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INVESTIGATION OF CURRENT SEISMIC DESIGN PROVISIONS FOR REINFORCED MASONRY SHEAR WALLS

by

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Ewing/Kariotis/Englekirk & Hart

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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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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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Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the National Science Foundation and/or the United States Government.

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INVESTIGATION OF CURRENT SEISMIC DESIGN PROVISIONS FOR REINFORCED MASONRY SHEAR WALLS

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ABSTRACT: Current seismic design provisions were investigated to determine if optimum designs resulted from their use. Forty-two components of recorded ground motions were used for the investigation. These ground motions were recorded in the Imperial Valley in 1940 and 1979. These ground motions were assumed to contain a consistent value of soils factor. The tabulation of the Imperial Valley ground motion revealed that these motions had mean velocity-acceleration ratios and velocity and acceleration amplification factors that exceeded those determined by prior studies. The investigation confirmed that a modified seismic design formula provided a more consistent dynamic behavior for reinforced masonry shear walls. The investigation also studied the influence of reinforcement ratios of masonry shear walls, the influence of under and excess strength, and the effectiveness of stiffness modification in lieu of strength modification. The investigations supported the conclusions that limitation of the vertical reinforcement ratio increased the displacement ductility of reinforced masonry shear walls, that the current NEHRP Recommended Provisions require a near optimum required strength, and that modification of the stiffness of the shear wall, in lieu of strength modification is highly beneficial in limiting the nonlinear displacements of reinforced masonry shear walls.

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INVESTIGATION OF CURRENT SEISMIC DESIGN PROVISIONS FOR REINFORCED MASONRY SHEAR WALLS

INTRODUCTION

Current NEHRP design provisions [1] specify a required strength of masonry shear walls by general formulae that include the effects of seismic zoning, site soil profile, and fundamental elastic period of the lateral load resisting system. The basic formula uses a ground velocity related seismic zone coefficient. This formula has prescribed upper bound values that are independent of soil profile and fundamental period and uses an acceleration related seismic zone coefficient. All of these formulae are modified by a response modification coefficient that is related to the structural elements, such as reinforced masonry shear walls, and to the building system. The relationship of the response modification coefficient to the building system requires a higher strength for a masonry shear wall in a bearing wall building than a shear wall in a building that has an independent vertical load carrying frame. The design provisions [1] also impose stiffness requirements in the form of drift limitations for the lateral load resisting system.

The majority of these provisions have a historical basis rather than an experimental or analytical basis. Recent studies [2] sponsored by the Building Seismic Safety Council (BSSC) suggest that predicted ground velocity rather than predicted ground acceleration be used in the required strength design formula. US-TCCMAR studies [3] [4] indicate that reinforced masonry shear walls are strongly affected by the frequency content of the ground motion and confirm this recommendation. The critical frequency range that is related to nonlinear displacement corresponds to the peak spectral velocity of the spectrum of the ground motion. This investigation extends previous research of

Task 2.3 of Category 2 research [5].

This report presents the results of an investigation where three formulae for the determination of required strength were evaluated; namely,

- Acceleration Based Design, NEHRP Design Provisions
- Peak Ground Velocity Based Design, Alternate NEHRP Design Provisions
- Spectral Velocity Based Design, Proposed Modification to the NEHRP Peak Ground Velocity Design

The investigation also included an evaluation of the proposed Spectral Velocity Design formula where two variations of the structure were considered; namely,

- Strength Variation
- Stiffness Variation

In the investigation a nonlinear dynamic model of a one story shear wall building was subjected to a series of time-history analyses using 42 earthquake records. The results of these analyses were used as the basis for evaluation of the design formulae and the strength and stiffness variation. The evaluation used the consistency of dynamic response displacements for the comparisons.

The foregoing discussion suggest that present procedures [1] may not be adequate or consistent with respect to the objective of producing reinforced masonry shear wall designs that would have the appropriate strength and stiffness characteristics considering the wide variability of probable ground motions. Basically, it is highly desirable that the damage levels of masonry systems caused by ground motions be similar in all seismic hazard zones of the United States. It is not clear that the present provisions [1] lead to this result.

PURPOSE

The overall purpose of this study was to investigate the viability of current design formulae to produce shear wall designs which respond in a similar fashion to average ground motions that are described by seismic zoning parameters. Ancillary objectives in support of the primary purpose were:

- To illustrate the extreme variability of ground motions caused by a single earthquake.
- To evaluate the consistency of possible earthquake damage to shear walls designed by alternative formulae
- To determine the influence of shear wall strength variations upon nonlinear response
- To determine the influence of stiffness variations upon nonlinear dynamic displacements
- To devise a procedure for estimation of dynamic displacements by pseudo-elastic calculations

PROCEDURE

This study used nonlinear models of reinforced masonry shear walls that were developed in the TCCMAR research [6]. The nonlinear properties of the shear walls were determined by a nonlinear finite element model [7]. The weight of the building that is coupled with the masonry shear wall for the dynamic analyses was determined by current design provisions [1] or by the proposed revised provisions.

The ground motions used in this investigation were recorded in the Imperial Valley of California. Nearly all were recorded in the Imperial County earthquake of October 15, 1979 [8]. Two horizontal components of the 1940 Imperial County earthquake were also included in this study as a standard of comparison. One of these components has been historically used as representing a probable strong ground motion. All of these ground motions were recorded in deep/stiff soils and were considered to include the soil profile factor in the recorded values.

The first part of this study used a nonlinear model of a shear wall, LPM [6], coupled with a building weight that was determined by the current NEHRP Provisions [1] and by alternative seismic design formulas, to make comparisons of the consistency of the probable earthquake damage. These studies used one shear wall to determine the effects of the variability of the recorded ground motions and the alternative design provisions. The ground motions used have a very wide range of energy. The values of Zero Period Acceleration (ZPA), the spectral acceleration at 25 Hz, varied from 5 to 85 percent of gravity. Other measures of energy such as Peak Ground Velocity (PGV) or spectral amplification factors, Q_a and Q_v, had a similar wide variation. It was assumed that a number of these records could represent the probable ground motion in any region of the United States that is zoned for the spectral values.

This study, the comparison of the dynamic behavior of buildings designed by current NEHRP Provisions and the alternative seismic design formulae, was initiated to determine the influence of the design provisions on the probable damage. Seismic provisions are generally developed without parametric studies such as these. In this study, the results of the analyses of the designs were compared and their effectiveness was judged on the basis of predicted damage. If a shear wall, designed for a ground motion parameter, has nearly the same peak dynamic displacement for each of the ground motions, the seismic design formula used would be the optimum formula.

In the second part of the investigation, the strength of the shear wall was systematically changed to determine the influence of variation of the required strength. The strength of the shear wall is a variable that can be used by the design engineer to modify the nonlinear response of the shear wall. This is accomplished by maintaining the size and shape factor of the shear wall and varying the quantity of the vertical reinforcement in the shear wall. The option of changing the yield and expected strength of the shear wall without revising the initial stiffness is available to designers of reinforced masonry shear walls. This option allows the investigation to determine an optimum reinforcement ratio. An optimum reinforcement ratio would produce a shear wall that has a capacity for large post-yield displacements.

The third part of this investigation was to change the stiffness of the shear wall in lieu of changing the strength of the shear wall. This part of the study also investigates the effect of design requirements that impose drift limits on the lateral load resisting system as well as a specification of a required strength. It is recognized that nonlinear dynamic displacements, as well as elastic displacements, are reduced by increasing the stiffness of the member. However, it is also commonly believed that the increase in the dynamic base shear of the stiffened element would negate the effect of increased stiffness. The shape factor and reinforcement ratio of masonry shear walls was independently varied to cause major changes in the stiffness of the shear walls while maintaining about the same design strength. A sub-task of this investigation was to devise a procedure for estimation of dynamic displacements by pseudo-elastic calculations.

GROUND MOTIONS

Prior research sponsored by the BSSC [2] has recognized that two factors should be included in the specification of standard spectra that are utilized as seismic loading in design provisions. These are the peak velocity/acceleration ratio, V/A, and the spectral amplification factors of the ground velocity and acceleration, Q_V and Q_A. These amplification factors are also used as a ratio, Q_V/Q_A. These factors are then combined to develop an index:

$$SR = (Q_V/Q_A) \bullet (V/A) \tag{1}$$

The group of ground motions selected for this study were the 40 records of the 1979 Imperial County earthquake and the 2 records of the 1940 Imperial County earthquake. Table 1 gives the peak acceleration and velocity values as recorded. Zero Period Acceleration is noted in this table for each component. This is taken from the five percent damped spectrum of the record as the spectral acceleration value at 25 hz. frequency. The velocity/acceleration (V/A) values are calculated from the recorded peak ground acceleration (PGA) and peak ground velocity (PGV) values. The spectral amplification values of acceleration and velocity were calculated by comparison of the peak spectral values of a five percent damped spectrum and the peak recorded values. The SR values were calculated as the product of the V/A ratio and the Q_V/Q_A ratios [2]. Mean values of V/A, Q_V, Q_A and SR, and the standard deviation of the data was determined and noted at the bottom of the columns of Table 1.

The mean values of V/A, Q_V, Q_A and SR taken from this data substantially differ from those reported in prior studies [2]. The purpose of this study was to specifically utilize this data source, the 1979 Imperial County earthquake as prior studies of nonlinear behavior [3] [4] has indicated that these ground motions would cause a wide range of nonlinear displacements. It is not necessarily concluded that this data is truly representative of probable ground shaking, such conclusions should be made by seismologist. This study uses the recorded data to indicate its impact on lateral load resisting elements that are designed by current and proposed seismic design requirements.

The spectral values given in Table 1 are assumed to have the amplification appropriate for a soil profile of S_2 (deep/stiff) type soils contained in the record. The prescribed value of S = 1.2 for this soil profile type was used in Equation [3] for determination of required strength.

DESCRIPTION OF THE ANALYSIS MODEL

The reinforced masonry shear wall used in this investigation was analyzed by use of a nonlinear finite element model (FEM) that was developed in the TCCMAR research [7]. The shear walls are representative of the lateral load resisting elements in a building that is described as follows.

- Single story
- Symmetrical
- Single shear wall on each face of the building
- Concrete filled steel deck roof diaphragm
- The gravity loads of the building are supported by a complete vertical load carrying frame
- The base of the shear wall is fixed by piling capable of resisting compressive and tensile loading.

The roof diaphragm is assumed to have an in-plane stiffness such that the tributary building weight is coupled with the shear wall without amplification. The reinforced masonry shear walls are twelve inches nominal thickness and of the dimensions noted in their description. The shear wall used for the basic investigations of required design strength formulae was 28 feet high and had a base dimension of 19.33 feet. The investigations of the effects of strength variation used shear walls of these dimensions and with variations in the vertical reinforcement.

The investigations of the effects of stiffness variation used reinforced masonry shear walls of the dimensions and reinforcement listed in Table 10. Horizontal reinforcement, #4 bars at 16 inches spacing, was used for the 28 foot high walls with variable strength and for the walls with a variable stiffness.

Each shear wall was analyzed by the FEM using the following assumptions:

- Compressive modulus of elasticity of masonry, 3×10^6 psi
- Peak compressive masonry strain, 0.0025 in./inch
- Compressive strength of masonry, 3,000 psi
- Unit weight, 120 lbs. per cu. ft.
- Tension stiffening model used
- No strain hardening in reinforcement
- Yield strength of horizontal and vertical reinforcement, 65 ksi
- Modulus of elasticity of reinforcement, 30,000 ksi.

Figure 1 indicates the results of a FEM analysis of a 28 foot high by 19.33 feet base dimension shear wall. This wall was reinforced with #5 bars vertical at 16 inch spacing and #4 bars horizonal at 16 inch spacing. The results of FEM analyses of the identical size walls (Wall A, B, C, D, and E) with reinforcement variation is given in Table 2.

Wall B, reinforced as shown in Table 2, was used for studies of the effectiveness of current seismic design formulae and to investigate modifications to these formulae. The FEM analyses were converted to design strengths by factoring the expected yield stress of the reinforcement, 65 ksi, to the minimum guaranteed yield stress of grade 60 reinforcement, 60 ksi. This yield strength was then modified from the nominal yield strength to a design strength by use of the prescribed capacity reduction factor of 0.8 [1]. The strength values for the masonry shear walls, A through E, are shown in Table 3.

The shear walls, analyzed by the nonlinear FEM [7], are represented for the dynamic studies by a nonlinear lumped parameter model (LPM) [6]. Figure 2 illustrates the materials behavior that was used for the dynamic studies. The hysteresis model used in the LPM has a curvilinear initial loading, a linear unloading stiffness that reduces as a function of the prior peak displacement, a reloading through a prescribed pinch to intersect a new reloading stiffness. The reloading stiffness aims at a stabilized load/displacement envelope that lies a prescribed distance below the virgin or monotonic load/displacement envelope.

