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RECOMMENDED PROCEDURE FOR CALCULATION OF THE BALANCED REINFORCEMENT RATIO

by

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PREFACE

This report presents the results of a research project which was part of the U. S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States - Japan Coordinated Masonry Research Program conducted under the auspices of the Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

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ABSTRACT

A method is proposed to calculate the balanced reinforcement ratio for concrete masonry shear walls. The proposed methodology uses the axial load on the wall and the dimensions of the shear wall to calculate a balanced reinforcement ratio. The proposed method recognizes that a moment in combination with an axial load can define the theoretical maximum elastic capacity of a shear wall. Current design requirements for reinforced masonry and concrete limit the vertical reinforcement of a shear wall to a percentage of the balanced reinforcement ratio. A large vertical reinforcement ratio combined with a relatively large axial load on a load bearing wall may cause the crushing of concrete masonry before most of the vertical reinforcement distributed along the length of the wall yields. Such a brittle performance is undesirable. A more ductile behavior is when all of the vertical reinforcement except for the reinforcement in and immediately adjacent to the compression zone yields before the masonry crushes. A criteria is proposed that will limit the amount of flexural reinforcement to a fraction of the balanced reinforcement ratio. The intent of this criteria is to avoid a brittle crushing failure at the toe of the shear wall and promote a more desirable "ductile" behavior of the shear walls. The limitation is also proposed for flanged shear walls. A comprehensive calculation procedure is presented, followed by examples that provide guidance for use of the proposed criterion.

RECOMMENDED PROCEDURE FOR CALCULATION OF THE BALANCED REINFORCEMENT RATIO

1.0 INTRODUCTION

The provisions of the 1991 Edition of the Uniform Building Code (UBC, 1991) for working stress design of a concrete masonry shear wall does not limit the flexural reinforcement ratio to a percentage of balanced reinforcement ratio. The only code limitation is that the area of the reinforcement cannot exceed 6% of the cell area. If the average net area of a concrete block masonry is fifty percent of gross area, this limiting ratio is about three percent of the wall area. This upper limit of a reinforcement ratio of about three percent does not depend upon the wall geometry nor the loading condition. Therefore, theoretically a wall with as much as a three percent reinforcement ratio could be designed. Other studies (Kariotis, 1990) show that such a shear wall would be an extremely over-reinforced section and have a very limited useful nonlinear displacement.

This research proposes a criterion for shear walls which limits the maximum vertical steel ratio. The criterion is based upon wall geometry and the axial load on the wall. It is recognized that the balanced condition, that is, the stress in masonry and the stress in reinforcement reach their maximum limits simultaneously, is a function of the axial load as well as the wall geometry. The reinforcement ratio corresponding to such balanced condition can be calculated by equating the compression forces to the tension forces in the reinforcement and the axial loading. The force resisted by the reinforcement in the compression zone is ignored for simplicity.

To calculate the compression force on the masonry, a linear stress-strain relationship of the expected peak compressive stress (f_{mc}) and the strain associated with this peak stress (e_{muc}) is assumed. All of the experimental studies of concrete masonry stress-strain relationships used concrete masonry prisms that were loaded by monotonically increasing strains until fragmentation of the prisms occurred. There is no experimental evidence that a monotonic uniaxial stress-strain curve can be retraced by cyclic reloading after the prism has been loaded to or near peak strain. Hegemeir (1985) has reported the results of testing of masonry by cyclic loading and concluded that uniaxial monotonic stress-strain curves for concrete masonry cannot be retraced under successive loading and unloading cycles. Therefore, it is appropriate to use a simplified linear constitutive law for the stress-strain relationship of concrete masonry for the lack of any better available relationship.

When the balanced steel ratio is determined, the maximum allowable steel ratio should be limited to a percentage of balanced steel ratio. A tentative Limit States Design Standard limits the maximum reinforcement ratio to 35 percent of balanced reinforcement ratio for reinforced masonry shear walls. The maximum reinforcement ratio has been limited to 75 percent of the balanced ratio for reinforced concrete members loaded by other than earthquake loading (ACI, 1989). The Commentary to the ACI-318 suggests that the maximum reinforcement ratio be limited to 50 percent of the balanced reinforcement ratio when the loading includes earthquake loading.

These maximum reinforcement ratios are specified for concrete or masonry members that have concentrated quantities of reinforcement at the ends of the flexural element. The current limit states design recommendations for masonry shear walls assume that the vertical reinforcement in the shear wall will be uniformly distributed along the length of the shear wall. This arrangement of the flexural reinforcement is desirable as the useful nonlinear curvature of the wall at its base is enhanced. In addition to this benefit, the reinforcement required to resist loading normal to the shear wall surface is fully utilized as part of the flexural reinforcement. If the reinforcement required for flexural capacity to resist normal loading is not calculated as part of the reinforcement ratio, over-reinforcement of the shear wall is likely.

The nonlinear curvature, and the displacement of the top of a shear wall with distributed reinforcement, at its peak strength, is greater than that of a wall, with the same strength, that has its flexural reinforcement concentrated at the edges. This greater curvature, which enhances the curvature or displacement ductility of the shear wall, is desirable. An arbitrary maximum ratio of reinforcement is recommended herein to increase the useful curvature ductility or displacement at the top of the shear wall. The limitations that are currently prescribed in the tentative standards do not refer to experimental research and have not been related to an expected useful nonlinear displacement of the top of the shear wall.

In many design situations, the structural requirements for lateral design in the two principal orthogonal directions of masonry structures will result in intersecting shear walls, creating shear walls having a flanged shape. Recommendations made herein are applicable to flanged shear walls. The reinforcement in the flange of the wall is treated as a concentrated quantity of reinforcement in the tension edge. The uniformly distributed reinforcement in the shear wall itself is included in the computation of a balanced reinforcement ratio as its contribution may be relatively significant. This approach is recommended for a shear wall that has one or more flanges. A balanced reinforcement ratio is calculated for this simplified assumption.

In this study, the material values used in the analyses and modeling are entered at the expected value. The expected value represents the mean value that has been determined by materials testing. For example, the expected value of yield stress of #4 to #6 size reinforcement is taken as 68 ksi rather than the minimum guaranteed value of 60 ksi that is specified by codes. The expected value approach reflects the "best estimate" value for the materials and thus eliminates any need for load factors, capacity reduction factors or safety factors in the calculation of a reinforcement ratio that corresponds to the balanced load condition.

2.0 DETERMINATION OF BALANCED STEEL RATIO

The structural mechanics involved in determining the balanced steel ratio and hence, the recommended maximum allowable reinforcement ratio for rectangular and flanged shear walls is described as follows:

2.1 Rectangular shear wall

Figure 2-1 shows a rectangular shear wall section with distributed flexural reinforcement and with a total expected axial load P_e and wall weight of P_{we}.

The neutral axis location is determined by assuming that the extreme compression fiber reaches maximum expected usable strain (e_{mue}) at the same curvature that the extreme reinforcement bar reaches its expected yield strain. The distance of the neutral axis from the extreme compression fiber is given by:

$$C_b = \frac{e_{mue} d}{e_{mue} + e_{ye}} \qquad (2\text{-}1)$$

where:

C_b = Distance of the neutral axis from the extreme compression fiber.

e_{mue} = f_{ye}/E_{se}, expected yield strain of reinforcement

d = Distance from extreme reinforcement bar to the extreme compression fiber.

Once the location of neutral axis is determined, the quantity of tension reinforcement required to satisfy the force equilibrium is given by Equation (2-2).

$$T_s = C_m - P_e - P_{we} \tag{2-2}$$

where:

P_e = Expected axial load

P_{we} = Expected weight of wall

C_m = Total compression force

T_s = Total tension force in the reinforcement

The masonry compression force can be calculated as:

$$C_m = 0.5\ f_{me} t C_b \tag{2-3}$$

where:

f_{me} = Expected peak compressive stress

t = Thickness of rectangular shear wall

and the tension force in the reinforcement is given by:

$$T_s = \rho_b\ (0.5\ f_{ye})\ t\ (d - C_b) \tag{2-4}$$

where:

ρ_b = Balanced reinforcement ratio

Substituting equation (2-3) and (2-4) into equation (2-2) and rearranging:

$$\rho_b = \frac{(0.5\ f_{me} t C_b - P_e - P_{we})}{0.5\ f_{ye}\ t\ (d - C_b)} \tag{2-5}$$

2.2 Flanged shear wall, flange in tension, only reinforcement in flange used

Figure 2-2 shows a shear wall with a flange in tension. The reinforcement in the flange is considered to be lumped at the center of the flange and the reinforcement in the shear wall is ignored. As discussed in Section 2.1, the distance from the extreme compression fiber to the neutral axis (C_b) can be determined by equation (2-6)

$$C_b = \frac{e_{mue} d}{e_{mue} + e_{ye}} \tag{2-6}$$

The force equilibrium equation is:

$$T_{sf} = C_m - (P_e + P_{fe} + P_{we}) \tag{2-7}$$

where:

T_{sf} = Total tension force in flange

P_{fe} = Expected axial load on flange of shear wall

and:

$$C_m = 0.5\ f_{me} t C_b \tag{2-8}$$

and:

$$T_{sf} = \rho_b t_f b_f f_{ye} = A_{sf} f_{ye} \tag{2-9}$$

where:

ρ_b = Balanced reinforcement ratio

t_f = Thickness of flange

b_f = Width of flange

A_{sf} = Area of reinforcement in flange

Substituting equations (2-8) and (2-9) into equation (2-7), we obtain:

$$\rho_b = \frac{(0.5\ f_{me}tC_b-(P_e+P_f+P_{we})}{t_f\ b_f\ f_{ye}} \tag{2-10}$$

2.3 Flanged shear wall, flange in tension, reinforcement in flange and shear wall.

Figure 2-3 shows a shear wall with a flange in tension. The reinforcement in the flange is considered to be lumped at the center of the flange and the reinforcement in the shear wall is uniformly distributed along its length. The reinforcement ratio in the flange is assumed to be equal to the ratio in the shear wall. As discussed in Sec. 2.1, C_b can be determined by equation (2-11).

$$C_b = \frac{e_{mue}d}{e_{mue}+e_{ye}} \tag{2-11}$$

The force equilibrium equation is:

$$T_{sf}+T_s = C_m-(P_e+P_{fe}+P_{we}) \tag{2-12}$$

where:

T_{sf} = Total tension force in flange

The masonry compression force is:

$$C_m = 0.5\ f_{me}tC_b \tag{2-13}$$

and the total tension force in the reinforcement is:

$$(T_{sf}+T_s)=\left[\ \rho_b t_f b_f f_{ye}+0.5\ f_{ye}t\ (d-C_b)\ \right] \tag{2-14}$$

substituting equations (2-12) and (2-13) into (2-14)

$$\rho_b = \frac{0.5\ f_{me}tC_b-(P_e+P_{fe}+P_{we})}{\{t_f b_f f_y+0.5\ f_{ye}t\ (d-C_b)\}} \tag{2-15}$$

2.4 Flanged shear wall, flange in compression.

Figure 2-4 shows a shear wall with the flange in compression. The balanced reinforcement ratio calculated for this condition probably will not be a critical condition for determination of the maximum allowable reinforcement ratio in the shear wall. If this condition allows a higher reinforcement ratio than that calculated by equation (2-15), the lesser maximum allowable reinforcement ratio should be calculated and used as a limitation. The distance of the neutral axis from the extreme edge is given by:

$$C_b = \frac{e_{muc}d}{e_{muc}+e_{ye}} \qquad (2\text{-}16)$$

The quantity of tension reinforcement is given by:

$$T_s = C_m+C_{mf}-(P_e+P_{fe}+P_{we}) \qquad (2\text{-}17)$$

where:

$$C_m = 0.5\ f_{me}tC_b \qquad (2\text{-}18)$$

$$C_{mf} = f_{me}t_fb_f \qquad (2\text{-}19)$$

where:

C_{mf} = Compressive force in flange

and the tension force in the reinforcement is:

$$T_s = \rho_b(0.5\ f_{ye})t(d-C_b) \qquad (2\text{-}20)$$

$$\rho_b = \frac{0.5\ f_{me}tC_b+f_{me}t_fb_f-(P_e+P_{fe}+P_{we})}{0.5\ f_{ye}t(d-C_b)} \qquad (2\text{-}21)$$

2.5 Shear Wall with flanges at each end

Figure 2-5 shows a shear wall with flanges in tension and compression. The reinforcement in the tension flange is assumed to be lumped at the center of the flange and the reinforcement in the shear wall is uniformly distributed along its length. The reinforcement ratio in the shear wall, as discussed in Section 2.1, can be determined by equation (2-22).

$$C_b = \frac{e_{muc} d}{e_{muc} + e_{ye}} \tag{2-22}$$

The force equilibrium equation is:

$$T_s + T_{sf} = C_m + C_{mf} - (P_e + P_{fct} + P_{fcc} + P_{we}) \tag{2-23}$$

where:

P_{fct} = Expected axial load on the flange in tension

P_{fcc} = Expected axial load on the flange in compression

The masonry compression force is:

$$C_m + C_{mf} = 0.5\ f_{me} t C_b + f_{me} t_{fc} b_{fc} \tag{2-24}$$

where:

t_{fc} = Thickness of the flange in compression

b_{fc} = Width of flange in compression

and the total tension force in the reinforcement is:

$$(T_{sf} + T_f) = \rho_b \left[t_{ft} b_{ft} f_{ye} + 0.5\ f_{ye} t\ (d - C_D) \right] \tag{2-25}$$

substituting equations (2-23) and (2-24) into (2-25)

$$\rho_b = \frac{0.5\ f_{me} t C_b + f_{me} t_{fc} b_{fc} - (P_e + P_{fct} + P_{fcc} + P_{we})}{\{t_{ft} b_{ft} f_{ye} + 0.5\ f_{ye} t (d - C_b)\}} \tag{2-26}$$

