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Simple Nonlinear Modelling of Earthquake Response in Torsionally Coupled R/C Structures

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A Preliminary Study

by

Mehdi Saiidi

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16. Abstract (Limit: 200 words)

Results are presented of a study to: 1) determine the seismic response of a torsionally coupled building based on multi-degree-of-freedom (MDOF) and single-degree-of-freedom (SDOF) nonlinear models and 2) to develop a simple SDOF nonlinear model to calculate displacement history of structures with eccentric centers of mass and stiffness. It is shown that planar models are able to yield qualitative estimates of the response of the building. The equivalent SDOF model developed as part of this study, the Q-Model-13, is described. The model is used to estimate the response of a hypothetical six-story frame-wall reinforced concrete building with torsional coupling, using two different earthquake intensities. It is shown that the Q-Model-13 can lead to a satisfactory estimate of the response of the structure in both cases.

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SIMPLE NONLINEAR MODELLING OF EARTHQUAKE RESPONSE IN TORSIONALLY COUPLED R/C STRUCTURES - A PRELIMINARY STUDY

by

Mehdi Saiidi

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ABSTRACT

This report presents the results of a preliminary study (1) to determine the seismic response of a torsionally coupled building, namely, the Imperial County Services Building, based on MDOF and SDOF planar nonlinear models and (2) to develop a simple SDOF nonlinear model to calculate displacement history of structures with eccentric centers of mass and stiffness.

It is shown that the planar models were able only to yield a qualitative estimate of the response of the County Building. This observation is made based on the study of correlation between the analytical results and the measured response of the building during the earthquake of October 1979.

The equivalent SDOF model developed as part of this study, called the Q-Model3, is described in detail. The model was used to estimate the response of a hypothetical six-story frame-wall reinforced concrete building with torsional coupling. Two different earthquake intensities were used. It is shown that the Q-Model3 led to a satisfactory estimate of the response for the structure in both cases. The basis for evaluation of the Q-Model3 was from the results of a MDOF nonlinear model.

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CHAPTER ONE

INTRODUCTION

1.1 Object and Scope

Earthquake response of structures with unsymmetric distribution of mass and stiffness may include significant contribution from torsional vibration, even if only one translation component of earthquake is applied. The extent of contribution from torsion depends on, in part, the eccentricity between the center of mass and stiffness. While the eccentricity remains constant in systems with elastic response, its magnitude varies in inelastic structures because the center of stiffness generally changes as nonlinear deformations are developed.

Inclusion of torsional effects in multi-degree-of-freedom (MDOF) nonlinear analytical models leads to a substantial increase in the size and cost of such models and their application [13]. For planar systems with or without uniform distribution of stiffness along the height, an approximate equivalent single-degree-of-freedom (SDOF) model (called the Q-Model) has been developed and has been found to satisfactorily simulate the nonlinear displacement history [21,34,36,37]. The primary objective of the preliminary study presented in this report was to examine the possibility of predicting the nonlinear response in torsionally coupled buildings using a simple model comparable to the Q-Model. The new model, referred to as the Q-Model3 in this report, was evaluated for a six-story frame-wall structure with unsymmetric distribution of stiffness. The measured response of the Imperial County Services Building during the earthquake of 1979 [14] was used in initial parts of the study to determine whether the response of an unsymmetric structure may be predicted based on the models for planar systems.

1.2 Review of Previous Research

The elastic response of structures with torsional coupling has been the subject of numerous analytical and experimental studies [2,3,-11,17,18,22,32,33]. These studies provided detailed information about the elastic behavior of structures. However, buildings designed based on current design codes are expected to develop significant yielding in the event of a severe earthquake and the results of research on elastic response may not provide adequate information about the actual behavior. Several investigators have included the nonlinearity of structures in their studies as described below [5,9,12,16,19,20,27,31].

Shibata, et.al., studied the torsion effect on one-story unsymmetric building models [40], using both analytical models and physical small-scale test specimens. An elasto-plastic hysteresis model was used. The torsion effect in this study was found to be very significant. A parameter study showed that, by using an appropriate combination of stiffness and strength of columns, a uniform distribution of ductility may be achieved. Kan and Chopra conducted an analytical study on a one-story torsionally coupled structure [19]. The stiffness variation at the joints were idealized by an elasto-plastic yield surface. In comparing the linear and nonlinear response of this system, it was found that the nonlinear response was less sensitive to the torsional effects.

Irvine and Kountouris used a simple system, with eccentric center of mass and stiffness, in their analytical study of the nonlinear response [16]. A bilinear hysteresis model was used in the modeling. An extensive parameter study on 3500 cases was carried out which included systems with different periods, eccentricities, and base motions. The peak ductility demand was computed for systems with different eccentricities, and was found somewhat insensitive to the eccentricity. Gillies and Shepherd used a three-dimensional nonlinear response history analysis of six-story reinforced concrete structures to determine the shortcomings of planar system analyses [12]. An elasto-plastic hysteretic behavior was assumed for the beams. A substantial contribution from torsion was noted. Kan and Chopra used a single-element nonlinear model to predict the response of torsionally coupled systems [20]. An elasto-plastic hysteresis model and a simplified yield surface was used to idealize the column inelastic behavior. Results were compared with those calculated based on a multi-element model and satisfactory correlation was observed.

The inclusion of torsional effect, even in structures subjected to unidirectional earthquakes, results in biaxial bending of columns. Several analytical and experimental investigations have aimed at developing a better understanding of the biaxial behavior of columns and structures subjected to two or more components of earthquakes [1,7,9,24-27,31,39,42,44]. The results have indicated a significant increase in forces and deformations due to the biaxial effects. It has been pointed out in many of these studies that exclusion of simultaneous earthquake components, at best, can provide a qualitative estimate of the response.

A comprehensive nonlinear seismic analysis requires complex computer programs and large-capacity computers in addition to lengthy data preparation and knowledge of advanced structural behavior. A recent study by Kan and Chopra [20] was aimed at simplification of the analysis of torsionally coupled structures. The study was concentrated on one-story structures having several columns and a rigid diaphgram, with eccentric center of mass and stiffness. The structures were represented by equivalent single-element models having overall characteristics as those of the multi-column structure. The model was found to yield a reasonable approximation to the response.

1.3 Acknowledgments

The research described in this report was part of an investigation aimed at the development of simple analytical models for nonlinear seismic response of reinforced concrete structures. The study was sponsored by the National Science Foundation under an extension of Grant PFR-80-06423. Any opinions, findings, and conclusions expressed in this publication are those of the writer and do not necessarily reflect the views of the National Science Foundation.

The writer is grateful to the program manager, Dr. Michael Gaus of the National Science Foundation, for his support and criticism. Thanks are due to Mike Kreger, graduate research assistant at the Civil Engineering Department at the University of Illinois for providing the measured data of the County Building.

Mrs. Ruthe Berryman is specially thanked for typing this report.

The CDC 6400 and CYBER 172 computers at the University of Nevada, Reno, were used throughout the course of this study. Ellen Jacobson of the Computer Center provided valuable consulting service.

1.4 Notations

The following symbols are used in this report:

C = damping factor

K_e = stiffness calculated based on the hysteresis model

M_e = equivalent mass

M_t = total mass of structure

m_i = mass at level i

N = summation of number of floors at all constituent frames and walls

X_e = abscissa of centroid

$\ddot{X}_g$ = base acceleration

x = displacement relative to base

$\dot{x}$ = velocity relative to base

$\ddot{x}$ = acceleration relative to base

Y_e = equivalent height

y_i = height at level i measured from base

ϕ_i = normalized displacement at level i of any frame or wall

CHAPTER TWO

DESCRIPTION OF BUILDINGS

2.1 Introductory Remarks

Two structures were analyzed in the course of the study presented in this report. One was the Imperial County Services Building (ICSB) which was severely damaged during the earthquake of 1979 and had to be demolished, and the other was a hypothetical frame-wall reinforced concrete structure designed for the purpose of this study. This chapter presents a brief description of ICSB and the earthquake instrumentations, in addition to information regarding the hypothetical structure.

2.2 The Imperial County Service Building (ICSB)

(a) Description of ICSB - This building was a six-story reinforced concrete structure with three spans in the north-south direction and five spans in the east-west direction (Fig. 2.1). The structural system consisted of columns, shear walls, beams, slab, and joists. Lateral stiffness in the east-west direction was provided by frame action of the beams and columns, but in the north-south direction the stiffness was primarily provided by shear walls. Four shear walls were used at different locations in the first story in an unsymmetric pattern (Fig. 2.1). In the second story and higher, two exterior shear walls were built which were approximately three times as wide as the first story walls. While the structure would qualify as being uniform and symmetric in the east-west direction, the lack of symmetry of the walls in the first story and the sudden change in stiffness from the first story to the second made the structure irregular and nonsymmetric in the north-south direction. The behavior in this direction was of relevance to the present study.

The specified 28-day compressive strength of concrete for walls, slabs, beams, and joists was 4000 psi and for columns was 5000 psi. Grade 40 reinforcement was used. Testing of steel and concrete samples taken from the building has shown that the actual strengths for both the steel and concrete are considerably higher than the specified values [38]. Detailed information on member dimensions and reinforcement is presented in Ref. 10.

The foundation of the building consisted of single footings supported on pile groups of six, nine and twelve piles. The pile lengths were 40 to 45 ft. The soil consisted of primarily medium to stiff silty clay.

(b) Instrumentation - The ICSB was instrumented by 13 accelerometers connected to a 13-channel data recording system. In addition, three instruments were placed at a distance of approximately 100 yards from the building. All the instruments were triggered by the earthquake of October 1979. The location of the accelerometers are shown in Fig. 2.2. The response in the north-south direction was measured at west, middle, and east parts of the second floor and the roof, in addition to the west and east parts at the ground floor.

2.3 Hypothetical Frame Wall Structure

The hypothetical structure used in the study was a six-story unsymmetric frame-wall structure called SUFW (Fig. 2.3). Reasonable values were assumed for dimensions and reinforcement (Fig. 2.4). The slab thickness was taken equal to nine inches. The assumed material properties are listed in Table 2.1.

The dead load at each floor was assumed to consist of the weight of slab, wall, and columns associated to that floor and was found to be equal to 120 kips. The beam reinforcement was assumed to be the same at the top and bottom to facilitate the input data preparation for the computer analysis to be followed. Had the top and bottom bars been different (usually the case in ordinary reinforced concrete buildings), the negative and positive moment-curvature values would have to be averaged.

With the assumed location of the wall and the frame, significant eccentricity (about forty percent of the east-west dimension) was accomplished for loadings in the north-south direction. Dimensions and reinforcements are comparable to those of the ICSB so that the behavior would be similar to the type of behavior experienced by the ICSB. Unlike the ICSB which was actually subjected to three simultaneous components of an earthquake, the SUFW was assumed to be loaded only in the north-south direction.

CHAPTER THREE

ANALYTICAL MODELS

3.1 Introductory Remarks

Two models for nonlinear seismic analysis were used in this study. One was a multi-degree-of-freedom (MDOF) model, and the other was a single-degree-of-freedom (SDOF) model (Q-Model). Original versions of these models were developed previously and are described elsewhere [35,36]. Only planar structures or structures which may be represented as planar systems can be analyzed by the original versions. As part of the study presented in this report, minor modifications were made in the MDOF model to be able to obtain an approximate calculation of the response of unsymmetrical structures with flexible diaphragms.

A new equivalent SDOF model was developed for the analysis of unsymmetric structures. The model, called Q-Model3, uses the same principles as those used in the original version of the Q-Model.

This chapter describes the modifications made in the MDOF model as well as the methodology used in the Q-Model3.

3.2 MDOF Modelling

The original version of the MDOF model used in the present study was developed for nonlinear response history analysis of planar systems. Under unidirectional seismic loads, symmetric structures with reasonably rigid diaphragms experience the same lateral displacement at different locations of the same floor and may be idealized as a series of coupled frames (Fig. 3.1). The response of unsymmetric structures, however, is associated with rotation of the floors with respect to a vertical axis, and different displacements within the same floor and some out-of-plane motion are expected (Fig. 3.2). The extent of rotation, in part, depends on the flexibility of the floors. In extremely "soft" floors, the rotation is very small because individual frames (and walls) displace independently and no interaction can occur among them. On the other hand, very "stiff" diaphragms tend to couple different frames and a rigid-body displacement of the floor takes place. The actual behavior of diaphragms falls in between these two extremes. Even reinforced concrete floors, customarily treated as rigid diaphragms, deform under severe earthquakes [29].

Response of unsymmetric structures with eccentric centers of mass and stiffness includes out-of-plane deformations, even though the earthquake or loading acts in one direction. Such structures require a three-dimensional analysis which takes into account the biaxial bending of columns, and deformations of floor systems. The development of such a model was beyond the scopes of this study. The available analytical models either incorporate element hysteresis models which are not necessarily the most "realistic", or they treat the floor systems as rigid elements [13]. It was, therefore, decided to modify the planar analytical model described in Ref. 25 and the related computer program, LARZ [34].

Because out-of-plane displacements were not of interest in this study, no attempt was made to calculate them. As a result, the model was set to compute displacements only in the direction of loading.

The model, in its old form, was capable of calculating response of several frames assuming that the frames are uncoupled and displacements at the same level of different frames are different (Fig. 3.2). This assumption is valid only for structures with extremely flexible floors. To account for the coupling effect of semi-rigid floors, an elastic flexible diaphragm element was added to the model. The horizontal stiffness of this element depends on flexural and shear stiffnesses of the floor in addition to torsional stiffness of the supporting vertical elements. The element may be idealized as shown in Fig. 3.4. The stiffness of the end springs is equal to the torsional stiffness of frames or walls at each end of the floor. Each element represents the floor system in between any two adjacent frames or walls.

To avoid unnecessary complication of the computer code, the program was set so that the stiffness of the floors is part of the input to the program. The updated user's manual for the program is presented in Appendix A and B in Ref. 34.

3.3 SDOF Modelling

(a) The Q-Model - The dynamic response of MDOF systems may be determined from the analysis of an equivalent SDOF model with a generalized mass, stiffness and damping [6,8]. Energy principles and an assumed displaced shape is used to calculate the properties of the equivalent system. Idealization of nonlinear systems by an equivalent system is more complicated because the assumed displaced shape needs to take into account the effect of nonlinearity. In addition, a hysteresis model is needed which can estimate the variation of overall stiffness of the structure reasonably well. An important point to note is that the representation of a MDOF system by an equivalent SDOF system is an attempt to simplify the anlaysis, and any method used to determine the displaced shape and stiffness variations would have to be relatively simple to be compatible with the idea of using the SDOF system.

The Q-Model is a nonlinear equivalent SDOF model which has shown to result in a reasonbale estimate of the displacement response of planar systems [21,36]. The model is described in detail in Ref. 34-36.

(b) The Q-Model3 - A method, incorporating the basic principles of the Q-Model, was developed for the analysis of reinforced concrete structures with eccentric centers of mass and stiffness. The hysteresis model used in the new method is the same as that used in the original version of the Q-Model. The new version is considered a preliminary model and, as it is discussed in the following section, further refinement in the model is necessary.

The displaced shape in the original version of the Q-Model was determined from a nonlinear static analysis of the multistory structure assuming that the structure is subjected to monotonically increasing lateral loads applied at floor levels. The loads were assumed to be proportional to the mass and height from the base, leading to a triangular load distribution for structures with the same mass at different floors. This distribution was chosen because it is simple and is similar to the load distribution recommended by the Uniform Building Code [15]. Because the analysis was carried out for planar systems, a tributary mass corresponding to each floor was used.

In unsymmetric structures, the distribution of seismic forces among different frames depends, to a great extent, on the relative stiffness and strength of the frames. Stiffer frames tend to absorb larger forces. As some of the elements yield, stiffness changes and, consequently, the distribution changes.

In the Q-Model3, it is assumed that lateral loading is such that the apparent yielding of different frames occurs simultaneously. The distribution of forces within each frame is assumed to be identical to that in the Q-Model. The assumption of simultaneous yielding may not be completely valid but it is perhaps reasonable. As the loads act on the structure, some of the frames yield, while others are still elastic. Beyond this stage, the yielded frames stop absorbing any significant additional forces and the new loads are primarily transferred to the unyielding frames. As the loading continues, all the frames reach the apparent yielding stage. It should be noted that this approach assumes that structural elements are capable of developing large ductilities.

(i) Distribution of Loads - To determine the load distribution among different frames, a limit analysis is carried out assuming that each frame is subjected to horizontal loads proportional to floor mass and height from the base. The magnitude of the mass is not important, rather, it is the value of the mass at each floor relative to masses at other floors which enters the computation. The behavior of structural elements are considered elasto-plastic for this stage. Several collapse mechanisms are considered for each frame and the minimum collapse load is determined [28]. The collapse loads for different frames establish the relative values of lateral loads to be applied to the structure at a later stage.

(ii) Nonlinear Static Analysis - The structure is analyzed for a series of monotonically increasing lateral loads distributed as described in the previous sections. A MDOF nonlinear static analysis, with appropriate consideration of floor flexibilities, is carried out and horizontal floor displacements at different frames are computed. The new version of a computer code, called LARZ2, may be used for this purpose [34]. The base moment for the structure is plotted in terms of displacements at a floor with the largest displacements (top floor of the "softest" frame). The resulting curve is idealized by a bilinear curve as shown in Fig. 3.5. Engineering judgement is used to idealize the curve. A study reported in Ref. 34 has shown that the results are insensitive to the location of the break point, as long as the point is chosen at a reasonable position which is representative of the apparent yielding of the structure. The bilinear curve, called the primary (skeleton) curve, is assumed to represent the force-deformation characteristics of the structure during the initial loading.

(iii) Deflected Shape - Associated to each point of the calculated moment-displacement curve, a deflected shape may be found. Previous studies on the static response of planar systems showed that no significant variation in the shape occurred beyond yielding [35]. In the original version of the Q-Model, therefore, the shape corresponding to the apparent yield point was used. In the course of developing the Q-Model3, preliminary studies of SUFW (see Sec. 2.3) revealed that deflected shapes vary significantly as the apparent yield point has passed. The quantitative identification of reasons for the variation was beyond the scope of this study. Qualitatively, however, the changes may be attributed to the fact that, at the apparent yield point in the moment-displacement curve, some of the frames may develop only minor nonlinearities. As the apparent yield point is passed, nonlinearity in these frames may become significant causing a large but gradual change in the deformed shape.