CORRELATION OF THE ANALYTICAL MODEL WITH SEISMIC DESIGN PROVISIONS

The analytical model of the masonry shear wall is coupled with a building weight for the dynamic time-history calculations. Since the shear wall dimensions, strength and stiffness have already been determined, the coupled weight is inferred (i.e., back calculated by the required strength formula). The weight, W, at the top of the shear wall is related to seismic zone parameters such as zero period acceleration, peak ground velocity, or spectral velocity. Current seismic design formulae [1] prescribe the required strength of a shear wall by the following formula:

$$V = C_s W \tag{2}$$

where:

C_s = seismic design coefficient defined in Equation 3 or 4

W = coupled building weight

and:

$$C_s = 1.2\ A_v S/RT^{2/3} \tag{3}$$

and:

$$C_s \text{ maximum} = 2.5\ A_a/R \tag{4}$$

Figure 3 indicates how Equation 4 functions as a cutoff value on Equation 3 values. When these formulae are used as a design procedure and the design strength is known, the total coupled load, W, is also known. In this investigation which uses uniform height shear walls as the model, the allowable design shear, V, is known and the theoretical tributary building weight, W, can be calculated for any values of A_a, A_v, and T. The effectiveness of Equation 4 for the specification of the required strength of a reinforced masonry shear wall was the subject of the first phase of the investigations (acceleration based design).

In first phase of the investigation of design formulae, a tributary building weight was calculated using by Equation 4. The ZPA of the record was taken as A_a, R is specified as 4.5 by reference [1], and V is the design shear value given in Table 3 for Wall B. Maximum values of calculated dynamic displacements of the top of the 28' x 19.33' shear wall versus ZPA are plotted in Figure 4 for each component of the ground motions. The period, T, given in Table 4, was calculated as an effective period that approximates the effective dynamic period of a masonry shear wall. The secant stiffness used for effective period determination was calculated at a top displacement of about twice of first yield displacement. The force-displacement values of Wall B for this secant stiffness are 126 kips and 0.7 inches. The secant stiffness is 180 kips per inch or about 10% of the initial stiffness. Comparison of this calculated stiffness with the FEM analysis of this wall, Figure 1, indicates that this assumes the displacements that are caused by the earthquake are in the order of 1/480th of the height of the shear wall. Experimental testing of masonry shear walls indicate that cracking is barely discernible at this relative displacement. Peak strengths are generally attained at a top displacement of 1/100th of the height of the shear wall.

The second phase of the investigation of current seismic design formula used an alternative formula recommended by the 1988 Edition of the NEHRP Recommended Provisions (peak ground velocity based design) [5]. The alternative formula uses effective peak ground velocity in lieu of a velocity related acceleration coefficient. This change was proposed to utilize the tentative seismic mapping of predicted velocity in rock that is included in the 1988 Edition of the NEHRP Recommended Provisions [1]. The procedure is fully described in reference [2], only the proposed formula for C_s is given here.

$$C_s = 0.013vS/RT^{2/3} \tag{5}$$

Where:

v = peak ground velocity in cm./sec.

For the calculation of coupled building weight, W in Formula (5), the value of S was taken as 1.2, v was the peak ground velocity taken from the ground motion record as given in Table 1, R was 4.5, and T was calculated for each coupled building weight by the following iterative procedure.

$$T = 2\pi(W/k_s g)^{1/2} \tag{6}$$

Where:

k_s = secant stiffness in kips/inch

And:

T is assumed to be 1.0 second
Calculate W for this assumption
Using W, recalculate T
Revise value of W

Figure 3 indicates that the acceleration based design coefficient, Equation (4), provides a cutoff for Equation (3). For this phase of the investigation, a partial cutoff similar to Equation (4) was provided by limiting the least value of the period, T, to 0.4 seconds. It was only a partial cutoff as the values of the peak ground velocity were not limited, only the values of T. Only 8 of the 42 calculations of the coupled weight given in Table 5 were modified by this cutoff value of minimum period.

Table 5 presents the calculated weights, effective periods, and nonlinear displacements resulting from use of Equations (2) and (5)for Wall B. The maximum dynamic displacement of the top of the shear wall that was caused by the ground motion is plotted versus the recorded peak ground velocity in Figure 5. One inch top displacement corresponds to a drift ratio of 1/336. The displacement used for the definition of the secant stiffness used for calculation of period, T, was 0.7 inches.

The third phase of the investigation of the effectiveness of seismic design formulae modified Equation 5 to the following:

$$C_s = 0.013\ (S_v/2.3)/RT \tag{7}$$

Where:

S_v = peak spectral velocity of the record

The spectral velocity used in this equation is normalized by dividing by the mean spectral velocity amplification of all of the records given in Table 1. T in lieu of $T^{2/3}$ was used in the formula as it appropriately represents the variation in spectral values and removes the nonconservatism in C_s values for periods of less than 1 second. The spectral velocity values that were taken from the records were assumed to contain a soils factor, thus S is considered to be included in the recorded spectral values. R was again 4-1/2, and the values of T were calculated by the iterative procedure previously described.

Two cutoff values of C_s were used in this investigation. For ground motions with a spectral velocity value of less than 120 cm/sec, the minimum value of C_s was taken as 0.22. This corresponds to an assumption of $A_a = 0.4$ in Equation (4) and assumes these ground motions are representative of those probable within the highest acceleration zoning contour of the NEHRP mapping. The cutoff value of 0.28 of the coupled weight was used for spectral velocities greater than 120 cm/sec and corresponds to an assumption that a higher intensity zone would be mapped in the near-fault regions that have the highest predicted ground velocities. And that this zone would have an A_a of 0.5. Table 6 indicates the values of spectral velocities, coupled weights, effective periods, and the calculated nonlinear displacements related to use of Equation (7). Figure 6 is a plot of the maximum spectral velocity versus the top displacement of the shear wall. A drift ratio of 1/100 is equivalent to a top displacement of 3.36. Reinforced masonry shear walls generally attain this drift ratio without strength degradation.

RESULTS OF THE INVESTIGATION OF DESIGN FORMULAE

Design based on acceleration (ZPA) - Figure 4 indicates that the nonlinear displacements of reinforced masonry shear walls designed by Equation (4) tend to increase with increasing ZPA. In addition to this trend, two ground motions with large V/A values were not constrained by the design formula. Outliers of the general trend of the data are to be expected with any formula. Ground motions are expected to be influenced by random phenomena and are described by a probabilistic statement. Exceedance of the intensity values recorded in the Imperial Valley is expected. The goal of a design formula is to cause a relatively common nonlinear displacement of the shear wall for all of the ground motions. Attainment of this goal would indicate that the masonry shear wall would have near identical nonlinear displacement in all seismic zones.

Designs that were based on acceleration of about 0.2 g had average drift ratios of about 1/400. Experimental testing of shear walls indicates this is an unnecessary conservatism.

Design based on peak ground velocity - Figure 5 indicates that this design formula causes an increasing conservatism with increasing peak velocity. The extreme outliers of the data are the Bonds Corners records. The spectral amplifications, Q_v, of these two records are 3.3 and 4.0. The third outlier, Dogwood Road, 360° component, has a Q_v of 3.0. This data could be constrained to a more common displacement by use of a limit on the required strength such as that used in the spectral velocity design procedure. However, this would not relieve the conservatism indicated for peak ground velocities of 20 cm per second or less.

Design based on spectral velocity - Figure 6 has a more desirable vertical trend than that shown on Figures 4 and 5. The maximum nonlinear displacement, with the exception of two outliers, is less than a drift ratio of 1/100. The drift ratio of the two outliers, the Bonds Corners components, is less than 1/50. Masonry shear walls displaced to this drift ratio are severely damaged but are capable of sustaining self weight and applied vertical loading.

Conclusion of the investigation of seismic design formulae - Design formulae that utilize peak spectral velocity as the seismic zone parameter show the most adaptability. The constant of 0.013, the value of spectral amplification that is related to the soil profile and limitations on required strength need further investigation. These studies used a reinforced masonry shear wall that had a secant period that varied from 0.41 to 1.6 seconds. Comparative studies of elements with longer periods, 1.0 to 3.0 seconds effective period, should be made. The best values for the design parameters must consider the dynamic behavior of all classes of structures.

R values of 4.5 were used for these shear wall studies. These studies indicate that the traditional relation of R to a structural system is not necessarily appropriate. If drift angles greater than 1/100 are contemplated by design procedures, strength degradation becomes a very critical factor. Studies of masonry shear walls displaced in the strength degradation range [10] indicate that the probable nonlinear displacement is highly sensitive to amplification by the horizontal diaphragm, number of cycles and the sequence of ground motions. If the ground motion pulses have an increase in intensity with time, versus a decrease, very large nonlinear displacements will result. Studies of stiffness and strength degrading structural materials, such as the composite of masonry and reinforcement, suggest that dynamic displacements in the strength degradation region are not acceptable for probabilistic based seismic design procedures. If the design procedure includes a conservatism to limit possible nonlinear displacement, a variation in the value of R that is related to the gravity load carried by the shear wall seems inappropriate.

STUDIES OF THE EFFECTS OF REINFORCEMENT RATIO AND STRENGTH VARIATION

These studies used the five masonry wall shear walls that are described in Table 3. The walls are the same size, 28' high by 19.33' base, and have reinforcement ratios that vary from 0.0011 to 0.0056. The shear wall, B, used in the investigation of optimum seismic design formulae had a reinforcement ratio of 0.0017. All walls, except E, have a flexural mode of strength degradation. The calculated shear capacity of these walls was about 236 kips. Examination of the nonlinear analysis of Wall E indicated that yielding of the horizontal reinforcement occurred prior to yielding of the vertical reinforcement.

The two components of six representative ground motions were selected for these comparative studies. The ground motions were the outliers, Bonds Corners, and three that caused dynamic drift angles of about 1/200. The coupled building weight and effective period were calculated as previously discussed and are tabulated in Table 7. The maximum displacement of each wall, designed by the recommended formula (6), is summarized in Table 8.

The trend of data clearly shows that a low percentage of vertical reinforcement is desirable, even for the very damaging ground motions. The displacement when compressive strain at the base of the shear wall is 0.0025 inches per inch, e_p, is also indicated in Table 8. The mean of the nonlinear displacements of Walls D and E exceeds e_p. The maximum reinforcement ratio of walls with adequate behavior is 0.00245. The minimum reinforcement ratio used in these studies was 0.0011. Current provisions require that a minimum percentage of 0.0007 be used as vertical reinforcement.

A second part of the strength variation study investigated the effect of varying the strength of the shear wall without modifying the coupled building weight. The wall that conforms to the design provisions is Wall B and is reinforced with #5@16 vertical. The weight that was calculated for Wall B was used for the analyses of walls A, C, D, and E. Table 9 summarizes the result of that study. The data indicates that excess strength, in the order of 1.5 times design strength (Wall D), provides a capacity that can cope with the ground motions that are outliers of the average of those recorded. Added strength beyond an increase of 1.5 does not provide equivalent benefits. Figure 7 plots the data to indicate the most effective zone of strength modification. The hazard of under-design in seismic zones that have a probability of ground motions with high V/A ratios, James Road 230, is also shown. The anomaly of Wall A and the Bonds Corners 140 ground motion is due to the effective period of the shear wall, it falls outside of the frequency content of that ground motion. The opposite is true for Wall A and James Road 230.

STUDIES OF THE EFFECTS OF STIFFNESS VARIATION

Several researchers in the field of structural response to earthquakes have suggested that nonlinear response can be more effectively controlled by variation of the stiffness rather than variation in the elastic strength of the structure. This phase of the investigation uses masonry shear walls of near-equal yield strength and with a wide range of secant stiffness to determine the effect of seismic design requirements that would specify a required stiffness as the primary seismic design criteria.

It is recognized that both adequate stiffness and strength are structural requirements for lateral load resisting elements. Current Provisions [1] require that a required strength be calculated. The structure, sized to conform to the required strength, is then analyzed for elastic deformations. The specified loading used for this deformation analysis is the same as that used for the strength design. If the calculated elastic deformation exceeds the specified limits, the stiffness of the structure must be increased to meet the specified drift limits. If the structural system was structural steel, increases in member stiffness will also increase the strength, but reinforced masonry shear walls can be stiffened without commensurate increases in strength. The previous phase of this investigation indicated that increasing the strength of the shear wall by increasing the reinforcement ratio, but not providing excess strength, did not necessarily provide equivalent relative displacement control. To clearly illustrate this statement, assume the following. The building weight is fixed by the owners requirements and the design task is to select adequate shear walls. One Wall D, with a reinforcement ratio of 0.0033 will meet the base moment strength requirement of 3,561 ft. kips, (Table 3). Or two Walls A with an individual strength of 1,685 ft. kip will also meet the strength requirements. The ratio of the relative displacements of these two choices will be in the order of 2.17/0.81 or about 2.7. If a maximum drift ratio of 1/200 was specified, the use of the two Walls A would be required.