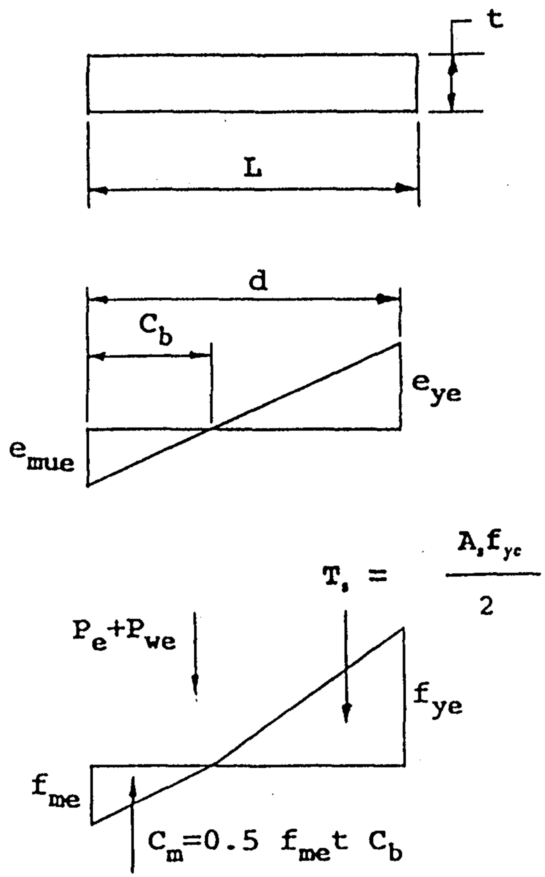


FIGURE 2-1

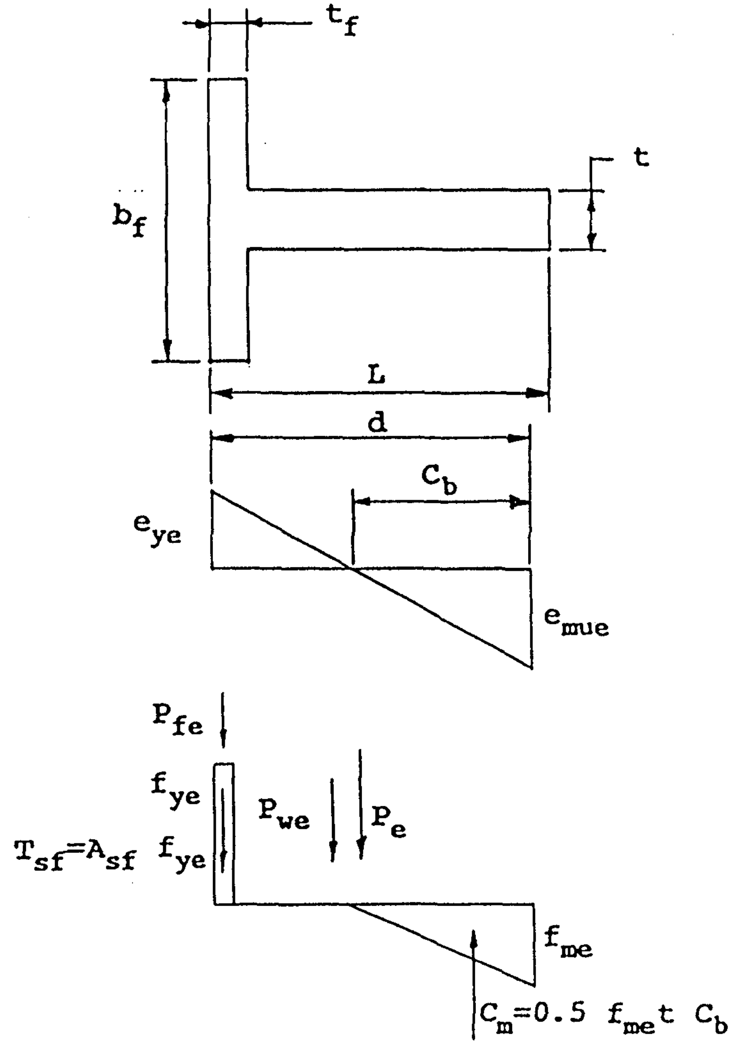


FIGURE 2-2

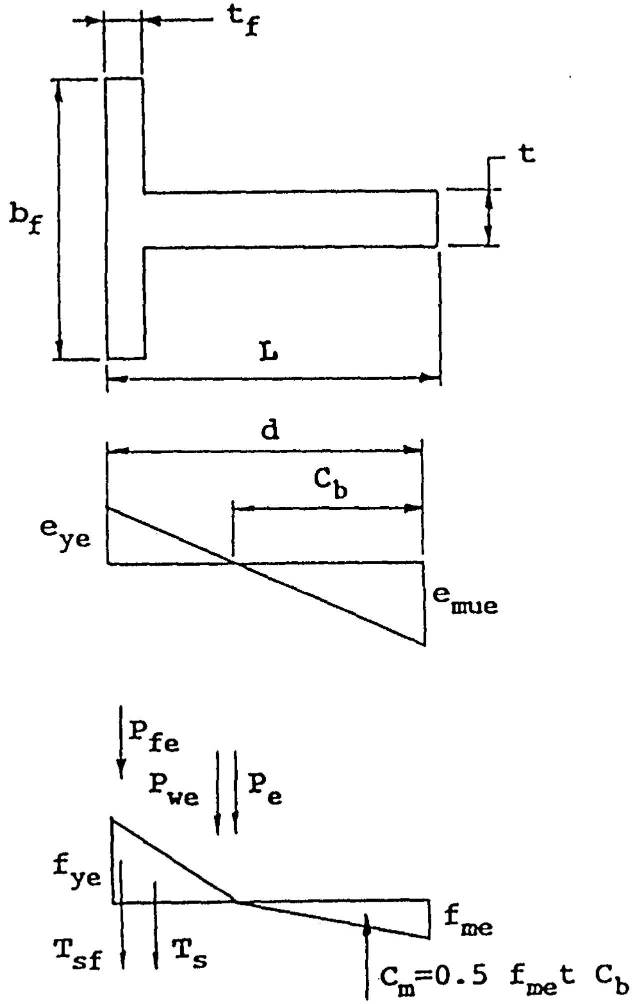


FIGURE 2-3

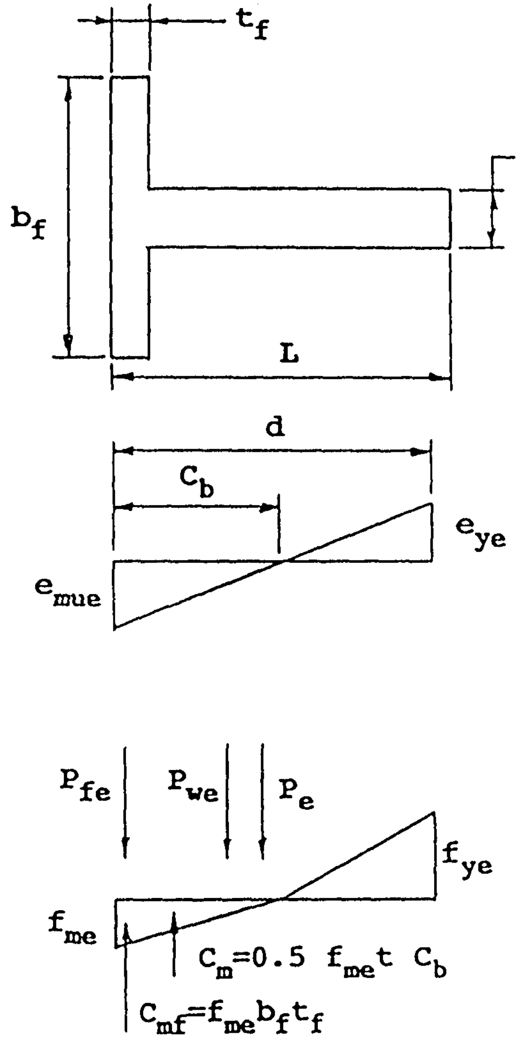
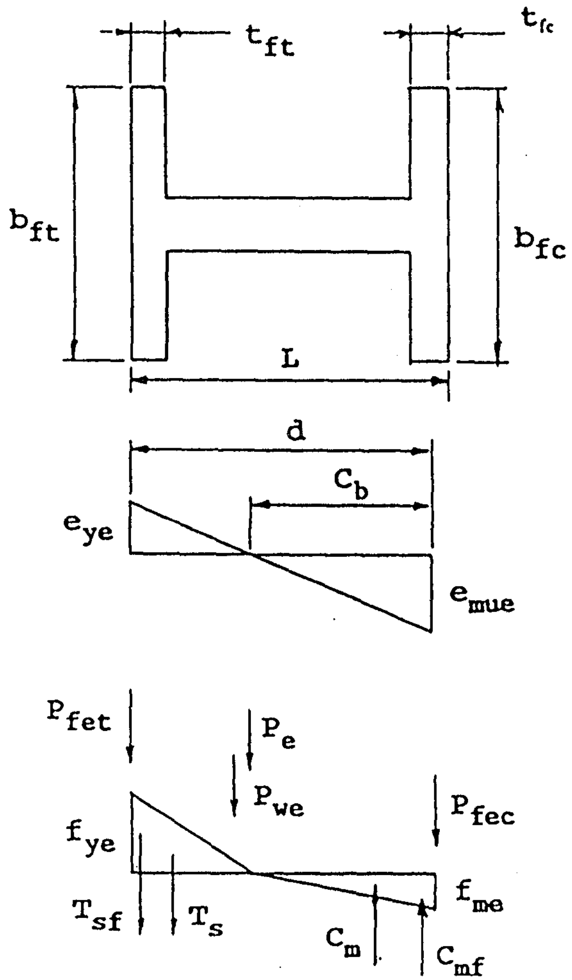


FIGURE 2-4



t_{ft}
t_{fc}
b_{ft}
b_{fc}
L
d
C_b
e_{ye}
e_{mue}
P_{fet}
P_e
P_{we}
P_{fec}
f_{ye}
f_{me}
T_{sf}
T_s
C_m
C_{mf}

FIGURE 2-5

3.0 RECOMMENDED PROCEDURE FOR CALCULATION OF BALANCED REINFORCEMENT RATIO

The following procedure is recommended for calculation of the balanced reinforcement ratio of a reinforced masonry shear wall.

1) Determine total expected axial load on the wall and its flanges, if any. The combination of axial load effects is given by the NEHRP Recommended Provisions (NEHRP, 1988) as:

$$(1.1+0.3\ S_{A(1.0)})\ Q_D+1.0\ Q_L+1.0\ Q_S \qquad (3\text{-}1)$$

where:

$S_{A(1.0)}$ = The coefficient representing spectral acceleration at 1.0 second period as given in Sec. 1.4.1 of the NEHRP Recommended Provisions

$Q_D, Q_L \& Q_S$ = As defined in Sec. 2.2 of the NEHRP Recommended Provisions. Q_S may be reduced as permitted by Sec. 2-1 of the Provisions.

2) Determine dimensions of the shear wall and its flanges. The width of the flange considered effective in tension or compression on each side of the web of the shear wall should be taken equal to 1/3 of the wall height or should be equal to the actual flange on either side of the web wall, whichever is less. If the flange is not used for calculation of the balanced reinforcement ratio, the transfer of shear between walls should be prevented.

3) If the shear wall is rectangular, use equation (2-1) through

(2-5) to determine the balanced reinforcement ratio, ρ_b, and hence calculate maximum allowable reinforcement ratio in accordance with the recommendations given in Sec. 6.0.

4) If the shear wall has a flange at one end, use Sec. 2.3 and 2.4 to calculate the balanced reinforcement ratio corresponding to the two directions of possible application of moment. The lesser balanced reinforcement ratio should be used to calculate the maximum allowable reinforcement ratio.

Sec. 2.2 may be used to approximate the balanced reinforcement ratio for loading in the plane of the wall when the length, L, of the shear wall is small in comparison with the flange width, b_f. Caution should be used as this balanced reinforcement ratio overstates the ratio and therefore the maximum allowable reinforcement ratio. Sec. 2.2 is appropriate for calculation of the balanced reinforcement ratio for a masonry wall loaded normal to the plane of the wall.

5) Using the maximum allowable reinforcement ratio, calculate the expected moment strength, M_e, of the shear wall for inplane loading or M_e for loading of the masonry normal to the wall surface. The expected moment, M_e, should be calculated with reduced dead load and no live or snow load. This combination of axial load effects is given by the NEHRP Recommended Provisions as:

$$(0.9-0.3\ S_{a(1.0)})Q_D+Q_E \tag{3-2}$$

6) Compare the expected moment, M_e, multiplied by the recommended capacity reduction factor, ϕ, with the required moment

capacity. If the expected strength exceeds the required strength, no further calculations are necessary. If the expected strength is less than the required strength, the dimensions of the masonry wall must be increased. This increase in dimensions allows additional vertical reinforcement to be added to the element thus increasing strength but not exceeding the maximum allowable reinforcement ratio.

4.0 **EXAMPLES OF CALCULATIONS**

The calculation of a balanced reinforcement ratio for four commonly occurring conditions are made herein. These calculations were made by a calculator. Calculation routines can be programmed for repetitious computations. These calculations use a value of 0.0026 for e_{muc}. This value was derived from a large number of prism tests (Atkinson, 1990). The recommended value for concrete block masonry is 0.0025 and should be used unless additional testing of the masonry material indicates otherwise.

4.1 Rectangular shear wall

The following material values and dimensions were used in this calculation. See Figure 2-1 for explanation of notation.

e_{muc} = 0.0026 inches per inch

f_{mc} = 2500 psi

e_{yc} = 0.0023 inches per inch

f_{yc} = 66,000 psi

L = 16 feet (192 inches)

h = 30 feet

t = 11.62 inches

d = 188 inches

P_c = 16,000 lbs.

P_{wc} = 57,600 lbs.

Equation (2-1)

$$C_b = \frac{e_{muc}d}{e_{muc}+e_{yc}} = \frac{0.0026 \times 188}{0.0026 + 0.0023} = 99.76 \text{ inches}$$

Equation (2-5)

$$\rho_b = \frac{(0.5\ f_{mc}tC_b - P_c)}{0.5\ f_{yc}t(d-C_b)} = \frac{0.5x2500x11.62x99.76-73,600}{0.5x66,000x11.62(188-99.76)}$$

$$= 0.0406 \text{ or } 4.1 \text{ percent}$$

4.2 <u>Shear wall with flange in tension</u>

The following materials values and dimensions were used in this calculation. See Figure 2-3 for explanation of notation.

e_{muc} = 0.0026 inches per inch

f_{mc} = 2,500 psi

e_{yc} = 0.0023 inches per inch

f_{yc} = 66,000 psi

L = 20.0 feet (240 inches)

h = 30.0 feet

t = 11.62 inches

d = 236 inches

t_f = 7.62 inches

b_f = 8.0 feet (96 inches)

P_c = 20,000 lbs.

P_{fc} = 4,000 lbs.

P_{wc} = 90,480 lbs.

Equation (2-11)

$$C_b = \frac{e_{muc}d}{e_{muc}+e_{yc}} = \frac{0.0026x236}{0.0026+0.0023} = 125.22 \text{ inches}$$

Equation (2-15)

$$\rho_b = \frac{0.5\ f_{me}tC_b-(P_e+P_{fe}+P_{we})}{\{t_f b_f f_{ye}+0.5\ f_{ye}t(d-C_b)\}}$$

$$\rho_b = \frac{0.5x2500x11.62x125.22-114,480}{\{7.62x96x66,000+0.5x66,000x11.62(236-125.22)\}}$$

$$= 00.0128 \text{ or } 1.3 \text{ percent}$$

4.3 <u>Shear wall with flange in compression</u>

All materials values and dimensions of the wall used in this calculation are identical to those used in Sec. 4.2. This calculation is made for comparison with the example shear wall used in Sec. 4.2.

Equation (2-16)

$$C_b = \frac{e_{muc}d}{e_{muc}+e_{ye}} = 125.22 \text{ inches}$$

Equation (2-21)

$$\rho_b = \frac{0.5\ f_{me}tC_b+f_{me}t_f b_{f}.(P_e+P_{ef}+P_{we})}{0.5\ f_{ye}t(d-C_b)}$$

$$= \frac{0.5x2500x11.62x125.22+2500x7.62x96-114,480}{0.5x66,000x11.62\ (236-125.22)}$$

$$= 0.0832 \text{ or } 8.3 \text{ percent}$$

The balanced reinforcement ratio for a shear wall with the flange in compression exceeds the balanced reinforcement ratio for the flange in tension by more than four times. Since shear walls are loaded by full reversing cyclic moments, the lower ratio calculated in Sec. 4.2 should be used for calculation of a maximum allowable reinforcement ratio.

4.4 Wall loaded normal to its surface

The procedure recommended in Sec. 2.2 is appropriate for the wall loaded normal to its surface. The reinforcement is in a single line either in the center of the wall or on two lines adjacent to the face of the masonry. In the second case, two lines of vertical reinforcement, the effect of the reinforcement adjacent to the compression face may be neglected.

The material values and dimensions used in this calculation are as follows:

e_{muc} = 0.0026 inches per inch

f_{mc} = 2,500 psi

e_{yc} = 0.0023 inches per inch

f_{yc} = 66,000 psi

L = 11.62 inches

h = 30.0 feet

d = 5.86 inches

t = 12.0 inches

t_f = 11.62 inches

b_f = 12.0 inches

P_e = 1,000 lbs./ft.

P_{we} = 120 #/sq.ft x h/2

Equation (2-1)

$$C_b = \frac{e_{mue}\ d}{e_{mue} + e_{ye}} = \frac{0.0026 \times 5.86}{0.0026 + 0.0023} = 3.11 \text{ inches}$$

Equation (2-10)

$$\rho_b = \frac{0.5\ f_{me}\ t\ C_b - (P_e + \frac{P_{we}}{2})}{t_f\ b_f\ f_{ye}} = \frac{0.5x2500x12x3.11-2800}{11.62x12X66,000}$$

$$= 0.0048 \text{ or } 0.5 \text{ percent}$$

If the reinforcement is placed in two lines adjacent to the face shell of the masonry unit, the depth, d, is 9.0 inches.

Equation (2-1)

$$C_b = 4.78"$$

Equation (2-10)

$$\rho_b = 0.0075 \text{ or } 0.75 \text{ percent}$$

Equation (2-10) calculates a balanced reinforcement ratio that uses one-half of the vertical reinforcement in the wall. The total vertical reinforcement in the wall could be twice this ratio times the wall area or:

area/linear foot = 0.075 x 2 x 11.62 x 12.0 = 2.09 sq. in./lin.ft.

In comparison, the balanced reinforcement ratio calculated in Sec. 4.1 for this wall loaded in the plane of the wall was 0.0406 or a quantity of reinforcement of 5.66 sq.in./L.F. This comparison indicates that the balanced reinforcement ratio calculated for normal loading on the wall may be the critical limitation on the maximum allowable reinforcement ratio of the shear wall.

5.0 **STUDIES OF THE MAXIMUM ALLOWABLE REINFORCEMENT RATIO**

5.1 Procedure for making the studies

Limitations on the reinforcement ratio are used in current design requirements for reinforced concrete elements (ACI,1989) and for reinforced masonry walls loaded normal to the plane of the wall (UBC, 1991). The specified maximum allowable reinforcement ratio is 0.5 ρ_b in both of these references. However, the method of calculation of the balanced reinforcement ratio implied by these references is significantly different than recommended herein. These dynamic response studies will use masonry shear walls, with a percentage of the balanced reinforcement as the variable. The balanced reinforcement ratio of these shear walls will be determined by these recommended procedures.

The loading on these walls will be a dynamic loading derived from the draft of the Limit States Design Standards. The expected strength of the shear wall varies with the ratio of vertical reinforcement. The seismic design recommendations of the Limit State Design Standards are used to back-calculate a weight (mass) that is coupled with the shear wall at its top edge in accordance with its expected strength.

Two types of walls will be used in these studies, a rectangular shear wall of 12 inches nominal thickness and 16 feet in length, and a shear wall with a flange. The flanged shear wall has a 12 inch nominal web thickness and is 20 feet in depth. The flange has a nominal thickness of 8 inches and a width of 8 feet. These walls correspond to the examples used in Sec. 4.1, 4.2 and 4.3.

Each of the shear walls will be analyzed by a nonlinear finite element program (Ewing, 1987) to determine an envelope of force-displacement relationship for monotonic loading. The flanged wall will have an asymmetric behavior. A dynamic behavior model will be fit to the envelope curve that is calculated by the finite element analysis and a nonlinear dynamic model (Kariotis, 1992) will be used to determine the displacement at the top of each shear wall. The hysteritic nonlinear behavior of the model has been validated by experiments. The experiments determined characteristics such as stiffness degradation on recycling, unloading and reloading stiffnesses, and pinching of full and partial reversing cycles. Damping of dynamic model is hysteretic only. No viscous damping is added to the model.

5.2 Design of the masonry shear walls

The rectangular shear wall used as an example in Sec. 4.2 is 12 inch nominal thickness and has a length of 16 feet. The height of the shear wall is 30 feet. The shear wall is assumed to be fixed at its base. The percentages of the balanced reinforcement ratio used in this study are 50, 35, 25 and 15 percent. These percentages are converted to the vertical reinforcement given in Table 5-1. The expected strength of each rectangular shear wall is given in Table 5-1.

TABLE 5-1

DESCRIPTION OF RECTANGULAR SHEAR WALL

Wall No.	% of ρ_b	Area of Vert.Reinf. sq.in./L.F.	Size and Spacing of Vert. Reinf.	Expected Moment Capacity ft. kips
R-1	50	2.83	2#9@8"(3.00)	15,210
R-2	35	1.98	#7 and #8@8"(2.09)	12,213
R-3	25	1.41	#9@8"(1.50)	9,560
R-4	15	0.85	#7@8"(0.90)	6,354

The expected strength is calculated as recommended in the Draft Limit States Standards.

" 8.3.1.1 Calculation of the expected strength of a member for flexure or flexure and axial loading shall include the effects of masonry cracking, the expected yield stress of the reinforcement and the yielding of the reinforcement distributed throughout the depth of the member. The expected strength for flexure or flexure and axial loading shall be taken as the value when strain in masonry is 0.0025.

8.3.1.2 In lieu of a nonlinear analysis to determine the expected strength, the expected strength may be estimated by assuming a compression zone of dimensions calculated by equating a uniform stress of 0.85 f_{mc} to the sum of expected forces of all of the flexural reinforcement outside of the compression zone at yield stress and the axial load ".

The rectangular shear wall has 24 vertical bars with the spacing beginning at 4 inches from the ends of the wall. The expected strength is calculated by the iterative procedure given in Sec. 8.3.1.2 of the Draft Standards. The axial load on the wall used in the example of Sec. 4.1 is 16,000 pounds and the weight of the wall is 57,600 pounds. These loads are used in conjunction with the reinforcement quantity to calculate the expected moment. The procedure is given by the following equation:

$$0.85\ Ad\ t\ f_{me} = (P_c+P_{wc})+n\ A_s\ f_{yc} \tag{5-1}$$

where:

Ad	=	Estimated length of compression zone measured along length of wall
t	=	Thickness of wall
f_{me}	=	Expected compressive strength of masonry
P_c	=	Expected axial dead load in wall
P_{wc}	=	Expected weight of wall
n	=	Number of vertical reinforcing bars outside of the compression zone, Ad
A_s	=	Area of each reinforcing bar
f_{yc}	=	Expected yield of the reinforcement

The solution of the iterative procedure for wall R-3 is Ad = 48.8" and n = 18 bars. The expected moment capacity is calculated by taking moments about the center of the assumed rectangular compression block.

$$M_c = (P_c+P_{wc})(0.5\ L-0.5\ Ad)+\Sigma[A_s f_{yc}(x-0.5\ Ad)] \tag{5-2}$$

where:

x = Distance of any reinforcing bar from the extreme compression fiber

The inplane shear capacity of the wall is required to be equal to or exceed the flexural capacity. The required shear capacity of each wall is given in Table 5-2.

TABLE 5-2

SHEAR REINFORCEMENT OF RECTANGULAR WALLS

Wall No.	Req. Shear Capacity, Kips	Horizontal Reinf.	Calc. Shear Capacity, Kips
R-1	507	2#5@8"	496
R-2	407	2#4@8"	418
R-3	319	#5@8"	373
R-4	212	#5@16"	290

A proposed formula for shear capacity called the "Matsumura" formula (1987) was used for determination of the shear reinforcement. The effectiveness of this formula has been evaluated by Fattel (1991) and found to be the better of the formulae evaluated. The formula is in SI units and is:

$$V_u = \left[k_u k_p \left(\frac{0.76}{(h/d)+0.7}+0.012\right)\sqrt{f_{me}}+0.18\gamma\delta\sqrt{\rho_h f_{ye}\, f_{me}}+0.2\sigma_o\right] t \times j \times 10^3$$

where:

V_u = Expected shear in KN

k_u = 1.0 for fully grouted masonry

k_p = 1.16 $\rho_t^{0.3}$, ρ_t = a_t/td in percent, a_t = area of edge tension bar

h = Height of shear wall

d = Distance from edge bar to extreme compression fiber

γ = 0.8, factor for anchorage of horizontal reinforcement

δ = 1.0, factor for loading method

ρ_h = Horizontal reinforcement ratio

σ_o = Axial stress due to applied load and ½ of the weight of the wall

t = Thickness of wall

j = 7/8 d

The properties of the materials of the shear wall are f_{mc} = 2500 psi (17.2 MPa) and f_{ye} = 66,000 psi (4.55 x 10^2 MPa). The effective depth of the shear wall is 188 inches (4.775 m), the thickness is 11.62 inches (0.30m). The axial stress is 7.17 psi (0.0495 MPa).

The flanged shear walls are designed by the same procedure. Table 5-3 provides the description of the dimensions of the walls, the vertical reinforcement, and the expected moment capacity. The flanged shear wall is the same as used for an example in Sec. 4.2. The balanced reinforcement ratio for this wall is 2.0 percent.