The Q-Model3 incorporates a predetermined structure ductility factor to specify the point on the moment-displacement curve at which the deflected shape is chosen. The structure ductility factor is defined as the ratio of maximum displacement and the apparent yield displacement (displacement at the break point of the primary curve). For earthquakes likely to cause nonlinear deformations in the building, the ductility value may be taken from two to six depending on the intensity of the earthquake and strength of the structure. Because the analysis based on the Q-Model3 is simple, a trial and error method may be used for the cases with no prior knowledge of the expected ductility.

The deflections are normalized with respect to the displacement at the floor with the largest displacement to obtain the deflected shape (surface) to be used at later stages of the analysis.

(iv) Equivalent Height - The mass of the equivalent SDOF system is assumed to be positioned at the weighted centroid of the deflected surface determined from (see Fig. 3.6)

$$X_e = \sum_{i=1}^{N} m_i \phi_i x_i / \sum_{i=1}^{N} m_i \phi_i \qquad (3.1)$$

$$Y_e = \sum_{i=1}^{N} m_i \phi_i y_i / \sum_{i=1}^{N} m_i \phi_i \qquad (3.2)$$

in which

X_e = abscissa of centroid,

Y_e = ordinate of centroid (equivalent height),

N = summation of the number of floors at all frames (walls),

m_i = mass at level i,

ϕ_i = normalized displacement at level i,

y_i = height of level i measured from the base.

Note that N is the total number of floors summed over all the frames and walls. For SUFW (see Sec. 2.3), N = 12 even though the structure is a six-story building. A tributary mass for each level of each frame may be used.

(v) Equivalent Mass - The equivalent (generalized) mass is calculated based on energy principles, and is simplified in the following form [6,36].

$$M_e = \left(\sum_{i=1}^{N} m_i \phi_i^2 / \sum_{i=1}^{N} m_i \phi_i \right) (M_t) \tag{3.3}$$

in which

M_e = equivalent mass, and

M_t = total mass of the structure.

(Other parameters are described following Eq. 3.2)

This relationship is the same as that used in the original formulation of the Q-Model.

(vi) Normalized Displacement at Equivalent Height - The deflection at the centroid is determined from a linear double interpolation of the normalized displacements at the adjacent floors and frames (Fig. 3.6). This value is used to modify the displacements in the idealized bilinear curve shown in Fig. 3.5, to obtain a curve which represent the base moment of the structure in terms of the displacement at the centroid. The Q-hyst model is used to idealize the hysteretic behavior of the moment-displacement relationship [36].

(vii) Equation of Motion - The equation of motion has the same form as that used in the original version of the Q-Model. The equation is written for a SDOF oscillator with mass located at the centroid of the deflected surface:

$$M_e \ddot{x} + C\dot{x} + K_e x = - M_t \ddot{X}_g \tag{3.4}$$

in which

C = damping coefficient,

K_e = stiffness calculated from hysteresis model,

$\ddot{X}_g$ = base acceleration,

$\ddot{x}$, $\dot{x}$, and x = relative acceleration, velocity, and displacement at the centroid (relative to base).

The viscous damping component in the equation accounts for energy dissipation during elastic stages. The damping coefficient is found using routine methods [8] and based on the pre-yielding frequency of the equivalent SDOF system. The choice of the damping factor is left to the analyst. However, because the nonlinear response is relatively insensitive to the viscous damping [34], the value used for this factor is not critical. A damping factor ranging from two to five percent appears to be reasonable.

In applying Eq. 3.4, it should be noted that the masses used in the left- and right-hand sides are not the same.

Equation 3.4 is written in incremental form and integrated based on one of the numerical techniques for integration. Newmark's β method [23] has been used in previous studies on the Q-Model and has led to very satisfactory results. The displacement history obtained from Eq. 3.4 is normalized according to the displaced surface to calculate the displacements at any desired floor of different frames.

CHAPTER FOUR

RESPONSE OF THE IMPERIAL COUNTY BUILDING

4.1 Introductory Remarks

This chapter presents a short discussion on the measured response of the Imperial County Services Building (ICSB) followed by the results of nonlinear modeling of the response in the north-south direction. Detailed information on the behavior of the building during the earthquake of October 1979 is provided in Ref. 29 and 45.

The modeling methods described in Sec. 4.2 and 4.3 are very approximate, and are performed to determine whether the north-south response could be predicted based on the available nonlinear models for planar structures. Estimated response in the east-west direction, in which the building was almost symmetric, is discussed in Ref. 37 and will not be repeated in this report. In all of the discussions on ICSB, the first four seconds of the original record [14] are ignored because the amplitudes were relatively small.

4.2 Measured Response

The building was instrumented with a 13-channel acceleration recording system (Fig. 2.2). All the instruments were triggered by the earthquake of October 1979. The measured accelerations were processed and integrated to obtain the velocity and displacement histories [14]. The term "measured displacements" used in this report refers to the doubly-integrated measured acceleration.

The building experienced significant accelerations and displacements during the earthquake. Yielding of steel and crushing of concrete at the bottom (over the ground floor slab) of east columns resulted in a pronounced settlement of the building at the east side. It is reported that axial forces caused by overturning moment had a substantial contribution to the failure of the columns [45]. Shear cracks were developed in some of the first-story columns. There was also indications of yielding of steel in some of the second floor beams. No visible damage was reported in the shear walls. The building did not collapse during the earthquake, but had to be demolished at a later time.

The extent of damage to the building was an indication that the response was nonlinear. The degree of nonlinearity, however, was different for the east-west and north-south directions. The response in the east-west direction was controlled by frame action, and the local failure of the east columns had a more pronounced contribution to the nonlinearity than it did to the response in the north-south direction, in which the lateral resistance was primarily provided by the walls. Nevertheless, the north-south response in the east side of the building was affected by the column failures.

The ground floor acceleration experienced by the building is of interest to analytical investigations. The north-south acceleration was measured at two locations of the ground floor (Fig. 2.2), one at the west edge (Trace 10) and the other at approximately 32 feet from the east edge (Trace 11). Figure 4.1 shows these traces. It can be seen that, except for slight differences in a few of the peaks, the two traces were almost identical. To examine the two records in more detail, the acceleration, velocity, and displacement spectra for five percent damping were obtained for the two records and superimposed (Fig. 4.2). Only minor differences were observed between the results for Traces 10 and 11. The fundamental period of ICSB in the north-south direction was approximately 0.4-0.5 sec [29]. In this range, the displacement and velocity responses for the two records were the same, while the acceleration response varied slightly. It is apparent that the ground floor did not experience any significant rotation with respect to vertical axis. Because the distance from the top of the ground floor and top of pile caps was only 2'-2", it is reasonable to assume that the acceleration at the pile caps was very similar to the ground floor acceleration.

More detailed discussions of the behavior of ICSB during the earthquake may be found in Ref. 29 and 45.

4.3 Multi-Degree-of-Freedom Response

Structures are three-dimensional systems responding accordingly when subjected to earthquakes. The customary method of considering the behavior in one direction at a time is not meant and does not represent the behavior in three dimensions [12]. To be absolutely realistic, a space-frame analysis should be carried out with three translation and three rotation components of the earthquake. Such an analysis should allow for nonlinearity due to shear, flexure, and axial load. Effects of dead load on the internal forces, and effects of non-structural elements on lateral stiffness need to be included. The analysis should also allow for floor deformations, even though, traditionally concrete floors are treated as rigid diaphragms. The need to include floor flexibilities is suggested by the relatively large deformations of the second floor in ICSB during the 1979 earthquake [29]. An analytical model with these capabilities and the capacity required for ICSB would be extremely complicated and lengthy. Such a model has not been developed.

An alternative method, widely used in research, is to consider the response in one direction at a time, treating the structure as a planar system with due consideration of the nonlinear behavior. This approach assumes that all the points on each floor experience the same lateral displacements, and lack of symmetry is, in effect, ignored. Typically, these models assume no shear strength deterioration in the view of the fact that, buildings properly designed to resist earthquakes, are expected to develop only flexural yielding. The results from these models should not be expected to show the "true" response because of all the simplifications incorporated in them. In the absence of the sophisticated models explained above, the question to be addressed is that whether the planar models can produce an estimate of response which has the same general characteristics of the true response. The usefulness of the planar models should not be overlooked. The results from these models can provide an indication of adequacy of the structure in resisting ground motion forces.

Against the above background, the response of ICSB was calculated using a planar model with Takeda hysteresis model [43] used for joint behavior idealization. The results of this analysis in relation to the measured response of the structure are discussed in the following sections.

(a) Material Properties - Results of the nonlinear analysis are sensitive to the assumed material properties, especially the yield stress of steel and compressive strength of concrete. The specified material properties are usually inadequate to determine the yield strength of structural members. There is no simple method available to estimate the actual strength of concrete and steel in the structure based on the specified values. Because ICSB had to be demolished after the earthquake, it was possible to obtain samples of concrete and steel and perform standard tests in the laboratory. This task is being carried out by Selna at the University of California at Los Angeles, and the data are expected to become available in September 1982 [38].

In the analysis performed for this study, steel yield stress was assumed to be equal to 50 ksi to account for the fact that the specified yield stress (40 ksi) defines the minimum yield stress, and the actual value is larger. The compessive strengths of concrete were assumed to be the same as the specified values.

A post-yielding slope of 870 ksi (three percent of modulus of elasticity) was assumed in the stress-strain diagram for steel. The concrete was assumed to reach its compressive strength at strain of 0.002, and crush at 0.003.

(b) Idealization of Structure - The structural system in the north-south direction consists of several frames and walls. To idealize the structure, all the frames were combined into one frame and all the walls were combined into one wall (Fig. 4.3). The first-story wall idealizes all the four walls in the first story of the building. The "beams" in the frame represent the corresponding slab and joists. Due to lack of information on the effectiveness of slab-joist systems in resisting lateral forces, the entire width of the slab was assumed to contribute to stiffness and strength.

The effect of soil-structure interaction was neglected in the analysis, and the structure was assumed to be fixed at the pile caps. The contribution of nonstructural elements to lateral stiffness was also ignored. It was assumed that the "beams" in the idealized frame are initially cracked due to the service load effects.

(c) Base Acceleration - With soil-structure effect ignored, it was reasonable to use the ground floor acceleration as the input motion. Because there were minor differences between some of the peak values of TR10 and TR11, the two records were averaged (Fig. 4.4). The first four seconds of the original record [14] had very small amplitudes, and were ignored in the analysis.

(d) Measured and Calculated Responses - Shown in Fig. 4.4 are the calculated and measured north-south response of ICSB in the west building. The calculated results are shown only for those floors which were instrumented in the north-south direction.

In comparing the calculated and measured displacement histories, it is evident that the correlation was unsatisfactory in terms of time and magnitude of peaks, period content, and waveforms. The same is true when the analytical results are compared with the measured curves at the middle and west edge (Fig. 4.5). The measured responses include the three-dimensional effects of the earthquake, while the calculated response has the effect of only the north-south component. It would be expected that the calculated response should yield the lower bound value. Yet, the calculated maximum displacement is larger than the measured value and, judging based on the effective periods of the waves, the effective stiffness is smaller than the actual stiffness of the building. The discrepancy can be attributed to the fact that the assumed material properties were smaller than the actual value [38] and that the effect of partition walls was ignored.

The correlation between the calculated and measured roof acceleration histories is also unsatisfactory. During the first seven seconds, the part with large peaks, the analytical results generally underestimated the peaks; a further indication that effective stiffness was underestimated by the model.

The calculated acceleration response at the second floor showed excellent agreement with the measured response. Comparison of the calculated second floor acceleration with the ground floor acceleration shows that, because the first story of the structure was very stiff, the two responses are identical. This observation is not surprising because there was a relatively stiff wall at the first story near the west end. In the analytical model, apparently the stiffness of the first story was idealized reasonably well, and, as a result, the very close agreement on the second floor was seen. It should be noted that in the analytical results, no significant degradation of stiffness was seen at the first story despite yielding of the east columns caused by the earthquake. Because the frames were combined, the analytical model was unable to single-out the yielding of these columns.

4.4 Single-Degree-of-Freedom Response

The unsatisfactory performance of the MDOF model was an indication that an equivalent SDOF model would not lead to acceptable results, because the SDOF model is, in effect, a simplified representation of the MDOF model. Nevertheless, the original version of the Q-Model [36] was used to estimate the displacement history and to determine the extent of shortcomings of the Q-Model in predicting the response of unsymmetric structures.

The idealized system explained in Sec. 4.3.b was substituted by its equivalent SDOF model using the material properties and the base acceleration explained in Sec. 4.3. The measured and calculated displacement histories are shown in Fig. 4.6. It can be seen that analytical results showed poor correlation with the measured curves during the first eight seconds. The lack of agreement was evident in peaks, period content, and waveforms. During the last seven seconds when small amplitudes occurred, the correlation was close. This is attributed to the possibility that the model estimated the average stiffness close to the effective stiffness of the structure. In view of the substantial disagreement in the first eight seconds, the good correlation in the low-amplitude part of the response is not considered to be significant.

4.5 Discussion

The above observations indicate that the idealization of a structure, with eccentric centers of mass and stiffness, by a planar model is inadequate to provide descriptive information about the seismic behavior of the structure. The analytical models, both the MDOF and the SDOF models, did lead to a qualitative estimate of the response. The maximum displacements, although overestimated, could provide information about the order of magnitude of the expected ductility demand which can be useful in design. It can be concluded, therefore, that planar representation of structures, at best, is a very approximate process.

CHAPTER FIVE

RESPONSE OF THE HYPOTHETICAL STRUCTURE

5.1 Introductory Remarks

The Q-Model3, described in Sec. 3.3, could not be used for ICSB because the computer model needed for nonlinear static analysis of the complete structure was not available. To evaluate the Q-Model3, a hypothetical structure (SUFW) was used which had the basic characteristics of ICSB, but it was considerably smaller (Sec. 2.3). This chapter describes the application of the model in the analysis of SUFW. The results are examined against those obtained from the MDOF model described in Sec. 3.2.

5.2 MDOF Results

The mode shapes and vibration periods of SUFW were found using program SAPIV [4]. The mode shapes in the north direction of the structure for the first three modes are shown in Fig. 5.1. The periods for these modes were found equal to 0.72, 0.25, and 0.20 seconds, respectively.

Structure SUFW was analyzed for the first 15 seconds of El Centro 1940 NS record using the modified MDOF model described in Chapter Three. The analysis was carried out for two earthquake intensities, one with a maximum acceleration of 0.3g (run 1) and the other with 0.5g (run 2). The structure was assumed to be uncracked prior to each earthquake run. The two values were used to examine the effect of different degrees of nonlinearity.

Member flexural properties were calculated based on the material properties listed in Table 2.1 using the method described in Ref. 28. The resulting moment curvature values are shown in Table 5.1. Bond slip deformations were taken into account using the formulation presented in Ref. 41 (Table 5.2). Bond slip rotations at cracking were ignored. The values in the table represent rotation corresponding to the unit length element cantilever length with an assumed inflection point at the center for the beams and columns and at the roof for the wall. Diaphragm effective stiffnesses were estimated from a three-dimensional static analysis using program SAPIV [4], although they could also be found using the model shown in Fig. 3.4.

The displacement histories were found and are shown in Fig. 5.2 and 5.3. In each case, the frame and wall displacements at different floors were separately grouped. To compare the frame response relative to the wall, the top-floor responses were superimposed. It can be seen that, in both cases, different floors were generally in phase, indicating that the response was dominated by an apparent fundamental mode. No major yielding was developed during the first run, but many of the beams and column bases experienced moments well beyond the cracking moment. During the second run, however, all the beams either yielded or developed moments which were very close to their yield moment. The wall base yielded in both runs.

5.3 Q-Model3 Results

(a) Variation in Deflected Shape - A key issue in the success of an equivalent SDOF model is whether a single deflected shape can be found which is representative of the deformed shape of the structure during the earthquake. The response obtained in the MDOF analysis was studied to determine if such a shape could be found. The displacements at different instances near major peaks where normalized with respect to the roof displacement of the frame. It can be seen in Fig. 5.2 and

5.3 that the peaks at different floors were not always simultaneous, but, because the times of their occurrence were very close (within $\pm$ 0.1 sec.), they were assumed to occur at the same time. The normalized values are listed in Tables 5.3 and 5.4, and plotted in Fig. 5.4 and 5.5. The boken lines in the figures show the plane of the structure in the east-west direction, and the solid lines show the displacements normal to the plane, which are in effect the north-south displacements.

The first column of data in the tables corresponds to the case before any major cracking was developed. It can be seen that displacements of the wall at each floor were considerably smaller than the frame displacements. The ratios became notably larger when major cracking and some yielding occurred. Between T=2.7 to 12.5 sec of the first run, the ratio of roof displacement at the wall to that of the frame varied drastically and ranged from 0.49 to 0.79. This can be quantitatively observed by comparing columns (b) and (d) in Table 5.3. The extent of variation in the shapes during run 2 was considerably smaller. The relatively large variations in the shapes in run 1 can be attribtued to the sensitivity of the response to torsional effects. The torsional effects in run 2 became less significant after t=2 sec, because the wall developed major cracking and yielding and, as a result became "softer" reducing the eccentricity between the center of mass and stiffness.

The above observations suggest that the equivalent SDOF approach is likely to be more successful for structures with a higher degree of nonlinearity.

(b) Deflected Shape Based on the Static Analysis - The deflected shape in Q-Model3 is found from a nonlinear static analysis of the structure subjected to a series of lateral loads with a distribution explained in Sec. 3.3. To determine the loads, a limit analysis of the wall and frame was carried out assuming that they were subjected to lateral loads proportional to the mass and height of each floor from the base. Because all the floors had the same mass, the loading was triangular. The collapse load for the wall was the load which caused yielding of the wall base. For the frame, the collapse load was reached for the mechanism shown in Fig. 5.6. These loads established the relative values of loads for frame and the wall used in the static nonlinear analysis.

The static analysis was carried out using program LARZ2 [34] based on the flexural properties explained in the previous section. Bond slip rotations were ignored at this stage, but were taken into account implicitly in the idealization of the primary curve by keeping the bilinear curve inside the calculated curve (Fig. 5.7). The calculated curve was constructed by applying the lateral loads in fifty load increments. The total load was sufficiently large to cause significant post "yield" deformations.

To determine the extent of changes in the deformed shape as deformations pass the apparent yield point (the break point in the bilinear curve), deflection at this point and at two other points beyond yielding were normalized with respect to the displacement at the frame roof (the floor with the largest displacement). The values are identified by loading numbers encircled in Fig. 5.7 and are listed in Table 5.5. It is evident that the changes in the shapes of the frame relative to wall were substantial. The roof displacement of the wall at the yield point was 43 percent of the frame, while at load 40, when the displacement was five times the yield displacement (displacement ductility of 5), the ratio increased to 96 percent. The changes in the shape within the frame or wall were minor, an observation which verifies the assumption made in the original version of the Q-Model.