This phase of the investigation used four walls (Walls B, F, G and H) with near equal strength and with a wide variation in secant stiffness to examine the influence of stiffness variations on dynamic displacement control. The shape factor, h/d, of the walls selected for the investigation had a range of 1.88, 1.45, 1.0, and 0.67. Wall B, with a shape factor of 1.45, was selected as the standard for comparative strength. A computer program RCCOLA [11], was selected to determine vertical reinforcement patterns for Walls F, G, and H that would have a strength near equal to that of Wall B. RCCOLA was selected in lieu of using the FEM and an iterative procedure. This program assumes that plane sections remain plane during flexural rotation but was used as it can be used efficiently by engineering offices on personal computers. The results of the RCCOLA analyses and the vertical reinforcement quantities obtained from these analyses is shown in Table 11 for the shear walls of dimensions shown in Table 10. The moment values determined by the RCCOLA and FEM analysis assume a yield stress of the vertical reinforcement of 65 ksi, no strain hardening, masonry strain of 0.0025 inches per inch at peak compressive stress, peak compressive stress of masonry of 3,000 psi, and an elastic modulus of the reinforcement of 30,000 ksi. The moment values given in Table 10 are when the compressive strain in the masonry reaches a strain of 0.0025 inches per inch. Shear warping of wall cross sections is the principal reason for the differences of moments predicted by RCCOLA and the FEM.

Table 10 presents the results of the FEM analyses of the shear walls, F, B, G and H. The initial stiffness of the walls have a variation of 929 to 8,220 kips per inch, a ratio of 9.5. The top displacement at a compressive strain of 0.0025 inches per inch varies from 2.93 to 1.75 inches, a ratio of 1.67. The stiffer walls are more brittle but the critical top displacement is not linear with the variation in initial stiffness.

Table 12 presents the nominal and design yield strengths of the walls with stiffness variation. This table indicates that the capability of RCCOLA to select a vertical reinforcement percentage that corresponds to an equal expected moment strength was not verified. The FEM analysis indicated that shear warping of wall cross sections becomes important for most shear walls. Expected strength, M_e, is defined as the value of the predicted strength that is calculated by use of expected properties of masonry and reinforcement. The ratio of the design moment, to the expected moment as predicted by the FEM analysis, is given in Table 12 and is reasonably consistent.

The dynamic displacements of the walls were calculated by use of the LPM [6]. The building weight that was coupled with the shear wall element was calculated by Equation 6 and the cutoff values that limit the upper bound of C_s. The secant stiffness of the shear wall that was used for the calculation of period was taken as 10 percent of the initial stiffness. Table 13 gives the results of the dynamic analyses and maximum displacements are summarized in Table 14. The mean of the dynamic displacements, excluding Bonds Corners data, is plotted versus initial stiffness in Figure 8(a). The mean displacements of the walls with constant and variable stiffness is shown in Figure 8(b). The plot of average displacement versus design strength is shown for each stiffness alternative. The "constant stiffness" is that data given in Table 8 for a 28' x 19.33' shear wall with strength variations. The "variable stiffness" is that data given in Table 14 for shear walls that have a limited variation in strength but have a range of initial stiffness from 929 to 8,220 kips per inch.

Analyses were made of shear walls with varying stiffness and strength that are coupled with the building weight calculated for Wall B. This study was to delete the effect of the increase in strength (and coupled weight) that was associated with the reinforcement patterns that were chosen for each shape of shear wall. The results of that study is presented in Table 15. This study was made to indicate the effect of combined stiffness and strength increases. As previously discussed, selection of member sizes for increased stiffness commonly provides an increase in member strength.

CONCLUSIONS OF THE STUDY

Development of a proposed seismic design formula - This investigation evaluated a formula that was presented for comment in the 1988 Edition of the NEHRP Recommended Provisions [1]. The formula substituted predicted ground velocity for velocity related acceleration, A_v. That proposed Equation (5) was modified in this investigation to Equation (7) that uses a pseudo-ground velocity that is normalized from spectral velocity. The method for the calculation of an effective T was significantly revised from current methods of fundamental period calculation and it was recommended that this effective T be used in Equation (7). The use of T to the 2/3 power was not recommended. Maximum values of C_s were proposed for limitation of required base shear. These maximum values of base shear correspond to the NEHRP Recommended Provisions for A_a values of 0.4 and 0.5 g. The selection of the change in the limitation of C_s was based on the value of spectral velocity being above or below 120 cm/sec. All spectral values were taken from a 5% damped spectrum. These investigations used ground motions that were recorded in the Imperial Valley earthquakes of 1940 and 1979. It was assumed that a soil profile factor for deep/stiff soils is contained in the recorded values. The proposed Equation (7) was used in methods that are equivalent to current seismic design procedures to establish a required strength that corresponds to the spectral velocity values of each component of

21 ground motions. The nonlinear shear wall model, using this required strength, gave a better distribution of calculated displacements than the current or proposed ground velocity formulae. The comparative data is shown in Figures 4, 5 and 6. An optimum distribution would correspond to nearly equal nonlinear displacements when plotted versus the chosen seismic parameter.

The recommended formula that uses spectral velocity can be used with the seismic zone mapping of the NEHRP Recommended Provisions [1] that give contours of horizontal velocity in rock. The velocity values in rock can be multiplied by spectral amplification factors, Q_v, that are appropriate for the site soil profile. Suggested ratios of spectral velocity amplification factors for soil profiles S_1, S_2, and S_3 are 1.7, 2.3, and 3.7 respectively [12].

This investigation of the dynamic behavior of shear walls should be supplemented by studies using ground motions that were recorded on both rock/stiff and deep/soft soil profiles. This study used the value (0.013) of the constant suggested by reference [2] in the base shear equation. Similar studies should be made using moment frames and braced frames of materials that retain initial stiffness and those that have strength degradation. These studies would determine a value of the constant that was used in Equation (7) that would be applicable to all structural materials.

Further studies of reinforced masonry shear walls are required to determine the best values of the limitation of C_s and the response modification coefficient, R.

Influence of strength variation - These studies clearly indicate that the ratio of the vertical reinforcement to the gross area of the shear wall should be limited. Table 8 indicates that for shear walls of the shape factor used, a reinforcement ratio greater than 0.0025 is undesirable and smaller ratios are more appropriate. Figure 8 indicates that increasing the stiffness of the shear wall is a much more effective solution for limiting nonlinear displacements than increasing the quantity of reinforcement. Use of over-strength shear walls to reduce relative nonlinear displacements was shown to be effective, Figure 7. However, provision of overstrength in excess of about 1.5 times the required strength is relatively ineffective.

Influence of stiffness variation - These studies clearly indicate that increasing the stiffness of shear walls was an effective method of minimizing probable nonlinear displacements. Provision of a drift limit to insure adequate stiffness of a shear wall requires the specification of methods for calculation of deflections. Generally the loading used for these calculations is that specified for seismic design loads. The drift limit may be set very low to approximate the elastic deformation, or the stiffness of the shear wall may be described as a secant stiffness and the calculated displacements will be equivalent to the nonlinear displacements. For this case, the specified drift limit should approximate the probable nonlinear displacement.

Current recommended provisions use elastic stiffness properties, loading that is used for strength design, and a multiplier of the calculated pseudo-elastic deflection. The NEHRP Recommended Provisions use a displacement amplification factor, C_d, to increase the pseudo-elastic displacement to an approximation of the probable nonlinear displacement.

Investigation of methods for calculation of dynamic displacements - Four ground motions were used in an attempt to discover if a relationship between a calculated pseudo-elastic and the dynamic relative displacement exists. Table 16 presents the data that was used for the calculation of relative displacements. Only three walls and four ground motions were used for these comparisons. Walls F and B have a reasonable scatter of the data and the ratio of nonlinear displacement to elastic displacements is nearly equal to R. If the C_d of 4, as given in the NEHRP Recommended Provisions, was used as a multiplier of the pseudo-elastic displacement, the calculated pseudo-elastic drift of Wall F would be 2.0 inches or about a 1/180 drift ratio. Wall F has e_p of 2.93 inches and a mean calculated nonlinear displacement of 2.18 inches. The calculated drift of Wall B would be 1.2 inches which has a reasonable relationship with the mean of the calculated nonlinear displacements of 1.38 inches. The comparisons of data in Table 15 of mean dynamic displacement and calculated pseudo-elastic displacement is plotted in Figure 10. This data indicates that a consistent relationship does not exist when the initial stiffness of the wall is in excess of the required stiffness.

These studies indicate that drift limits for masonry shear walls will not have a significant influence on the seismic design requirement unless set at low limits such as 1/200 of the shear wall height. A drift limit of 1/200 would require that the seismic capacity of Wall F be modified. If the reinforcement ratio was also limited to 0.0025, the base width of Wall F would have to be increased to meet the combined stiffness and strength design criteria. Additional studies should be made to determine the validity of drift limits and if valid, the values of acceptable drift and acceptable reinforcement ratios need be recommended.

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Any opinions, findings, conclusions, or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the National Science Foundation and/or the United States Government.

APPENDIX I - BIBLIOGRAPHY

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APPENDIX 2 - NOTATION

A = peak ground acceleration

A_a = coefficient representing Effective Peak Acceleration

A_v = coefficient representing Effective Peak Velocity-Related Acceleration

C_s = seismic design coefficient

e_p = displacement of top of shear wall when masonry strain equals 0.0025 inches per inch

F_p = peak load applied at top of shear wall

K_i = estimated initial stiffness of the shear wall prior to tension cracking

K_s = secant stiffness of shear wall in the post-yield region

M_e = expected moment strength

Q_A = amplification factor of peak ground acceleration for a median 5% damped spectra

Q_v = amplification factor of peak ground velocity for a median 5% damped spectra

R = response modification coefficient

S = coefficient for site soil profile type

SR = S_V/S_A, a ratio of pseudo velocity to pseudo acceleration

S_v = maximum spectral velocity of a 5% damped spectrum

T = secant period of the shear wall with stiffness degradation

v = mapped ground velocity in cm/sec. for use in Eq. (5)

V = peak ground velocity

W = Total dead load

TABLE 1 - Records of the Imperial Valley earthquakes of 1979 and 1940

Station	Comp.	PGA cm/s^2	PGV cm/s	PGD cm	ZPA g	Epicen. Dist. km	Dist. Fault km	V/A cm/s/g	S_v cm/s	Vel. Ampl.	S_a g	Accel. Ampl.	Q_v/Q_a	SR= (Eq. 1)
Borchard Ranch	140	139.35	14.58	5.48	0.17	37	22	102	32.8	2.30	0.58	4.10	0.56	57.12
Borchard Ranch	230	136.23	11.23	4.90	0.15	37	22	81	23.4	2.10	0.43	3.10	0.68	55.08
Keystone Road	140	309.30	31.21	9.93	0.33	31	16	99	89.4	2.90	1.54	4.90	0.59	58.41
Keystone Road	230	405.49	26.51	12.99	0.44	31	16	64	54.6	2.10	1.63	3.90	0.54	34.56
Pine Union	140	261.74	46.32	15.35	0.27	28	13	173	98.3	2.10	1.00	3.70	0.57	98.61
Pine Union	230	218.13	36.80	17.91	0.22	28	13	165	57.9	1.60	0.84	3.80	0.42	69.30
Anderson Rd.	140	483.64	37.05	11.91	0.49	26	7	75	115.0	1.31	1.04	2.10	1.48	111.00
Anderson Rd.	230	349.65	77.65	48.02	0.39	26	7	218	141.0	1.81	0.84	2.40	0.75	163.50
James Rd.	140	517.19	43.99	21.84	0.59	28	4	83	121.9	2.80	1.21	2.30	1.22	101.26
James Rd.	230	367.21	86.56	51.85	0.39	28	4	231	199.1	2.30	1.33	3.60	0.64	147.84
Huston Rd.	140	368.67	63.13	26.94	0.55	27	1	168	158.7	2.50	1.21	3.20	0.78	131.04
Huston Rd.	230	428.09	108.71	55.16	0.46	27	1	249	243.1	2.20	1.10	2.50	0.88	219.12
Imp. Valley College	140	326.78	44.96	19.52	0.34	26	1	135	103.1	2.30	0.80	2.40	0.96	129.60
Imp. Valley College	230	453.65	107.83	41.36	0.47	26	1	233	201.7	1.87	1.37	3.00	0.62	144.46
Cruickshank Rd.	140	598.25	53.43	22.25	0.61	27	4	87	124.5	2.30	1.23	2.00	1.15	100.05
Cruickshank Rd.	230	457.37	47.71	29.34	0.58	27	4	102	98.0	2.10	1.43	3.10	0.68	69.36
Community Hospital	140	221.69	42.18	16.69	0.23	27	9	186	100.3	2.40	0.68	3.00	0.80	148.80
Community Hospital	230	168.21	44.28	27.13	0.18	27	9	258	86.4	2.00	0.59	3.40	0.59	152.22
McCabe School	140	355.41	35.01	14.15	0.38	27	13	96	92.2	2.60	2.10	5.80	0.45	43.20
McCabe School	230	374.54	39.21	13.38	0.39	27	13	102	76.4	2.00	1.52	4.00	0.50	51.00
Brockman Rd.	140	138.68	17.52	9.63	0.15	30	18	124	45.0	2.60	0.40	2.80	0.93	115.32
Brockman Rd.	230	113.36	19.38	8.45	0.12	30	18	167	35.3	1.80	0.38	3.30	0.55	91.85
Strobel Residence	140	114.63	14.63	5.86	0.12	34	22	125	33.0	2.30	0.52	4.40	0.52	65.00
Strobel Residence	230	136.22	14.21	6.04	0.14	34	22	102	26.0	1.80	0.49	3.50	0.51	52.02
Bonds Corners	140	575.73	43.63	12.19	0.64	6	3	74	142.0	3.30	1.80	3.10	1.06	78.44
Bonds Corners	230	770.42	44.07	14.64	0.85	6	3	56	174.5	4.00	2.50	2.90	1.38	77.28
Parachute Test	225	106.86	17.15	9.17	0.12	47	15	157	32.5	1.90	0.39	3.60	0.53	83.21
Parachute Test	315	200.20	14.62	7.87	0.20	47	15	72	34.3	2.30	0.49	2.40	0.96	69.12
Brawley Airport	225	162.17	35.29	18.66	0.19	42	7	213	68.6	1.90	0.61	3.70	0.51	108.63