TABLE 5-3

DESCRIPTION OF FLANGED SHEAR WALLS

Wall No.	% of ρ_b	Vertical Reinforcement sq in/lin ft. Flange	Web	Size & Spacing Flange	Web	Expected Moment Capacity ft. kips
F-1	50	0.59	0.89	#8@16"(0.59)	2#7@16(0.90)	13,292
F-2	35	0.41	0.62	#7@16"(0.45)	#8@16"(0.59)	11,504
F-3	25	0.29	0.45	#4@8"(0.30)	#7@16(0.45)	8,857
F-4	15	0.18	0.27	#6@24"(0.22)	#8@32"(0.29)	6,843

The inplane shear capacity is required to be equal to or exceed the flexural capacity. The expected shear, based on the expected moment strength, is given in Table 5-4. The horizontal reinforcement in the web wall and the calculated expected shear capacity is also shown in Table 5-4.

TABLE 5.4

SHEAR REINFORCEMENT OF FLANGED WALLS

Wall No.	Req. Shear Capacity, Kips	Horizontal Reinf.	Calc. Shear Capacity, Kips
F-1	443	2#4@8"	477
F-2	383	#5@8"	423
F-3	295	#5@16"	380
F-4	228	#4@16"	362

5.3 Determination of the Earthquake Loading

The earthquake loading of a reinforced masonry shear wall is used to specify a required strength. This required strength is not intended to provide sufficient strength to cause the lateral load resisting system, the shear wall, to respond to the design level earthquake and have internal displacements that are less than an elastic limit state. Seismic design requirements (UBC, 1991) or recommended provisions (NEHRP, 1988) assume that nonlinear behavior of a shear wall with earthquake loading will occur when earthquake intensities are equal to or approach the intensity specified in codes and provisions as "design intensities".

These studies have established the expected strength of the shear wall, R1 through R4, and F1 through F4, by a criteria that limits the quantity of flexural reinforcement. The earthquake loading is calculated in this section to fully utilize the expected strength.

The earthquake loading is specified in the NEHRP Recommended Provisions by the following equation:

$$V = C_s W \tag{5-4}$$

where:

V = Earthquake loading

W = The total dead load specified in the Draft Limit States Design Standards

The variable, C_s, is calculated for each shear wall by the following method:

$$C_s = \frac{S_{a(1.0)} S}{R \, T^n_{cs}} \qquad (5\text{-}5)$$

and:

$$C_s = \frac{S_{a(0.3)}}{R} \qquad (5\text{-}6)$$

where:

n = 1.0 for $T_{cs} \leq 1.0$ sec. and $n = 2/3$ for $T_{cs} > 1.0$ sec.

$S_{a(1.0)}$ = The spectral acceleration at 1.0 seconds period of the design spectrum

$S_{a(0.3)}$ = The spectral acceleration at 0.3 seconds period of the design spectrum

S = Soil profile coefficient. For these examples S_2 profile was assumed and $S = 1.0$

R = The response modification coefficient. For these examples a value of 4½ was used

T_{cs} = The expected period of the shear wall. The effective secant stiffness of the shear wall at a displacement of about twice first yield displacement was used to calculated the expected period.

Since the value of the expected period is dependent on both the weight, W, and the secant stiffness, k_s, an iterative procedure must be used to calculate the related unknown values of T_{cs} and W.

The iterative procedure is as follows:

V = The required design loading based on the expected strength of the shear walls. The values are given in Tables 5-2 and 5-4 as required shear capacity. A capacity reduction factor of 0.9 was used for calculation of W.

$$W \geq \frac{VR}{S_{a(0.3)}} \quad \text{from equation (5-6)} \qquad (5\text{-}7)$$

where:

$S_{a(0.3)}$ = 1.0, assumed for these studies

and:

$$W = \frac{VRT^{n}_{cs}}{S_{a(1.0)}S} \qquad (5\text{-}8)$$

where:

$S_{a(1.0)}$ = 0.58, assumed for these studies

S = 1.0, assumed for these studies

and T_{cs} is determined by the following procedure:

$$T_{cs} = (W/k_s \cdot g)^{1/2} \qquad (5\text{-}9)$$

where:

k_s = Secant stiffness from Figure 5-1 for rectangular walls and Figure 5-2 for flanged walls.

g = 386 in/sec^2

T_{cs} is given an initial value of 0.5 sec. Then calculate W. Recalculate T_{cs} and then revise W.

The yield displacement and secant stiffness (effective stiffness) of the shear walls and the expected period is given in Table 5-5.

TABLE 5-5

STIFFNESS PROPERTIES OF SHEAR WALLS

Wall No.	Yield Displ.	k_i kips/in	k_{es} kips/in	T_{es} Sec.	V kips	W kips
R-1	0.125	850	675	0.59	507	2,088
R-2	0.135	850	580	0.55	407	1,562
R-3	0.120	850	540	0.47	319	1,047
R-4	0.095	850	555	0.42(1)	212	859(1)
F-1	0.108	2,065	1,300	0.40(1)	443	1,794(1)
F-2	0.085	2,065	1,315	0.37(1)	303	1,551(1)
F-3	0.062	2,065	1,900	0.27(1)	295	1,195(1)
F-4	0.020	2,065	2,065	0.23(1)	228	923(1)

(1) W calculated by Equation (5-7)

The stiffness properties given for the flanged walls are for the flange in tension. The stiffness and strength is less when the flange is in compression but the useful displacement for "flange in compression" indicates that dynamic displacement in that direction will not be critical. Comparison of Figures 5-2 and 5-3 clearly show this relationship of useful displacement, which is the nonlinear displacement prior to the masonry reaching peak strain.

The initial stiffness, k_i, given in Table 5-5 was taken from the FEM analysis. The initial uncracked stiffness of the rectangular masonry section calculated by engineering principles is:

$$k_i = \frac{3\ EI}{L^3} = 846 \text{ kip/inch}$$

where:

$E = 1.92 \times 10^3$ kips per sq. 2 inch

$$I = \frac{11.62 \times 192^3}{12} = 6.854 \times 10^6 \text{ inch}^4$$

$L = 360$ inches

The dynamic loading of the design level earthquake was a series of recorded ground motions that were scaled to a design spectra having a $S_{a(0.3)}$ of 1.0 and $S_{a(1.0)}$ of 0.58. This spectra is that given in the 1991 Edition of the Uniform Building Code as Figure No. 3 and in the Commentary to the NEHRP Recommended Provisions as Figure C1-10. The 5% damped spectrum of the records that were used for the analyses were amplitude scaled to that design spectrum for the range between 0.5 to 1.5 seconds period. The ground motions selected and the scale factor used are given in Table 5-6.

TABLE 5-6

GROUND MOTION USED FOR DYNAMIC ANALYSIS

Event	Station	Component	Scale Factor
Imperial Valley, 1940	El Centro	E-W	1.79
Imperial Valley, 1940	El Centro	N-S	1.31
Imperial Valley, 1979	Pine Union	140°	1.71
Imperial Valley, 1979	Cruickshank Road	230°	1.50
Imperial Valley, 1979	James Road	140°	1.39
Kern County, 1952	Lincoln School	S69E	2.06

Imperial Valley, 1979	Cruickshank Road	140°	1.20
Imperial Valley, 1979	Brawley Airport	315°	2.07
Imperial Valley, 1979	Keystone Road	140°	1.85

These ground motions were selected for an ongoing analysis of buildings designed by the Draft Limit State Design Standards. The fit of the average of the nine scaled ground motions to the design spectrum is shown in Figure 5-4.

5.4 Description of the Dynamic Model

The shear walls are modeled as a fixed-base single-degree-of-freedom oscillator. The nonlinearity of the response is represented as a hysteretic spring in a nonlinear dynamic analysis program, LPM/I, Version 1.03 (Kariotis, 1992). The hysteretic behavior was programmed to replicate cyclic tests of reinforced masonry walls. The behavioral model of the rectangular walls has symmetry, the behavioral model of the flanged wall is asymmetric. The envelope of the behavior model was determined by the nonlinear finite element analysis and is shown in Figures 5-1,5-2 and 5-3. Representative hysteretic behavior of the rectangular and flanged shear wall is shown in Figures Group 5-5 and Group 5-8.

5.5 Results of the dynamic studies

The dynamic studies of the effects of reinforcement on the nonlinear behavior of a shear wall designed in accordance with the Draft Standards consisted of the following steps:

- The behavioral envelope of the monotonically displaced shear wall was fitted with a modified Spring 11 (Kariotis, 1992). This spring has hysteretic damping, stiffness degradation on reloading, stiffness degradation on unloading for increasing displacements and pinching due to elongation of the tensile reinforcement. Shear deformation is included in the model.
- Each shear wall was excited with the North-South component of the 1940 El Centro record.
- After review of the response of the wall to the El Centro record, the shear wall was shaken by the nine earthquake records selected as representative of the seismic hazard zone.

The results are summarized for the rectangular walls and for the flanged walls in the following subsections.

5.5.1 Rectangular walls

The maximum displacement of the shear walls with the full range of the quantity of reinforcement, 50 to 15 percent of balanced reinforcement ratio, were excited by the N-S component of the 1940 El Centro record scaled as described in Sec. 5.3. The virgin envelope of the force-displacement relationship as determined by the FEM analysis, the hysteretic behavior of the shear wall and the top displacement is shown in Figures Group 5-5. The walls R1, R2, R3 and R4 have reinforcement quantities of 50,35,25 and 15 percent of the balanced reinforcement ratio respectively. Figures Group 5-6 shows the response of Wall R1 to the nine ground motions that were selected and scaled to the standard spectrum for S_2 soils profile and ZPA = 0.4 g. Figures Group 5-7 shows the response of Wall R4 to the same nine ground motions.

The results of the dynamic analyses of Walls R1 and R4 are summarized in Table 5-7.

TABLE 5-7

RESPONSE OF WALLS R1 AND R4 TO SELECTED GROUND MOTIONS

Ground Motion No.	Maximum Displacement (Inches)	
	Wall R1	Wall R4
1	9.72	8.87
2	9.28	6.28
3	7.39	6.53
4	8.63	6.54
5	7.54	6.45
6	3.53	4.02
7	8.73	7.94
8	6.40	6.67
9	7.64	6.85
Average	7.65	6.68
Std.Dev.	1.86	1.31

5.5.2 Flanged Walls

The four flanged walls with reinforcement quantities of 50, 35, 25 and 15 percent of the balanced minimum reinforcement ratio were analyzed by FEM. The minimum reinforcement ratio is that calculated for the flange in tension.

The results of the FEM analyses are shown in Figure 5-2 and 5-3. The consistency of the shape of the virgin envelope indicates that the analysis of the effects of the quantity of reinforcement can be limited to analysis of walls F1 and F4. The results of the FEM analyses, the fitting of Spring 11 to the FEM results, the hysteretic behavior of Spring 11 and the response of the shear wall to the N-S component of the 1940 El Centro record x 1.31 is shown in Figures Group 5 - 8. The dynamic response of Wall F1 to the nine scaled ground motions is shown in Figures Group 5-9, the same data for Wall F4 is shown in Figures Group 5-10. The top displacements of Walls F1 and F4 is summarized in Table 5-8.

TABLE 5-8

RESPONSE OF FLANGED SHEAR WALLS TO SELECTED GROUND MOTIONS

Ground Motion No.	Maximum Displacement (Inches)	
	Wall F1	Wall F4
1	8.63	8.62
2	6.91	6.25
3	5.96	5.75
4	6.80	5.52
5	6.37	6.87
6	3.47	3.54
7	7.83	8.28
8	6.52	8.13
9	6.98	6.21
Average	6.61	6.57
Std.Dev.	1.42	1.61

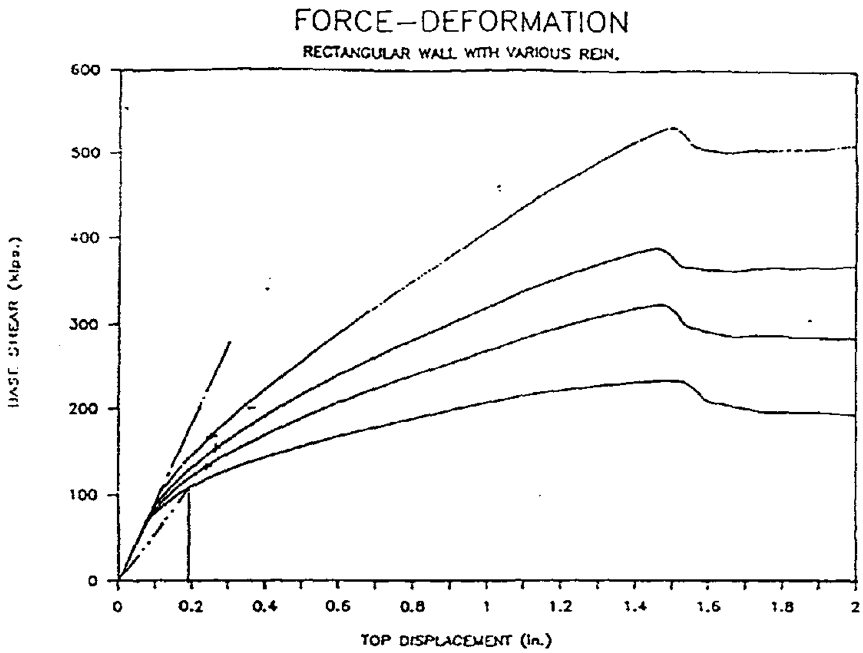


FIGURE 5-1

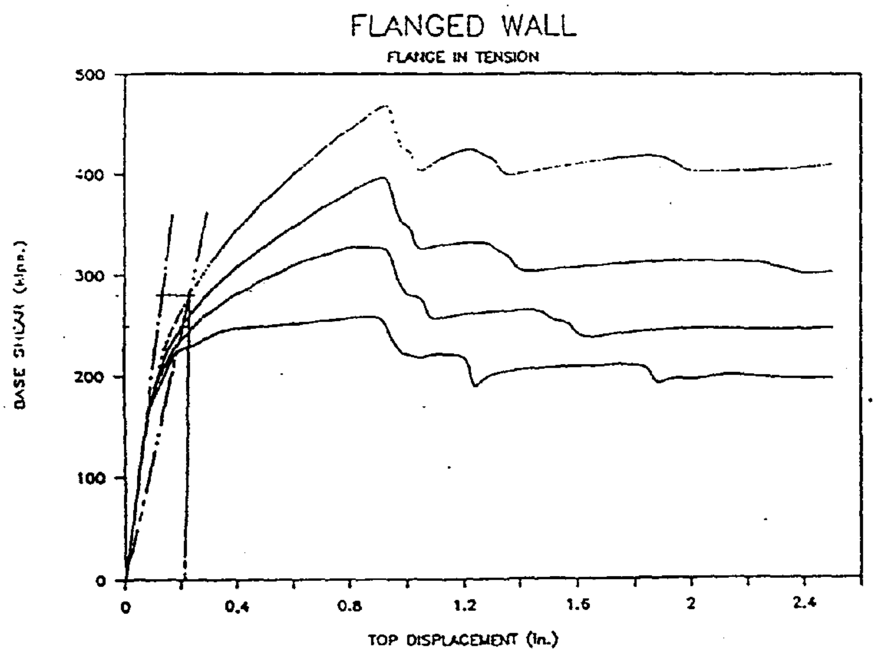


FIGURE 5-2

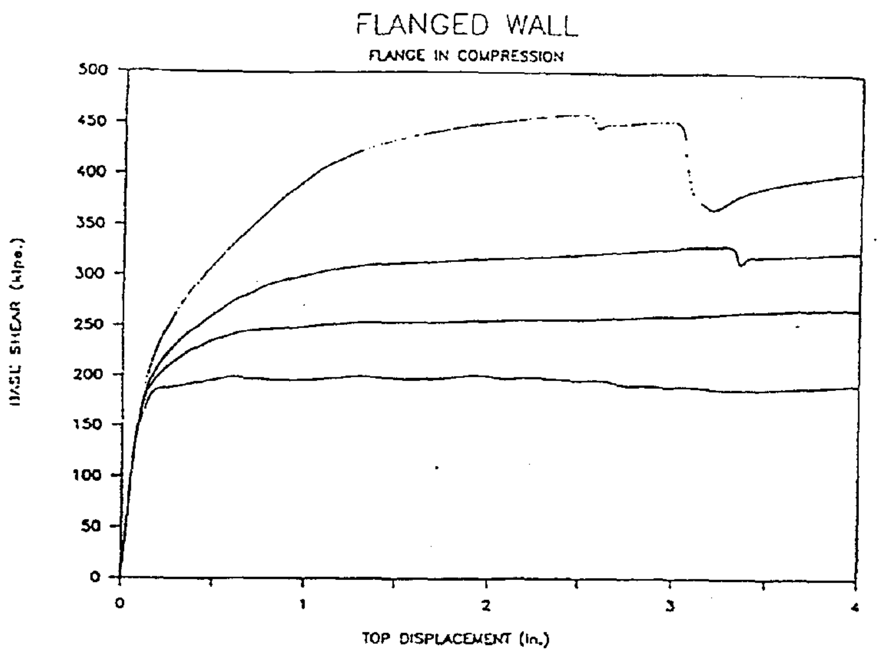


FIGURE 5-3

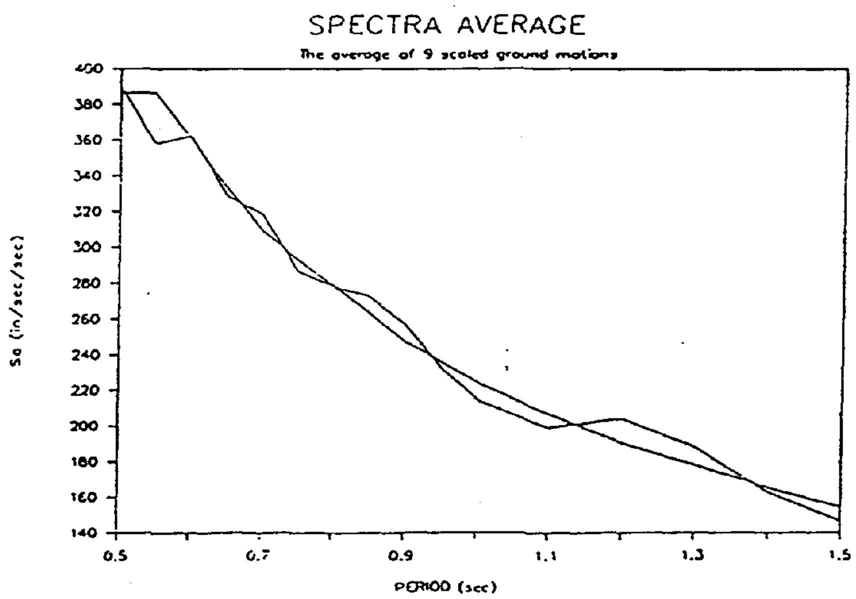
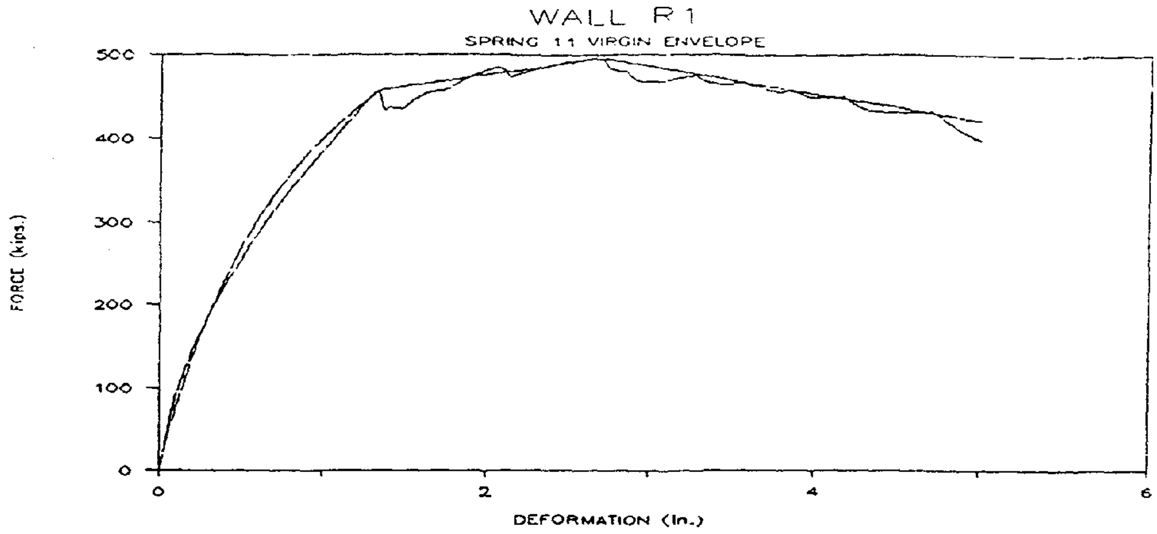


