen sections and each section into twenty fibers. In an average sense the model represented well the experimental results, but too many variables and uncertainties preclude having a point-to-point fit on the behavior (Armendariz, 1995).

FRAME LEVEL STUDIES

The fiber model and the incremental stiffness approach were used again to study the nonlinear dynamic response of a one-bay three-story building frame subjected to two components (N-S and vertical) of the 1940 El Centro earthquake record. Frame characteristics, cross-section properties and reinforcing steel distribution are shown in Fig. 2. The strengths of concrete used were 5,000; 8,000; 10,000 and 12,000 psi, assuming the same cross-section properties and reinforcing steel in the analysis. Although this does not seem realistic, the purpose was to isolate the effect of the concrete strength from the other parameters like cross-section size, column's confinement, type of damping, etc. in the dynamic response of the building.

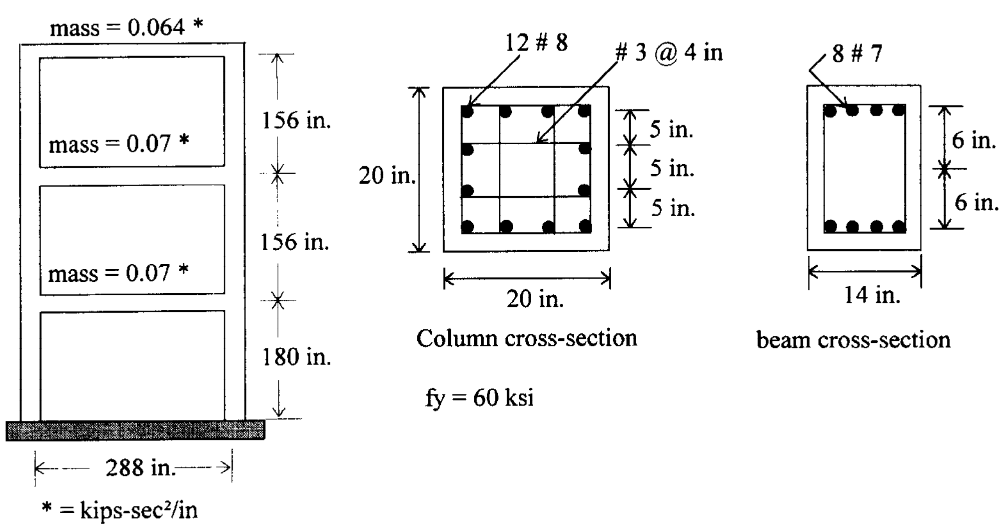

Fig. 2. One-bay three-story frame characteristics

The computer program developed had a number of options. In these applications it was used with a consistent mass formulation, the constant average acceleration method as integration scheme, mass proportional damping with 2% damping for the first natural frequency, and a time increment of 0.001 seconds. Each member was divided into 19 sections and each section into 10 concrete fibers. Both concrete and steel fibers were monitored through the hysteretic stress-strain curves at each step of the analysis.

Figures 3 and 4 show the horizontal displacements of the top story, right column when the concrete strength is 12000 psi and both components of motion (horizontal and vertical) are considered. The corresponding results for a concrete strength of 5000 psi are shown in Fig. 5 and 6. The maximum displacements are about the same in both cases but the duration over which the maximum displacements are reached is longer for the higher strength concrete. On the other hand the permanent horizontal displacement is smaller for the 12000 psi concrete. The results for 8000 and 10000 psi concretes were very similar to those for the 12000 psi case. Figure 7 shows the vertical displacement of the top joint for the 5000 psi concrete when only the horizontal component of motion is considered. It can be seen that the permanent displacement decreases in this case. Figures 8 and 9 show the axial force in the top column for the same concrete with the 2 components of motion or only one. It can be seen that the vertical earthquake component affects significantly the results. It should be noticed however that this is only the dynamic axial force without accounting for the gravity loads.