TABLE 1 - Records of the Imperial Valley earthquakes of 1979 and 1940 (Con't)

Station	Comp.	PGA cm/s^2	PGV cm/s	PGD cm	ZPA g	Epicen. Dist. km	Dist. Fault km	V/A cm/s/g	S_v cm/s	Vel. Ampl.	S_a g	Accel. Ampl.	Q_v/Q_a	SR= (Eq.1)
Brawley Airport	315	216.52	37.12	10.64	0.23	42	7	168	50.50	1.40	0.70	3.20	0.44	73.92
Calexico Fire	225	269.61	19.43	5.71	0.28	15	11	71	58.40	3.00	0.86	3.10	0.97	68.87
Fire	315	196.86	16.08	7.05	0.21	15	11	80	42.70	2.70	0.76	3.80	0.71	56.80
Calepatria Fire	225	125.65	14.99	7.05	0.13	57	21	117	28.20	1.90	0.39	3.00	0.63	73.71
Calepatria Fire	315	77.00	12.41	5.35	0.08	57	21	158	23.10	1.90	0.29	3.70	0.51	80.58
Coachella Canal #4	225	113.58	12.89	2.46	0.12	84	47	111	27.90	2.20	0.34	2.90	0.76	84.36
Coachella Canal #4	315	125.74	15.95	3.08	0.13	84	47	124	46.20	2.90	0.54	4.20	0.69	85.56
Plaster City	225	41.93	3.22	1.52	0.05	52	31	75	8.60	2.70	0.18	4.20	0.64	48.00
Plaster City	315	55.49	5.77	1.81	0.06	52	31	102	11.90	2.10	0.23	4.10	0.51	52.02
Dogwood Rd.	270	344.90	67.77	33.75	0.37	26	5	192	119.60	1.80	1.50	4.30	0.42	80.64
Dogwood Rd.	360	477.14	42.51	13.69	0.50	26	5	87	126.00	3.00	1.35	2.80	1.07	93.09
El Centro,1940	S00E	341.70	33.40	10.90	0.36	7		93	80.16	2.40	0.91	2.60	0.92	85.56
El Centro,1940	S90E	210.10	36.90	19.80	0.27	7		137	67.05	1.82	0.64	2.98	0.61	83.57
Values for all Records				Mean				131		2.31		3.35	0.73	90.31
					Std. Deviation			(55.5)		(0.51)		(0.78)	(0.26)	(38.27)
Values for 20 records within 10 km of fault				Mean				152		2.37		2.93	0.85	114.97
					Std. Deviation			(67.2)		(0.62)		(0.57)	(0.30)	(38.83)

TABLE 2 - CALCULATED PROPERTIES OF 28' X 19.33' SHEAR WALLS
WITH REINFORCEMENT VARIATION

Wall	Reinforcement Vertical	Reinforcement Horizontal	Initial Stiffness, K_i Kips/inch	F_p kips	e_p inches
A	#4@16"	#4@16"	1722	102.5	3.84
B	#5@16"	#4@16"	1738	140.1	2.93
C	#6@16"	#4@16"	1754	177.8	2.13
D	#7@16"	#4@16"	1744	221.0	1.87
E	#9@16"	#4@16"	1819	320.0	1.53

TABLE 3 - NOMINAL AND DESIGN STRENGTH OF 28'x 19.33' SHEAR WALLS

Wall	Reinforcement Vertical	Reinforcement Horizontal	Expected Yield Shear kips	Nominal Shear kips	Design Shear kips	Design Moment ft. kips
A	#4@16"	#4@16"	81.6	75.3	60.2	1,685.6
B	#5@16"	#4@16"	111.1	102.5	82.1	2,298.8
C	#6@16"	#4@16"	138.3	127.6	102.1	2,858.8
D	#7@16"	#4@16"	172.2	159.0	127.2	3,561.6
E	#9@16"	#4@16"	251.9	232.6	186.0	5,208.0

TABLE 4 - CALCULATED BUILDING WEIGHT, PERIOD AND DISPLACEMENTS FOR ACCELERATION BASED DESIGN USING WALL B

Station	Comp	PGV cm/s	ZPA %g	Distance to Fault km	W kips	T (sec)	Displacement inches -	Displacement inches +
Borchard Ranch	140	14.58	0.17	22	875.40	0.705	0.340	0.427
Borchard Ranch	230	11.23	0.15	22	992.00	0.750	0.171	0.212
Keystone Rd.	140	31.21	0.33	16	450.80	0.505	0.913	1.322
Keystone Rd.	230	26.51	0.44	16	338.10	0.438	0.502	0.298
Pine Union	140	46.32	0.27	13	676.20	0.619	0.449	0.805
Pine Union	230	36.80	0.22	13	551.20	0.559	1.553	0.885
Anderson Road	140	37.05	0.49	7	303.80	0.415	0.900	1.569
Anderson Road	230	77.65	0.39	7	381.30	0.465	1.050	0.709
James Road	140	43.99	0.59	4	252.00	0.378	1.010	0.766
James Road	230	86.56	0.39	4	381.30	0.465	3.159	5.219
Huston Road	140	63.13	0.55	1	270.50	0.392	0.181	0.500
Huston Road	230	108.71	0.46	1	323.40	0.428	1.174	0.733
Imp. Val. Coll.	140	44.96	0.34	1	437.70	0.498	0.666	0.712
Imp. Val. Coll.	230	107.83	0.47	1	316.52	0.424	2.970	2.054
Cruickshank Rd.	140	53.43	0.61	4	243.90	0.372	0.344	0.752
Cruickshank Rd.	230	47.71	0.58	4	256.30	0.381	0.488	0.423
Community Hosp.	140	42.18	0.23	9	646.93	0.606	0.629	0.621
Community Hosp.	230	44.28	0.18	9	826.80	0.685	0.798	0.479
McCabe School	140	35.01	0.38	13	391.40	0.710	0.710	0.716
McCabe School	230	39.21	0.39	13	381.30	0.465	1.170	0.870
Brockman Rd.	140	17.52	0.15	18	992.00	0.750	0.569	0.827
Brockman Rd.	230	19.38	0.12	18	1240.20	0.839	0.487	0.466
Strobel Resid.	140	14.63	0.12	22	1240.00	0.839	0.545	0.437
Strobel Resid.	230	14.21	0.14	22	1063.00	0.777	0.423	0.320
Bonds Corners	140	43.63	0.64	3	232.30	0.363	2.865	2.236
Bonds Corners	230	44.07	0.85	3	174.80	0.315	2.078	3.373
Parachute Test	225	17.15	0.12	15	1240.50	0.840	0.385	0.546
Parachute Test	315	14.62	0.20	15	744.30	0.650	0.504	0.489
Brawley Airport	225	35.29	0.19	7	783.19	0.667	0.267	0.286
Brawley Airport	315	37.12	0.23	7	646.90	0.606	0.869	0.903
Calexico Fire St.	225	19.43	0.28	11	531.50	0.549	0.791	1.056
Calexico Fire St.	315	16.08	0.21	11	708.69	0.634	0.878	0.523
Calepatria Fire	225	14.99	0.13	21	1144.80	0.806	0.243	0.212
Calepatria Fire	315	12.41	0.08	21	1860.50	1.028	0.292	0.562
Coachella Canal	225	12.89	0.12	47	1144.80	0.806	0.715	1.410
Coachella Canal	315	15.95	0.13	47	1240.20	0.839	0.727	0.499
Plaster City St.	225	3.22	0.05	31	2480.80	1.187	0.124	0.244
Plaster City St.	315	5.77	0.06	31	2976.80	1.300	0.157	0.176
Dogwood Road	270	67.77	0.37	5	402.20	0.478	0.418	0.899
Dogwood Road	360	42.51	0.50	5	297.60	0.411	1.549	0.915
El Centro 1940	90	36.92	0.27	7	551.30	0.559	1.700	1.860
El Centro 1940	00	33.44	0.36	7	413.50	0.484	1.520	1.033

TABLE 5 - CALCULATED BUILDING WEIGHT, PERIOD AND DISPLACEMENTS FOR PEAK GROUND VELOCITY BASED DESIGN USING WALL B

Station	Comp	PGV cm/s	ZPA %g	Distance to Fault km	W kips	T sec	Displacement inches -	Displacement inches +
Borchard Ranch	140	14.58	0.17	22	1583.4	0.95	0.648	0.491
Borchard Ranch	230	11.23	0.15	22	2324.7	1.15	0.326	0.488
Keystone Rd.	140	31.21	0.33	16	511.3	0.54	0.725	1.579
Keystone Rd.	230	26.51	0.44	16	651.2	0.61	0.614	0.627
Pine Union	140	46.32	0.27	13	284.9	0.40	0.483	0.474
Pine Union	230	36.80	0.22	13	400.6	0.48	0.294	0.239
Anderson Road	140	37.05	0.49	7	396.6	0.47	1.334	2.266
Anderson Road	230	77.65	0.39	7	166.8	0.40	0.089	0.049
James Road	140	43.99	0.59	4	307.5	0.42	1.264	0.706
James Road	230	86.56	0.39	4	149.6	0.40	0.081	0.117
Huston Road	140	63.13	0.55	1	205.0	0.40	0.099	0.157
Huston Road	230	108.71	0.46	1	119.1	0.40	0.057	0.077
Imp. Val. Coll.	140	44.96	0.34	1	297.7	0.41	0.234	0.317
Imp. Val. Coll.	230	107.83	0.47	1	120.1	0.40	0.063	0.056
Cruickshank Rd.	140	53.43	0.61	4	242.4	0.40	0.329	0.739
Cruickshank Rd.	230	47.71	0.58	4	271.4	0.40	0.452	0.445
Community Hosp.	140	42.18	0.23	9	327.3	0.43	0.308	0.357
Community Hosp.	230	44.28	0.18	9	304.5	0.42	0.140	0.088
McCabe School	140	35.01	0.38	13	431.3	0.49	0.662	0.792
McCabe School	230	39.21	0.39	13	364.6	0.46	1.132	0.783
Brockman Rd.	140	17.52	0.15	18	1202.8	0.83	0.764	1.001
Brockman Rd.	230	19.38	0.12	18	1035.8	0.77	0.490	0.408
Strobel Resid.	140	14.63	0.12	22	1575.6	0.95	0.771	0.667
Strobel Resid.	230	14.21	0.14	22	1652.0	0.97	0.590	0.334
Bonds Corners	140	43.63	0.64	3	311.3	0.42	4.131	1.841
Bonds Corners	230	44.07	0.85	3	306.7	0.42	2.120	5.109
Parachute Test	225	17.15	0.12	15	1241.5	0.84	0.385	0.546
Parachute Test	315	14.62	0.20	15	1577.1	0.95	0.375	0.660
Brawley Airport	225	35.29	0.19	7	426.2	0.49	0.163	0.316
Brawley Airport	315	37.12	0.23	7	395.5	0.47	0.572	0.303
Calexico Fire St.	225	19.43	0.28	11	1031.9	0.77	1.225	1.000
Calexico Fire St.	315	16.08	0.21	11	1374.5	0.88	1.216	0.700
Calepatria Fire	225	14.99	0.13	21	1521.2	0.93	0.325	0.313
Calepatria Fire	315	12.41	0.08	21	1998.4	1.07	0.421	0.645
Coachella Canal	225	12.89	0.12	47	1881.4	1.03	1.399	0.647
Coachella Canal	315	15.95	0.13	47	1390.7	0.89	0.619	1.405
Plaster City St.	225	3.22	0.05	31	15148.9	2.93	0.645	0.523
Plaster City St.	315	5.77	0.06	31	6300.8	1.89	0.714	0.450
Dogwood Road	270	67.77	0.37	5	191.1	0.40	0.351	0.417
Dogwood Road	360	42.51	0.50	5	323.5	0.43	2.025	1.198
El Centro 1940	90	36.92	0.27	7	398.6	0.48	0.338	0.621
El Centro 1940	00	33.44	0.36	7	461.7	0.51	1.570	1.470