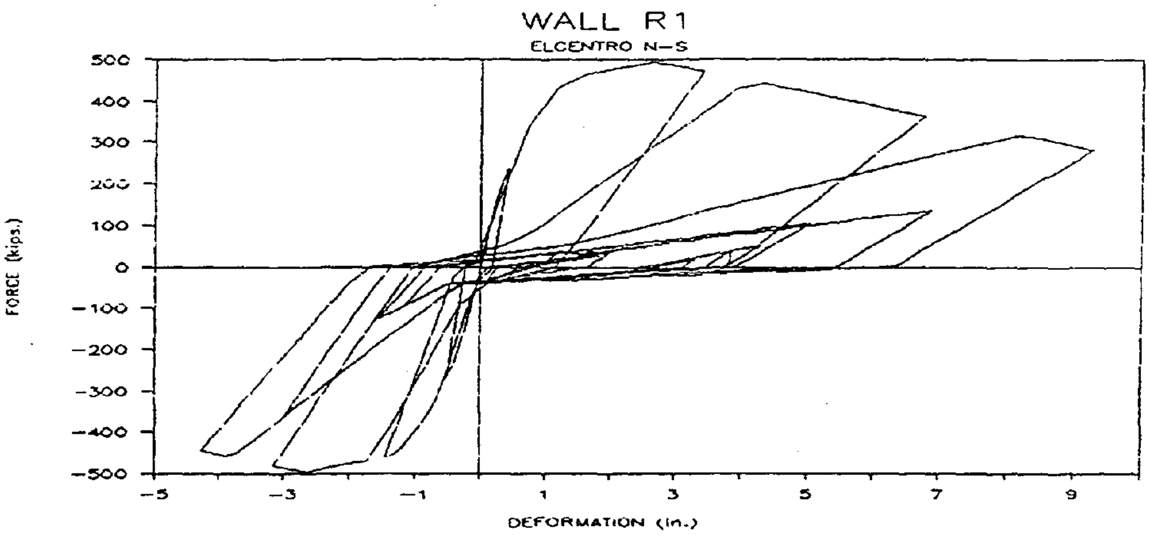
FIGURE 5-4

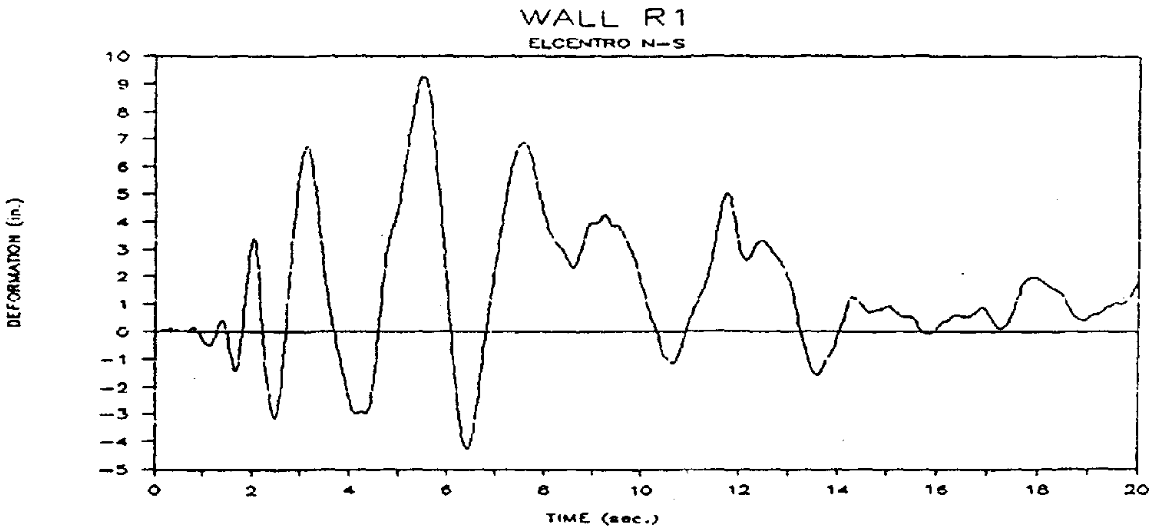
FIGURES GROUP 5 - 5

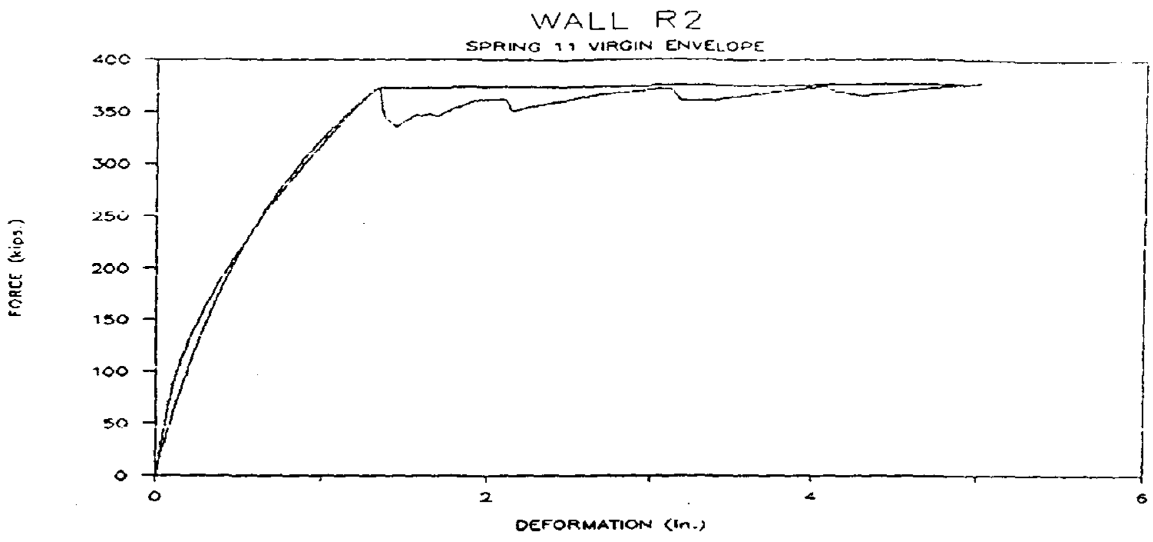
RECTANGULAR SHEAR WALLS, R1 THROUGH R4

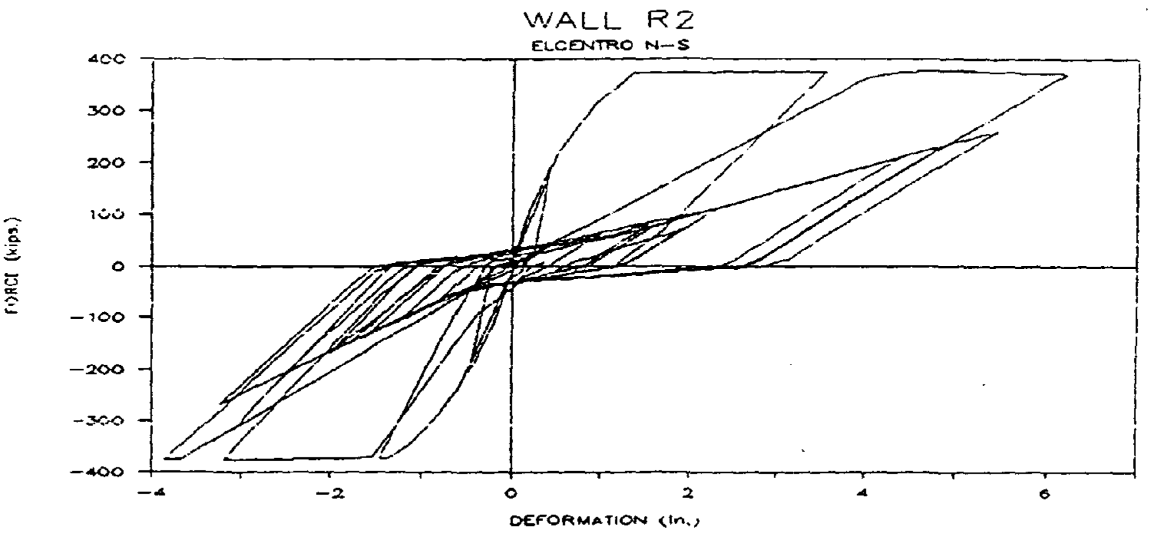
- RESULTS OF THE FEM ANALYSIS
- MODELING OF THE WALLS BY LPM/I
- DYNAMIC DISPLACEMENT CAUSED BY NS EL CENTRO X 1.31

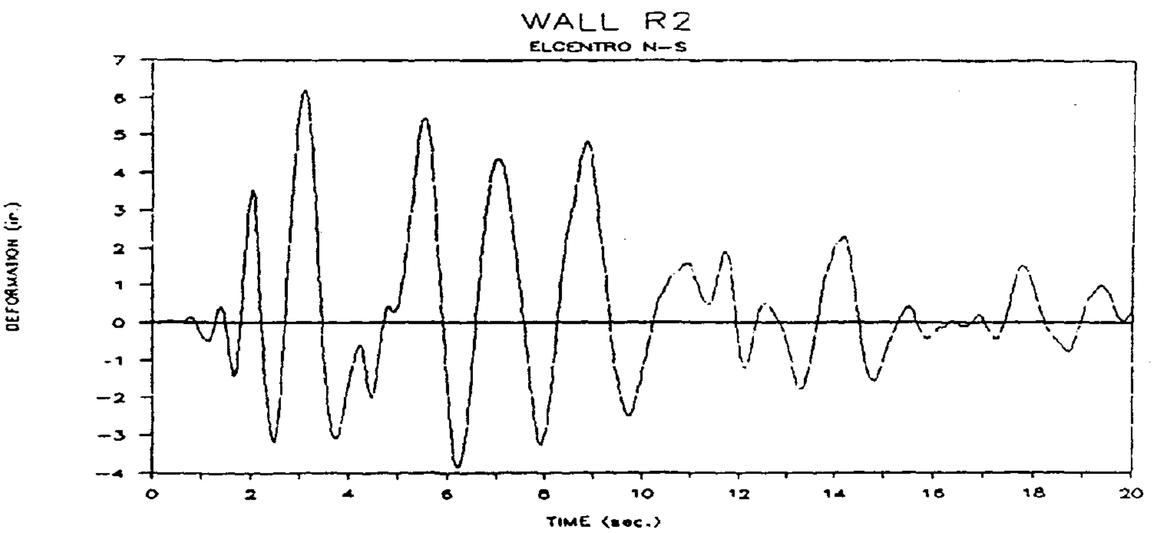

WALL R1
SPRING 11 VIRGIN ENVELOPE
FORCE (kips.)
DEFORMATION (in.)

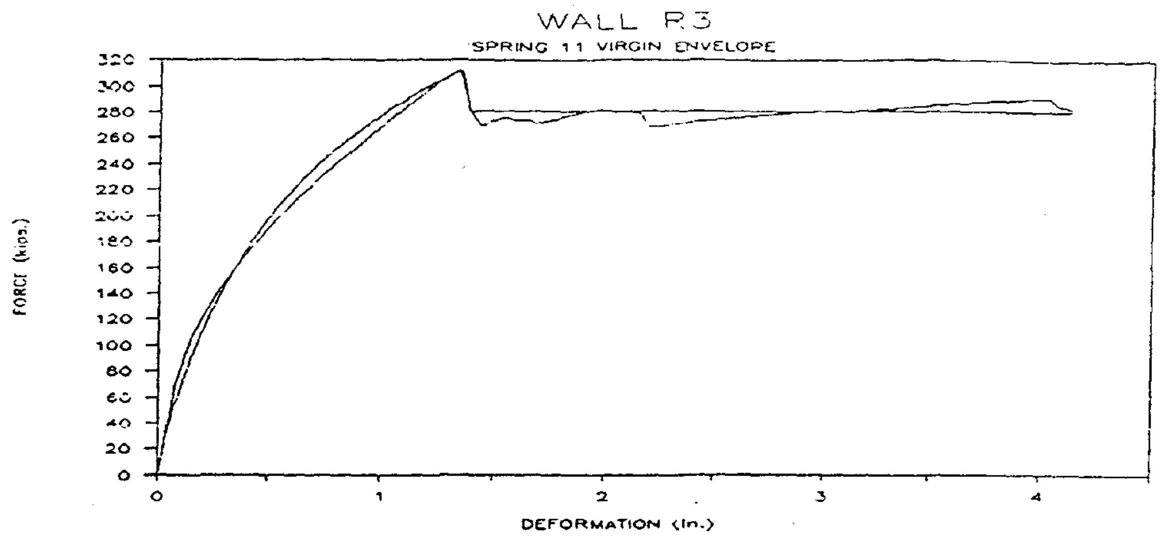

WALL R1
ELCENTRO N-S
FORCE (kips.)
DEFORMATION (in.)

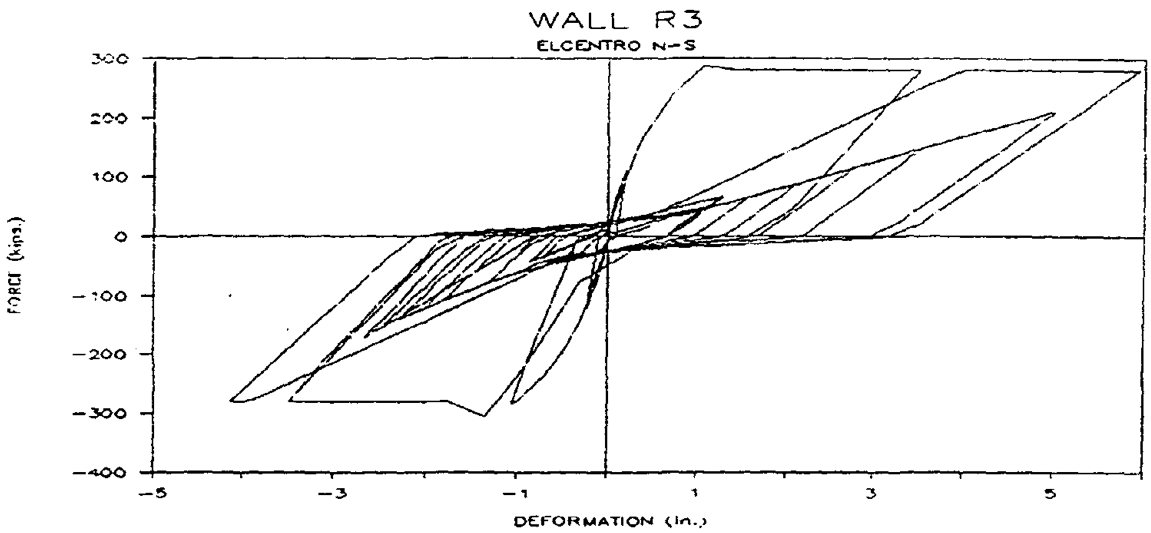

WALL R1
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)

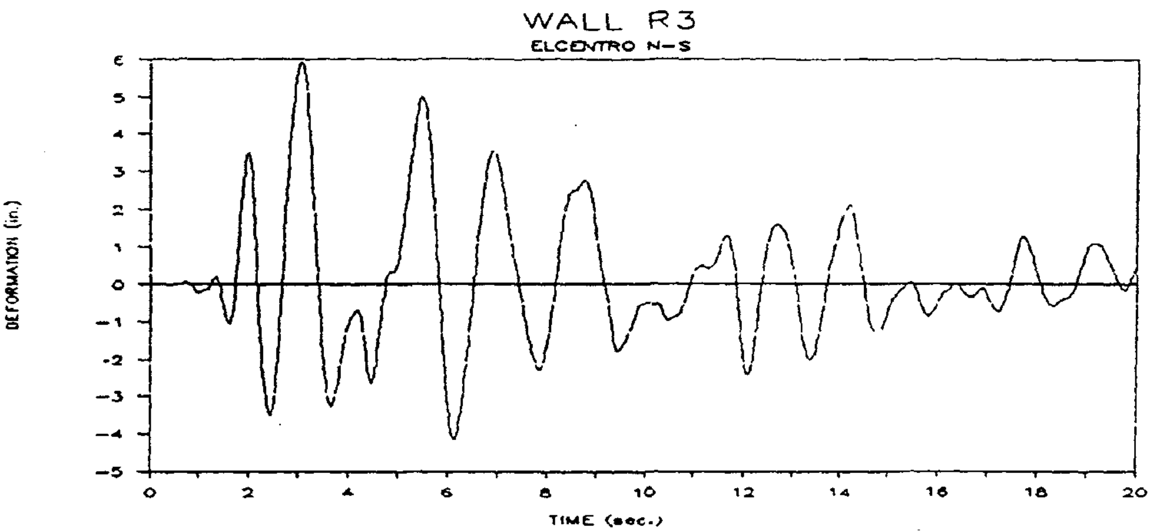

WALL R2
SPRING 11 VIRGIN ENVELOPE
FORCE (kips)
DEFORMATION (In.)

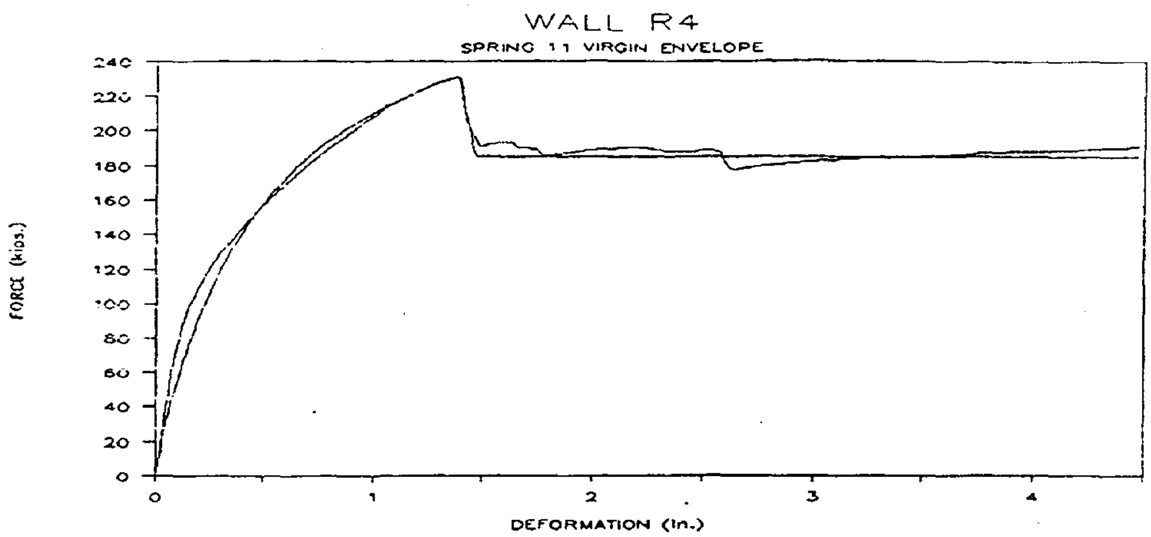

WALL R2
ELCENTRO N-S
FORCE (kips)
DEFORMATION (In.)