(c) Q-Model3 Properties - Corresponding to each deflected shape, the properties of an equivalent SDOF model was determined using the formulation presented in Chapter Three. The mass at each floor was assumed to be equally distributed between the wall and the frame. Each SDOF model was identified by the load number at which the displaced shape was found. The properties are listed in Table 5.6. It can be seen that the only parameter with significant variation was X_e. It should be noted, however, that the only effect of X_e is on the abscissa of the centroid which is used only to determine the normalized deflection at that point. Comparison of the normalized deflection for different cases shows that the changes in this parameter were very small. Other properties of the SDOF model, such as the equivalent mass, height (Y_e), and moment-displacement curve were insensitive to the deflected shape.

(d) Results for Run 1 - The Q-Model3 was used to analyze SDOF systems Q28, Q34, and Q40 subjected to the El Centro 1940 NS record with a maximum acceleration of 0.3g and 0.5g, resulting in six sets of results. A damping ratio of five percent was used. The response at the roof of the wall and the frame were considered to be adequate to demonstrate the quality of the results. To evaluate the responses, they were compared with the responses obtained in the MDOF analysis.

Figure 5.8 shows the responses for the first earthquake run. It can be seen that in terms of waveforms and frequency content the correlation between the SDOF and MDCF results was reasonably good for all three cases. With respect to the peaks in the frame responses, again the correlation may be considered satisfactory. The peaks in the wall responses, however, were underestimated in Q28, but were overestimated in Q34 and, at a higher degree, in Q40. It is clear that the changes in the deflected shape used in the Q-Model3 analyses has led to the differences.

According to the Q-Model3 (Sec. 3.3), the assumed displaced shape using the output from the static analysis, based on a preassigned displacement ductility ranging, say, from 2 to 6. The earthquake in the first run had a maximum acceleration of 0.3g and could be classified as a moderate earthquake, leading to a relatively small ductility demand, say, two. The displacement ductility for Q28 was unity (the shape was taken at the break point) and for Q34 was 2.7. It appears that, by following the Q-Model3 method, a displaced shape between those of Q28 and Q34 would have been chosen leading to a more acceptable agreement between the peaks in the SDOF and MDOF results.

Displacements of the structure at time of maximum frame roof response are shown in Fig. 5.9. Disagreement between the wall displacements obtained from the SDOF and MDOF model is evident.

(e) Results for Run 2 - The responses for the second earthquake run are shown in Fig. 5.10. Similar to what was observed in run 1, the correlation between the SDOF and MDOF results is very good in terms of response shapes and frequencies. The peaks in the wall response in Q28 were underestimated by the Q-Model3. The correlation between the peaks in other cases, however, was very close in most instances.

The earthquake maximum acceleration for run 2 was 0.5g, qualifying it as a strong earthquake. For such an earthquake, a ductility demand of 4 to 5 would be anticipated. It can be seen in Fig. 5.10 that both the Q34 and Q40 data (covering a ductility range of 2.7 to 5) led to responses in very good agreement with the MDOF results suggesting that any reasonable estimate of ductility demand would have yielded satisfactory results. The success of the Q-Model3 can be also verified in comparing the displaced shapes based on the Q-Model3 with those from the MDOF analysis (Fig. 5.11).

5.4 Discussion

The above observations show that the Q-Model3 was successful in estimating the displacement history of an unsymmetric torsionally-coupled structure. The satisfactory performance of the model was evident in all important characteristics of the response in cases with both the limited and severe nonlinearity.

Many researchers would agree that by controlling the lateral displacements of well designed and constructed structures, it is possible to keep the internal forces and deformations below the critical limits. The satisfactory performance of the Q-Model3 for SUFW suggests that the model has the potential to be used in conjunction with this design philosophy. Many more structures with different configurations need to be studied for various earthquakes before a conclusive statement about the performance of the Q-Model3 may be made. The promising performance of the model for SUFW, however, makes the model very worthwhile for further detailed studies.

CHAPTER SIX

SUMMARY AND CONCLUSIONS

6.1 Summary

The research presented in this report was a preliminary study to determine (1) the possibility of estimating the north-south response of the Imperial County Services Building (ICSB) using planar models, and (2) whether an equivalent single-degree-of-freedom (SDOF) nonlinear model may be developed to estimate the displacement history of structures with eccentric centers of mass and stiffness.

ICSB was a six-story reinforced concrete building which was severely damaged during the earthquake of October 1979. The lateral resistance of the building in the east-west direction was provided by a frame action while the resistance in the north-south direction was due to four shear walls in the first story, built in an unsymmetric pattern, and two exterior shear walls in the second through sixth story. The building was instrumented with thirteen channels of acceleration recording systems which were all triggered by the earthquake [14]. In the study reported here, the building was analyzed in the north-south direction, using SDOF and MDOF planar nonlinear models taking into account only the north-south component of base motion. Study of the two acceleration records measured at different locations of the ground floor showed that no significant torsional motion (with respect to the vertical axis) was present at the base. The structure was assumed to be fixed at the base. By idealizing the structure as a planar system an important feature of the structure, namely, the lack of symmetry of stiffness in the first story was eliminated.

In the second part of this study, a nonlinear SDOF model

(Q-Model3) was developed for displacement history analysis of structures with torsional coupling (Sec. 3.3). The Q-Model3 incorporates the basic principles used in the original version of the Q-Model [36]. A nonlinear static analysis is performed to obtain the initial stiffness variations (primary curve) of the equivalent SDOF system. The deflected shape of the structure is determined based on the results of the static analysis and an estimate of the expected displacement ductility, defined as the ratio of maximum displacement to the apparent yield displacement of the structure. The Q-Model3 was used to analyze the response of a hypothetical, six-story, torsionally coupled, frame-wall structure (SUFW). The analysis was carried out assuming that SUFW was subjected to El Centro NS, 1940, with maximum acceleration normalized to 0.3g in one set of analysis, and to 0.5g in another. Different deformed shapes were used to determine their influence on the response. The Q-Model results were evaluated by comparing the results with results obtained from a MDOF analysis.

6.2 Observations

The following important points were noted in this study.

1. The idealization of ICSB by a planar system led to responses which generally showed poor correlation with the measured results. This was noted in both the MDOF and SDOF results.

2. The calculated second floor acceleration was in a very good agreement with the measured results at the west end, despite the fact that lack of symmetry of the structure and three-dimensional effect of the earthquake were ignored. The close correlation can be explained as follows. The elastic stiffness in the first story was relatively large and, as a result, the second floor acceleration was very similar to the

ground floor acceleration. The analytical model, apparently, idealized the stiffness reasonably well, and because the response at the west side was not significantly affected by the torsional coupling, the model led to a satisfactory estimate of the response.

3. For SUFW, the Q-Model3 estimated the displacement histories reasonably well. Results obtained from the Q-Model3 were in good agreement with MDOF results in terms of amplitudes, frequency contents, waveforms, and deformed shapes.

4. The choice of expected ductility used in Q-Model3 does not need to be exact. A reasonable estimate of ductility demand was found to be adequate to arrive at an acceptable deflected shape. Because the model is very simple, even without a good estimate of ductility, the response may be determined through a trial-and-error method.

6.3 Conclusions

1. Models for the nonlinear seismic analysis of planar structures are not meant to, and do not, represent the "true" three-dimensional response of structures, especially if centers of mass and stiffness do not coincide. The study of ICSB presented in this report showed that planar systems were able to produce only a qualitative estimate of the response and the calculated results were generally different from the measured values. This was in agreement with observations made in previous investigations [12,24].

2. When possible, the actual, rather than the specified, material properties need to be used in the nonlinear analysis. At design stage, when the actual properties are not available, the best estimates of the properties supported by statistical data should be used.

3. The success of the Q-Model3 in calculating the response of

SUFW is an indication that the nonlinear response of torsionally coupled structures may be found by using an equivalent SDOF model. Considerably more research, experimental and analytical, should be carried out before a conclusive statement about the Q-Model3 can be made. The performance of the model for SUFW shows that the model is worthwhile for further research.

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Table 2.1 - Material Properties for SUFW

Concrete	
Compressive Strength (f'_c)	4 ksi
Modulus of Elasticity	3,600 ksi
Strain at f'_c	0.002
Ultimate Strain	0.003
Tensile Strength	0.47 ksi
Steel	
Yield Stress	60 ksi
Modulus of Elasticity	29,000 ksi
Strain-Hardening Slope	2,900 ksi
Strain at Start of Strain Hardening = Yield Strain	0.00207

Table 5.1 - Flexural Properties of Elements in SUFW

Element	Mom. of Inertia in^4	Crack. Mom. (k-in)	Yield Mom. (k-in)	Post-Yield Mom. (k-in)	Yield Curv. (1/in)	Post-Yield Curv. (1/in)
Beams (level 5-Roof)	10,667	506	2,256	2,704	0.000,166	0.000,820
Beams (level 2-4)	10,667	506	3,667	4,361	0.000,179	0.000,988
Columns (Story 4-6)	27,648	1,330	3,674	4,957	0.000,134	0.000,628
Columns (Story 1-3)	27,648	1,690	6,000	6,676	0.000,170	0.000,558
Wall	19,840,000	77,490	80,000	97,400	0.000,005,5	0.000,083,8

Note: Post-yielding quantities represent values for an arbitrary point beyond yield point.

Table 5.2 - Bond Slip Rotation in Elements of SUFW

Element	Rotation at Yield Point	Rotation at Post-Yield Point
Beams (level 5-roof)	11.2×10^{-6}	16.1×10^{-6}
Beams (level 2-4)	18.6×10^{-6}	26.3×10^{-6}
Columns (Story 4-6)	16.9×10^{-6}	30.7×10^{-6}
Columns (Story 1-3)	19.7×10^{-6}	24.4×10^{-6}
Wall	0.194×10^{-6}	0.287×10^{-6}

Table 5.3 - Normalized Displacements During Run 1

Approx. Time (Sec.)		(a) 2.3	(b) 2.7	(c) 4.6	(d) 6.	(e) 12.5
	Floor					
	Roof	1.0	1.0	1.0	1.0	1.0
	6	0.84	0.97	0.87	0.79	0.97
Frame	5	0.69	0.86	0.71	0.61	0.84
	4	0.50	0.69	0.54	0.46	0.61
	3	0.29	0.47	0.34	0.31	0.37
	2	0.13	0.23	0.15	0.15	0.12
	Roof	0.31	0.79	0.57	0.49	0.66
	6	0.24	0.66	0.47	0.40	0.55
	5	0.19	0.52	0.37	0.32	0.43
Wall	4	0.14	0.39	0.28	0.23	0.34
	3	0.09	0.26	0.15	0.16	0.18
	2	0.05	0.14	0.10	0.08	0.12

Table 5.4 - Normalized Displacements During Run 2

Approx. Time (Sec.)	Floor	(a) 2	(b) 2.5	(c) 3	(d) 6	(e) 12
Frame	Roof	1.0	1.0	1.0	1.0	1.0
	6	0.85	0.84	0.84	0.82	0.76
	5	0.69	0.64	0.66	0.62	0.54
	4	0.51	0.48	0.44	0.49	0.41
	3	0.30	0.28	0.26	0.34	0.23
	2	0.12	0.09	0.10	0.16	0.14
Wall	Roof	0.25	0.72	0.8	0.82	0.61
	6	0.21	0.59	0.67	0.68	0.50
	5	0.16	0.48	0.53	0.54	0.40
	4	0.12	0.36	0.40	0.42	0.31
	3	0.09	0.24	0.27	0.28	0.22
	2	0.07	0.13	0.14	0.16	0.12

Table 5.5 - Normalized Displacements at Different Loading Stage

Load Number	Floor	28	34	40
Frame	Roof	1.0	1.0	1.0
	6	0.91	0.88	0.88
	5	0.76	0.72	0.72
	4	0.57	0.54	0.54
	3	0.35	0.35	0.35
	2	0.15	0.16	0.17
Wall	Roof	0.43	0.86	0.96
	6	0.35	0.72	0.81
	5	0.27	0.58	0.65
	4	0.19	0.43	0.49
	3	0.12	0.29	0.34
	2	0.06	0.16	0.18

Table 5.6 - Properties of SDOF Models Used in Q-Model3

	Q28	Q34	Q40
Equivalent Mass (kip-mass)	1.20	1.29	1.35
X_e, measured from the west (in)	83.	136.	145.
Y_e, measured from the base (in)	715.	709.	707.
Normalized Deflection at Centroid	0.67	0.70	0.73
Moment at Break Point (k-in)	94,750.	94,750.	94,750.
Displacement at Break Point (in)	0.884	0.928	0.968
Post-Yielding Slope (k-in/in)	11,760.	11,190.	10,750.
Elastic Frequency (Hz)	1.78	1.68	1.61