TABLE 6 - CALCULATED BUILDING WEIGHT, PERIOD AND DISPLACEMENT FOR PROPOSED SPECTRAL VELOCITY BASED DESIGN USING WALL B

Station	Comp	S_v cm/s	ZPA %g	Distance to Fault km	W kips	T sec	Displacement inches -	+
Borchard Ranch	140	32.80	0.17	22	2289.57	1.140	0.870	0.568
Borchard Ranch	230	23.40	0.15	22	4498.52	1.600	0.777	0.695
Keystone Rd.	140	89.40	0.33	16	376.00	0.460	0.837	1.060
Keystone Rd.	230	54.60	0.44	16	826.26	0.690	0.785	0.540
Pine Union	140	98.30	0.27	13	376.00	0.460	0.762	0.695
Pine Union	230	57.90	0.22	13	734.76	0.650	0.488	0.781
Anderson Road	140	115.00	0.49	7	376.00	0.460	1.046	2.123
Anderson Road	230	141.00	0.39	7	295.00	0.409	0.481	0.159
James Road	140	121.90	0.59	4	295.00	0.409	1.270	0.694
James Road	230	199.10	0.39	4	295.00	0.409	1.130	1.350
Huston Road	140	158.70	0.55	1	295.00	0.409	0.242	0.706
Huston Road	230	243.10	0.46	1	295.00	0.409	0.940	0.642
Imp. Val. Coll.	140	103.10	0.34	1	376.00	0.460	0.575	0.696
Imp. Val. Coll.	230	201.70	0.47	1	295.00	0.409	1.980	1.390
Cruickshank Rd.	140	124.50	0.61	4	295.00	0.409	0.642	0.838
Cruickshank Rd.	230	98.00	0.58	4	376.00	0.460	0.597	0.789
Community Hosp.	140	100.30	0.23	9	376.00	0.460	0.317	0.363
Community Hosp.	230	86.40	0.18	9	376.00	0.460	0.230	0.235
McCabe School	140	92.20	0.38	13	376.00	0.460	0.702	0.545
McCabe School	230	76.40	0.39	13	422.00	0.490	1.190	1.140
Brockman Rd.	140	45.00	0.15	18	1216.40	0.830	0.897	1.040
Brockman Rd.	230	35.30	0.12	18	1976.75	1.060	1.060	1.180
Strobel Resid.	140	33.00	0.12	22	2261.90	1.130	1.030	1.190
Strobel Resid.	230	26.00	0.14	22	3643.80	1.440	2.160	1.700
Bonds Corners	140	142.00	0.64	3	295.00	0.409	4.530	1.570
Bonds Corners	230	174.50	0.85	3	295.00	0.409	2.110	5.090
Parachute Test	225	32.50	0.12	15	2332.04	1.150	1.250	0.812
Parachute Test	315	34.30	0.20	15	2093.70	1.090	0.380	0.786
Brawley Airport	225	68.60	0.19	7	523.42	0.550	0.180	0.274
Brawley Airport	315	50.50	0.23	7	965.87	0.740	2.110	2.620
Calexico Fire St.	225	58.40	0.28	11	722.23	0.640	0.792	1.260
Calexico Fire St.	315	42.70	0.21	11	1350.97	0.880	1.190	0.706
Calepatria Fire	225	28.20	0.13	21	3097.44	1.330	1.040	0.436
Calepatria Fire	315	23.10	0.08	21	4616.13	1.620	1.000	2.100
Coachella Canal	225	27.90	0.12	47	3164.41	1.340	1.340	0.540
Coachella Canal	315	46.20	0.13	47	1154.03	0.890	0.699	1.410
Plaster City St.	225	8.60	0.05	31	33304.65	4.360	0.772	0.877
Plaster City St.	315	11.90	0.06	31	17394.34	3.140	1.160	0.445
Dogwood Road	270	119.60	0.37	5	376.00	0.460	0.432	0.817
Dogwood Road	360	126.00	0.50	5	295.00	0.409	1.480	0.902
El Centro 1940	90	80.16	0.27	7	383.34	0.470	1.370	0.950
El Centro 1940	00	67.05	0.36	7	547.90	0.560	1.690	1.850

TABLE 7 - CALCULATED BUILDING WEIGHT, PERIOD, AND DISPLACEMENT FOR STRENGTH VARIATION

Wall	Station	Comp	S_v cm/s	W kips	T sec	Displacement inches -	+
A	Bonds Corners	140	142.0	215	0.35	3.85	1.94
	Bonds Corners	230	174.5	215	0.35	2.08	4.73
	James Road	140	121.9	215	0.35	1.16	0.67
	James Road	230	199.1	215	0.35	0.87	1.04
	Brockman Road	140	45.0	645	0.60	0.52	0.60
	Brockman Road	230	35.3	1048	0.77	0.54	0.54
	El Centro,1940	0	80.2	274	0.39	1.11	0.70
	El Centro,1940	90	67.1	291	0.41	0.23	0.38
B	Bonds Corners	140	142.0	295	0.41	4.53	1.57
	Bonds Corners	230	174.5	295	0.41	2.11	5.09
	James Road	140	121.9	295	0.41	1.27	0.69
	James Road	230	199.1	295	0.41	1.13	1.35
	Brockman Road	140	45.0	1216	0.83	0.90	1.04
	Brockman Road	230	35.3	1976	1.06	1.06	1.18
	El Centro,1940	0	80.2	383	0.47	1.37	0.95
	El Centro,1940	90	67.1	548	0.56	1.69	1.85
C	Bonds Corners	140	142.0	365	0.46	4.89	1.59
	Bonds Corners	230	174.5	365	0.46	2.17	5.21
	James Road	140	121.9	365	0.46	1.29	0.73
	James Road	230	199.1	365	0.46	1.31	1.50
	Brockman Road	140	45.0	1854	1.03	1.15	1.26
	Brockman Road	230	35.3	3013	1.31	1.44	1.36
	El Centro,1940	0	80.2	584	0.58	1.62	1.83
	El Centro,1940	90	67.1	835	0.69	2.30	1.80
D	Bonds Corners	140	142.0	454	0.51	5.16	1.51
	Bonds Corners	230	174.5	454	0.51	2.19	5.73
	James Road	140	121.9	454	0.51	1.41	0.82
	James Road	230	199.1	454	0.51	1.67	1.72
	Brockman Road	140	45.0	2,876	1.28	1.23	2.03
	Brockman Road	230	35.3	4,675	1.63	2.20	2.25
	El Centro,1940	0	80.2	907	0.72	2.92	2.77
	El Centro,1940	90	67.1	1,296	0.86	2.67	2.09
E	Bonds Corners	140	142.0	664	0.61	5.89	1.47
	Bonds Corners	230	174.5	664	0.61	2.48	4.90
	James Road	140	121.9	664	0.61	1.64	1.48
	James Road	230	199.1	664	0.61	2.18	2.35
	Brockman Road	140	45.0	6,156	1.87	3.00	1.16
	Brockman Road	230	35.3	10,004	2.38	1.57	1.14
	El Centro,1940	0	80.2	1,940	1.05	6.75	3.12
	El Centro,1940	90	67.1	2,773	1.25	2.02	4.12

TABLE 8 - MAXIMUM DISPLACEMENTS OF
28' X 19.33' SHEAR WALL WITH STRENGTH VARIATIONS

Ground Motion		Maximum Displacement: inches				
		Wall A #4@16	Wall B #5@16	Wall C #6@16	Wall D #7@16	Wall E #9@16
Bonds Corners	140	3.85	4.53	4.89	5.16	5.89
Bonds Corners	230	4.73	5.09	5.21	5.73	4.90
James Road	140	1.16	1.27	1.29	1.41	1.64
James Road	230	1.40	1.35	1.50	1.72	2.35
Breckman Road	140	0.60	1.04	1.26	2.03	3.00
Breckman Road	230	0.54	1.18	1.44	2.25	1.57
El Centro,1940	0	1.11	1.37	1.83	2.92	6.75
El Centro,1940	90	0.38	1.85	2.30	2.67	4.12
Mean of Displacements, Excluding Bonds Corners		0.87	1.34	1.60	2.17	3.24
e_p of shear wall		3.84	2.93	2.13	1.87	1.53
Reinforcement ratio		0.00111	0.00172	0.00245	0.00334	0.00556

TABLE 9 - MAXIMUM DISPLACEMENT OF 28' X 19.33'
SHEAR WALLS WITH UNDER AND EXCESS DESIGN STRENGTH

Ground Motion		Maximum Displacement: Inches				
		Wall A #4@16	Wall B #5@16	Wall C #6@16	Wall D #7@16	Wall E #9@16
Bonds Corners	140	3.18	4.53	3.33	0.92	0.41
James Road	140	1.49	1.27	0.89	0.57	0.36
Brockman Road	140	1.69	1.04	0.82	0.59	0.45
El Centro,1940	0	1.82	1.37	0.92	0.42	0.31
James Road	230	5.24	1.35	0.51	0.36	0.28
El Centro,1940	90	2.41	1.85	0.93	0.47	0.27
e_p of shear wall		3.84	2.93	2.13	1.87	1.53
Ratio of capacity to design moment		0.73	1.00	1.24	1.55	2.27

TABLE 10 - CALCULATED PROPERTIES OF SHEAR WALLS WITH STIFFNESS VARIATION

Wall	Dimensions Height Feet	Length Feet	Initial Stiffness, K_1 Kips/inch	F_P kips	e_p inches
F	30	16	929	114	2.93
B	28	19.33	1,738	140.1	2.93
G	24	24	3,968	183	2.36
H	20	30	8,220	268	1.75

TABLE 11 - MOMENT STRENGTH OF SHEAR WALLS WITH STIFFNESS VARIATIONS AS DETERMINED BY RCCOLA [11] AND FEM ANALYSIS

Wall	Reinforcement Vertical	Horizontal	Wall Weight kips	Moment, Ft. Kips RCCOLA Analysis	FEM Analysis
F	#6@16"	#4@16"	55.8	3,161	3,420
B	#5@16"	#4@16"	62.9	3,307	3,923
G	#6@40"	#4@16"	66.9	3,161	4,392
H	#4@32"	#4@16"	69.7	3,114	5,360

TABLE 12 - NOMINAL AND DESIGN YIELD STRENGTH OF SHEAR WALLS WITH STIFFNESS VARIATION

Wall	Wall Size Hgt. x Depth Feet	Reinf. Vertical	Reinf. Ratio	Nominal Shear kips	Design Shear kips	Design Moment ft. kips	% M_e
F	30 x 16	#6@16"[1]	0.00256	80.8	64.6	1,938	0.57
B	28 x 19.33	#5@16"	0.00172	102.5	82.1	2,298	0.59
G	24 x 24	#6@40"	0.00105	123.5	98.8	2,371	0.54
H	20 x 30	#4@32"	0.00057	178.3	142.7	2,854	0.53