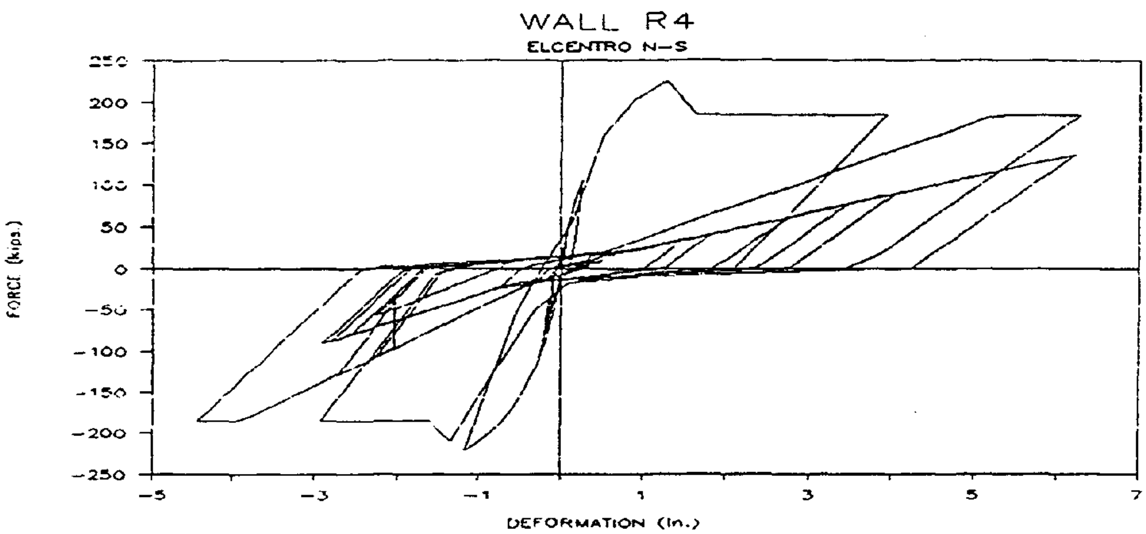

WALL R2
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)

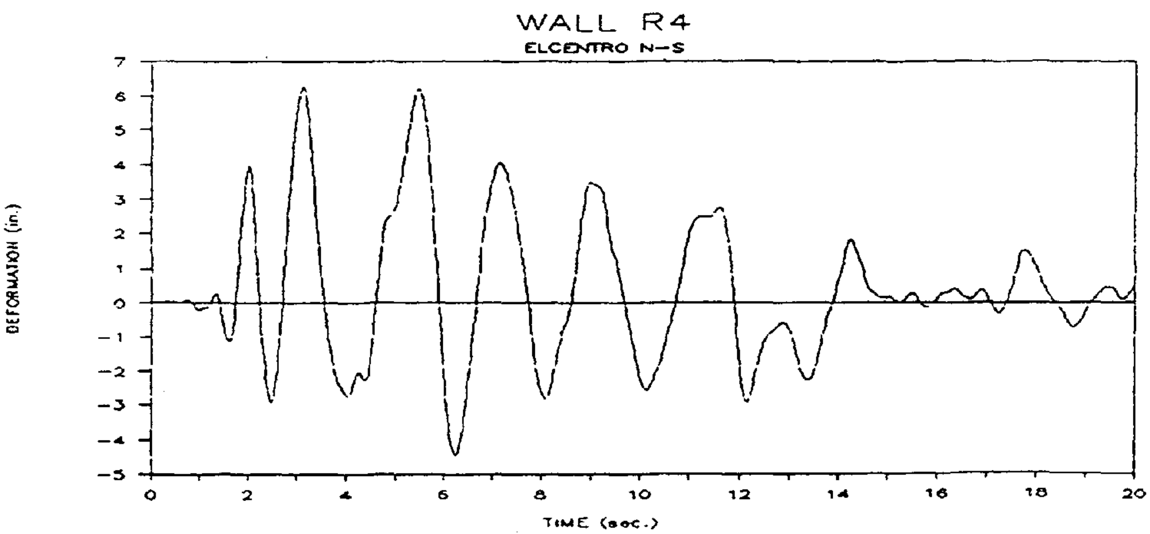

WALL R3
SPRING 11 VIRGIN ENVELOPE
FORCE (kips)
DEFORMATION (in.)


WALL R3
ELCENTRO N-S
FORCE (kips)
DEFORMATION (in.)


WALL R3
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)


WALL R4
SPRING 11 VIRGIN ENVELOPE
FORCE (kips)
DEFORMATION (in.)

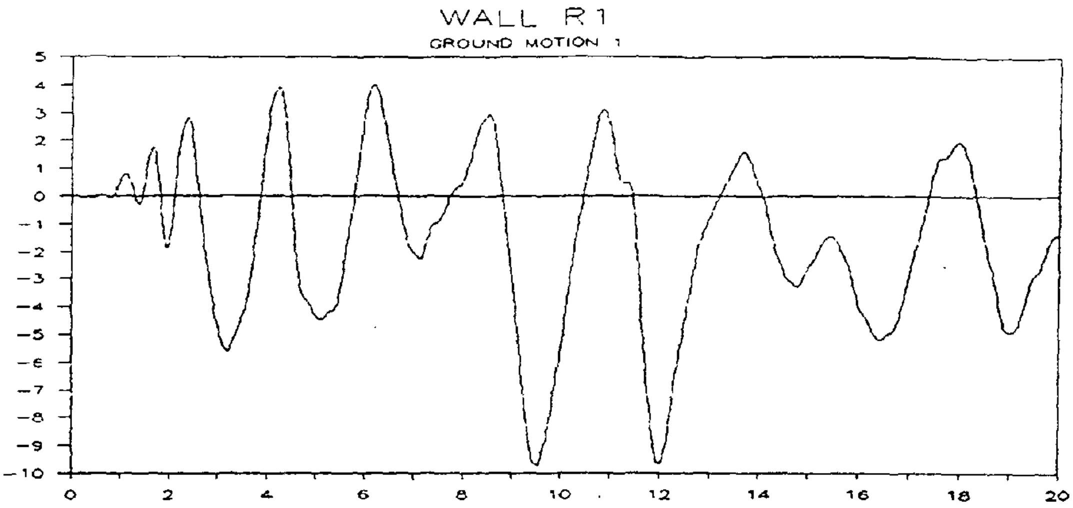

WALL R4
ELCENTRO N-S
FORCE (kips)
DEFORMATION (in.)

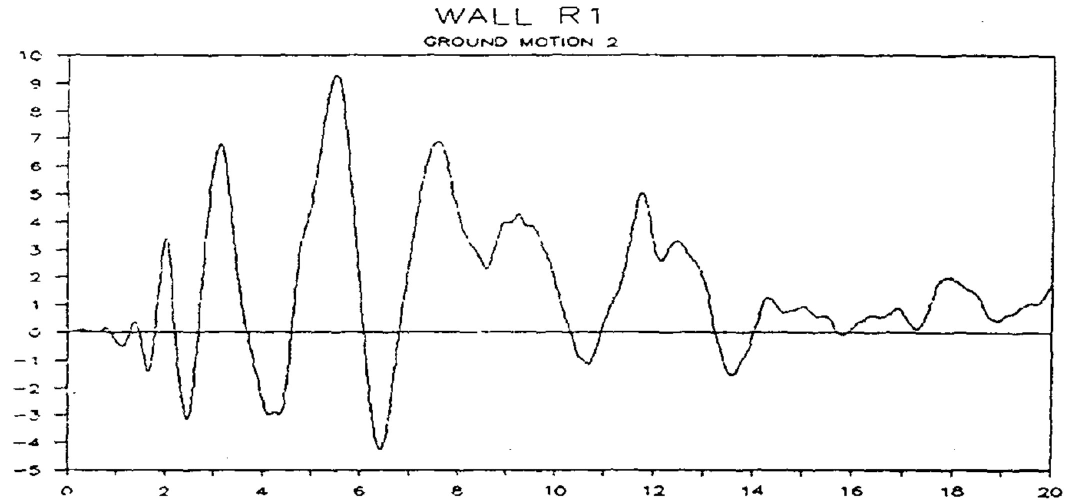

WALL R4
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)

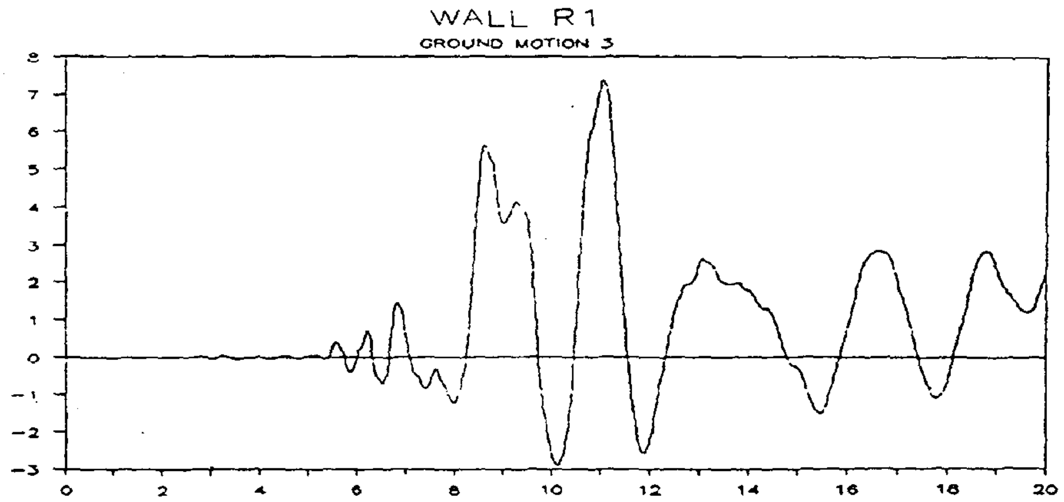
FIGURES GROUP 5 - 6

WALL R1

RESPONSE TO NINE SCALED GROUND MOTION RECORDS


WALL R1
GROUND MOTION 1
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20


WALL R1
GROUND MOTION 2
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
0
2
4
6
8
10
12
14
16
18
20


WALL R1
GROUND MOTION 3
8
7
6
5
4
3
2
1
0
-1
-2
-3
0
2
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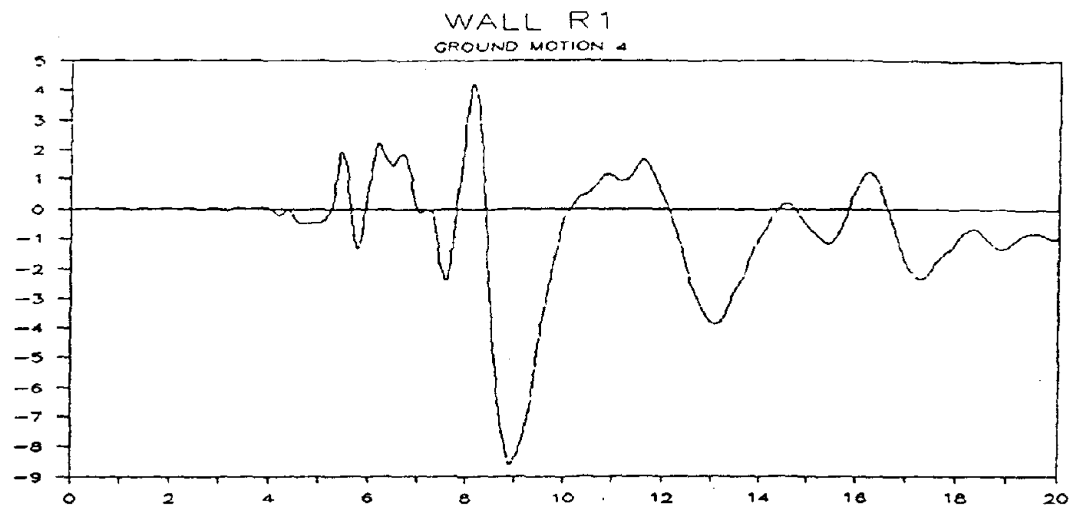
12
14
16
18
20


WALL R1
GROUND MOTION 4
5
4
3
2
1
0
-1
-2
-3
-4
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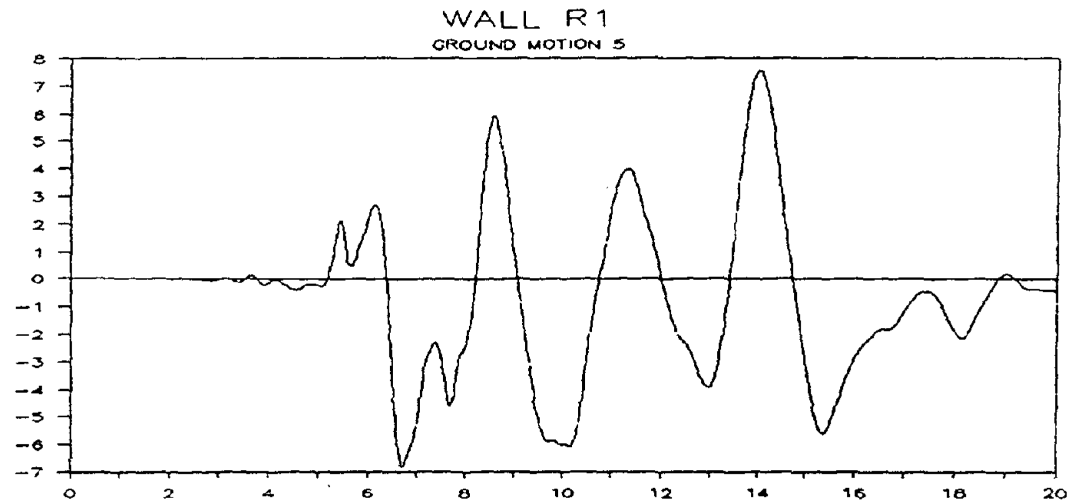
12
14
16
18
20


WALL R1
GROUND MOTION 5
8
7
6
5
4
3
2
1
0
-1
-2
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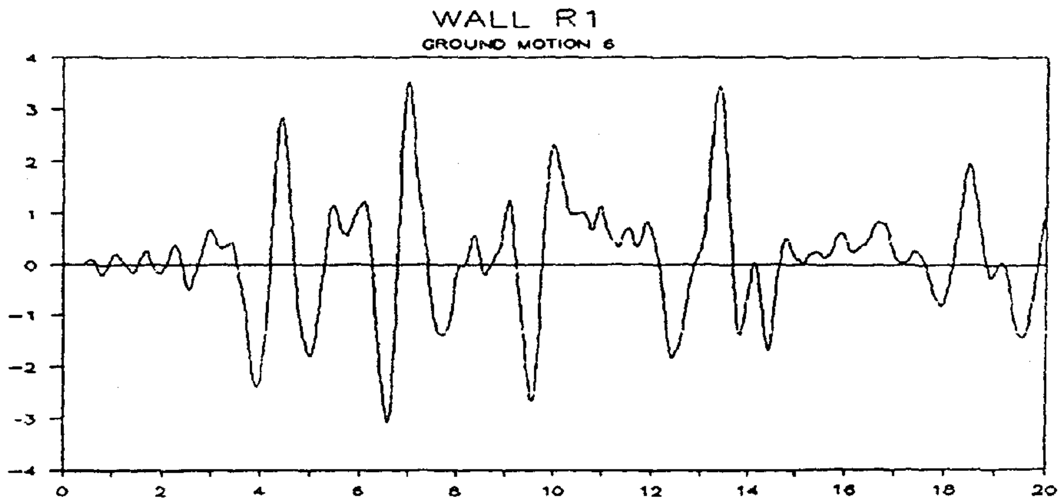
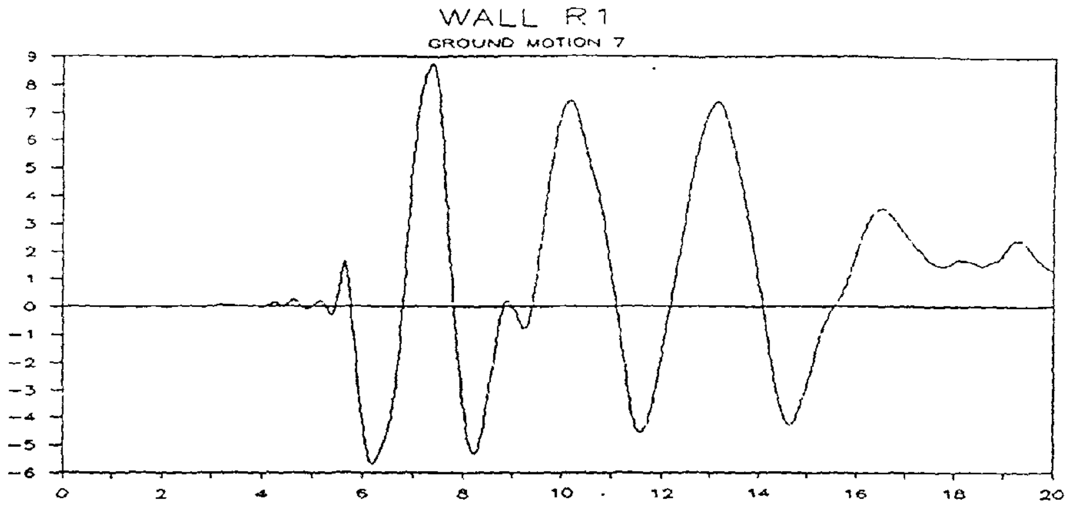
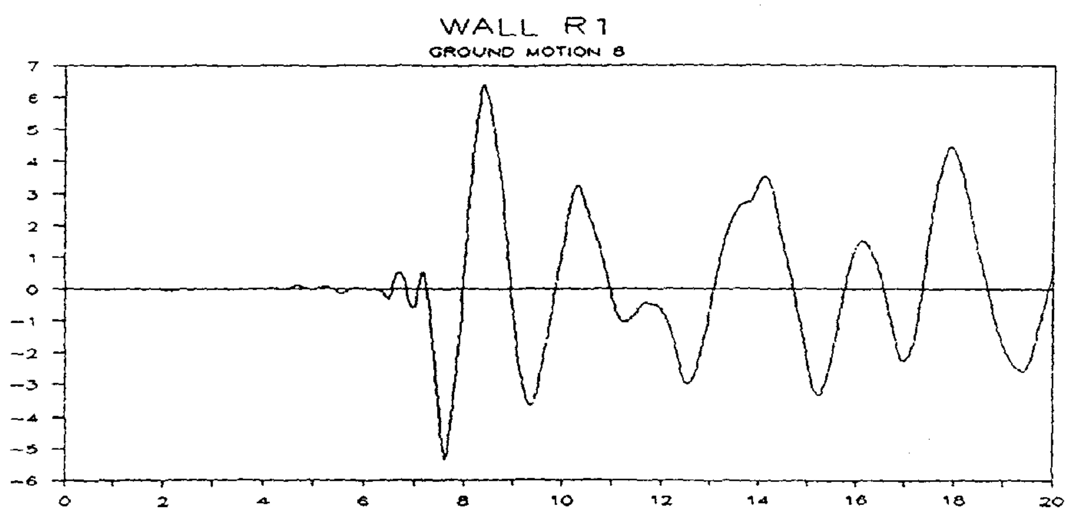
12
14
16
18
20