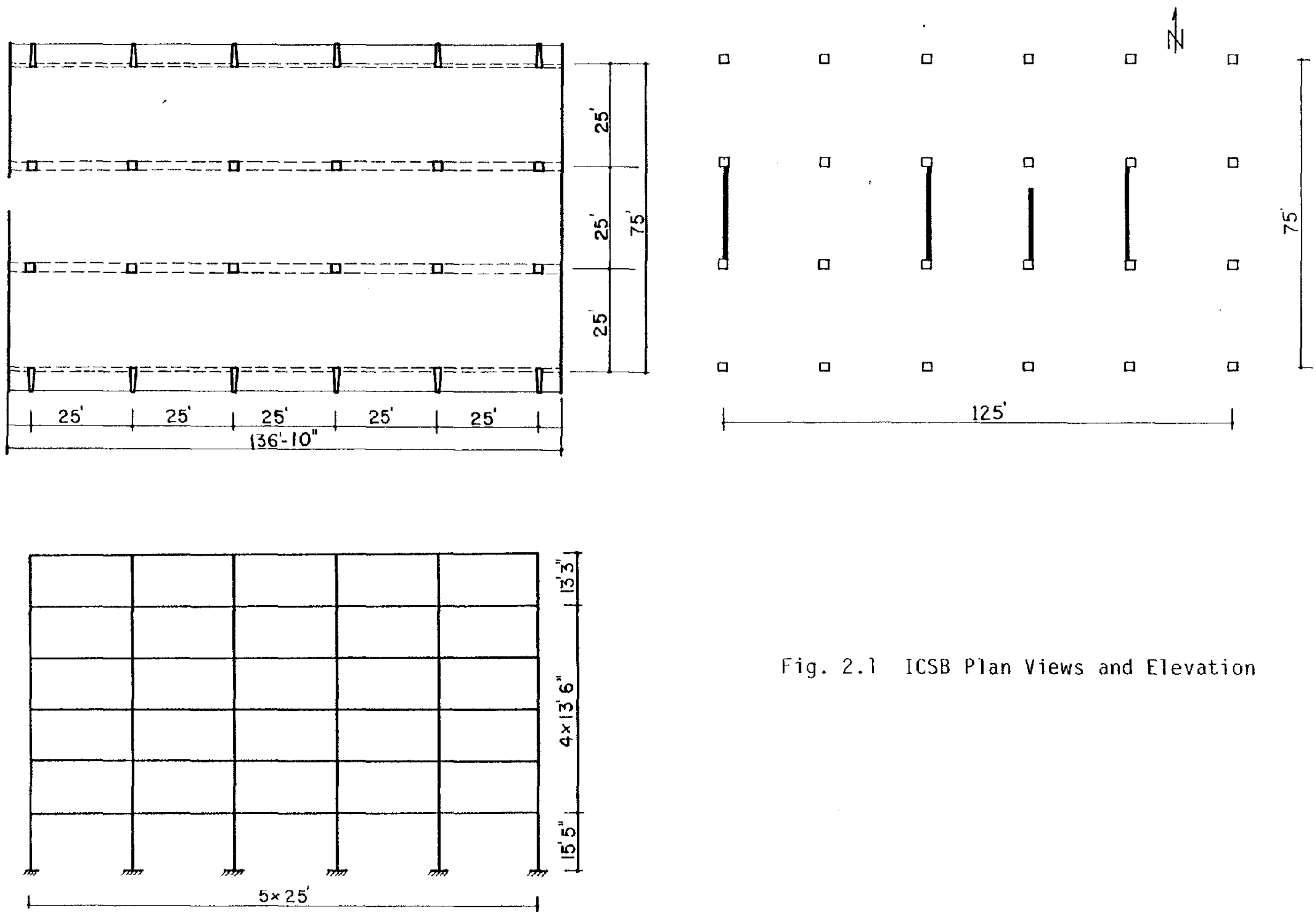


Fig. 2.1 ICSB Plan Views and Elevation

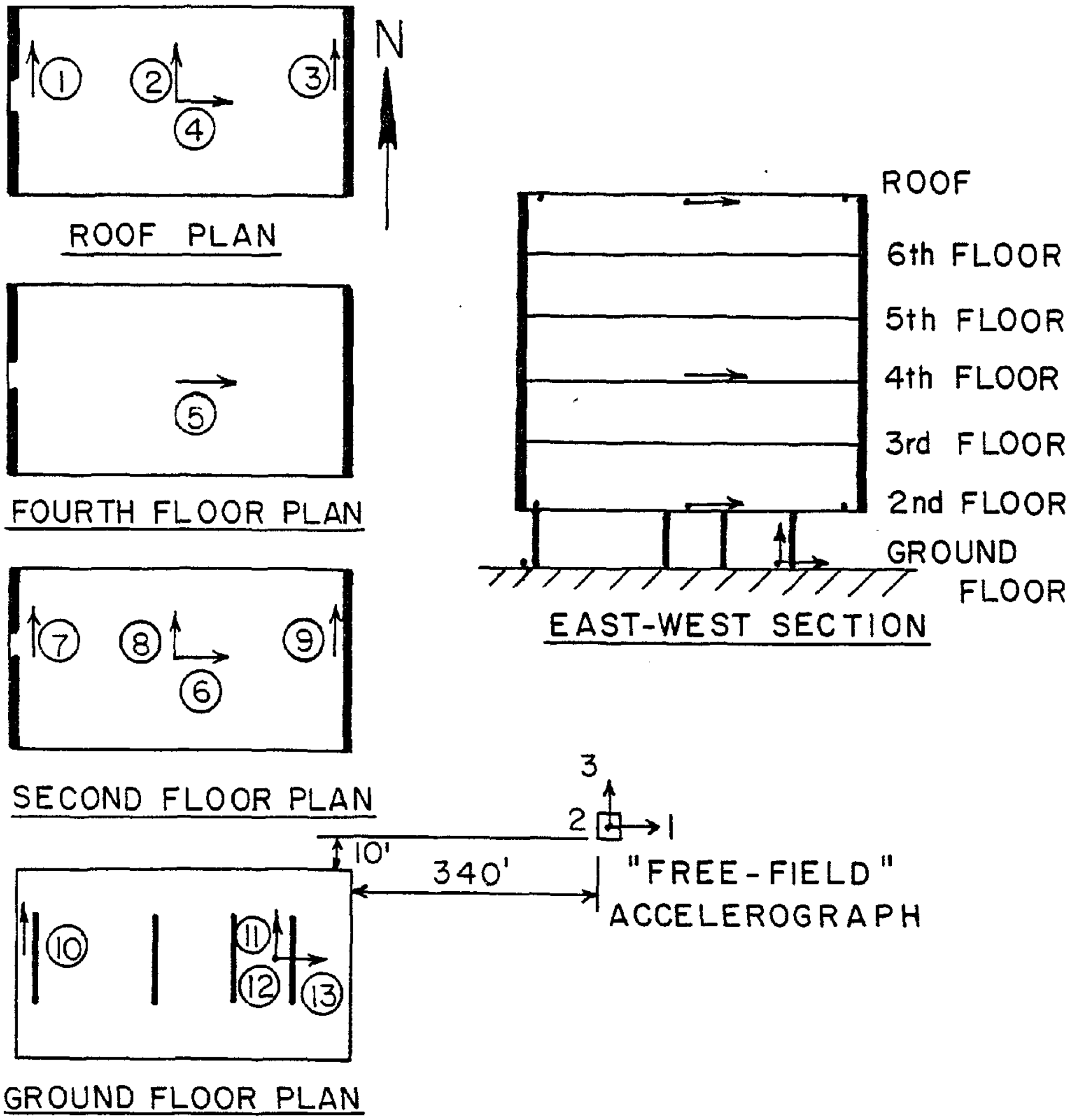


Fig. 2.2 Accelerometer Locations

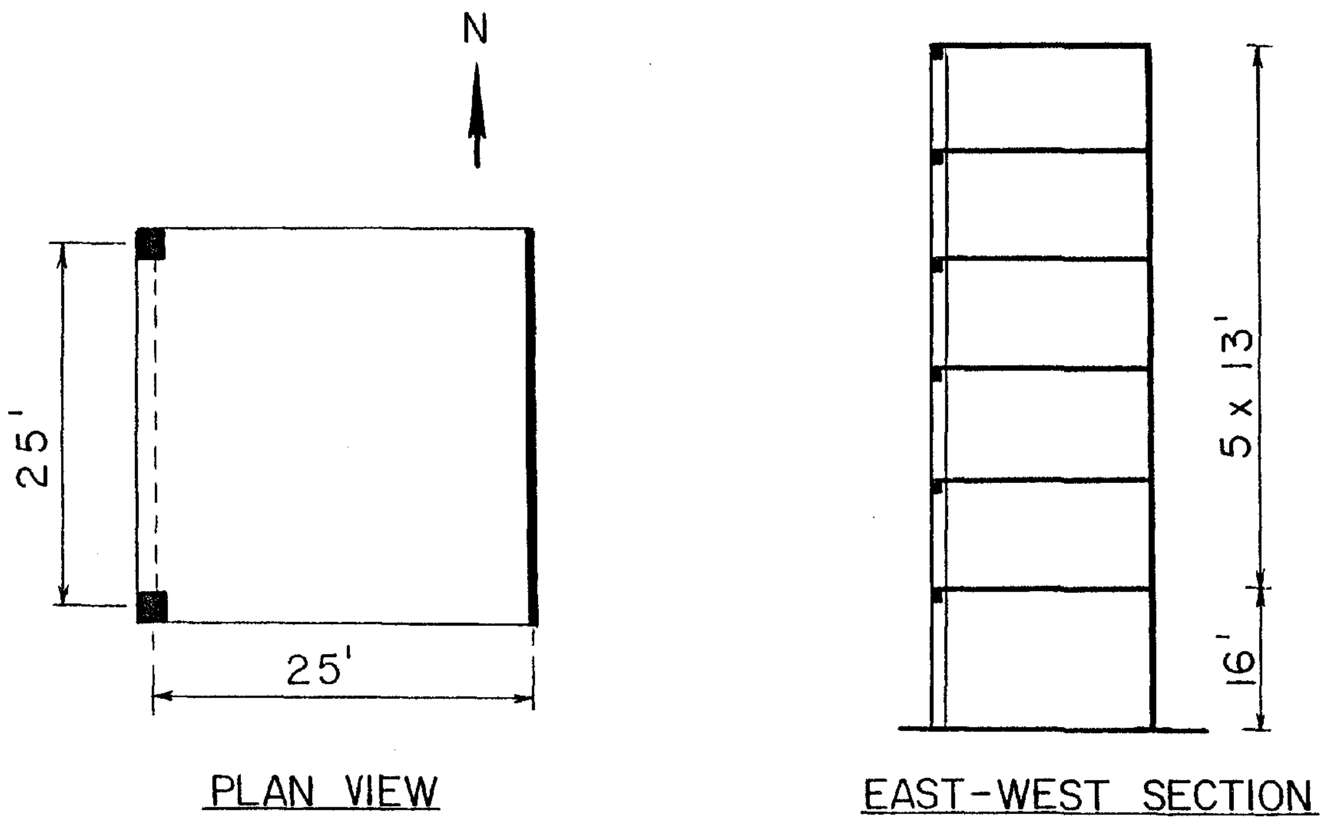
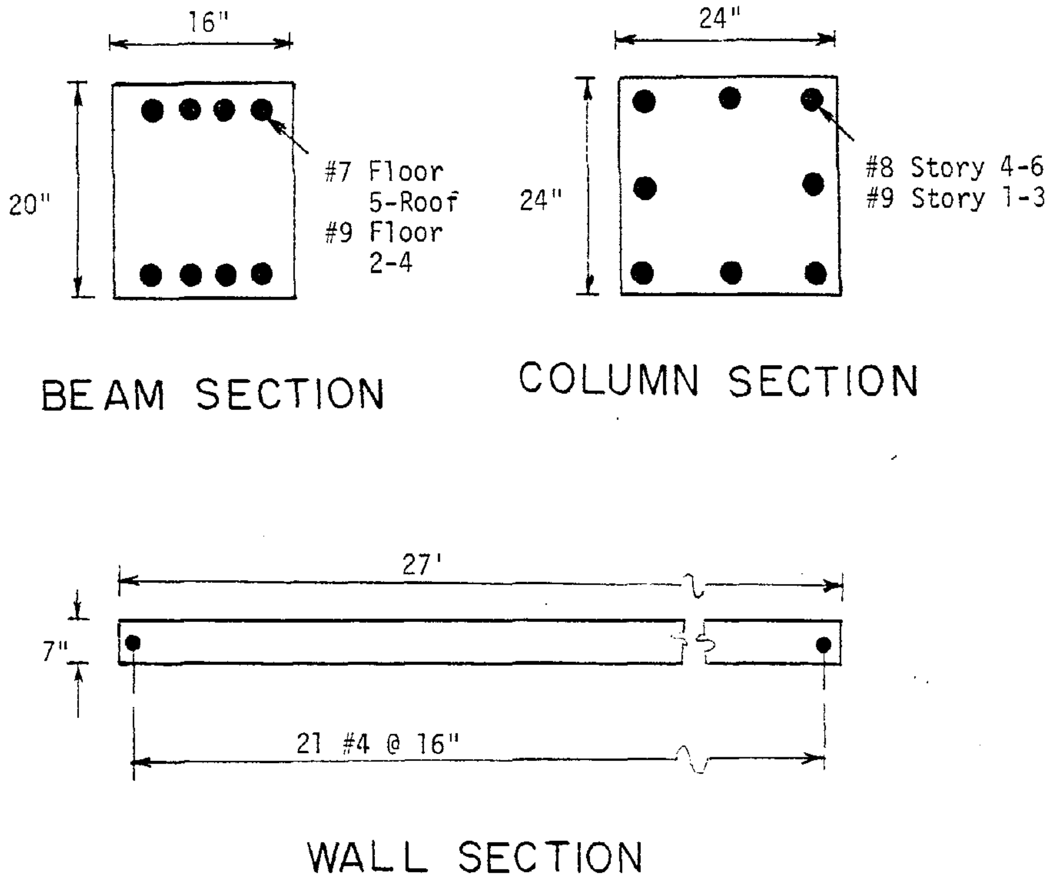


Fig. 2.3 SUFW Plan View and Elevation



Notes: - stirrups, ties, and wall transverse reinforcement not shown
- sections not to scale