[1] Spacing of first and second bars is 12 inches

TABLE 13 - CALCULATED BUILDING WEIGHT, PERIOD AND DISPLACEMENT FOR SHEAR WALLS WITH STIFFNESS VARIATION

Wall	Station	Comp	S_v cm/s	W kips	T sec	Displacement inches -	+
F	Bonds Corners	140	142.0	231	0.50	4.36	2.16
	Bonds Corners	230	174.5	231	0.50	2.20	5.00
	James Road	140	121.9	231	0.50	1.40	0.80
	James Road	230	199.1	231	0.50	1.65	1.72
	Brockman Road	140	45.0	1,437	1.26	1.22	1.98
	Brockman Road	230	35.3	2,335	1.60	1.80	1.80
	El Centro,1940	0	80.2	453	0.71	2.61	2.67
	El Centro,1940	90	67.1	647	0.84	2.66	2.07
B	Bonds Corners	140	142.0	295	0.41	4.53	1.57
	Bonds Corners	230	174.5	295	0.41	2.11	5.09
	James Road	140	121.9	295	0.41	1.27	0.69
	James Road	230	199.1	295	0.41	1.13	1.35
	Brockman Road	140	45.0	1,216	0.83	0.90	1.04
	Brockman Road	230	35.3	1,976	1.06	1.06	1.18
	El Centro,1940	0	80.2	383	0.47	1.37	0.95
	El Centro,1940	90	67.1	548	0.56	1.69	1.85
G	Bonds Corners	140	142.0	353	0.30	3.43	1.98
	Bonds Corners	230	174.5	353	0.30	2.09	5.36
	James Road	140	121.9	353	0.30	0.82	0.68
	James Road	230	199.1	353	0.30	0.39	0.53
	Brockman Road	140	45.0	787	0.45	0.24	0.23
	Brockman Road	230	35.3	1,279	0.57	0.29	0.29
	El Centro,1940	0	80.2	449	0.34	0.48	0.30
	El Centro,1940	90	67.1	449	0.34	0.21	0.16
H	Bonds Corners	140	142.0	509	0.25	3.20	1.86
	Bonds Corners	230	174.5	509	0.25	2.02	5.13
	James Road	140	121.9	509	0.25	0.51	0.52
	James Road	230	199.1	509	0.25	0.21	0.37
	Brockman Road	140	45.0	793	0.31	0.07	0.08
	Brockman Road	230	35.3	1,288	0.40	0.14	0.18
	El Centro,1940	0	80.2	648	0.28	0.34	0.21
	El Centro,1940	90	67.1	648	0.28	0.08	0.06

TABLE 14 - MAXIMUM DISPLACEMENTS OF
SHEAR WALLS WITH STIFFNESS VARIATION

Ground Motions		Maximum Displacement, inches Wall F 30' x 16'	Wall B 28' x 19.33'	Wall G 24' x 24'	Wall H 20'x 30'
Bonds Corners	140	4.36[1]	4.53	3.43	3.20
Bonds Corners	230	5.00	5.09	5.36	5.13
James Road	140	1.40	1.27	0.82	0.52
James Road	230	1.72	1.35	0.53	0.37
Brockman Road	140	1.98	1.04	0.24	0.08
Brockman Road	230	1.80	1.18	0.29	0.18
El Centro,1940	00	2.67	1.37	0.48	0.34
El Centro,1940	90	2.66	1.85	0.21	0.09
Mean of Displacements Excluding Bonds Corners		2.04	1.34	0.43	0.26
e_p of shear wall		2.93	2.93	2.36	1.75
Reinforcement ratio		0.00256	0.00172	0.00105	0.00057

[1] secant period of shear wall is in frequency range of diminishing energy

TABLE 15 - MAXIMUM DISPLACEMENT OF SHEAR WALLS
WITH EQUAL SEISMIC DEMAND AND STIFFNESS VARIATION

Ground Motions		Maximum Displacement, inches Wall F 30' x 16'	Wall B 28' x 19.33'	Wall G 24' x 24'	Wall H 20'x 30'
Bonds Corners	140	3.29[1]	4.53	2.11	0.24
James Road	140	1.58	1.27	0.39	0.11
James Road	230	4.70	1.35	0.22	0.04
Brockman Road	140	1.69	1.04	0.64	0.19
El Centro,1940	00	2.11	1.37	0.48	0.05
El Centro,1940	90	2.46	1.85	0.38	0.04
Ratio of capacity to design moment		0.87	1.00	1.12	1.37
e_p of shear wall		2.93	2.93	2.36	1.75

[1] secant period of shear wall is in frequency range of diminishing energy

TABLE 16 - DATA USED FOR CALCULATION OF COMPARATIVE STATIC AND DYNAMIC DISPLACEMENTS

Wall	Ground Motions	S_v cm/sec	W kips	T sec	V, Eq.6 kips	Max. Displ. inches LPM	Max. Displ. inches Static	Ratio
F	James Road 140	121.9	231	0.50	64.6	1.40	0.50	2.80
	Brockman Road 140	45.0	1,437	1.26	64.6	1.98	0.50	3.96
	El Centro,1940 00	80.2	453	0.71	64.6	2.67	0.50	5.34
	El Centro,1940 90	67.1	647	0.84	64.6	2.66	0.50	5.32
				Mean				4.36
				Std. dev.				1.22

Wall	Ground Motions	S_v cm/sec	W kips	T sec	V, Eq.6 kips	Max. Displ. inches LPM	Max. Displ. inches Static	Ratio
B	James Road 140	121.9	295	0.41	82.1	1.27	0.30	4.23
	Brockman Road 140	45.0	1,216	0.83	82.1	1.04	0.30	3.47
	El Centro,1940 00	80.2	383	0.47	82.1	1.37	0.30	4.57
	El Centro,1940 90	67.1	540	0.56	82.1	1.85	0.30	6.17
				Mean				4.61
				Std. dev.				1.14

Wall	Ground Motions	S_v cm/sec	W kips	T sec	V, Eq.6 kips	Max. Displ. inches LPM	Max. Displ. inches Static	Ratio
G	James Road 140	121.9	353	0.30	98.8	0.82	0.12	6.80
	Brockman Road 140	45.0	787	0.45	98.8	0.24	0.12	2.00
	El Centro,1940 00	80.2	449	0.34	98.8	0.48	0.12	4.00
	El Centro,1940 90	67.1	449	0.34	98.8	0.21	0.12	1.75
				Mean				3.64
				Std. dev.				2.34

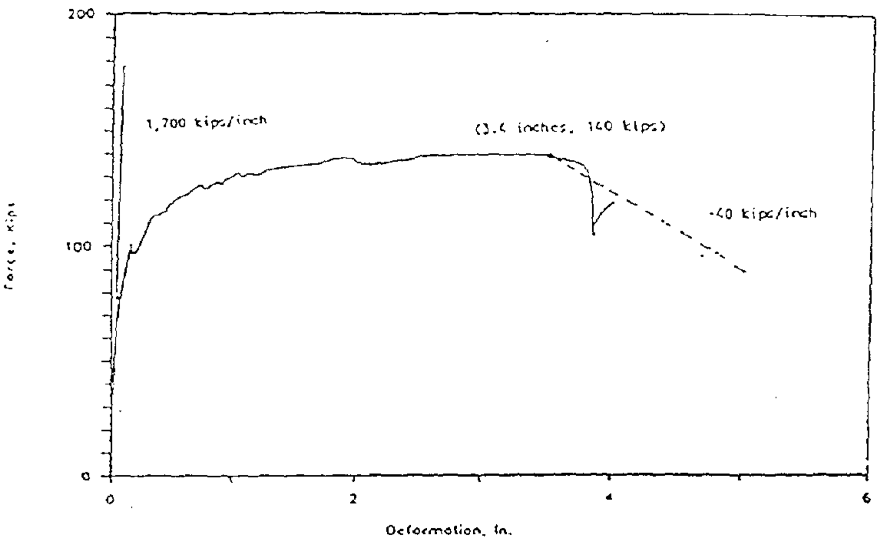

FIGURE 1. RESULTS OF FINITE ELEMENT ANALYSIS

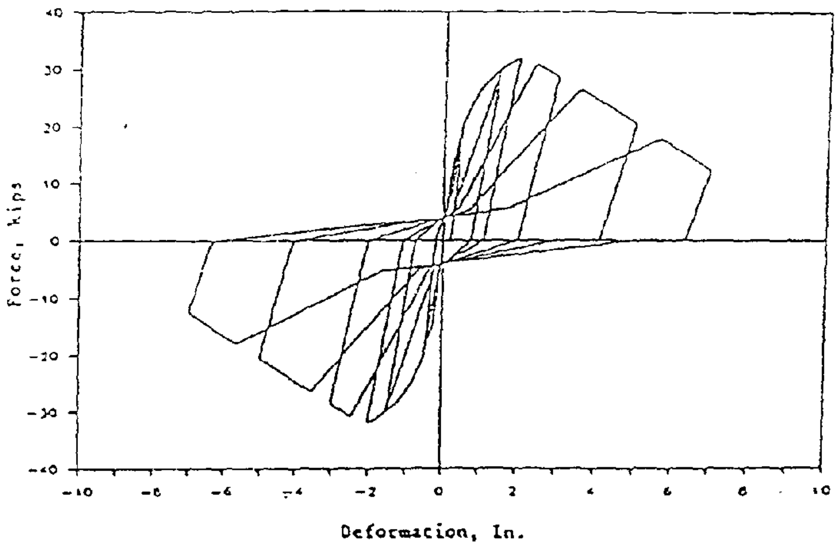

FIGURE 2. EKEH HYSTERESIS MODEL

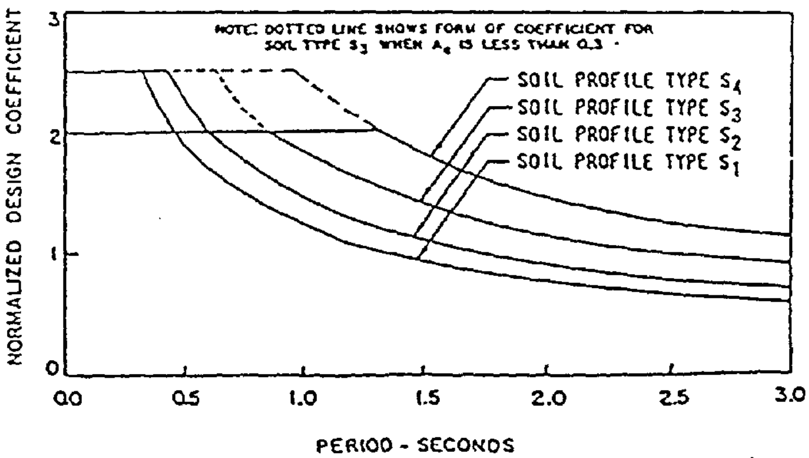

FIGURE 3. NORMALIZED LATERAL DESIGN FORCE COEFFICIENTS ($A_a = A_v = 1.0$).