WALL R1
GROUND MOTION 6
4
3
2
1
0
-1
-2
-3
-4
0
2
4
6
8
10
12
14
16
18
20


WALL R1
GROUND MOTION 7
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
0
2
4
6
8
10
12
14
16
18
20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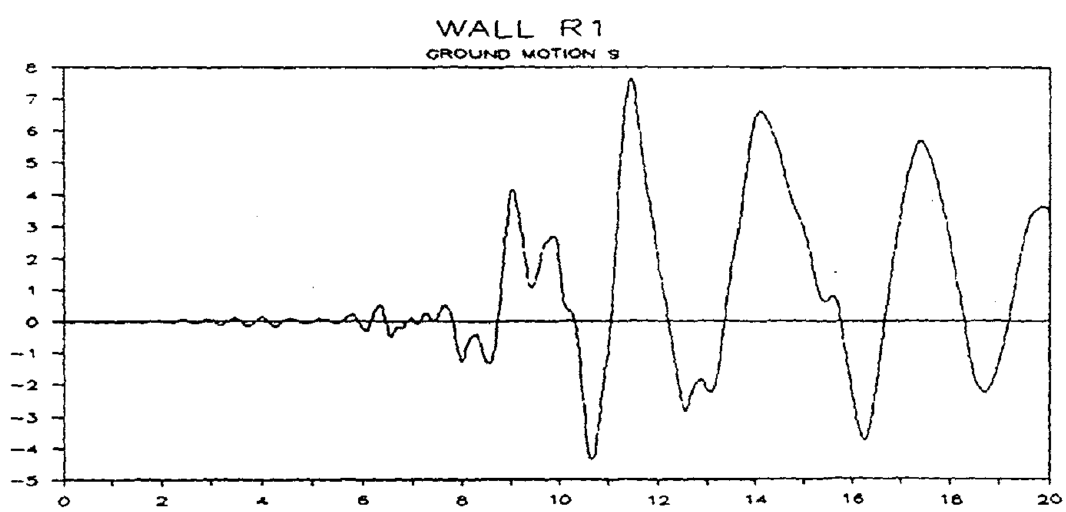
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GROUND MOTION 9
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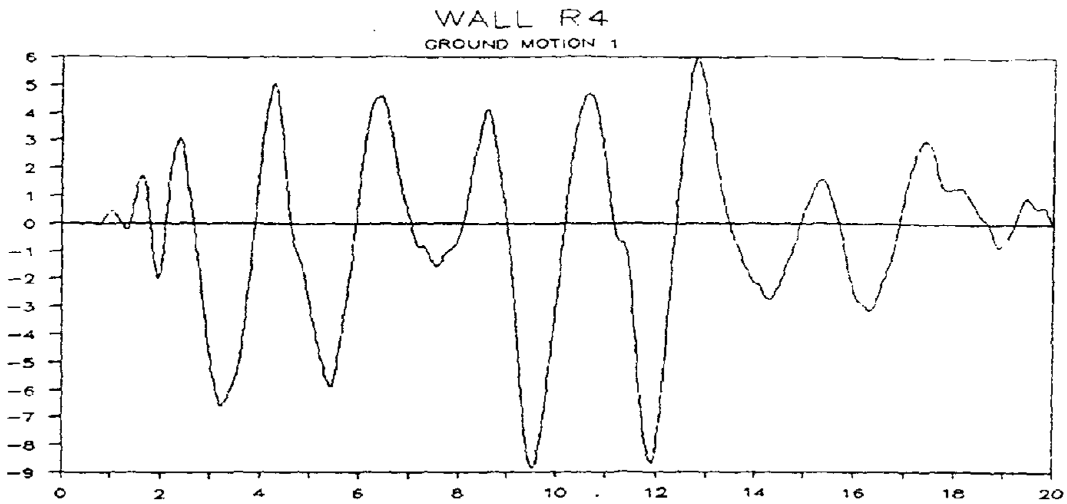
FIGURES GROUP 5 - 7

WALL R4

RESPONSE TO NINE SCALED GROUND MOTION RECORDS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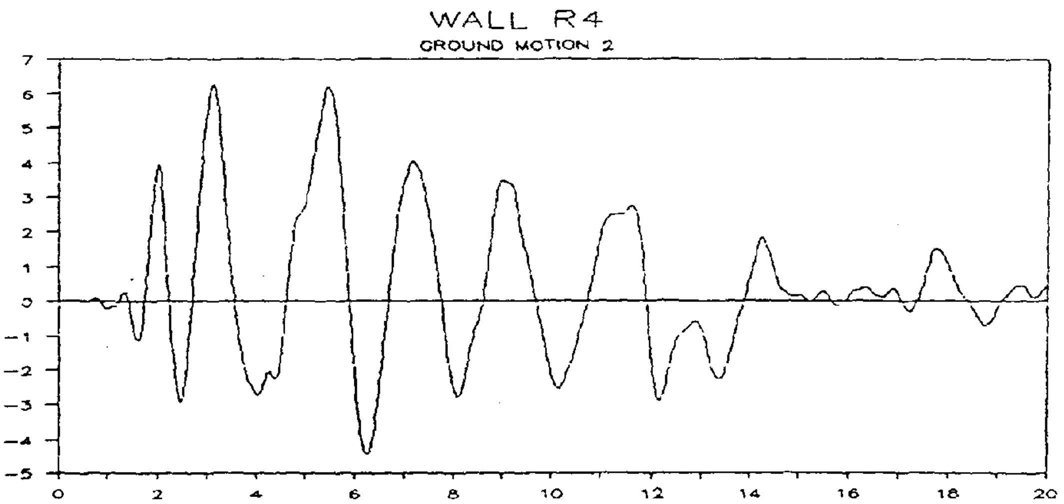
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GROUND MOTION 2
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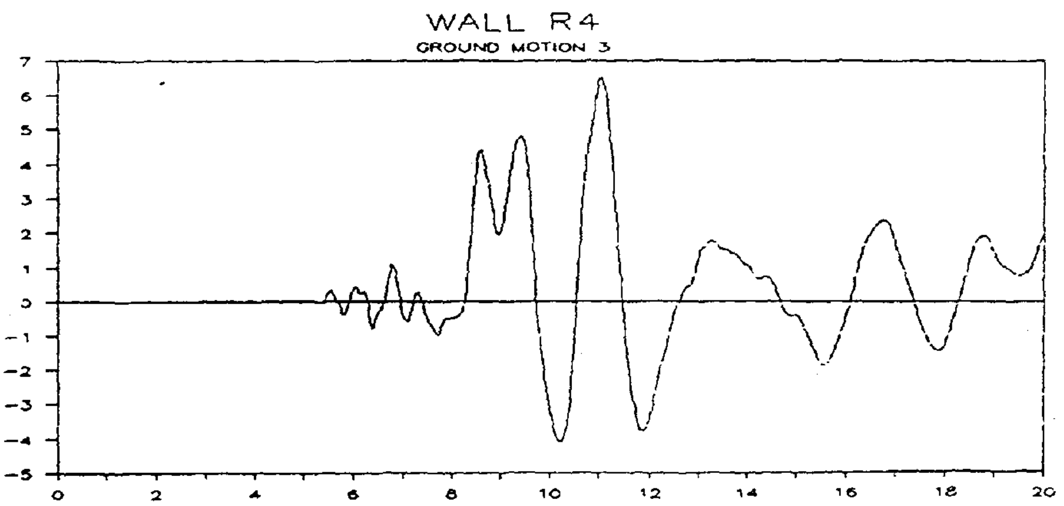
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GROUND MOTION 3
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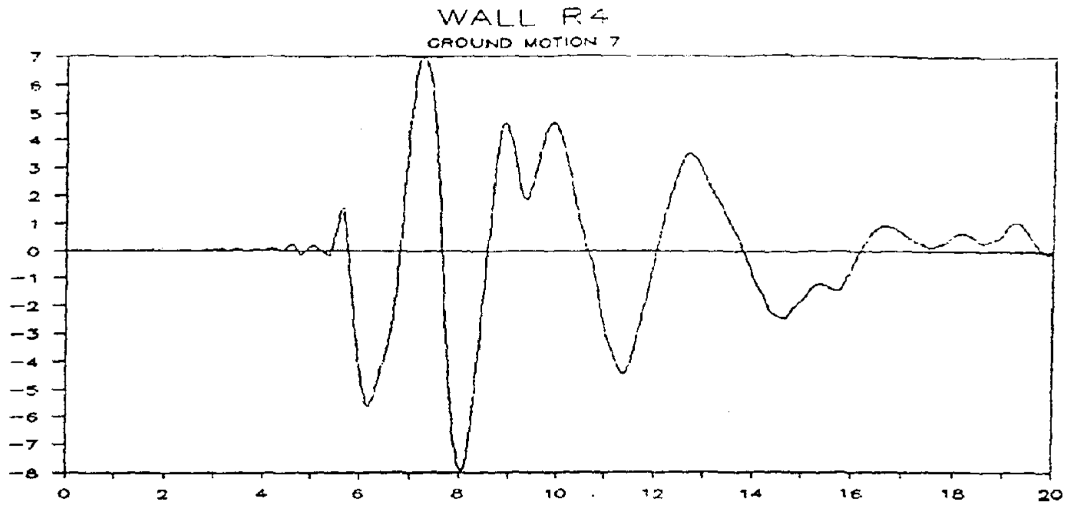
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GROUND MOTION 7
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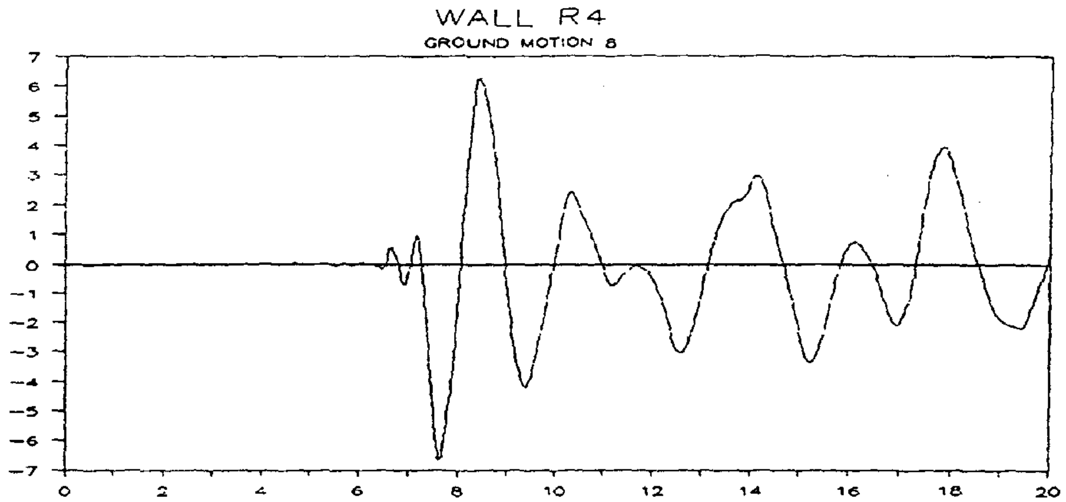
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GROUND MOTION 8
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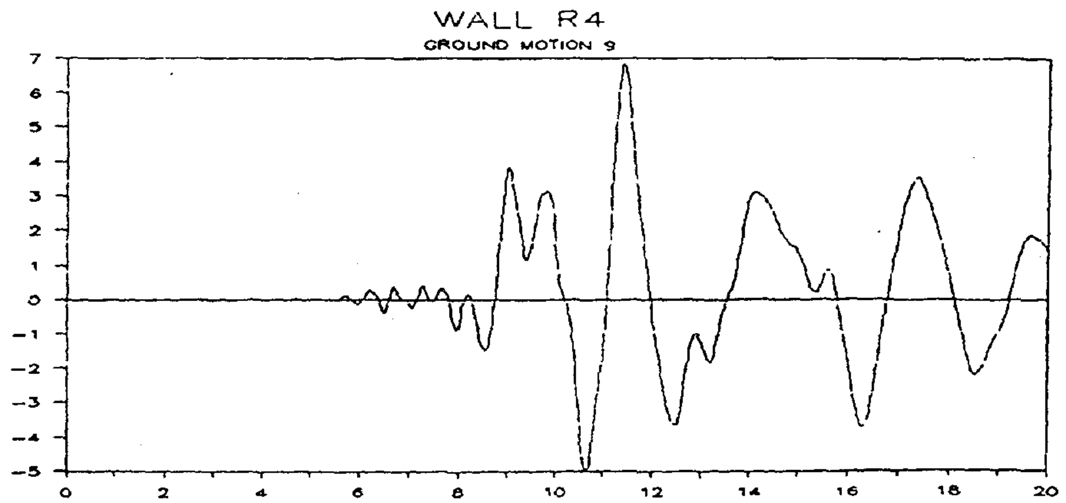
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GROUND MOTION 9
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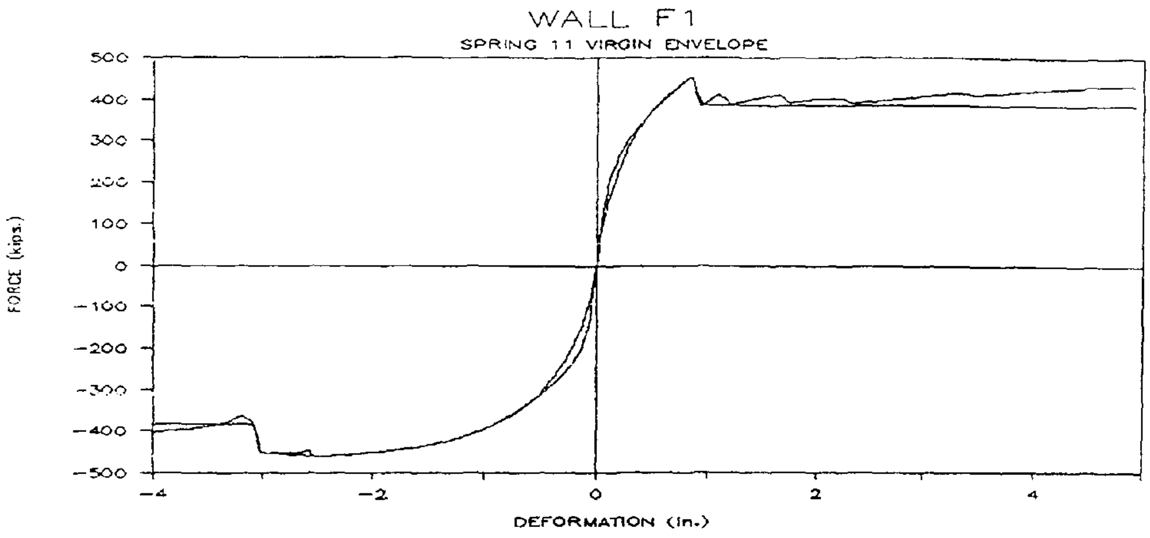
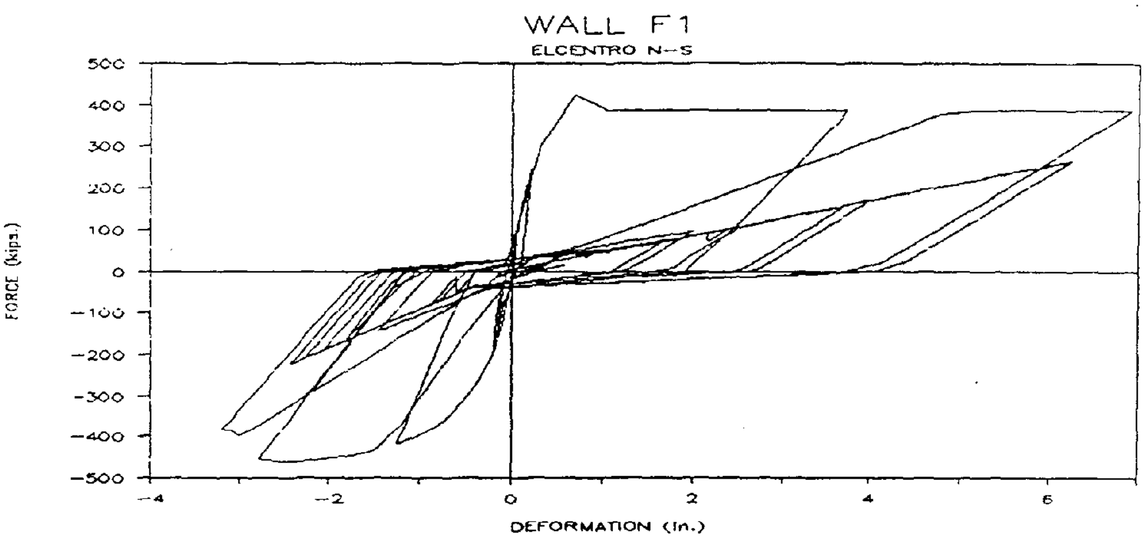
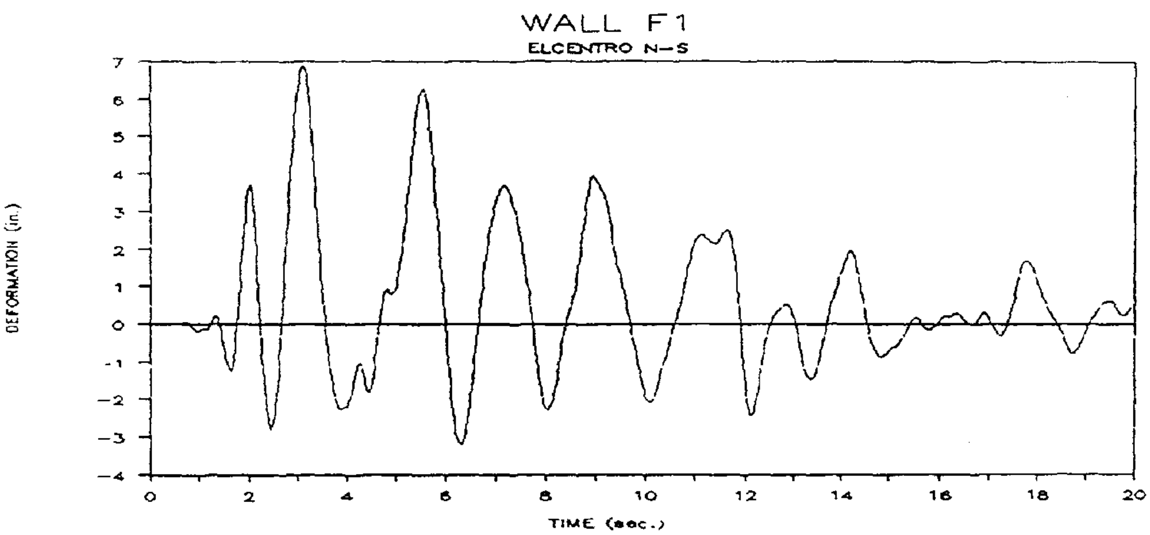

WALL F1
SPRING 11 VIRGIN ENVELOPE
FORCE (kips)
DEFORMATION (in.)

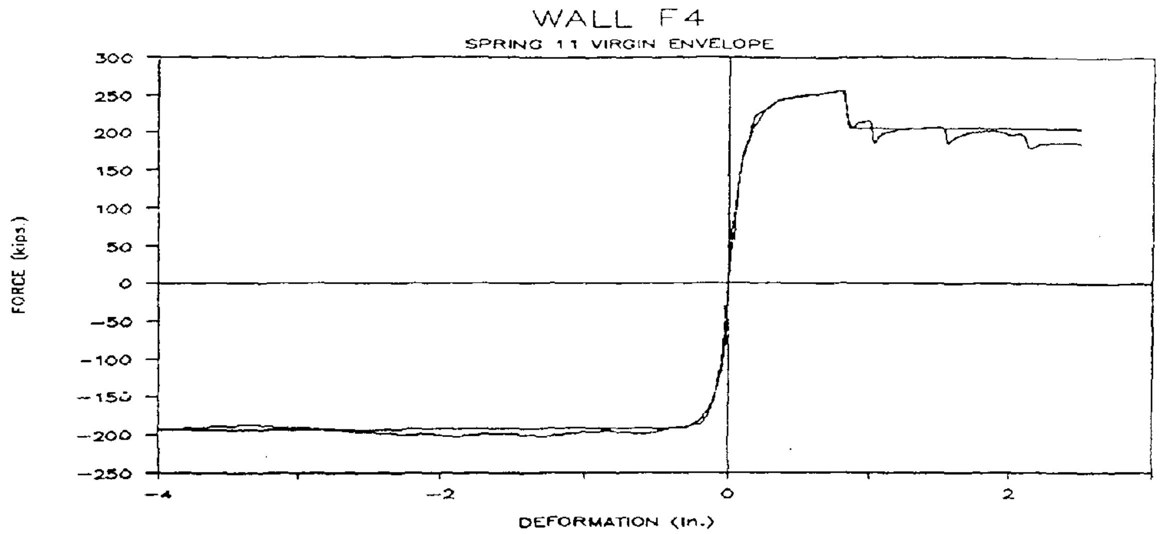

WALL F1
ELCENTRO N-S
FORCE (kips)
DEFORMATION (in.)


