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16. Abstract (Limit: 200 words)

This project developed and analyzed design information on the behavior of multi-story structures during earthquakes. Structural and ground motion parameters, their effects on dynamic response, the most significant parameters as design variables, and relationships among the design variables were examined. A computer program was used to investigate nonlinear response of coupled wall structures. The analytical modeling procedures used are given and the effects of structural parameters on dynamic inelastic response of coupled walls are discussed. Properties of structures with different mass for inertia are noted. General information is provided concerning earthquake resistant design and variables found to be significant in formulating a design procedure. Factors considered include the beam-to-wall stiffness ratio, wall strenth, beam-to-wall strength ratio, yield deformation level, and earthquake characteristics.

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Parametric Investigation and Design Procedure

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July 1981

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Parametric Investigation and Design Procedure

by

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R. A. Parmelee(4), and A. Scanlon(5)

INTRODUCTION

Behavior of multistory structures during recent earthquakes indicates that both protection of human life and good damage control can be attained if buildings are stiffened by properly proportioned an detailed structural walls. A common use of structural walls in multistory buildings is in the form of coupled walls. For functional reasons, reinforced concrete walls are usually pierced or connected to other walls by girders forming a "coupled wall system."

The performance of coupled wall structures during earthquakes provides convincing evidence that coupled walls possess both strength and deformation capacity beyond the elastic range. Superiority of coupled wall systems in resisting strong earthquakes results from their ability to dissipate energy by significant yielding in beams while overall structural stability and stiffness is maintained by walls.

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Although coupled wall systems have a long history of satisfactory use in stiffening multistory buildings, there is a need for information on the behavior of such structures under strong earthquake motions. Knowledge on magnitudes of deformations and forces expected at critical regions of coupled wall systems under specific combinations of structural and ground motion parameters is essential for aseismic design. Similarly, information on beam-to-wall strength and stiffness ratios is also required to control the sequence of plastification among members. Inelastic dynamic response analysis was carried out in this research project to clarify questions regarding non-linear response of coupled walls. Design information was developed for earthquake resistant coupled walls.

This investigation is part of a combined analytical and experimental program to develop design information for earthquake-resistant reinforced concrete structural wall systems, currently underway at the Construction Technology Laboratories of the Portland Cement Association.

OBJECTIVE AND SCOPE

The overall objective of the project is to develop design information for earthquake resistant coupled wall structures. Objectives of the portion of the project reported here were to:

1. Identify structural and ground motion parameters, investigate their effects on dynamic response, and select the most significant parameters as design variables.

2. Establish relationships among design variables, recommend a design procedure, and prepare sample design aids.

The following outlines the scope of the work:

1. Selection of a 20-story coupled wall structure for use in dynamic analysis.
2. Identification of structural ground motion parameters.
3. Dynamic analyses to investigate the effects of selected structural and ground motion parameters on elastic response of coupled walls.
4. Selection of design variables and determination of relationships among these variables.
5. Development of design procedure.
6. Development of sample design aids and illustration of their use by a design example.

DYNAMIC ANALYSIS PROCEDURE

It is generally agreed that under strong earthquakes inelastic behavior of structures is inevitable. Therefore, dynamic analysis of structures under earthquake excitations should consider inelastic action. Dynamic inelastic response analysis was employed in this investigation. A computer program was used to investigate nonlinear response of coupled wall structures.

A brief description of the computer program and the analytical modeling procedures used is given in the following sections. Detailed explanation of the modeling techniques and analytical procedures used are discussed in Ref. 1.

Computer Program DRAIN-2D

Dynamic response analysis was carried out using the program DRAIN-2D[2] developed at the University of California at Berkeley and later modified by the Portland Cement Association. The program has capabilities to analyze plane inelastic structures under seismic excitation.

The structural stiffness matrix is formulated by the direct stiffness method, with nodal displacements as unknowns. Each element in a structure is idealized as a planar discrete element. Degrees of freedom at each node are translations in the x and y directions and rotation about the z axis.