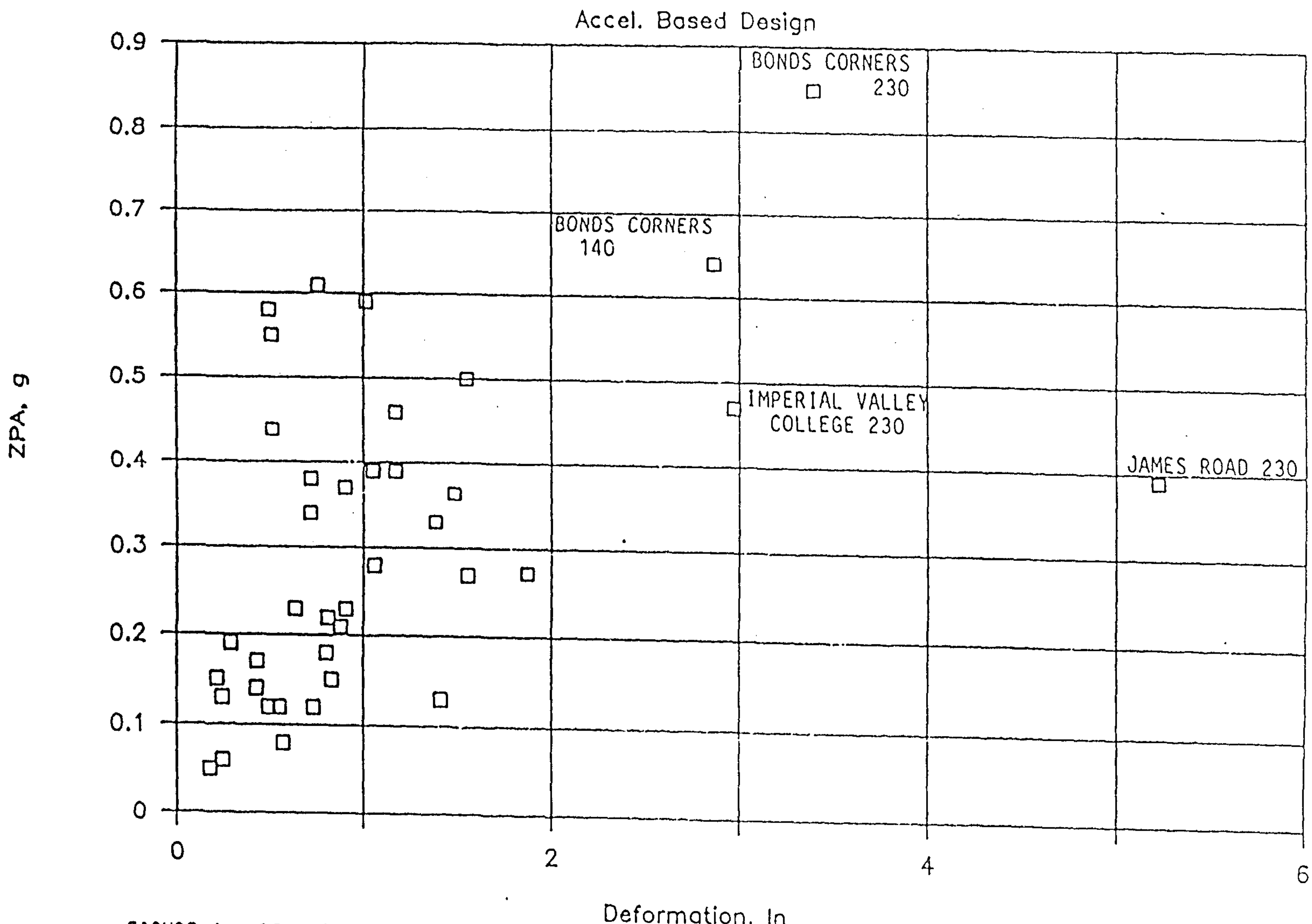

FIGURE 4. RELATIVE DISPLACEMENTS OF MASONRY SHEAR WALLS DESIGNED ON BASIS OF ACCELERATION

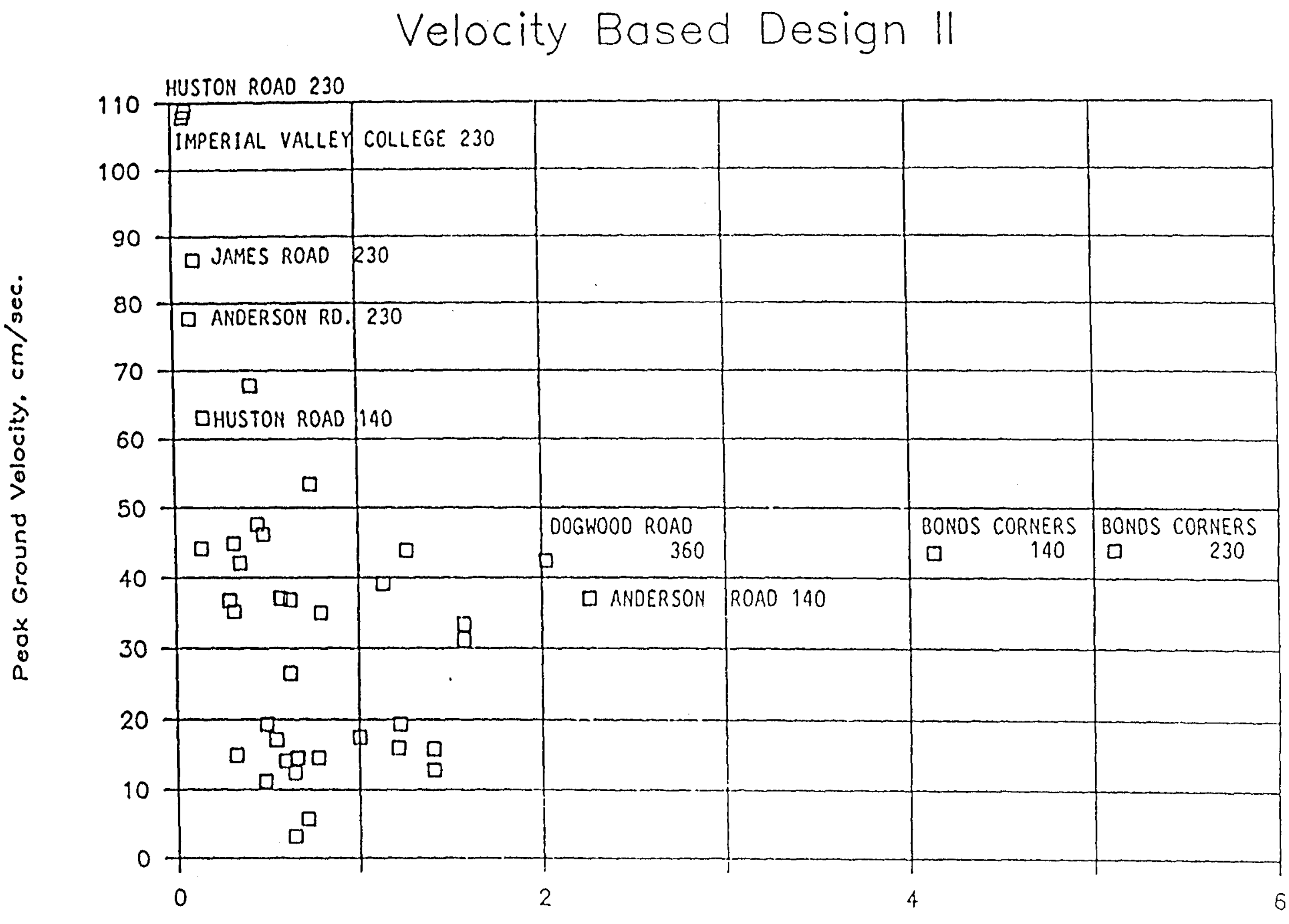

FIGURE 5. MAXIMUM RELATIVE DISPLACEMENTS OF MASONRY SHEAR WALLS DESIGNED ON BASIS OF PEAK GROUND VELOCITY

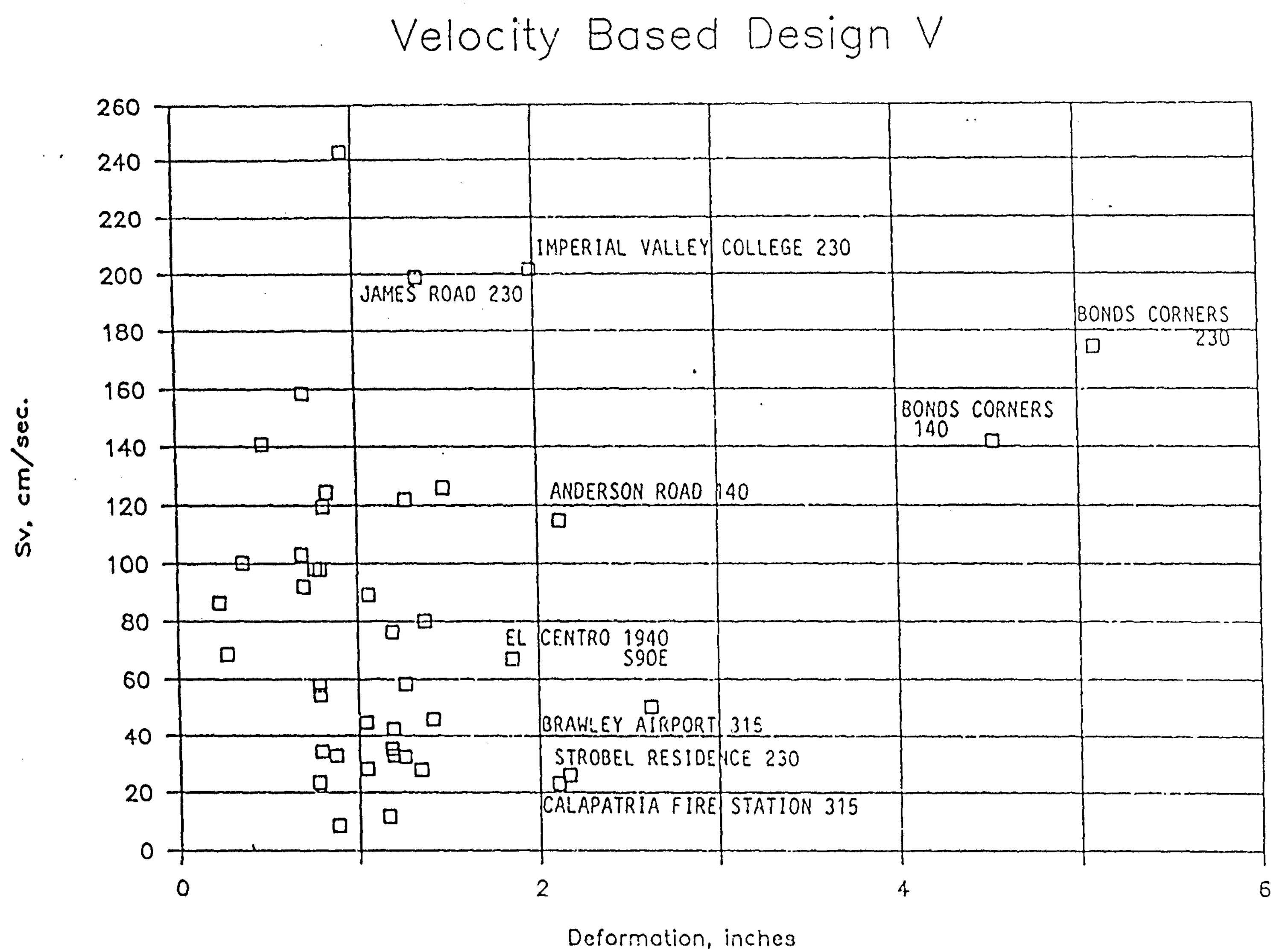

FIGURE 6. MAXIMUM RELATIVE DISPLACEMENTS OF MASONRY SHEAR WALLS DESIGNED ON THE BASIS OF MAXIMUM SPECTRAL VELOCITY

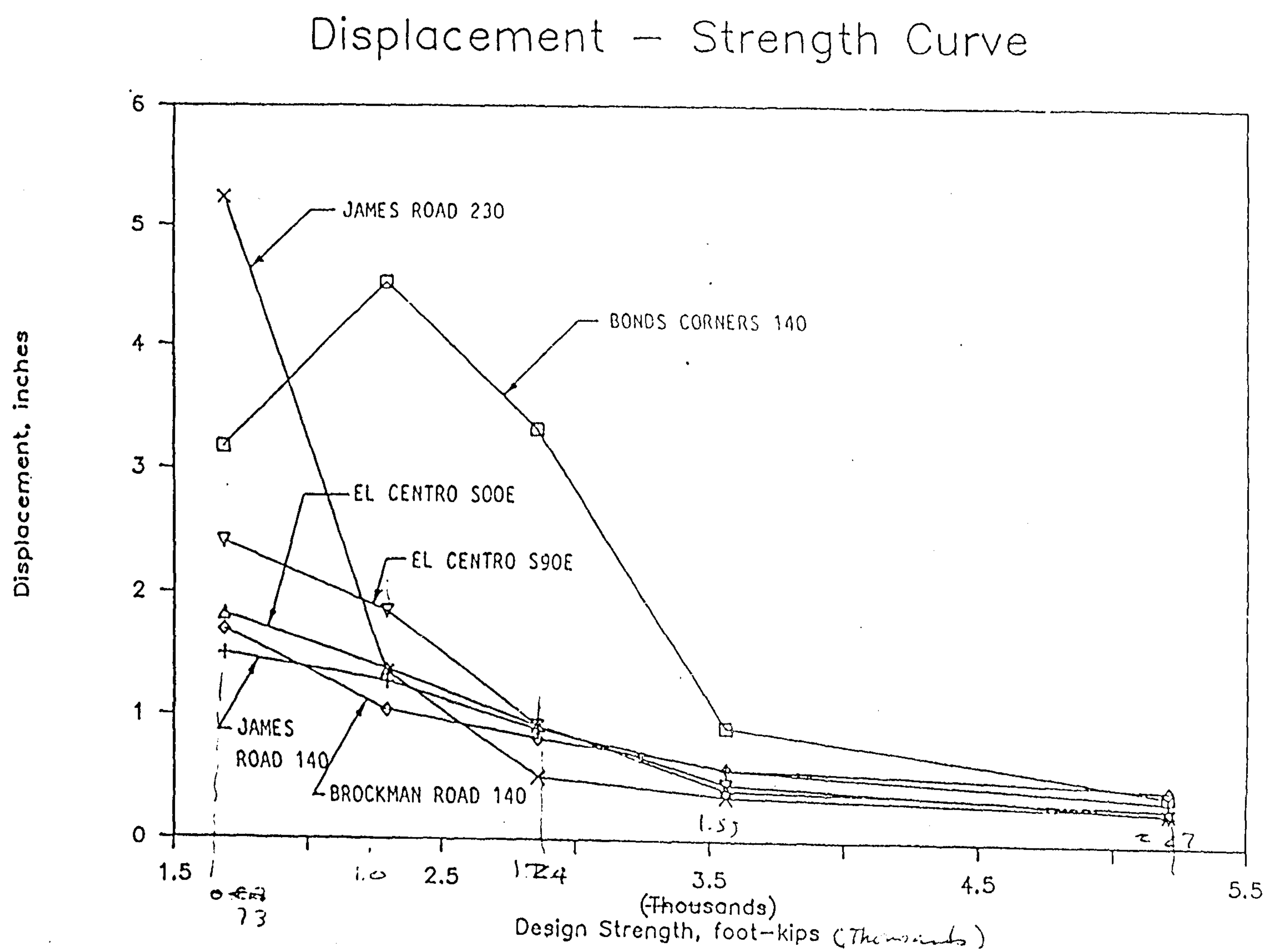