WALL F1
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)

FIGURES GROUP 5 - 8

FLANGED SHEAR WALLS, F1 AND F4

- RESULTS OF THE FEM ANALYSIS
- MODELING OF THE WALLS BY LPM/I
- DYNAMIC DISPLACEMENTS CAUSED BY NS EL CENTRO X 1.31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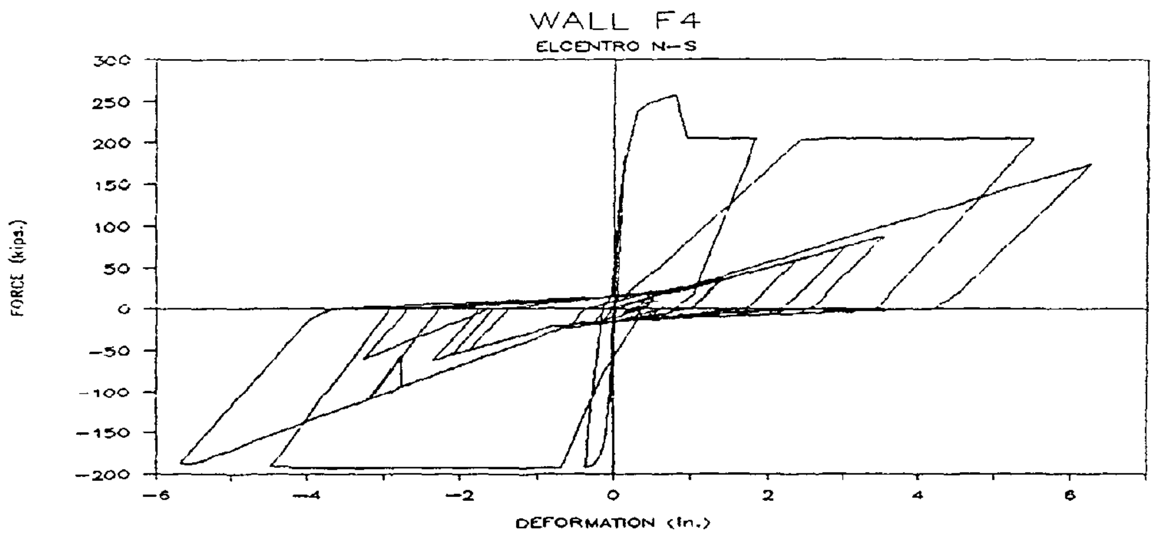
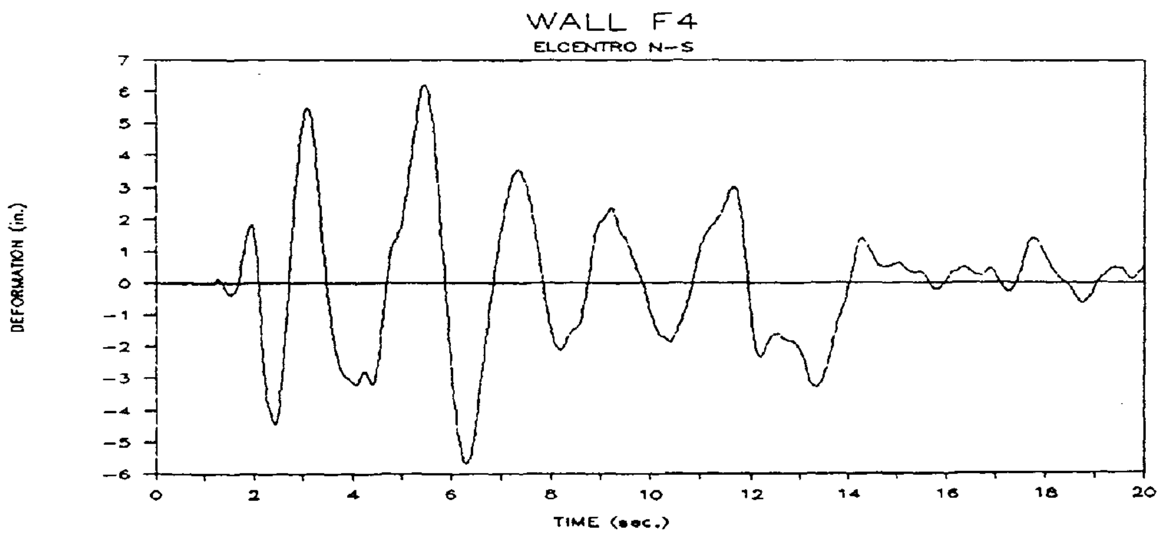
SPRING 11 VIRGIN ENVELOPE
FORCE (kips.)
DEFORMATION (in.)

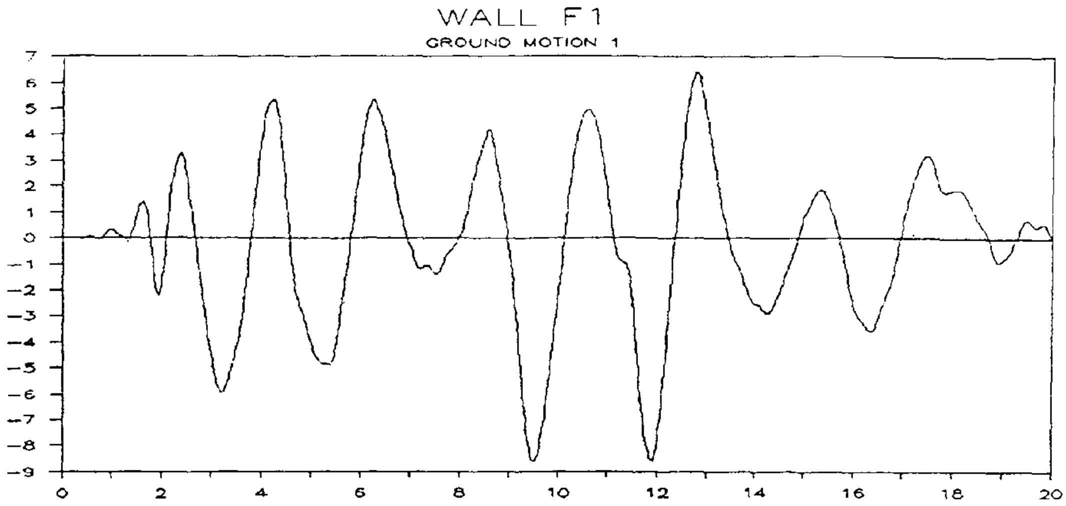

WALL F4
ELCENTRO N-S
FORCE (kips.)
DEFORMATION (in.)


WALL F4
ELCENTRO N-S
DEFORMATION (in.)
TIME (sec.)

FIGURES GROUP 5 - 9

WALL F1

RESPONSE TO NINE SCALED GROUND MOTION RECORDS


WALL F1
GROUND MOTION 1
7
6
5
4
3
2
1
0
-1
-2
-3
-4
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-6
-7
-8
-9
0
2
4
6
8
10
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14
16
18
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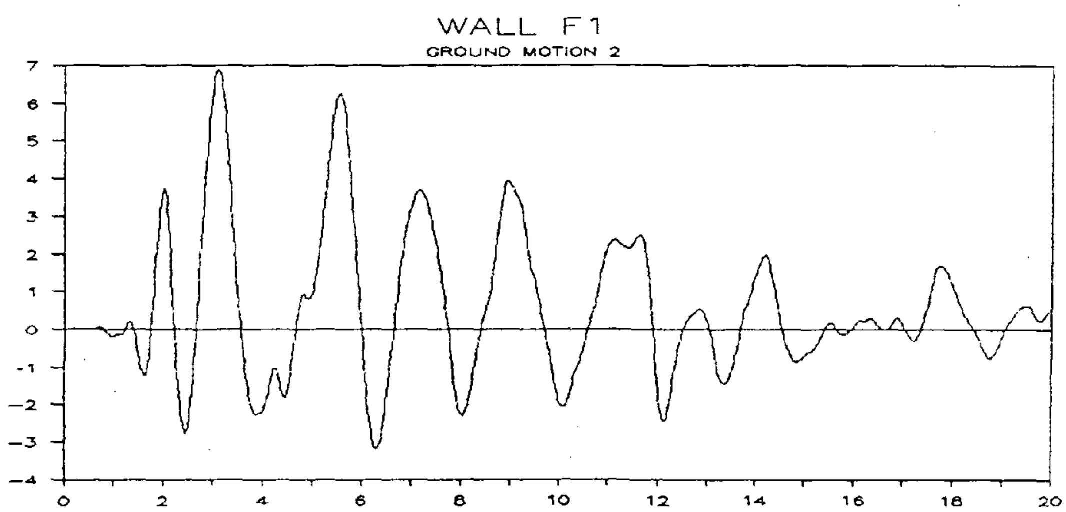
WALL F1
GROUND MOTION 2
7
6
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4
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4
6
8
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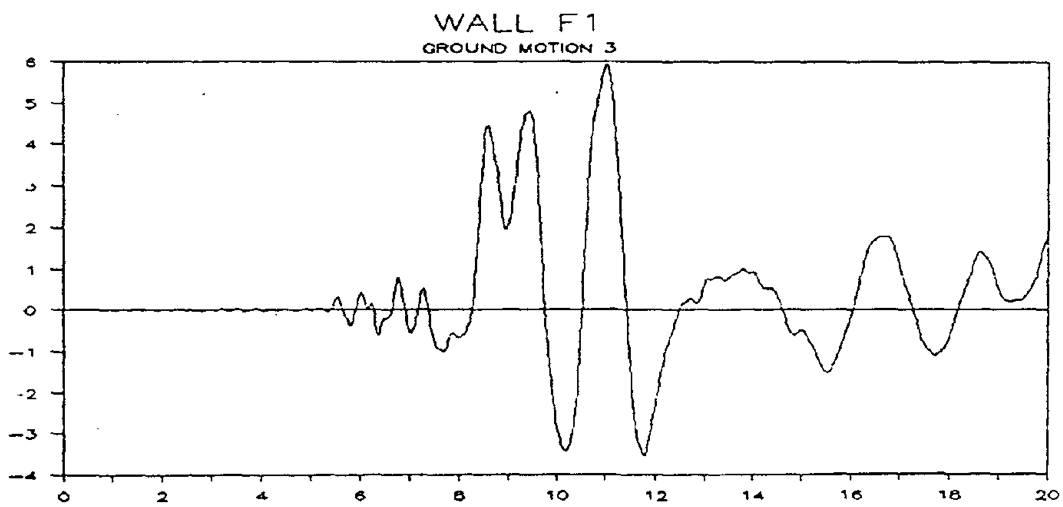
WALL F1
GROUND MOTION 3
6
5
4
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-2
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4
6
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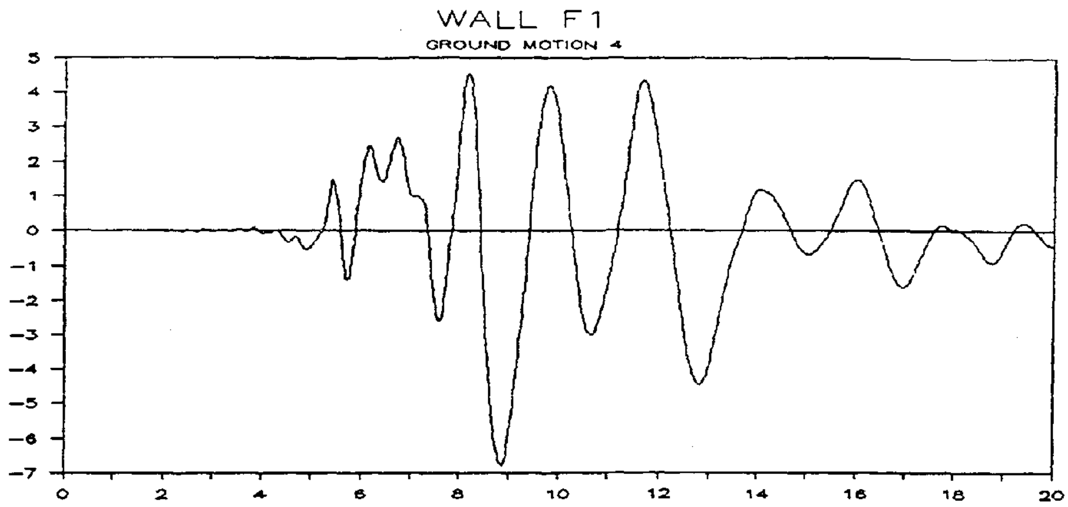
WALL F1
GROUND MOTION 4
5
4
3
2
1
0
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0
2
4
6
8
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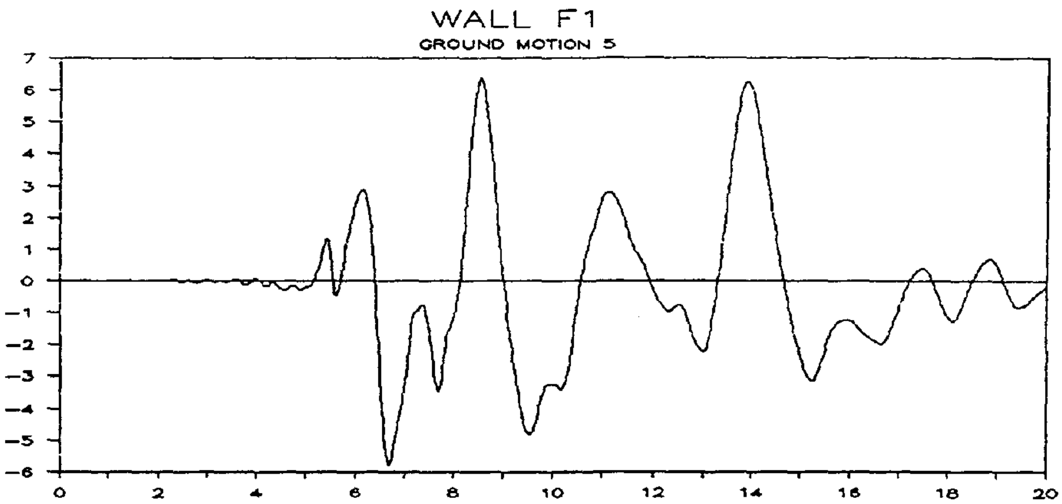
WALL F1
GROUND MOTION 5
7
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5
4
3
2
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0
−1
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−3
−4
−5
−6
0
2
4
6
8
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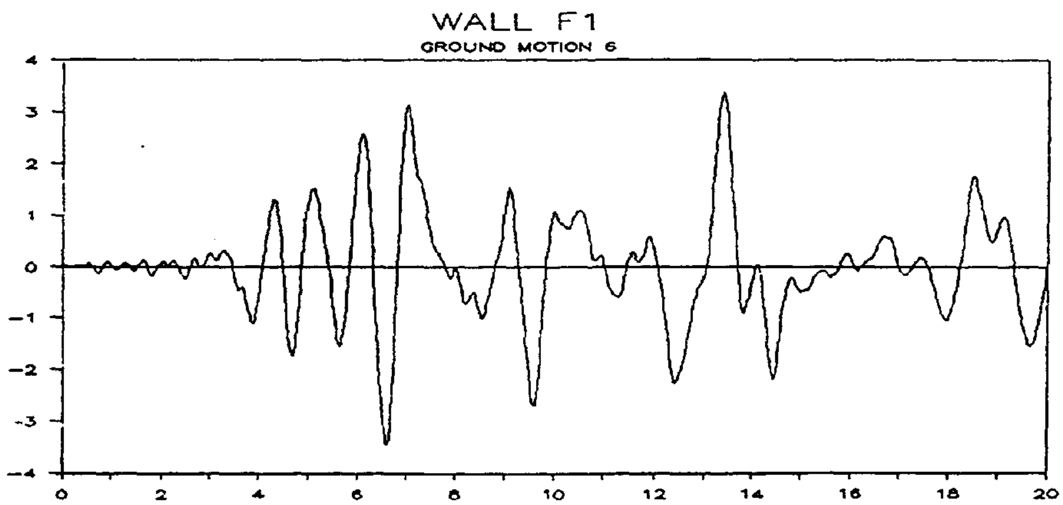
WALL F1
GROUND MOTION 6
4
3
2
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−1
−2
-3
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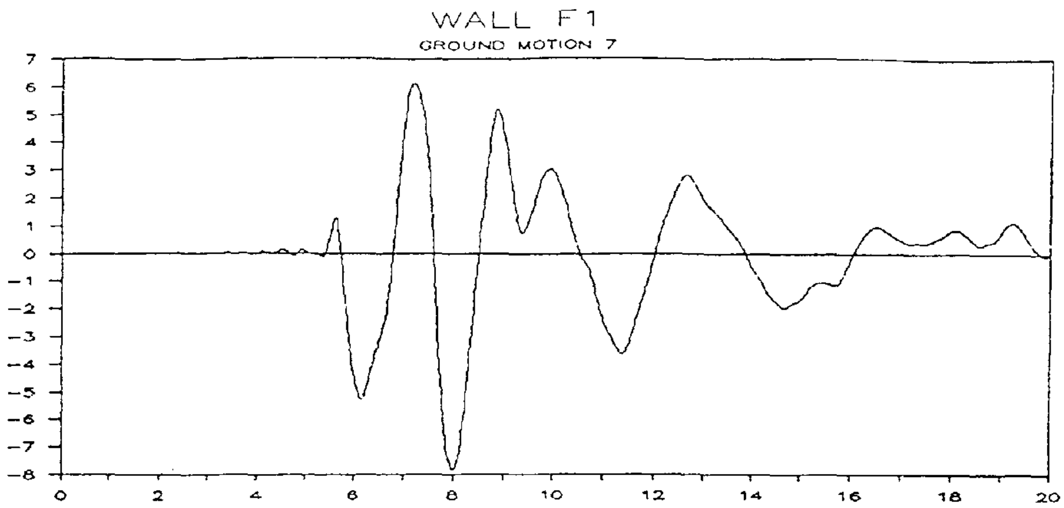
WALL F1
GROUND MOTION 7
7
6
5
4
3
2
1
0
-1
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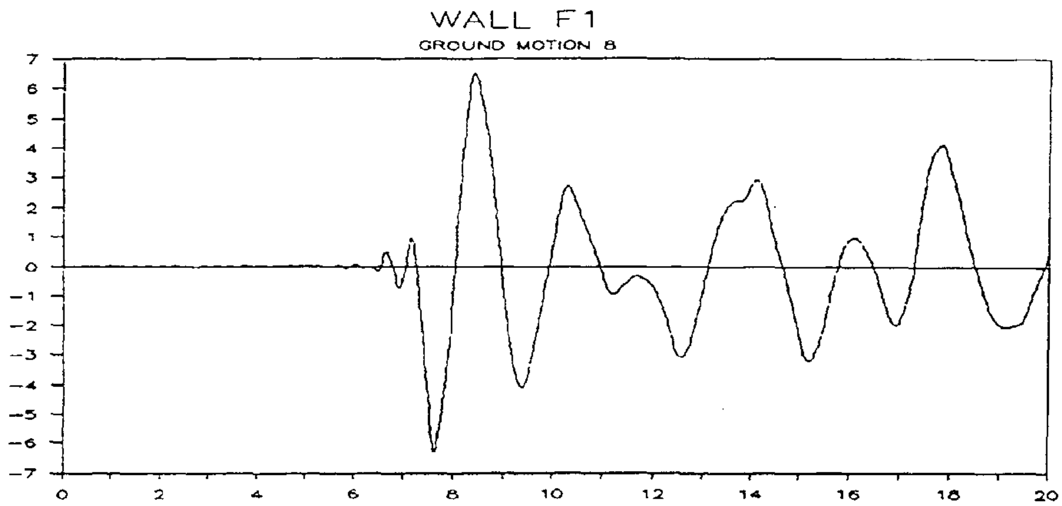
WALL F1
GROUND MOTION 8
7
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5
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3
2
1
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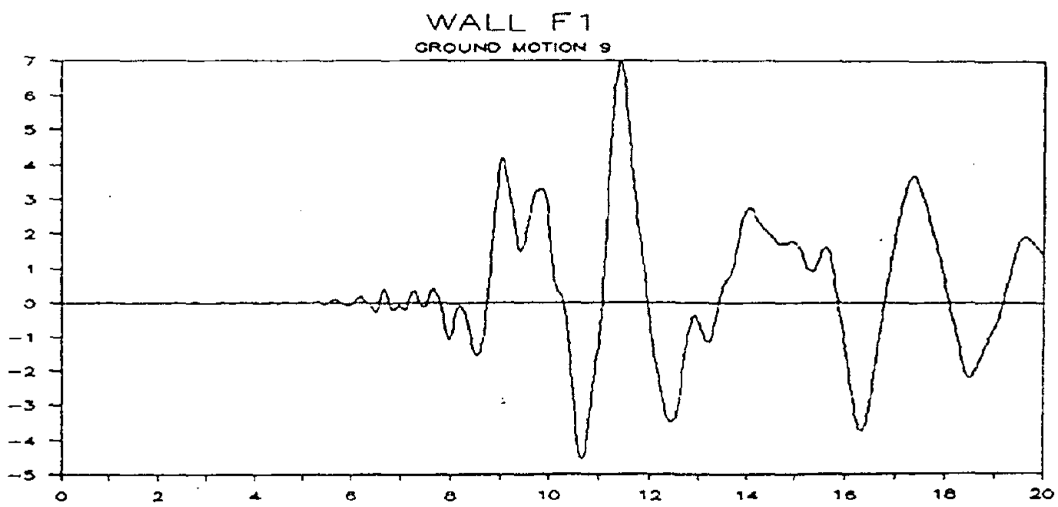
WALL F1
GROUND MOTION 9
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2
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-2
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-4
-5
0
2
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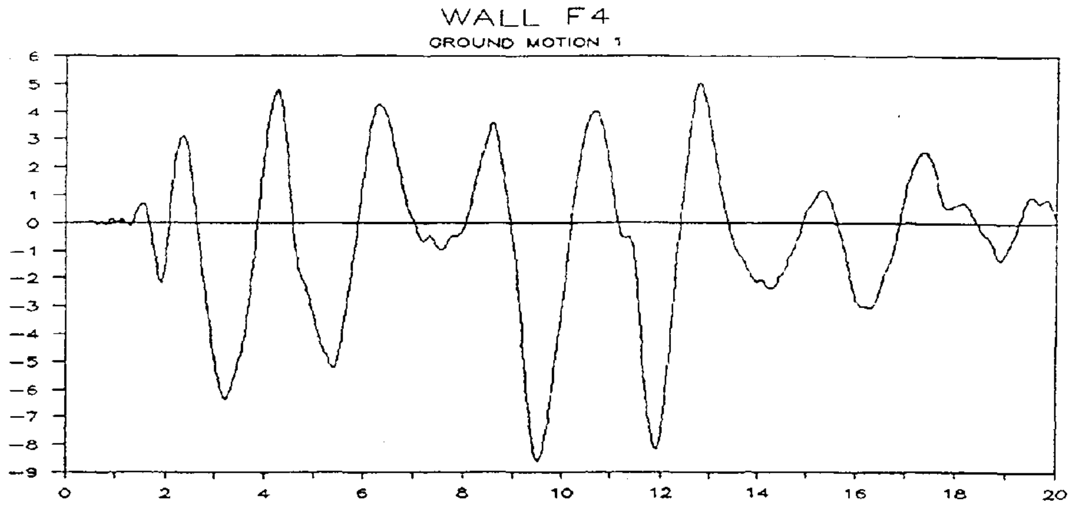
FIGURES GROUP 5 - 10