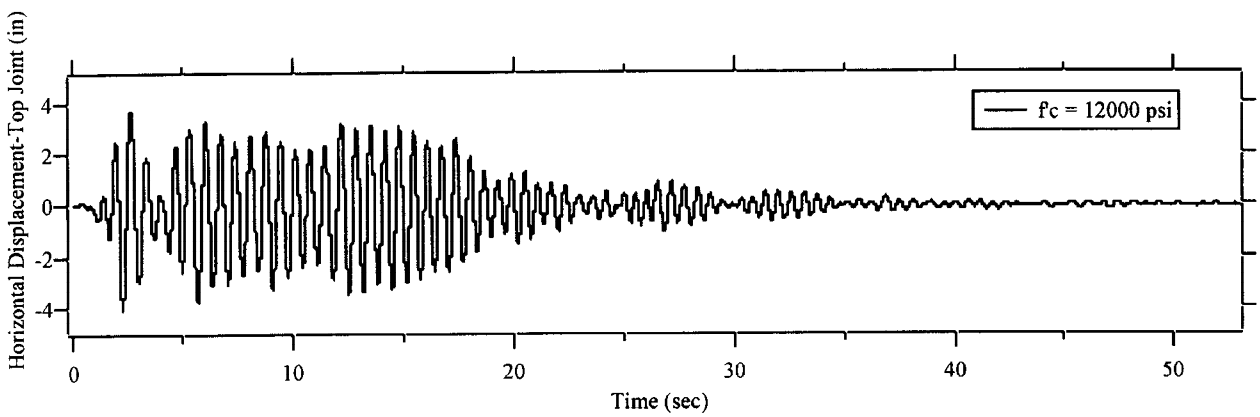

Fig. 3. Horizontal Displacement-Top Joint Two Earthquake Components

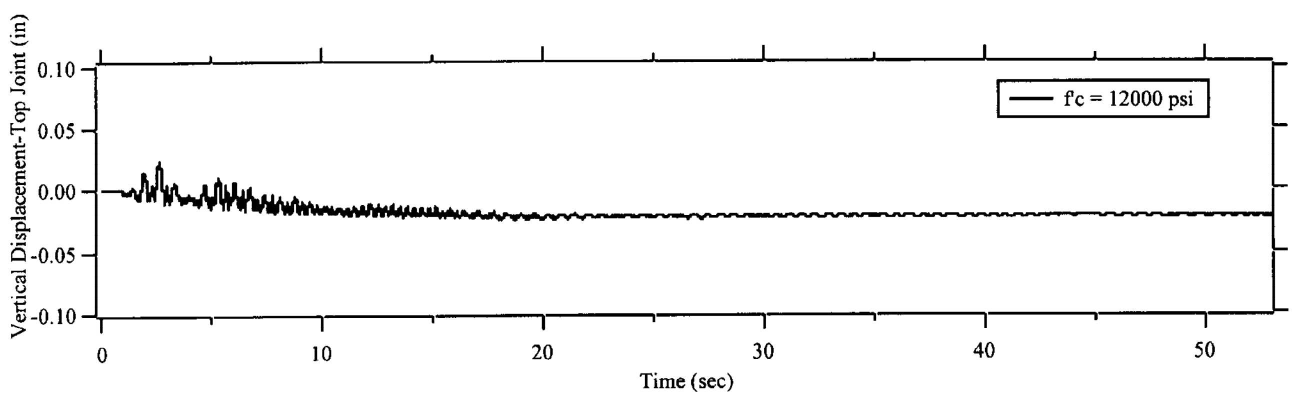

Fig. 4. Vertical Displacement-Top Joint Two Earthquake Components

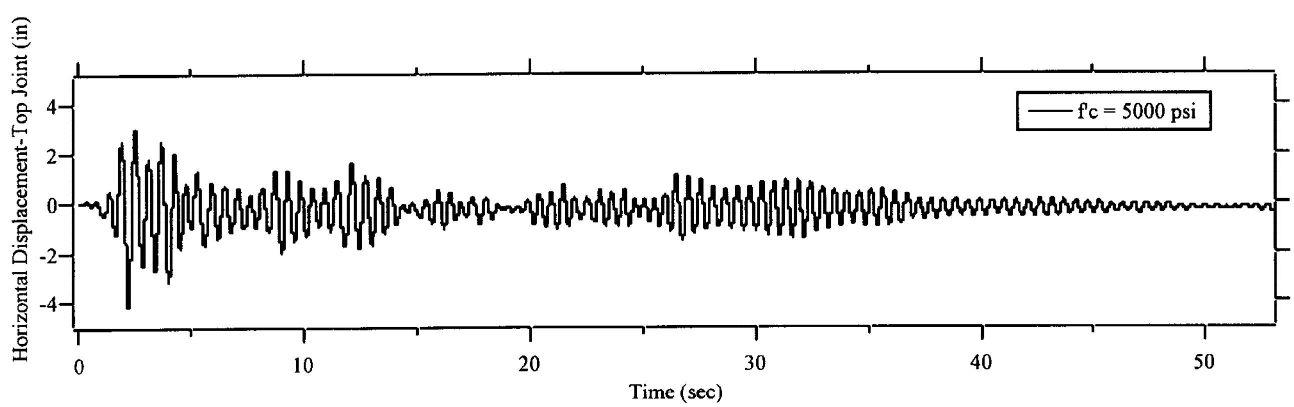

Fig. 5. Horizontal Displacement-Top Joint Two Earthquake Components

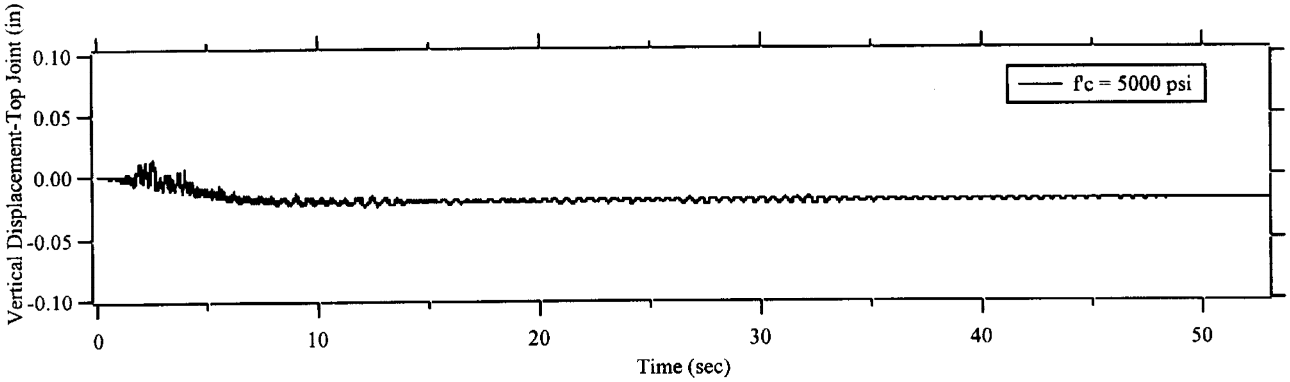

Fig. 6. Vertical Displacement-Top Joint Two Earthquake Components

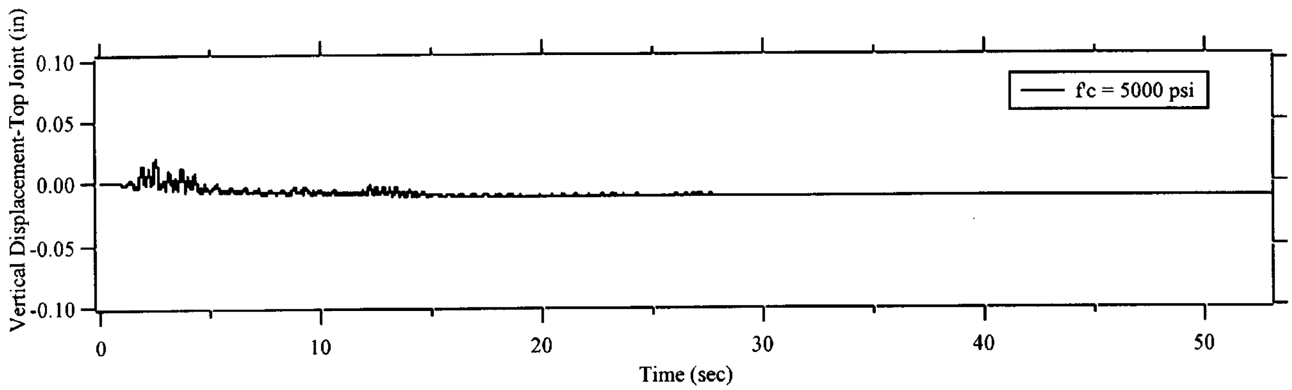

Fig. 7. Vertical Displacement-Top Joint Horizontal (N-S) Component

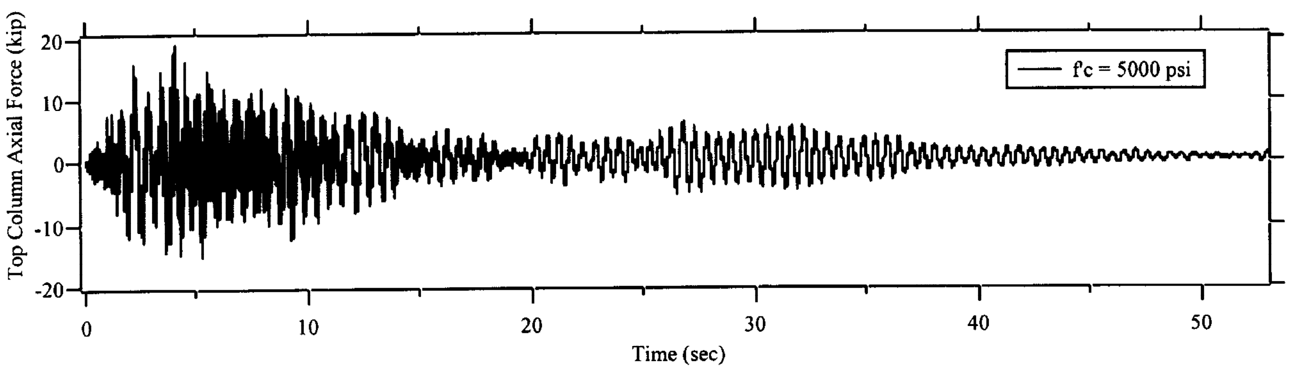

Fig. 8. Top column Axial Force (kips) Two Earthquake Components