Dynamic response is determined using step-by-step integration assuming a constant response acceleration during each time step. Viscous damping is assumed resulting from a combination of mass dependent and stiffness dependent effects.

Output options of the program include printouts of response quantities at prescribed time intervals. These quantities include displacement components, forces, and plastic deformations at member ends. Envelopes of basic response quantities are automatically printed at the end of each computer run. Plotting options are also available for response time histories and force deformation hysteresis loops. More detailed information on the program can be obtained from Ref. 2.

Element Idealization for Computer Analysis

For the purpose of analysis, each wall and beam element between joints is idealized as a line element. It is extremely important to specify properties of these line elements properly so that both elastic and inelastic behavior of individual members can be simulated accurately. While the load-deformation relationship for the elastic region is straightforward, representation of hinging regions of walls and beams requires special attention.

Inelastic action is introduced by allowing formation of plastic hinges at the ends of line elements. Thus, each element consists of an "elastic beam" and two potential "point hinges" at each end as shown in Fig. 1. Since the computer model and the actual structure should yield the same behavior, stiffnesses of elastic beam and point hinges should be specified such that total chord rotation of a line element in the model is equal to chord rotation of the actual members. If a member has not reached yield level, the point hinges are assigned infinitely large stiffnesses and hence do not rotate. In this range the member deforms on the basis of the elastic stiffness assigned to the elastic beam. If the force level exceeds the prescribed yield level, the point hinges at member ends are allowed to rotate to simulate yielding. Details of beam and wall elements are given in Ref. 1.

Force-Deformation Hysteretic Loops

In the analytical model, inelasticity is introduced through point hinges at member ends. Therefore, behavior of members under inelastic load cycles is simulated by assigning load-deformation hysteretic loops to the point hinges. Features of

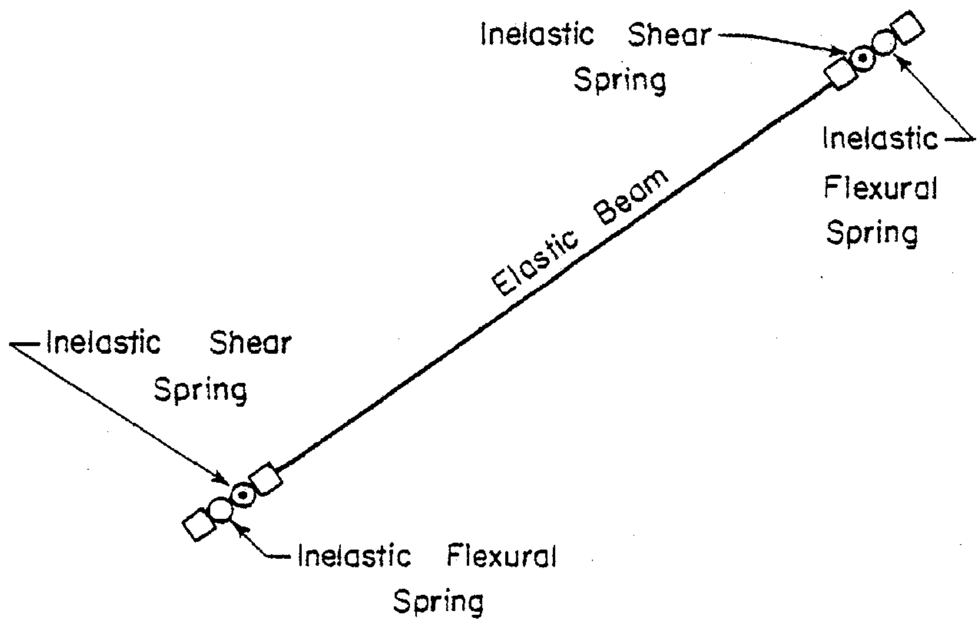

Fig. 1 Element Idealization

hysteretic behavior observed in laboratory tests were used to model member behavior. Detailed aspects of hysteretic loops were examined prior to performing dynamic analysis.[1]

The primary force-deformation relationship was idealized as a bilinear function. Figure 2 illustrates a moment-rotation curve obtained from tests on a reinforced concrete flexural member under monotonic loading. A bilinear idealization of the curve, used in the computer model is also shown in Fig. 2. The first line segment represents effective stiffness in the elastic range. The second line segment represents post yield stiffness.