WALL F4

RESPONSE TO NINE SCALED GROUND MOTION RECORDS


WALL F4
GROUND MOTION 1
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5
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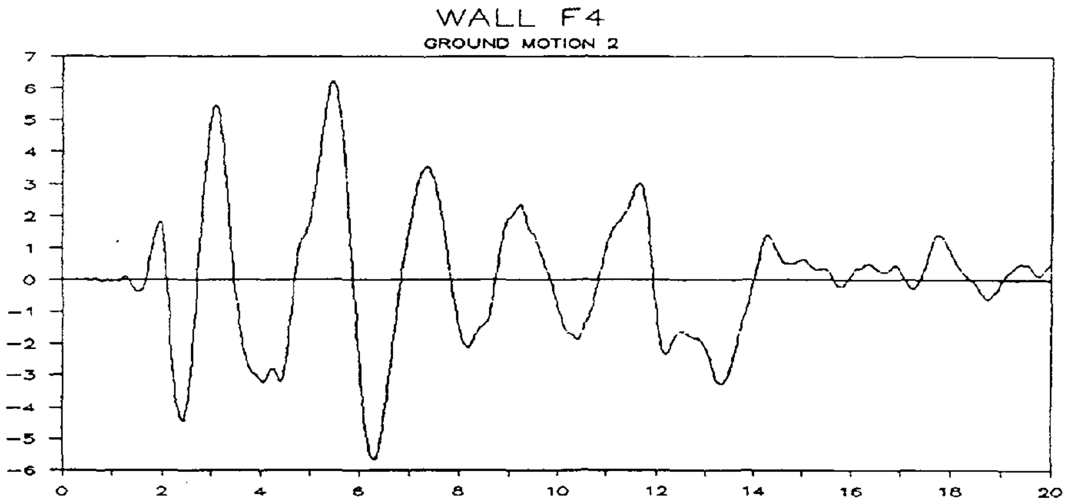
WALL F4
GROUND MOTION 2
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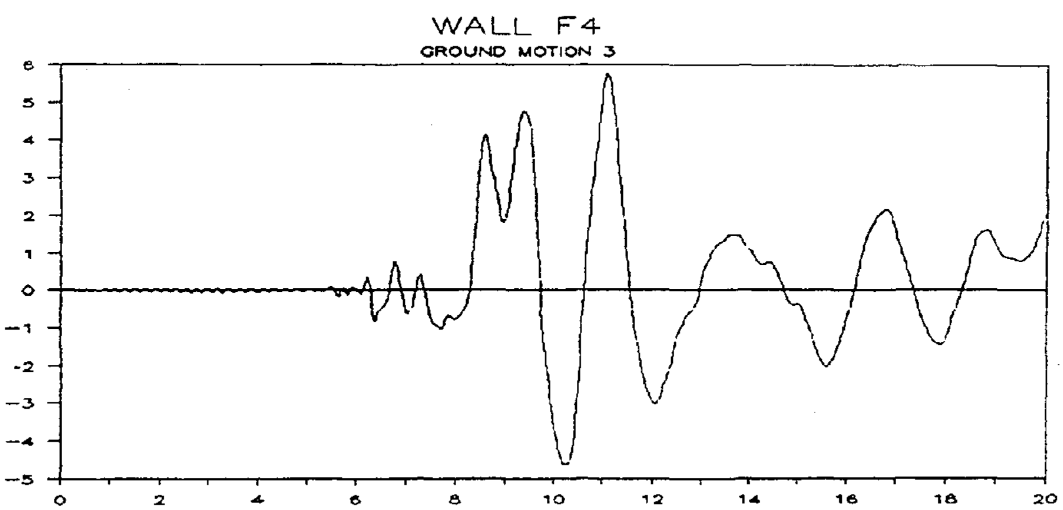
WALL F4
GROUND MOTION 3
6
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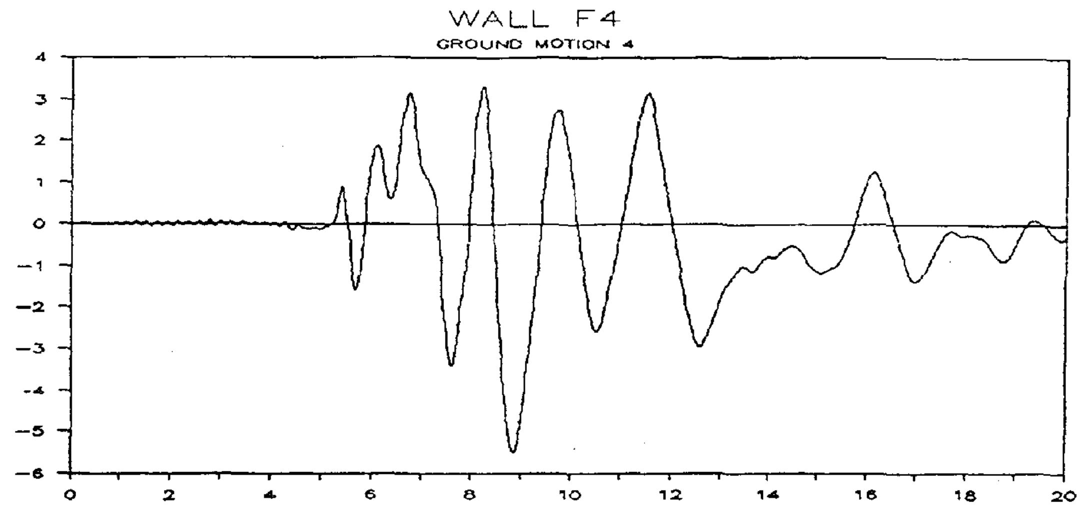

WALL F4
GROUND MOTION 4
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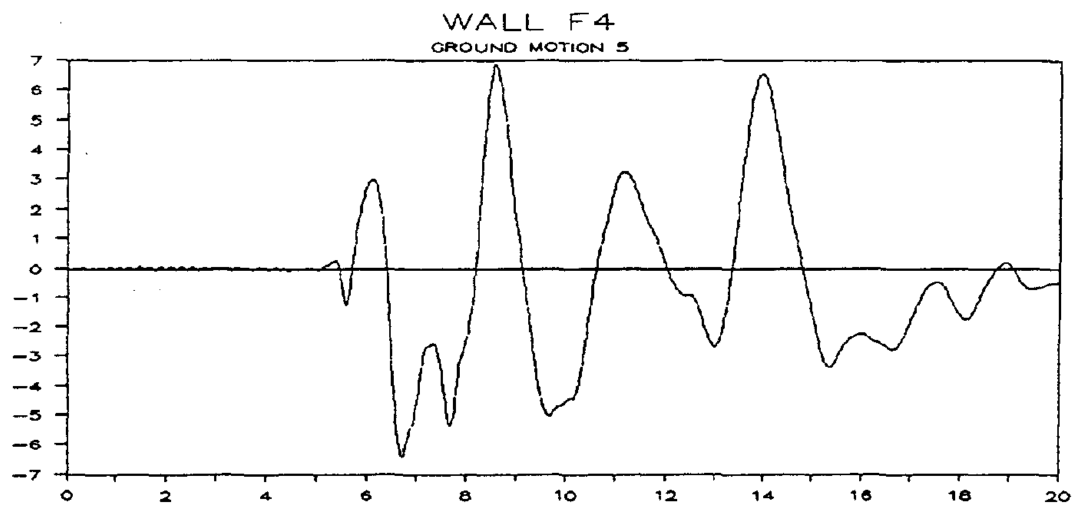

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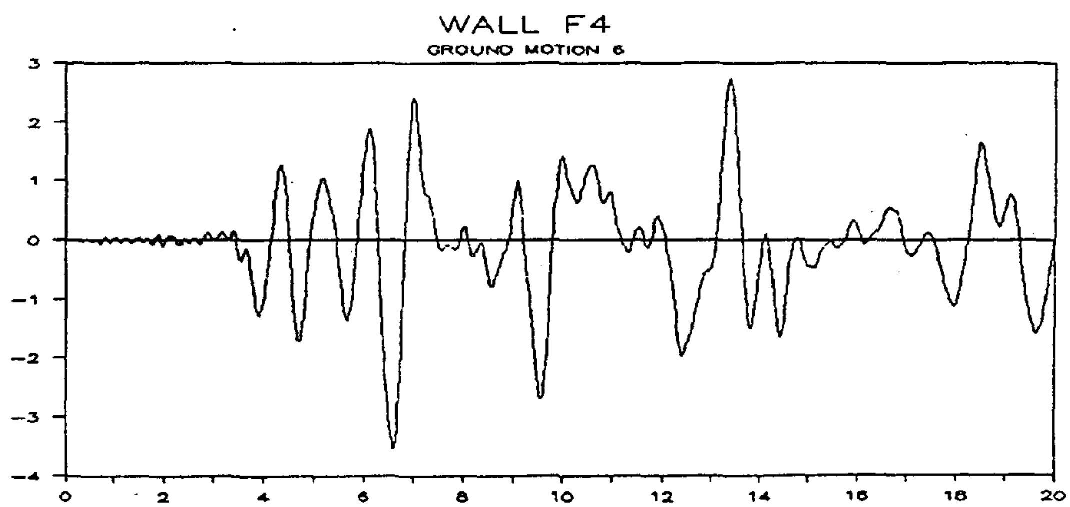

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GROUND MOTION 6
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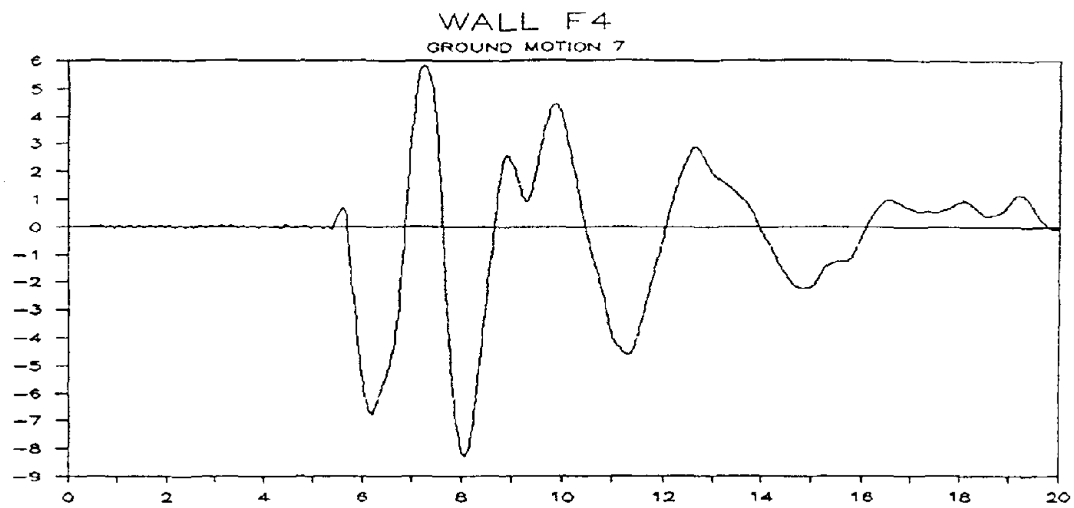
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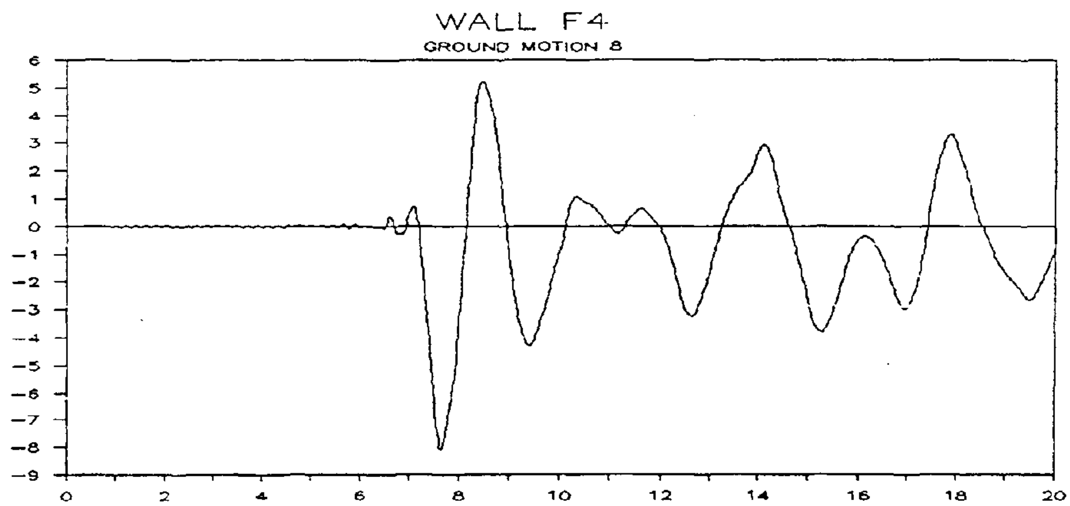
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GROUND MOTION 8
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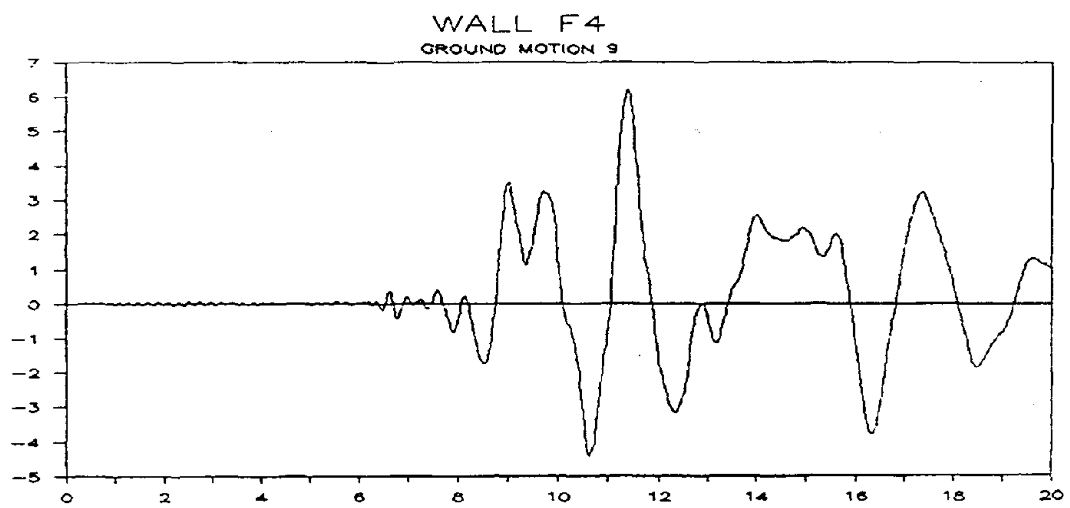
WALL F4
GROUND MOTION 9
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6.0 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations made in this section are those of the Principal Investigator of Category 2.3 of the TCCMAR coordinated research program. These studies were made to provide support for the development of the Draft Limit States Design Standards by TCCMAR.

6.1 Conclusions supported by the study

The studies of the quantities of reinforcement that would be allowed by a high percentage of a balanced design ratio indicate that the percentage must be limited for physical reasons alone. Studies of development of reinforcement in unit masonry show that the size of reinforcement should be limited to a relatively small percentage of the size of cell in the unit masonry. The reasons for this restriction was the physical difficulty in placing grout around reinforcement and the splitting effects of an unconfined grout core caused by development of large size reinforcement bars.

The data for flanged walls is inconclusive as to giving a strong indication as to an upper bound of the reinforcement quantity. Figure 5-2 shows that Wall F1, with 50 of the balanced reinforcement ratio, has a less desirable nonlinear behavior. This behavior did not have a significant effect on the top displacement when subjected to ground motion scaled to the standard spectrum.

The data obtained from the studies of the rectangular shear wall is more conclusive that use of 50 percent of the balanced reinforcement ratio causes a negative slope to the force-deformation envelope in the post peak strength region. This did not occur at 35 percent of the balanced reinforcement ratio. The difference between 35, 25 and 15 percent of the reinforcement ratio is too small to warrant any conclusions as to a desirability of the use of any of these ratios.

6.2 Recommendations for a maximum allowable reinforcement ratio

A maximum reinforcement ratio of 35 percent of the balanced reinforcement ratio is recommended. This recommendation is principally based on the data obtained from the studies of the rectangular shear walls. The data obtained from the studies of the flanged shear walls was inconclusive but the balanced reinforcement ratio for the flanged shear wall was 1.3 percent. This balanced ratio is very substantially less than the ratio of 4.1 percent calculated for the rectangular shear walls.

The limitation of the quantity of vertical reinforcement of shear walls could be accomplished by an arbitrary ratio rather than a percentage of the balanced ratio. Another study (Kariotis, 1990), that used much lower reinforcement ratios, indicated the desirability of use of minimum quantities of vertical reinforcement to obtain large displacements of the wall prior to crushing of the masonry. This study also indicated that limitation of the top displacement of a shear wall is more easily limited by increasing the physical dimensions of the wall rather than increasing the strength of the wall.

An upper bound of the reinforcement ratio of 35 percent of balanced design ratio is recommended as a maximum value. However, it is not a recommended ratio to be used in design. Use of reinforcement quantities significantly below this maximum quantity improves the dynamic behavior of reinforced masonry shear walls.

SECTION 7.0

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