Fig. 2.4 Reinforcement Detail for SUFW

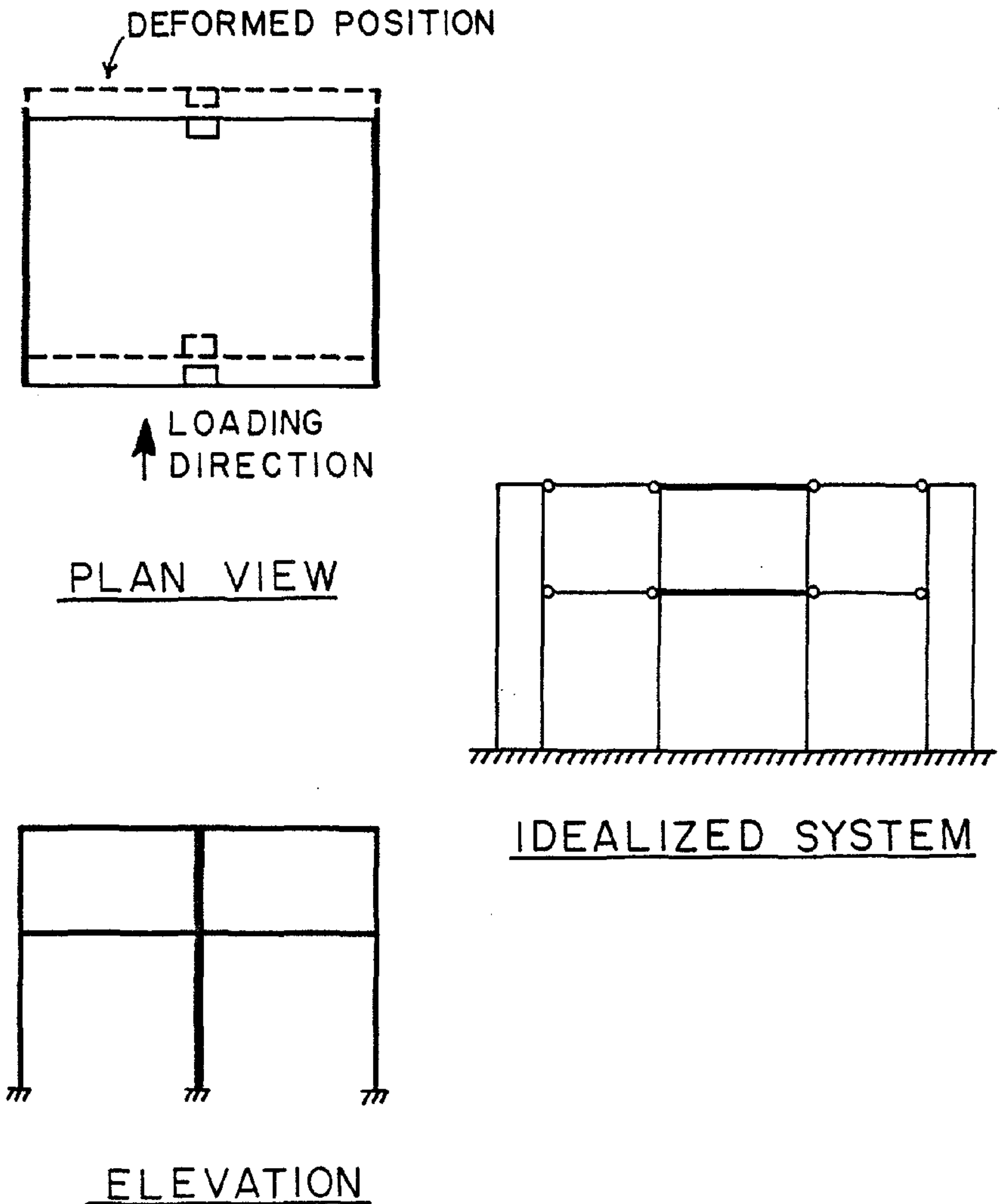


Fig. 3.1 Idealization of Coupled Symmetric Systems

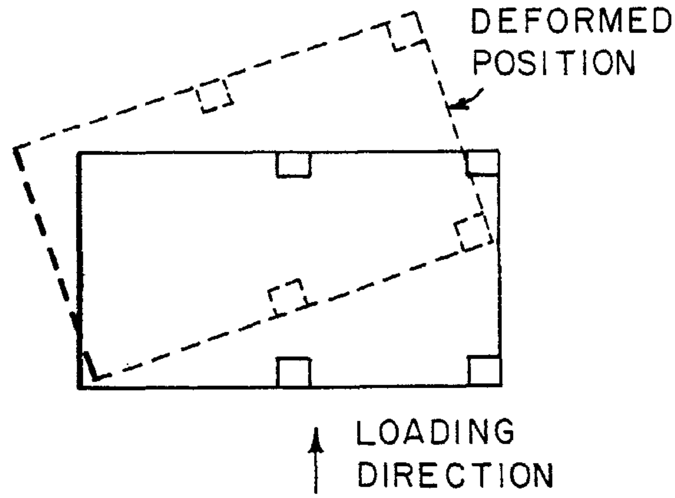


Fig. 3.2 Deformation of Unsymmetric Systems

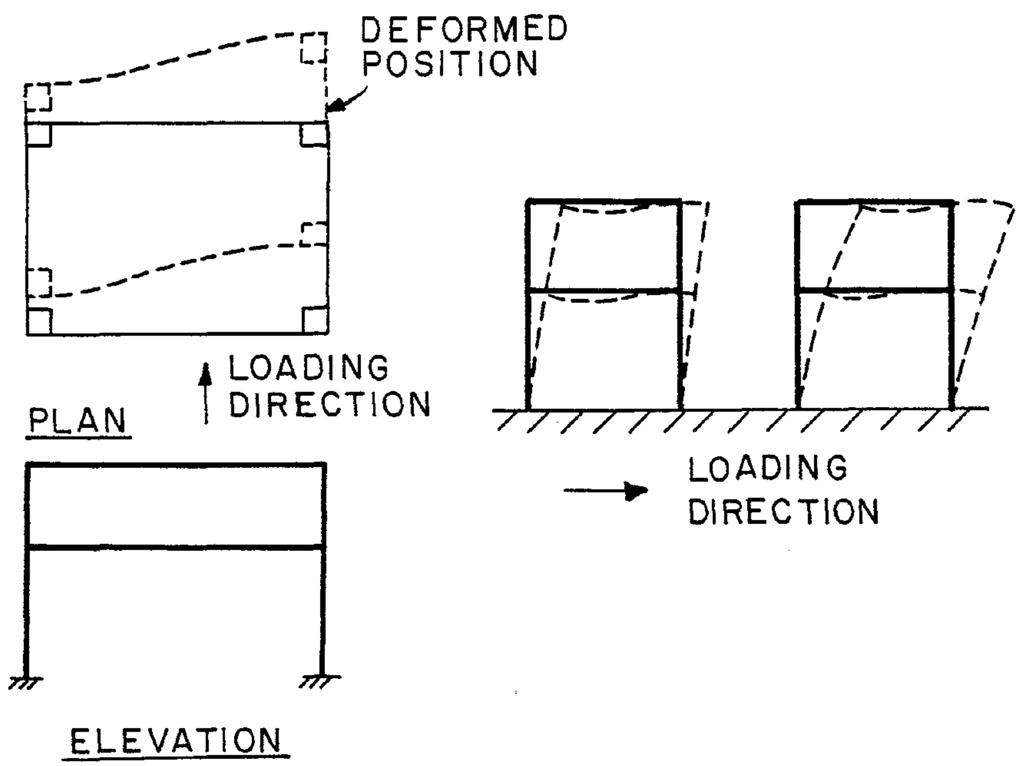


Fig. 3.3 Response of Uncoupled Frames

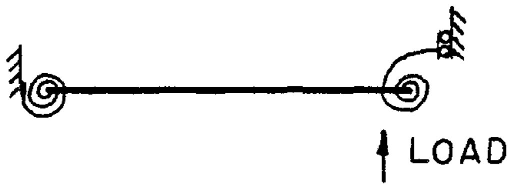


Fig. 3.4 Horizontal View of the Idealized Diaphragm Element

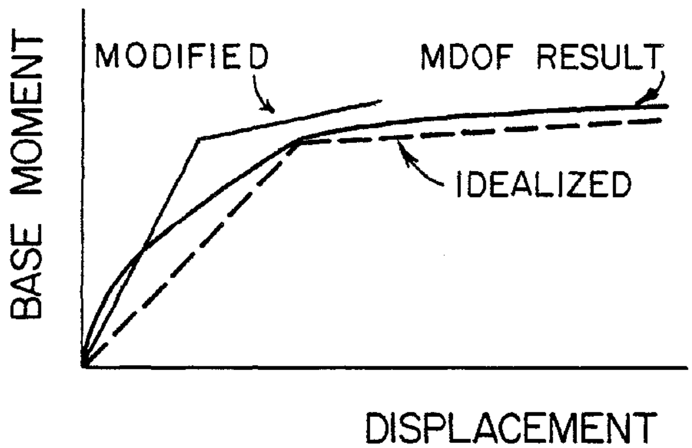


Fig. 3.5 Calculated and Idealized Moment-Displacement Relationship

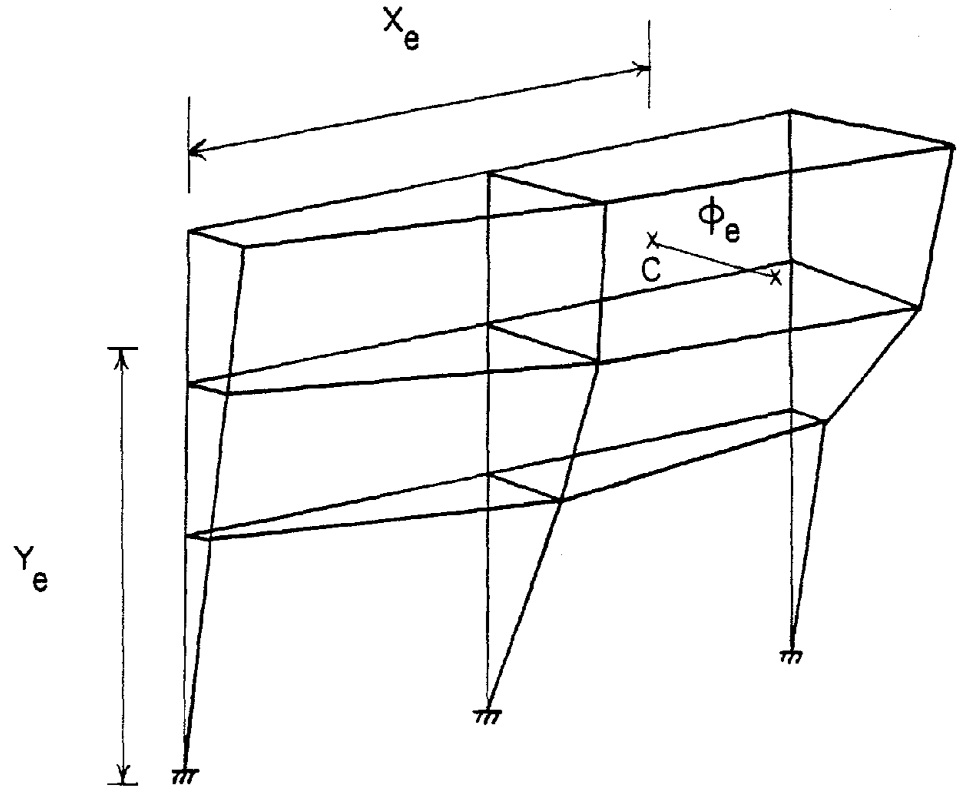


Fig. 3.6 Centroid of Deflected Surface

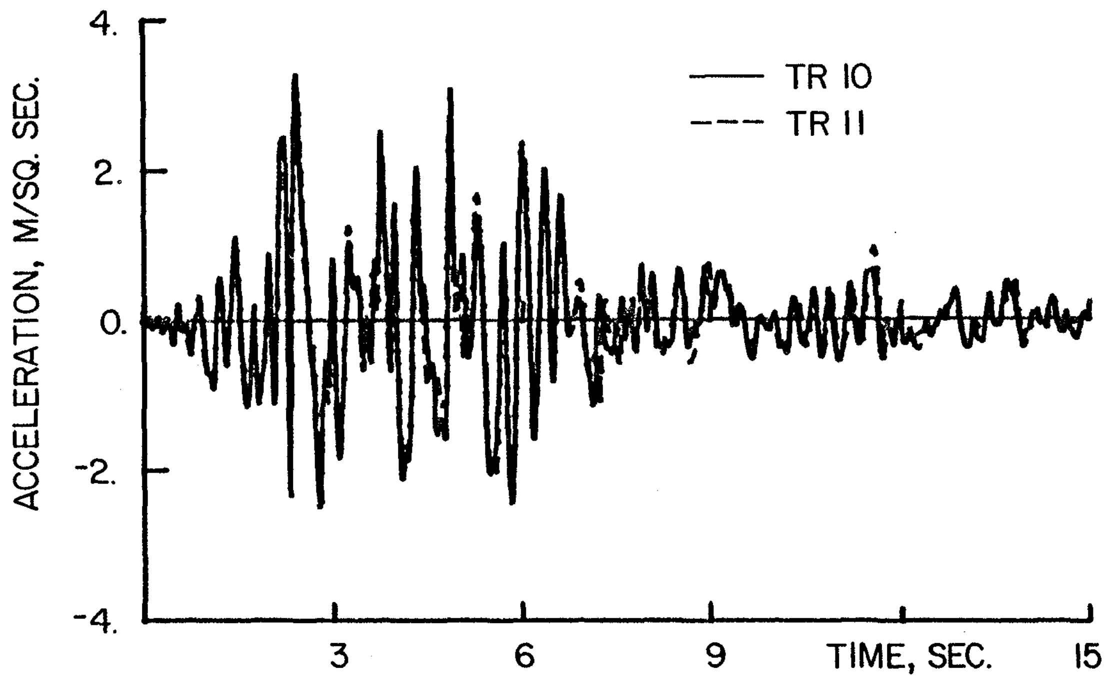


Fig. 4.1 Measured Acceleration in the North-South Direction of Ground Floor (1 M=39.37 IN)

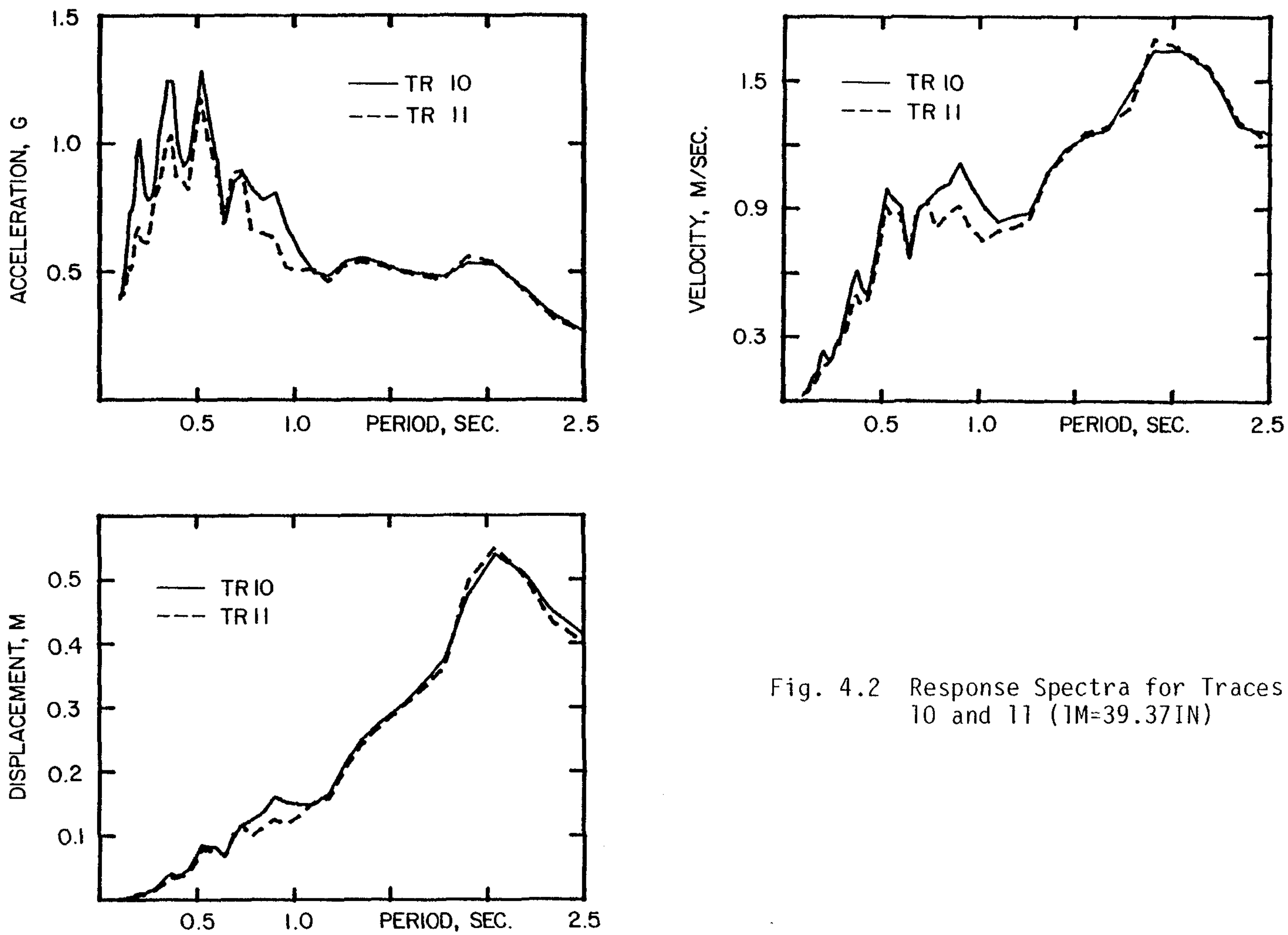


Fig. 4.2 Response Spectra for Traces 10 and 11 (1M=39.37IN)