Hysteretic loops were simulated using a degrading stiffness model incorporating effects of axial force-flexure interaction. Reinforced concrete members generally exhibit degrading stiffness properties when subjected to load cycles. Degradation in stiffness occurs during unloading and reloading. This behavior was modeled by Takeda[3] and was adopted here with some minor changes. The general form of Takeda's model is shown in Fig. 3.

Takeda's model was developed for members under constant axial force. However, coupled walls generally undergo substantial changes in level of axial force during response to earthquake motions. Because of this continuous change in axial force and the interaction between axial force and bending moment, the yield moment changes continuously. This interaction not only alters the initial yield level, but also affects the effective stiffness of the structure in the post-yield range. Therefore, the basic degrading stiffness model was modified for

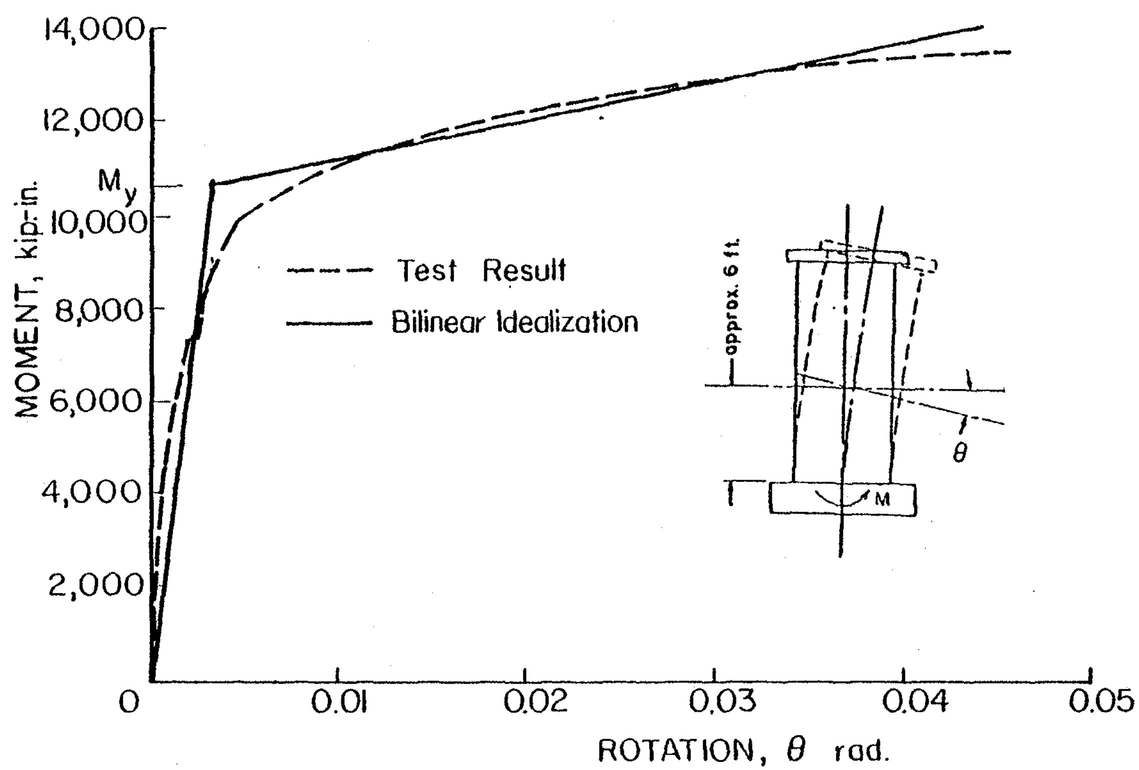

Fig. 2 Bilinear Idealization of Moment-Rotation Relationship

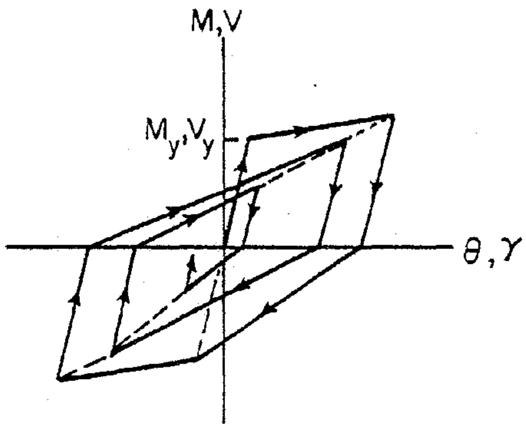

Fig. 3 Takeda's Hysteretic Loop

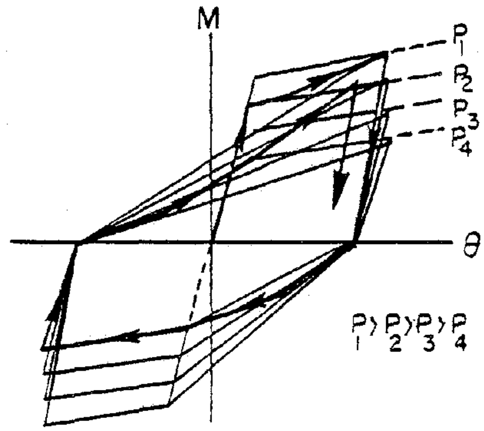

Fig. 4 Hysteretic Loop Under Changing Axial Forces

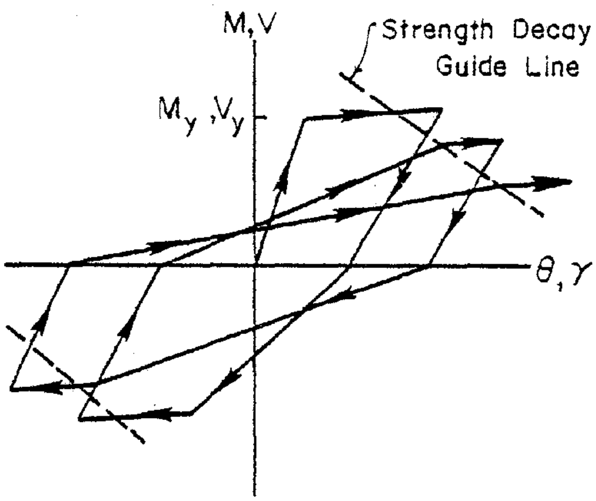

Fig. 5 Hysteretic Loop Showing Strength Decay

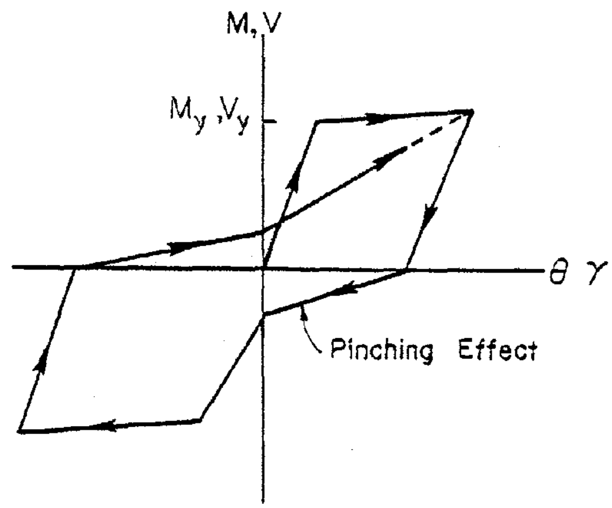

Fig. 6 Pinching of Hysteretic Loop

this investigation as shown in Fig. 4. Details are discussed in Ref. 1.