FIGURE 7. INFLUENCE OF UNDER AND EXCESS STRENGTH

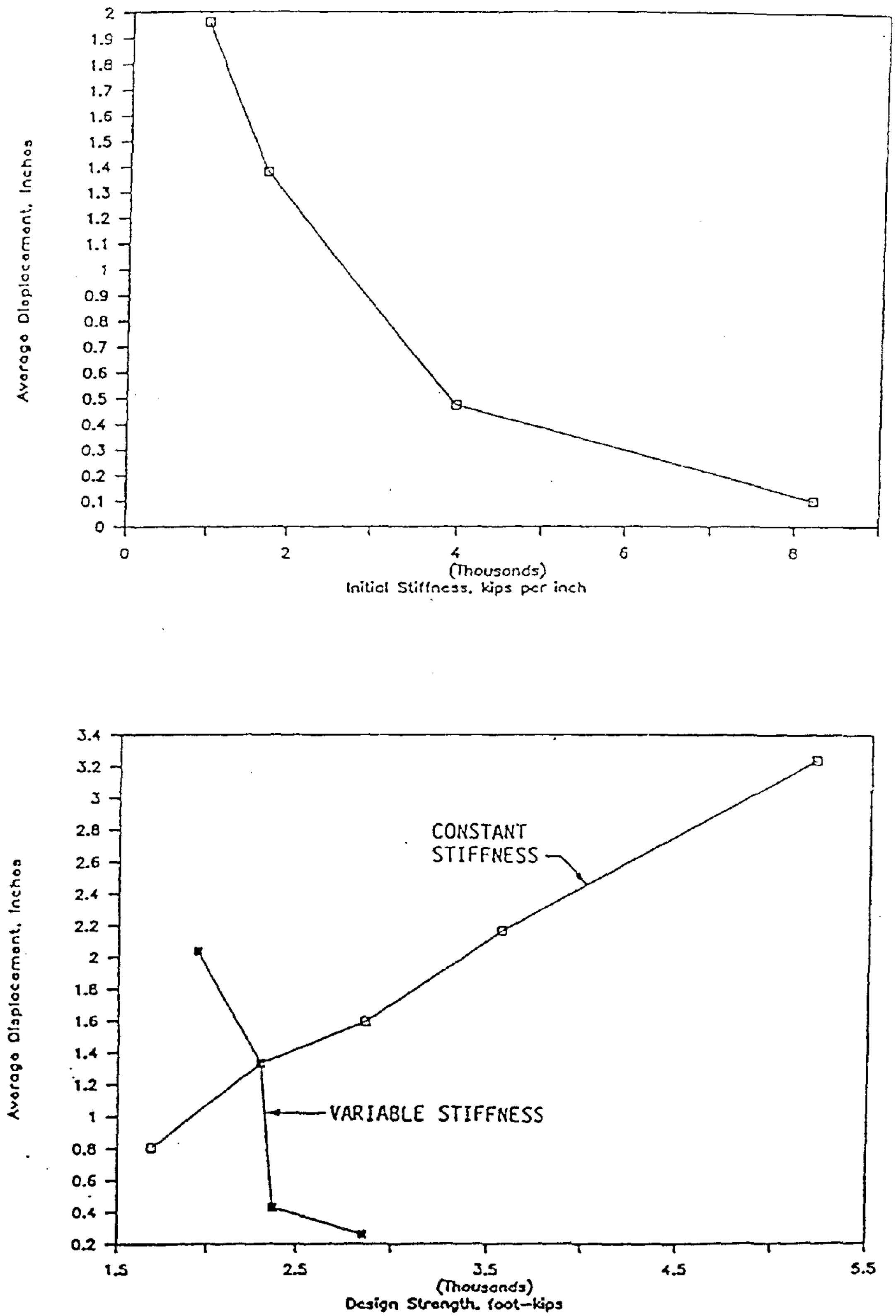

FIGURE 8. RELATIONSHIPS OF STIFFNESS, STRENGTH, AND DISPLACEMENT CONTROL

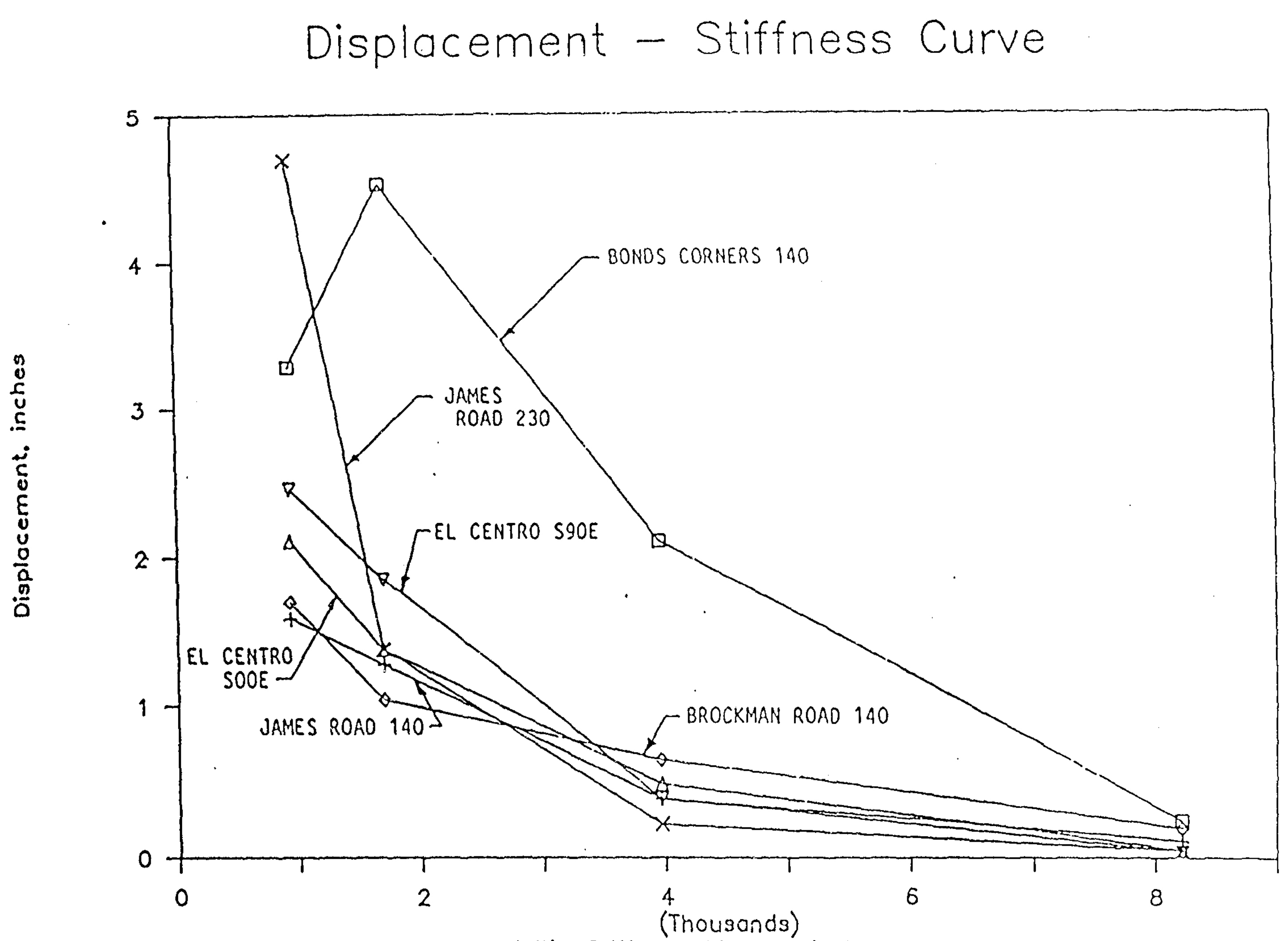

FIGURE 9. RELATIONSHIP OF STIFFNESS VARIATION ON DISPLACEMENTS OF SHEAR WALLS WITH EQUAL SEISMIC DEMAND

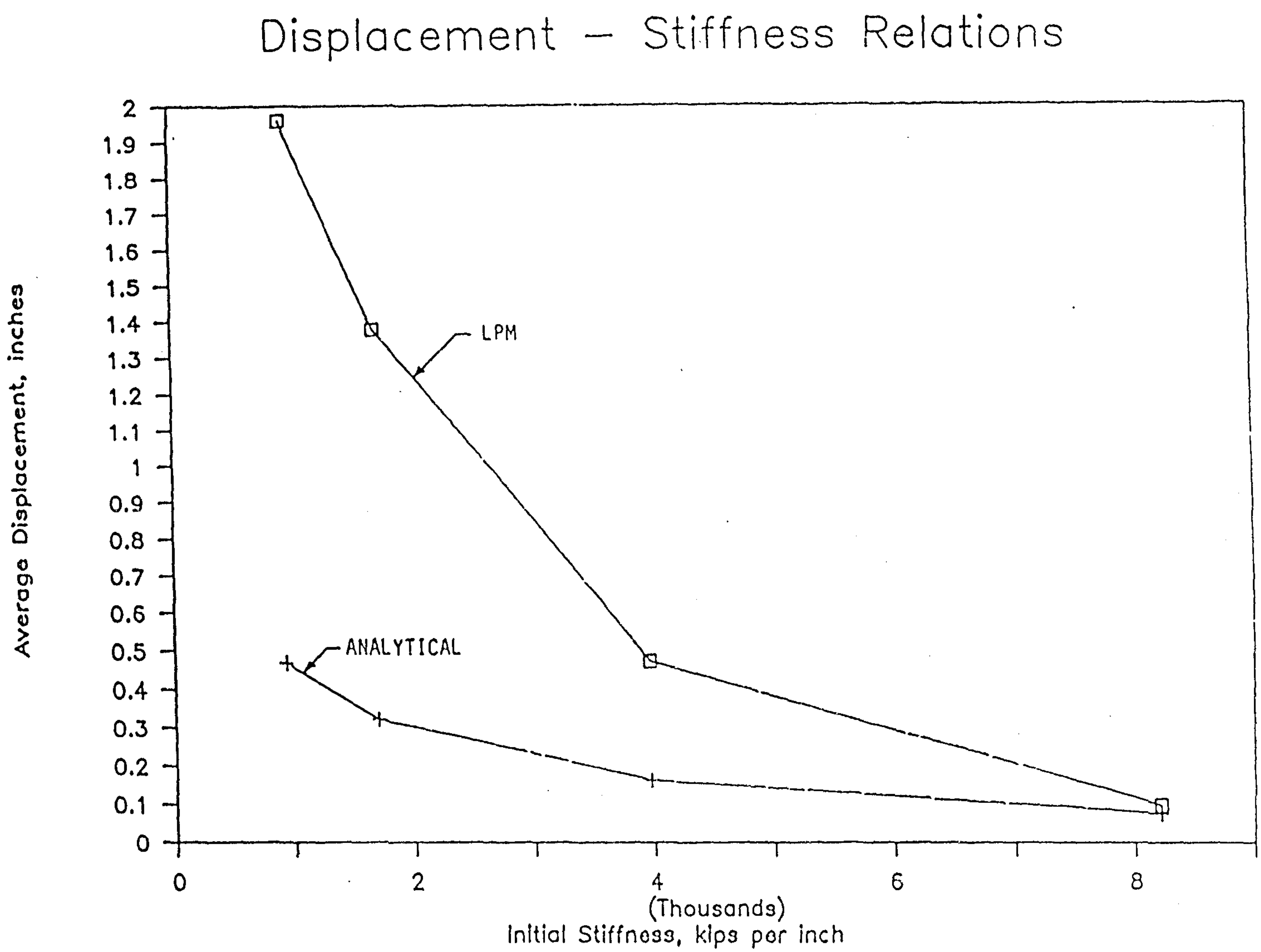

FIGURE 10. RELATIONSHIP OF CALCULATED DYNAMIC AND STATIC DEFLECTIONS FOR FOUR GROUND MOTIONS