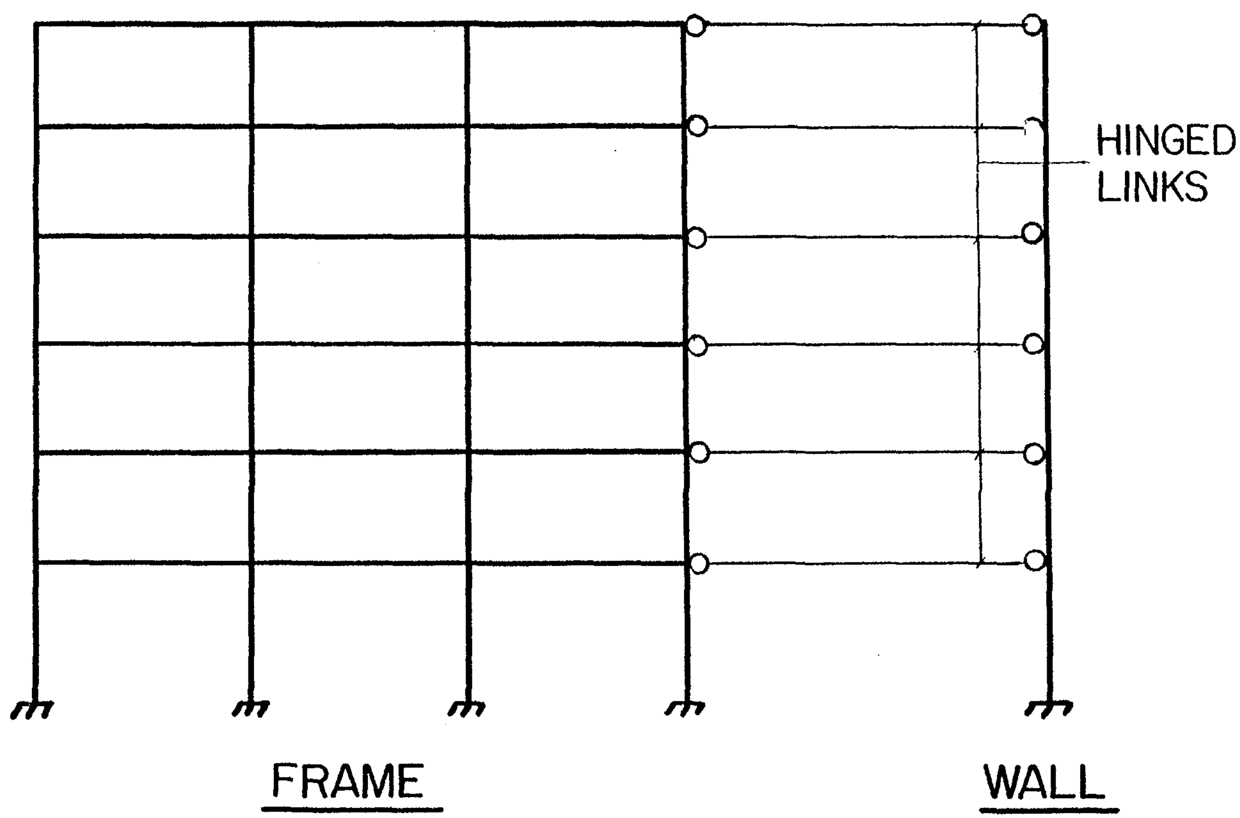


Fig. 4.3 Idealized MDOF System for ICSB in the North-South Direction

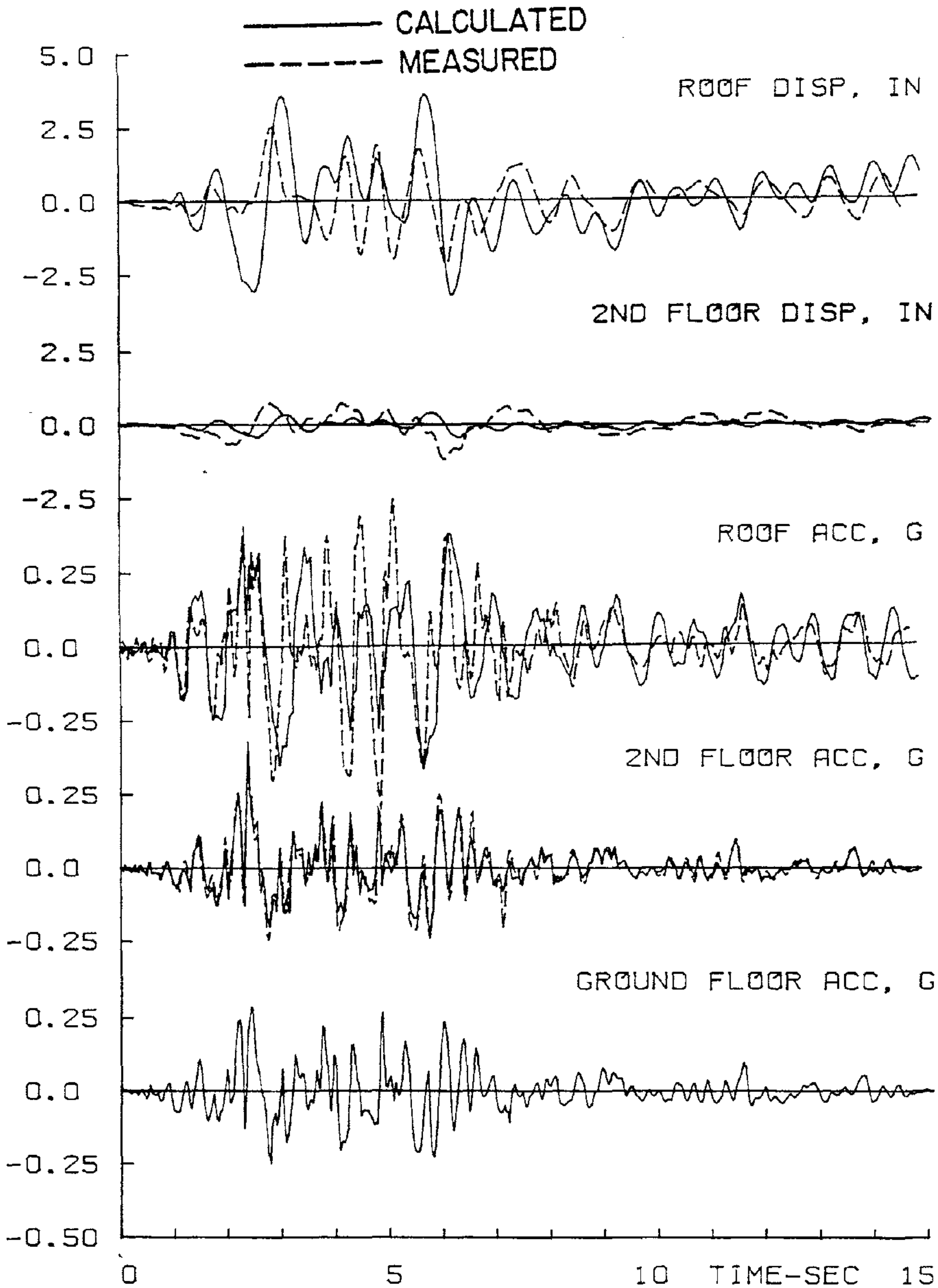


Fig. 4.4 Calculated and Measured MDOF Responses for ICSB

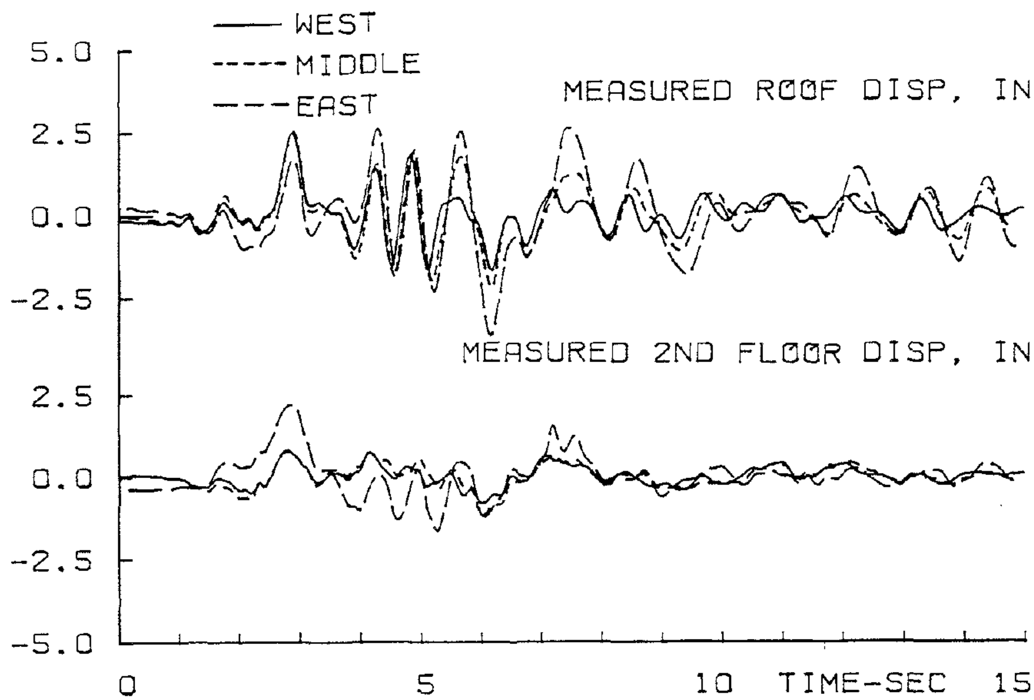


Fig. 4.5 Measured Displacement Histories at Different Locations

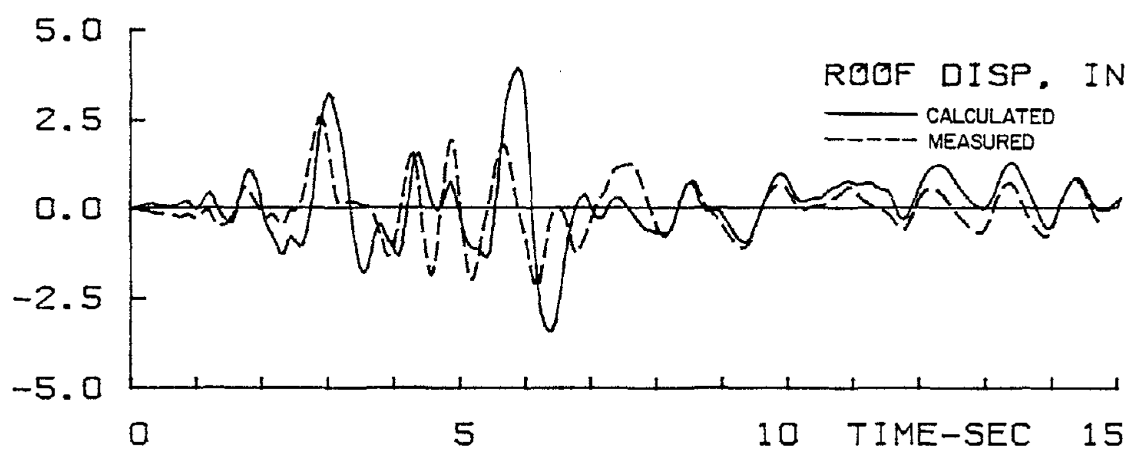


Fig. 4.6 Q-Model Response in the North-South Direction

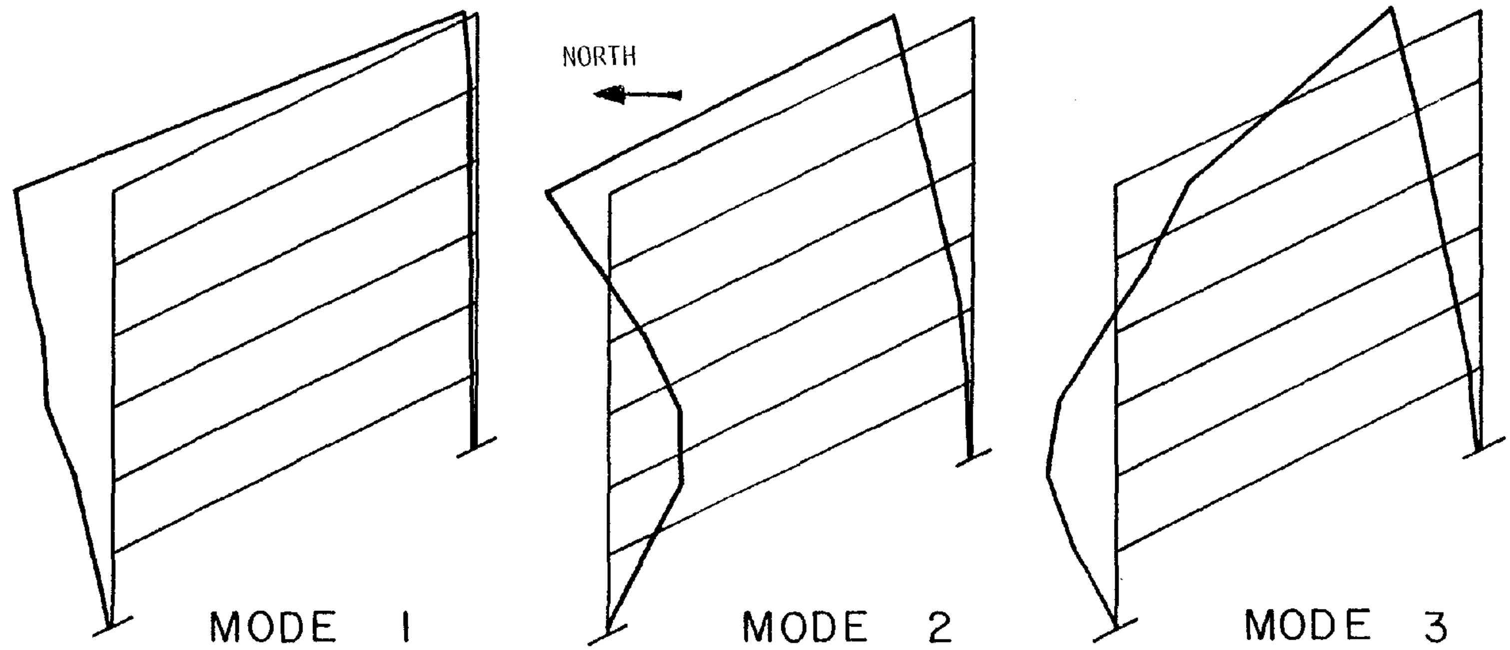


Fig. 5.1 First Three Mode Shapes for SUFW

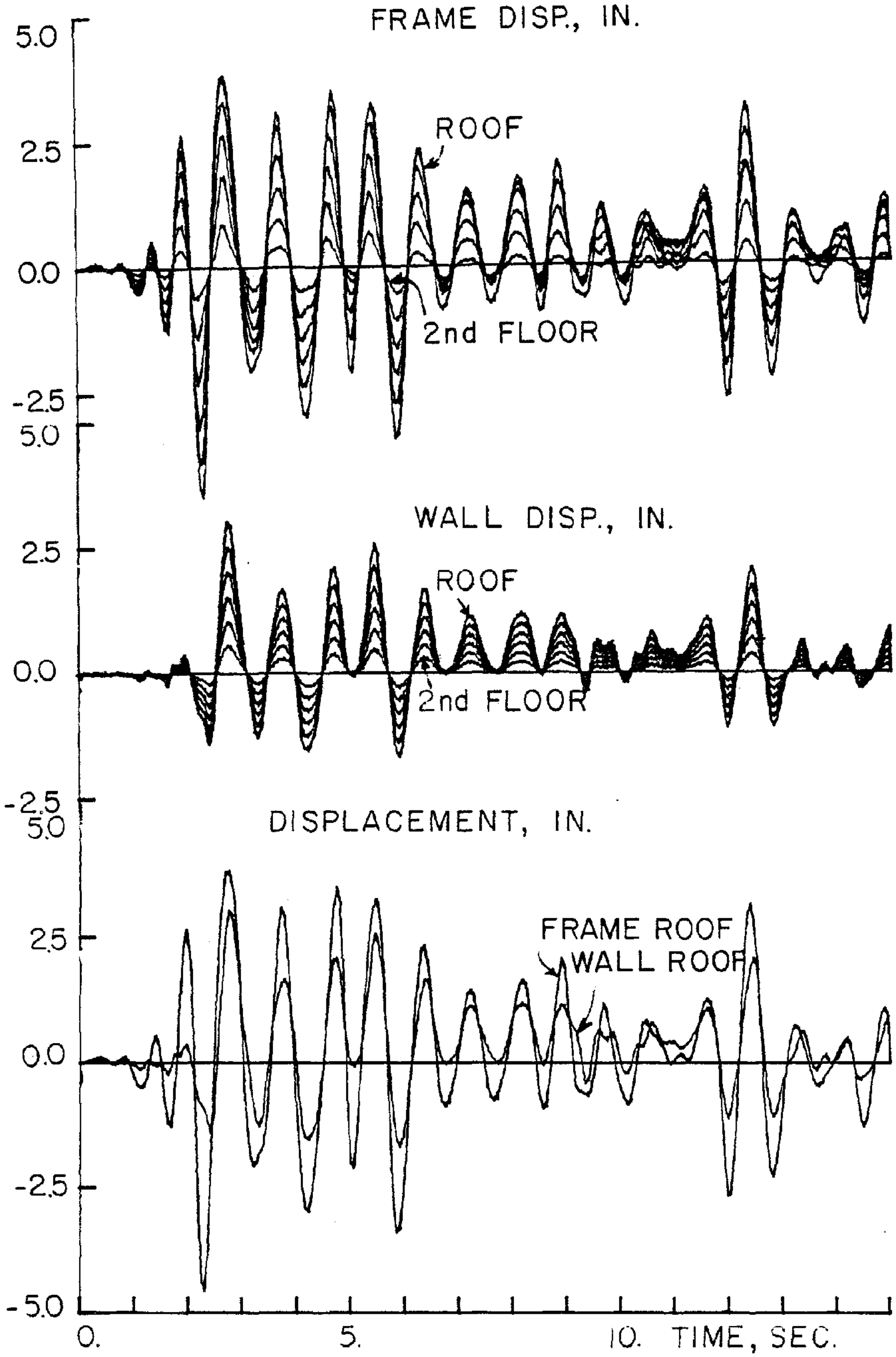


Fig. 5.2 MDOF Responses for El Centro Record Normalized to 0.3g

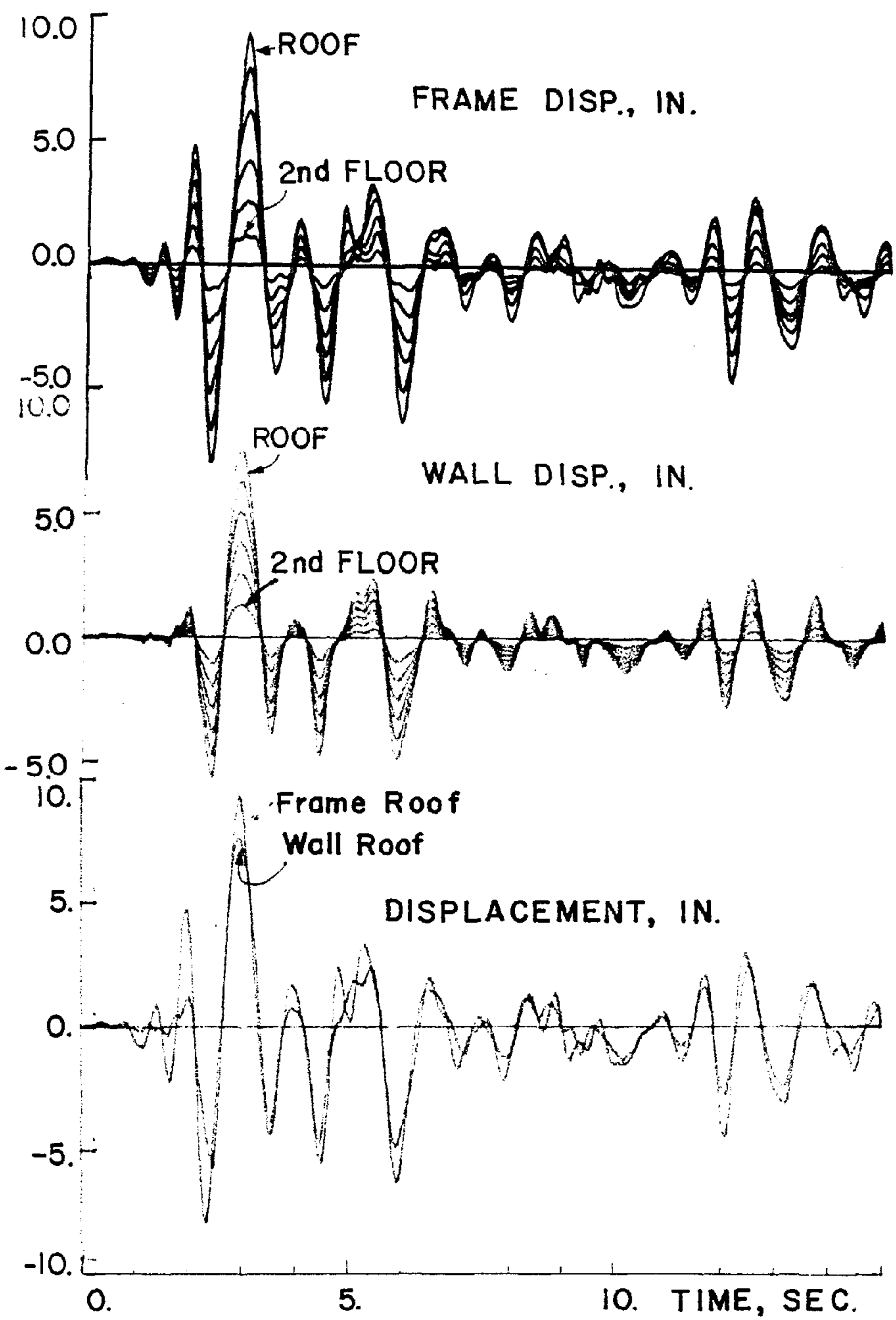


Fig. 5.3 MDOF Responses for El Centro Record Normalized to 0.5g