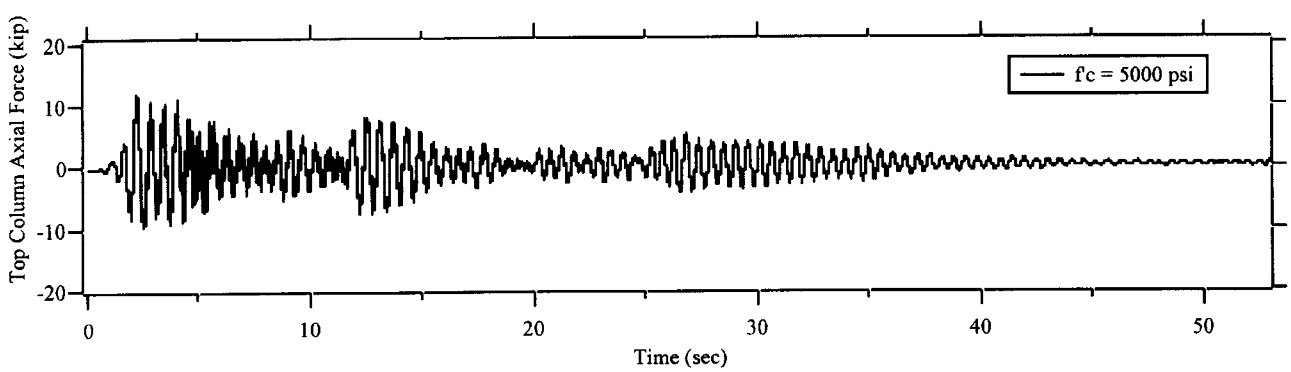

Fig. 9. Top column Axial Force (kips) Considering the N-S Component Only

CONCLUSIONS

The studies conducted suggest that the fiber model can reproduce reasonably well the behavior of HSC members. A point-by-point fit of the predicted and the experimental results was , however impossible (Armendariz, 1995).

The effect of concrete strength on the response was very difficult to isolate because of dealing with one particular earthquake record. However , from all the cases studied (Armendariz, 1995) it seems that as the concrete strength increased the permanent displacements tended to decrease. It was found that the vertical ground acceleration had an important effect on the vertical displacement history and the permanent deformations of the frame.

In the studies conducted to date only joint displacements and member forces were considered as output. A more interesting result would be the required ductilities. The selection of appropriate ductility measures (strain ductility, curvature or rotation ductility, interstory displacement ductility etc.) and the implementation of their calculation in the computer program are being conducted now.

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