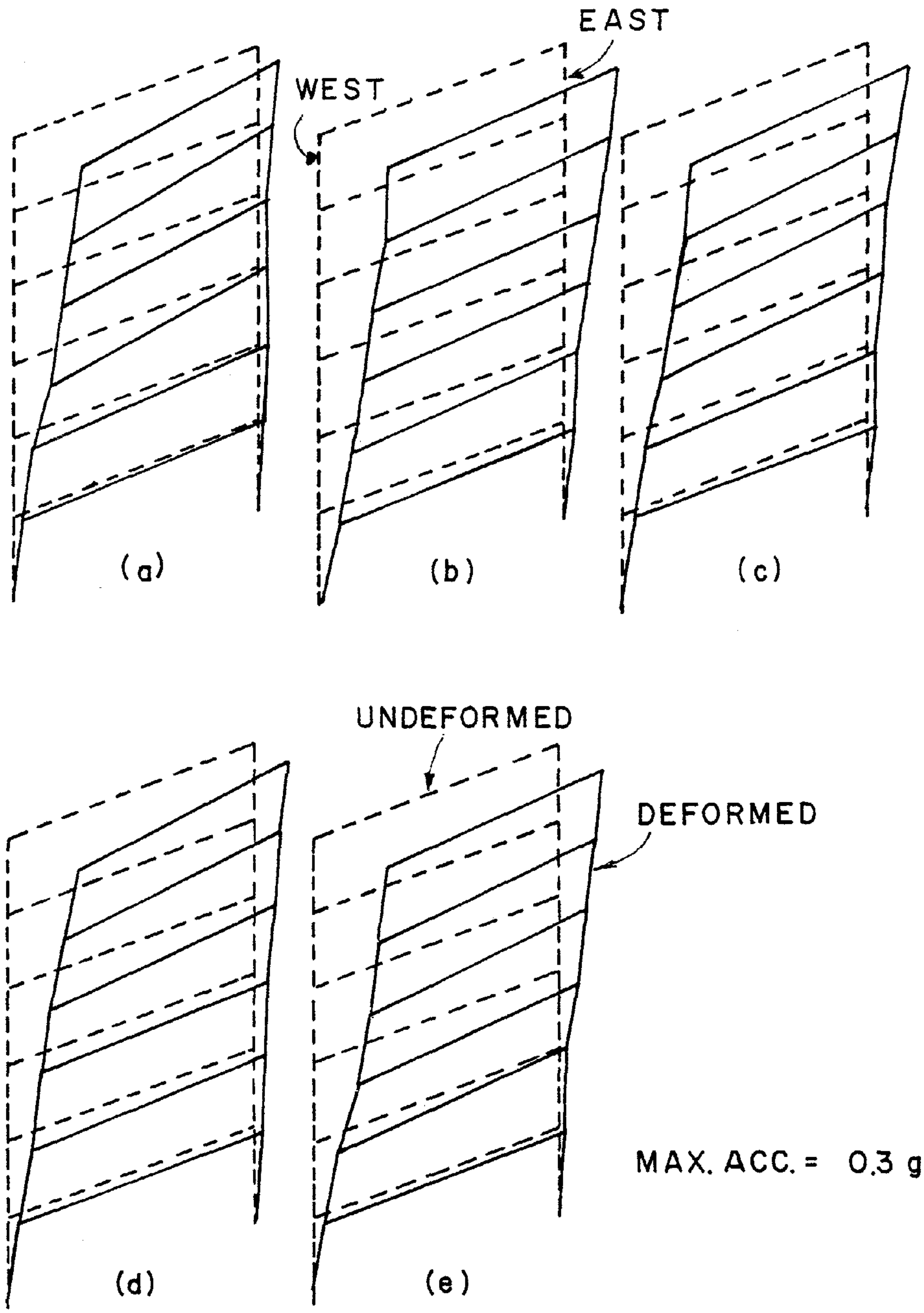


Fig. 5.4 Normalized Displacements During Run 1

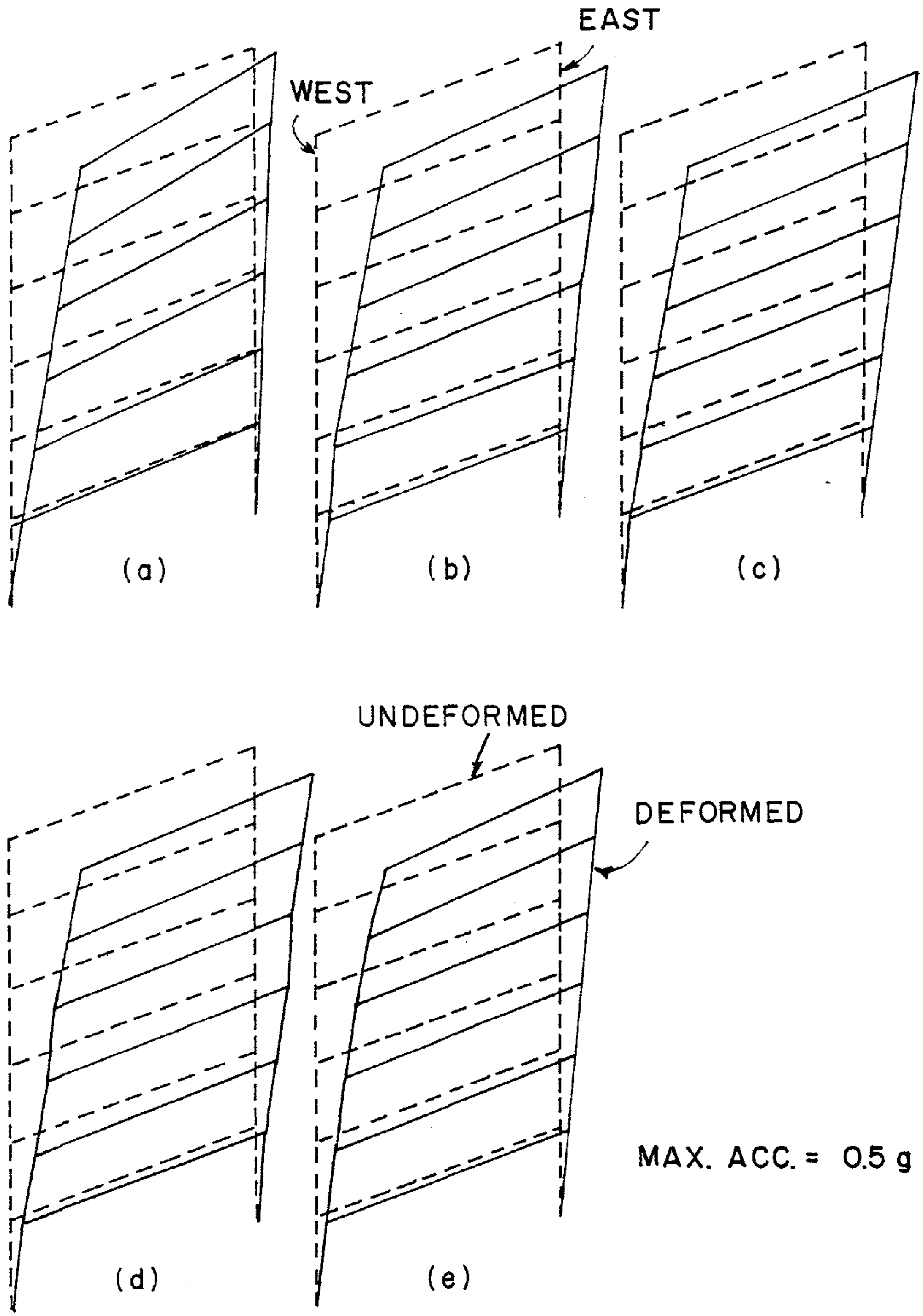


Fig. 5.5 Normalized Displacements During Run 2

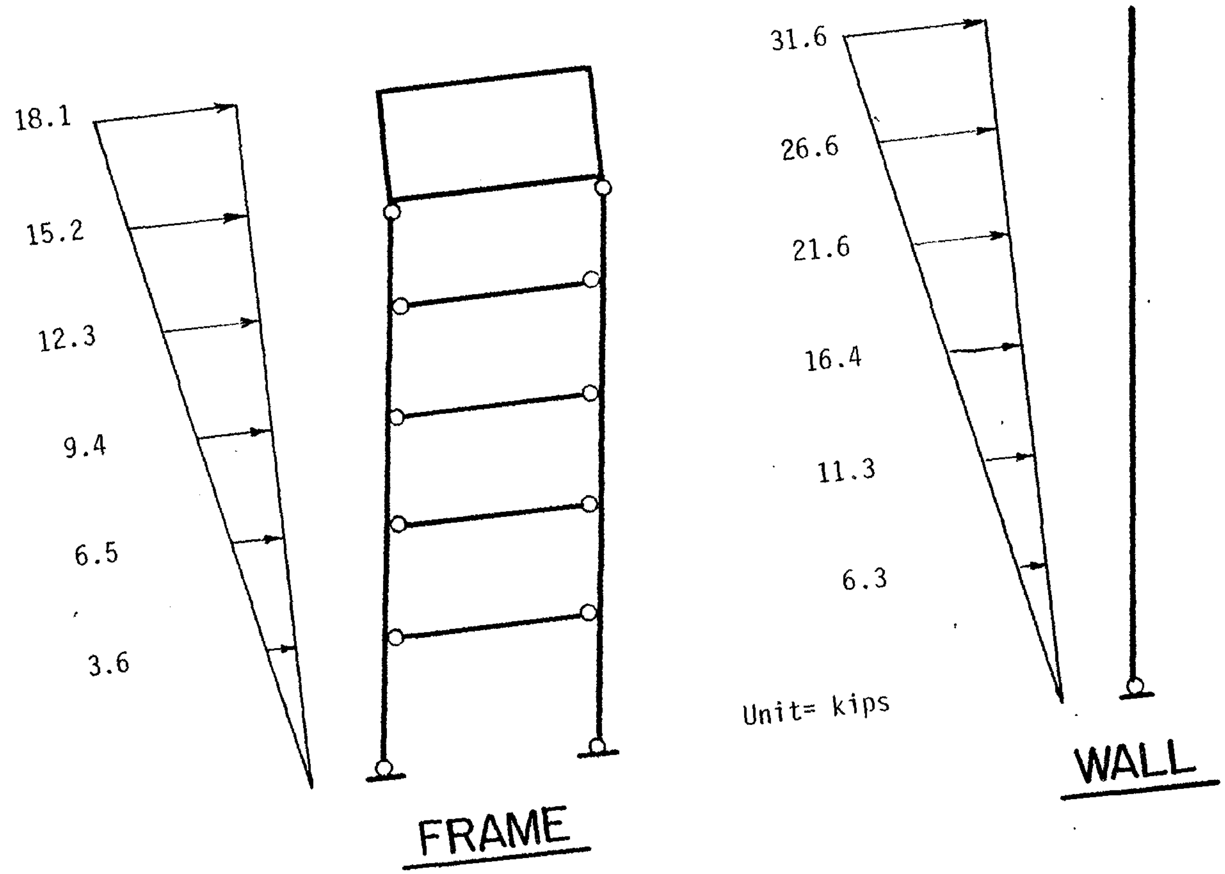


Fig. 5.6 Collapse Loads for SUFW

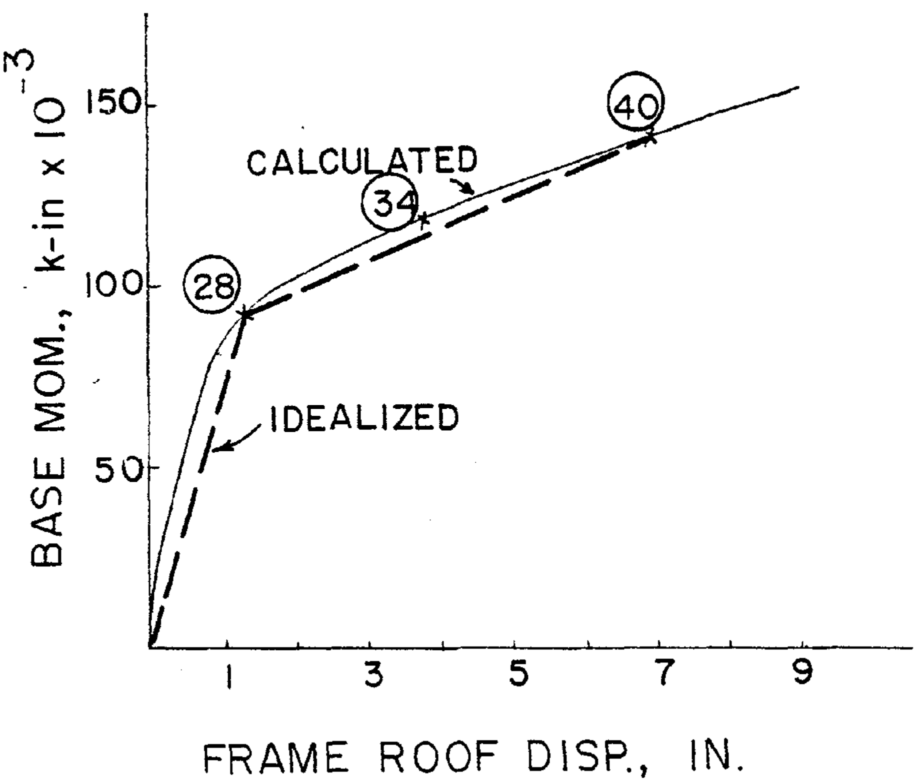


Fig. 5.7 Moment-Displacement Relationship for SUFW

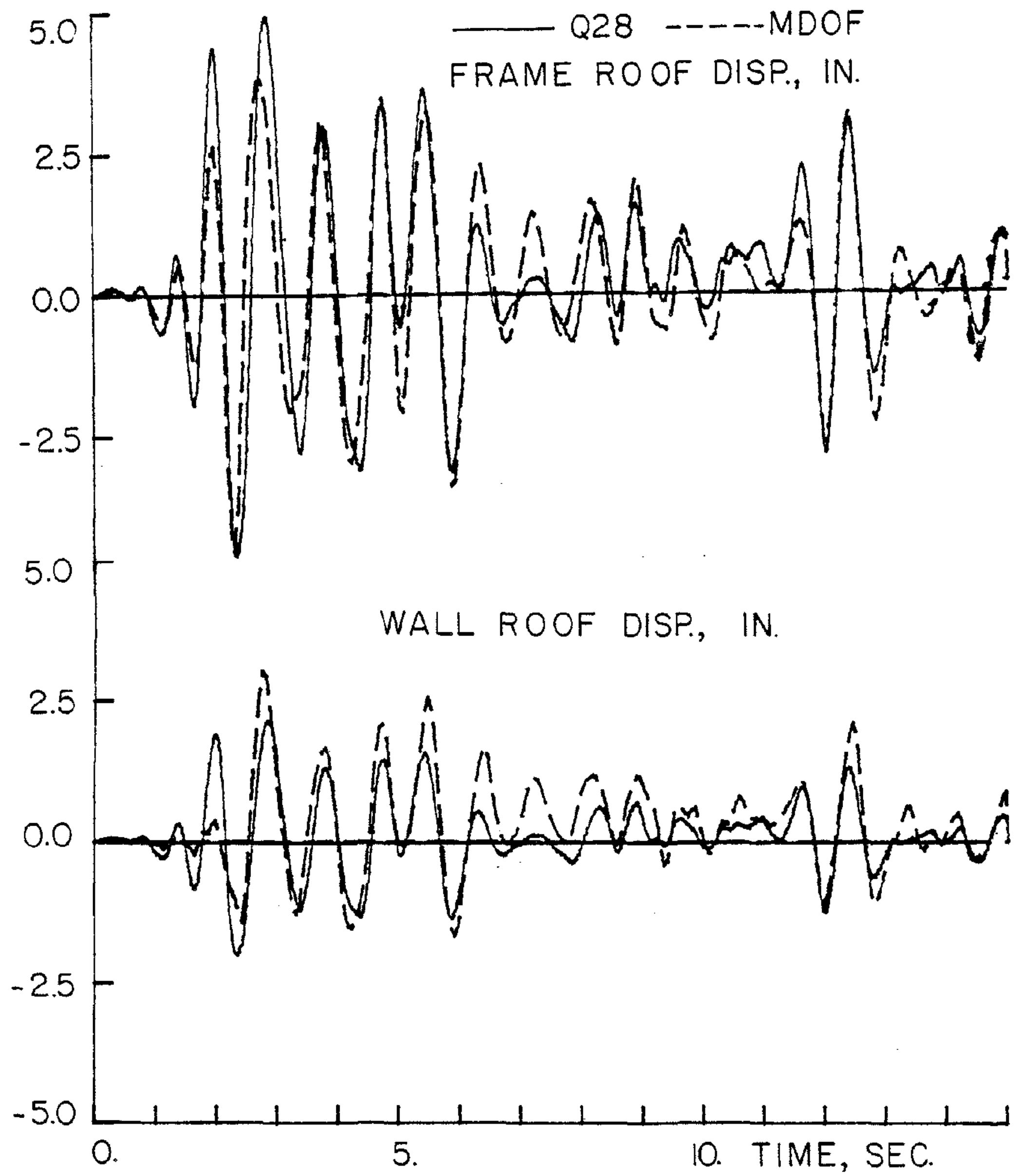


Fig. 5.8 Displacement Histories for Maximum Base Acceleration of 0.3g (Run 1)

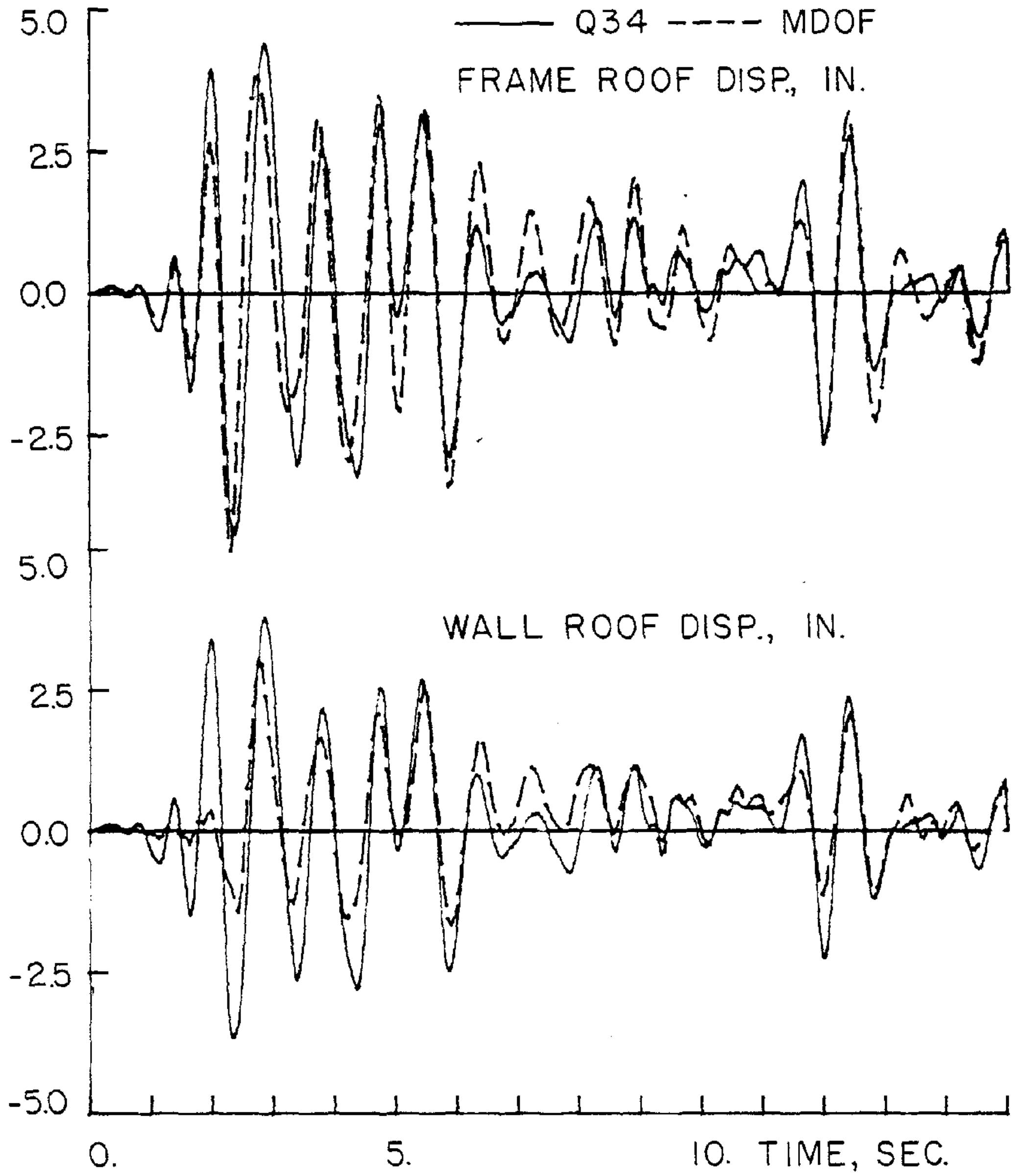


Fig. 5.8 (cont'd) Displacement Histories for Maximum Base Acceleration of 0.3g (Run 1)

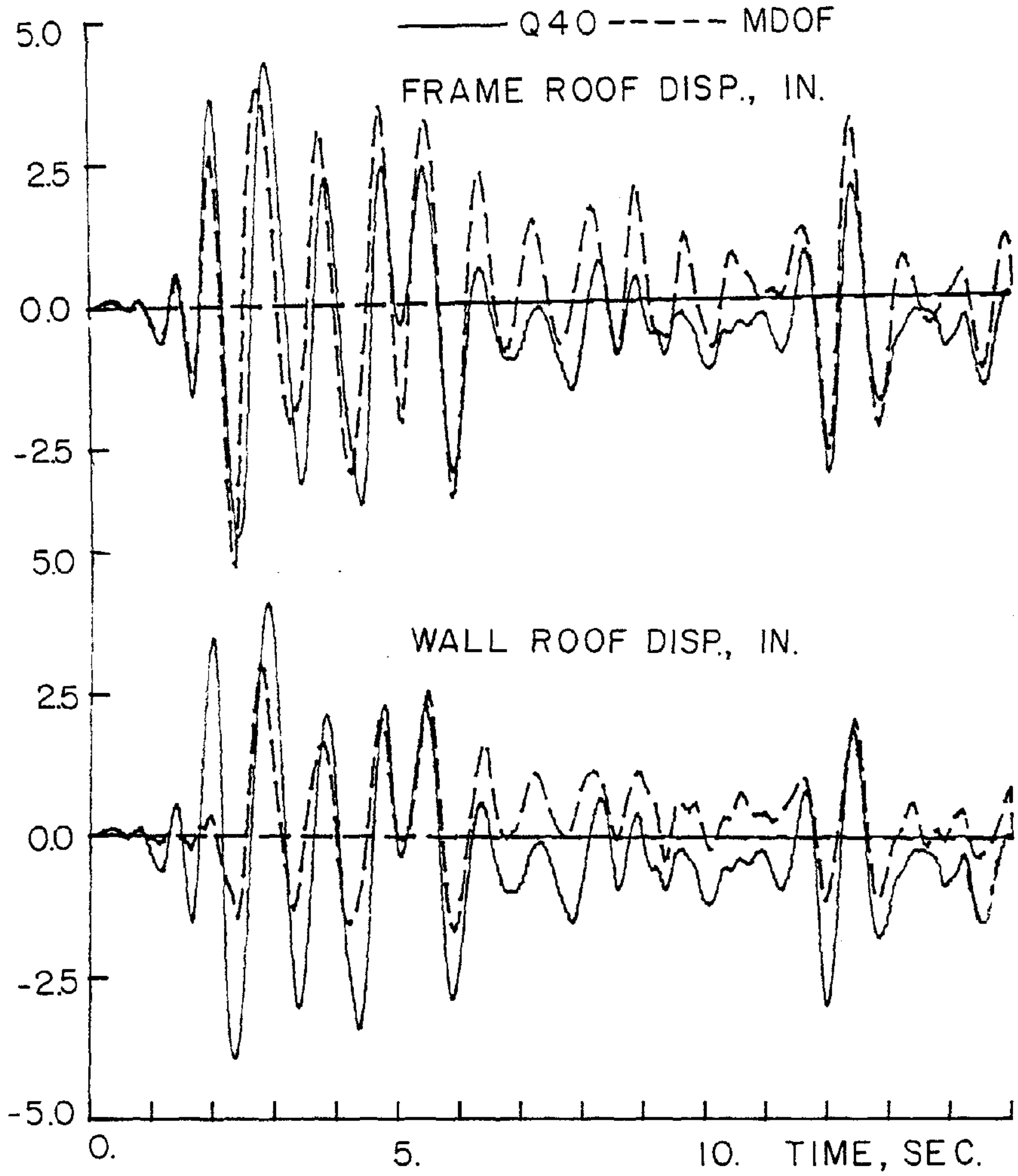


Fig. 5.8 (cont'd) Displacement Histories for Maximum Base Acceleration of 0.3g (Run 1)

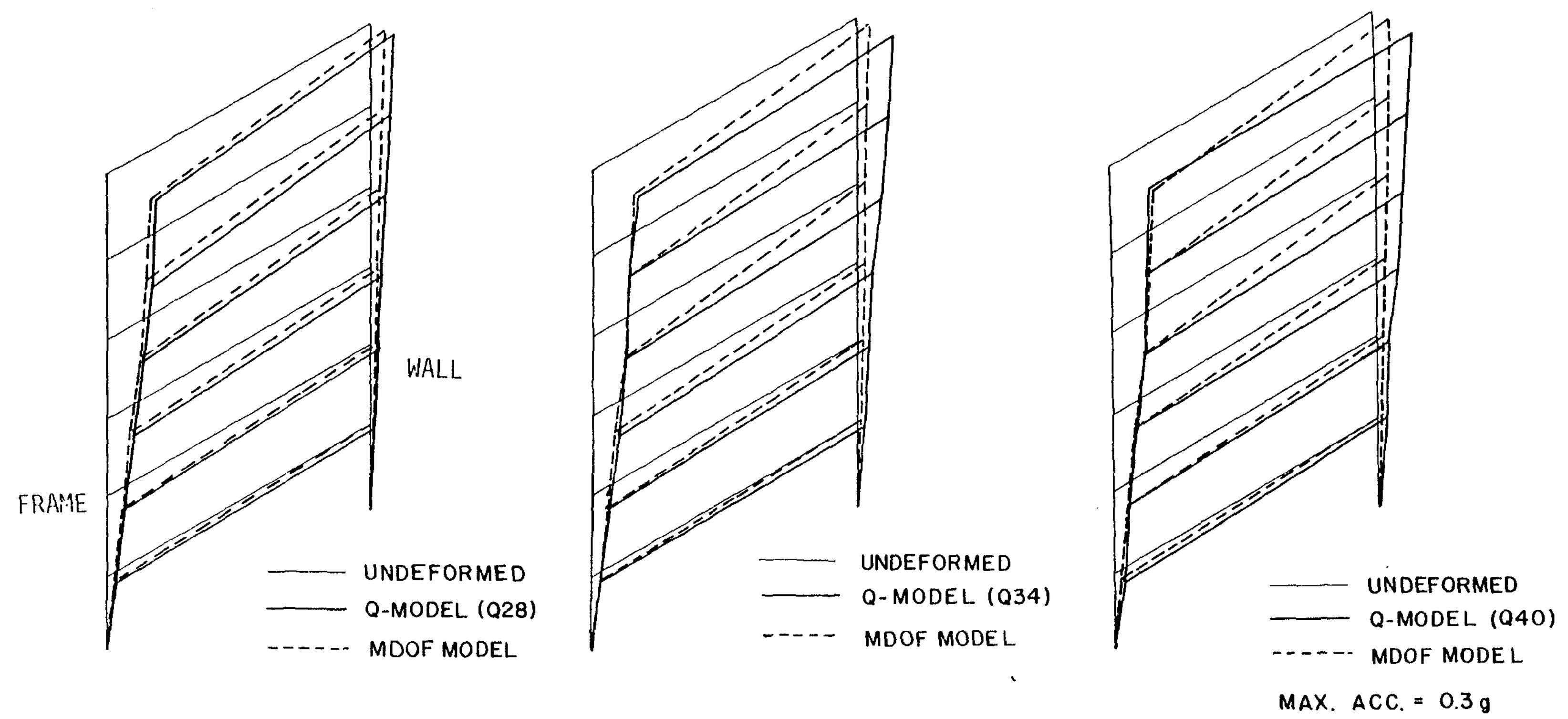


Fig. 5.9 Displaced Shape at Time of Maximum Frame Top Response in Run

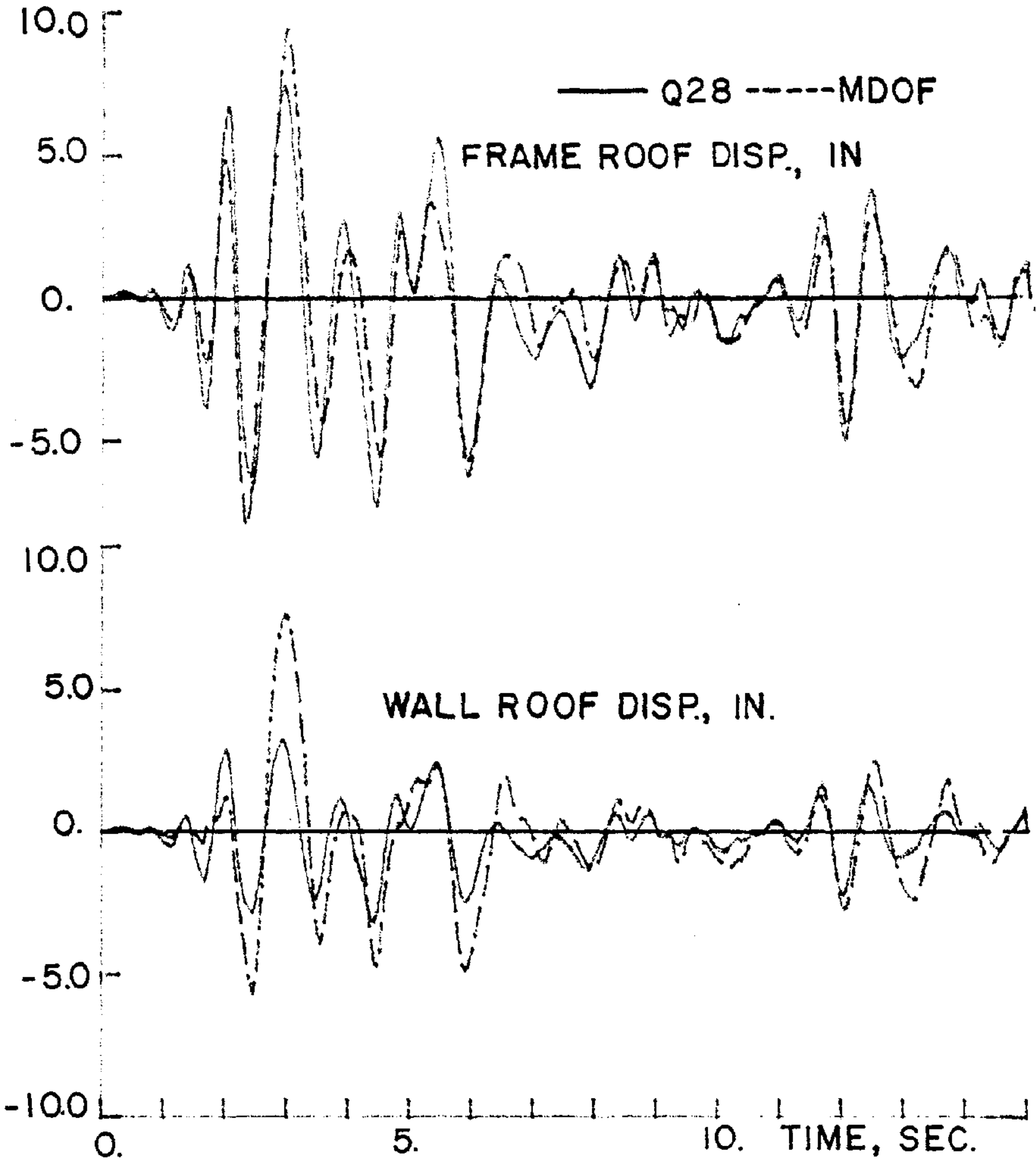


Fig. 5.10 Displacement Histories for Maximum Base Acceleration of 0.5g (Run 2)

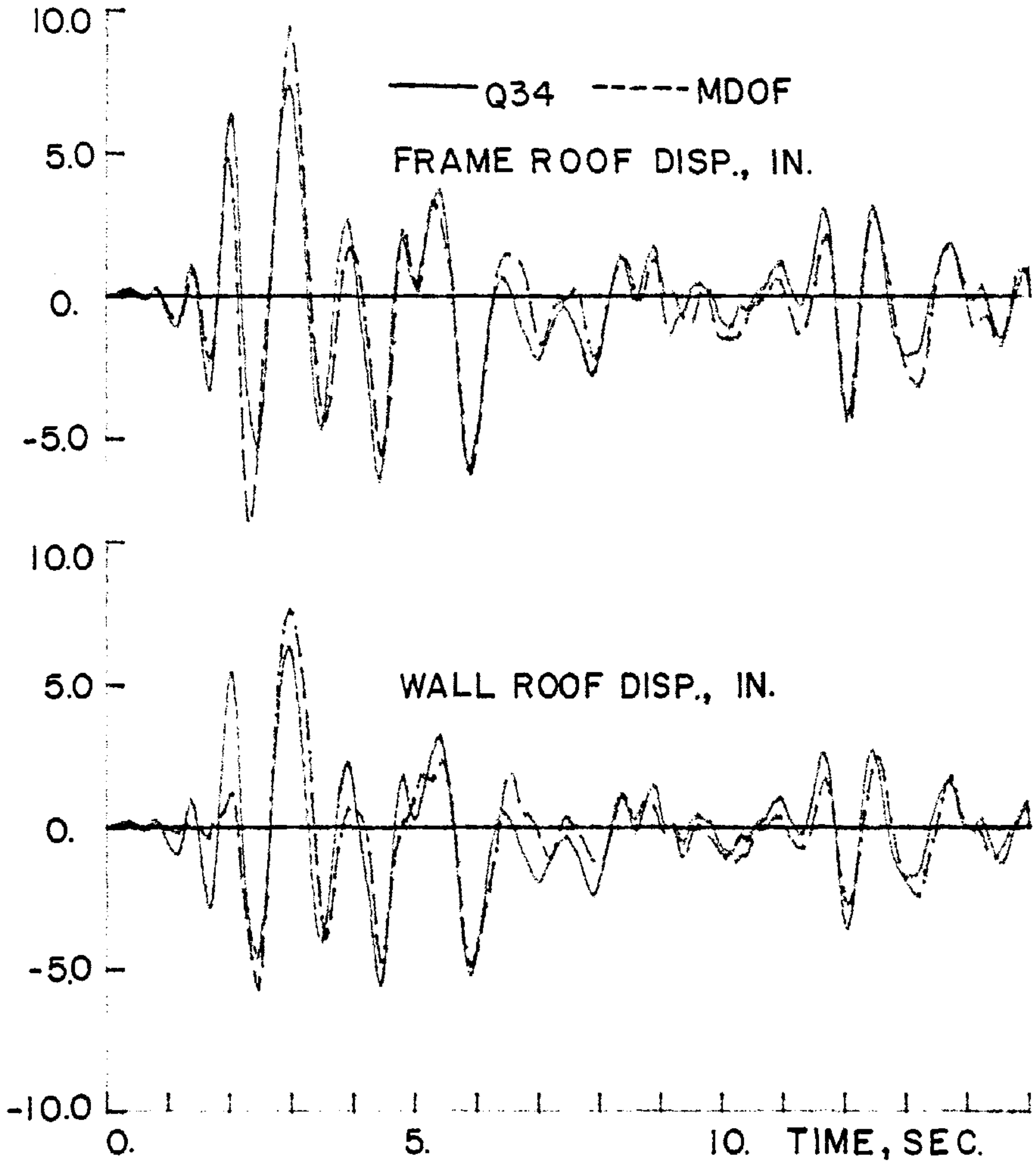


Fig. 5.10 (cont'd) Displacement Histories for Maximum Base Acceleration of 0.5g (Run 2)

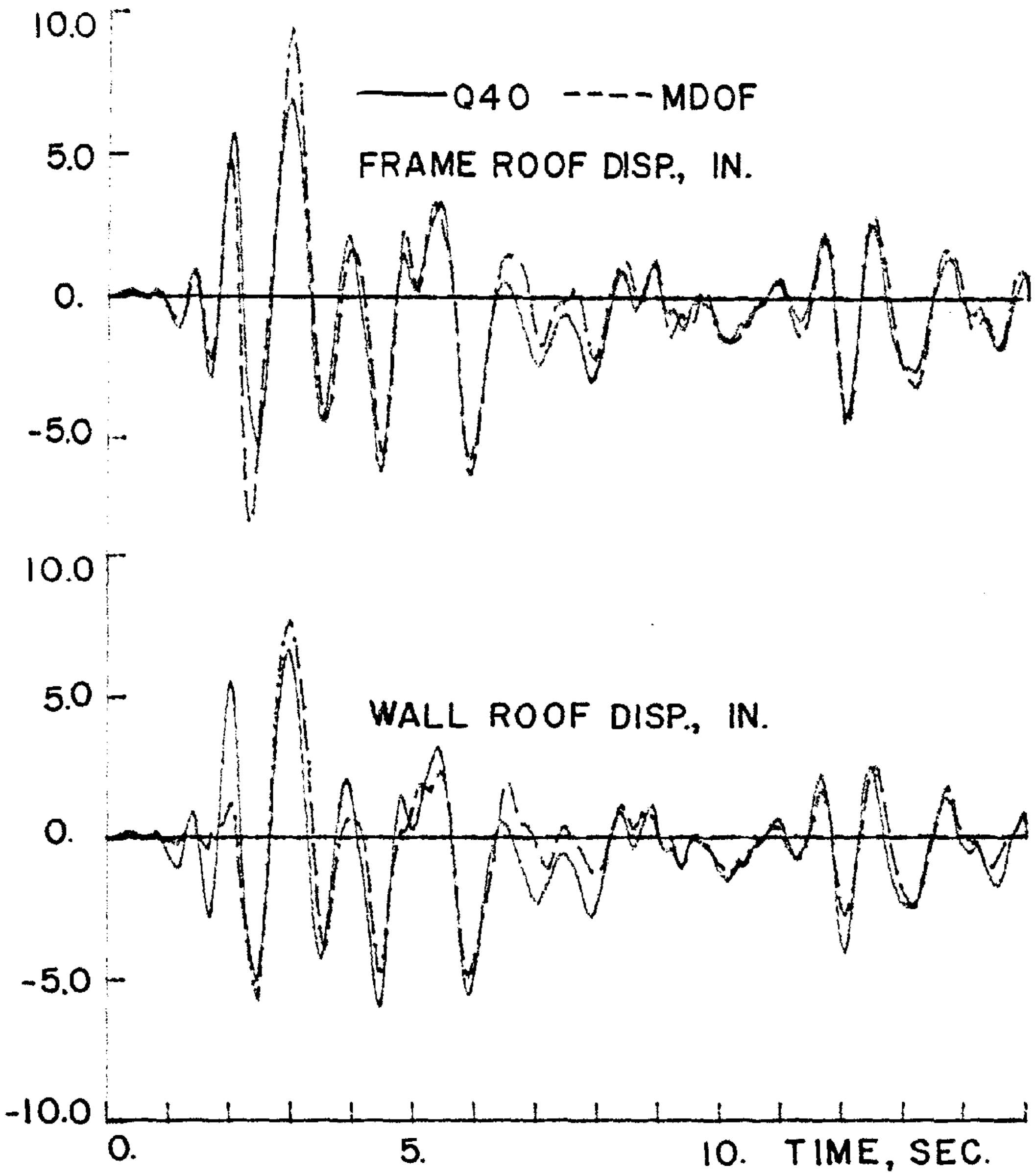


Fig. 5.10 (cont'd) Displacement Histories for Maximum Base Acceleration of 0.5g (Run 2)

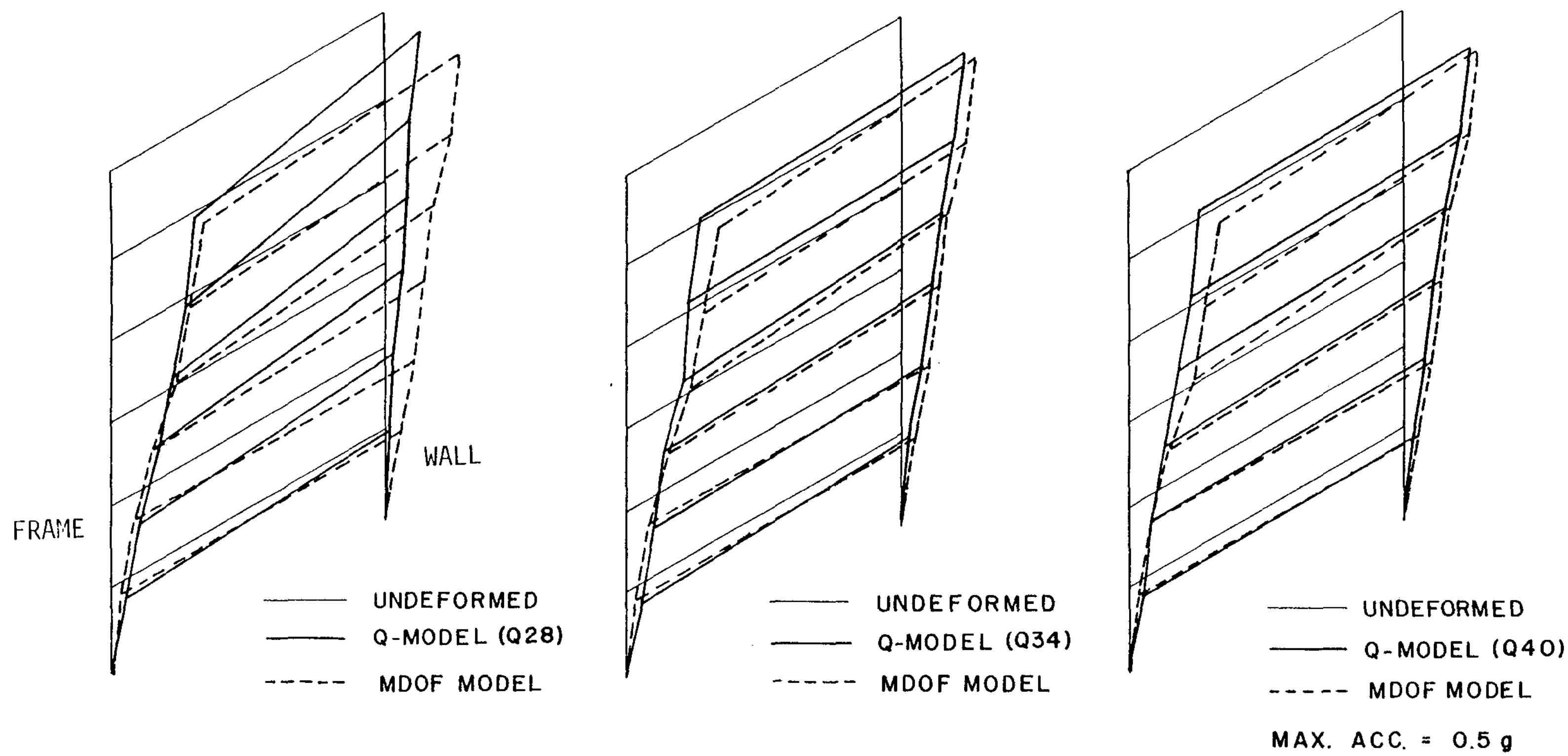


Fig. 5.11 Displaced Shape at Time of Maximum Frame Top Response in Run 2