Other features of force-deformation loops were also modeled and tested. Figures 5 and 6 show "strength decay" and "pinching" effects under reversed load cycles. Significance of these modeling features on dynamic response of coupled walls was investigated in the early part of this project.[1]

A simplified inelastic shear model was used to investigate the effect of shear yielding on dynamic response. Shear force-shear distortion relationship was modeled on the basis of Takeda's rules.[1,4] The significance of shear yielding was investigated prior to the parametric studies.[1]

Based on the results of the initial phase of this investigation, reported in Ref. 1, degrading stiffness model with axial force effects was used in this part of the investigation. The model used for moment-rotation hysteresis loops is shown in Fig. 4. Elastic shear-distortion and axial force-axial deformation relationships were employed throughout this part of the investigation.

Rotational Ductility Factor

Ductility can be defined as the ratio of maximum deformation to yield deformation. In this investigation rotational ductility factor was used as a measure of inelasticity in members. Rotational ductility factor is defined as:

$$\mu_r = \frac{\theta_{max}}{\theta_y}$$

where θ_{max} is maximum rotation and θ_y is yield rotation. Usually rotations referred to here are rotations of the hinging region.

In multistory structures, width of walls is generally equal to or greater than floor-to-floor height. If the hinging region height is approximately equal to the wall width[5,6] then an entire wall element between floors forms part of the hinging region. Therefore, in the case of wall elements, it is appropriate to consider ductility in terms of the entire element. Total rotation in the element is given by the sum of chord rotations at both ends of the element, as shown in Fig. 7a.

The same basic definition of ductility is used for coupling beams. However, because coupling beams are generally bent in double curvature, as compared to single curvature prevalent in walls, a slightly different method is used in calculating ductility. In a coupling beam bent in an antisymmetrical mode, hinges can form at each end but rotate in opposite directions. Because of this, hinge lengths in coupling beams are generally limited in extent to one-half the span. In this case, ductility at one end is based on chord rotation at that end rather than the sum of the chord rotations at both ends used for walls. This can be seen in Fig. 7a.

Further clarification of the definition of ductility for coupled wall structures may be in order due to the nature of hysteretic loops for this kind of structure. Because of the coupling action between the walls, magnitudes of axial forces at yielding and at the point of maximum rotation may be dif-

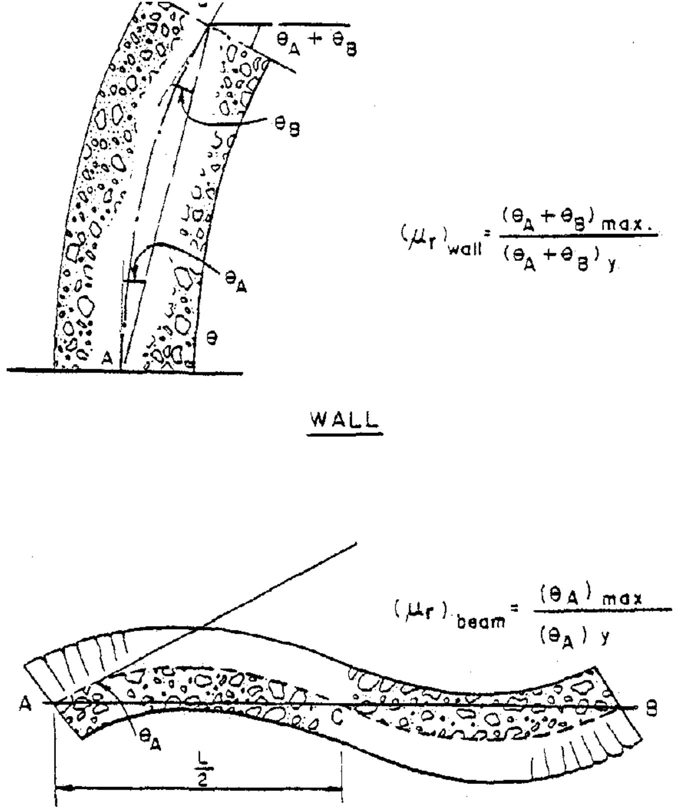

Fig. 7a - Rotations Used in Defining Ductility Factors

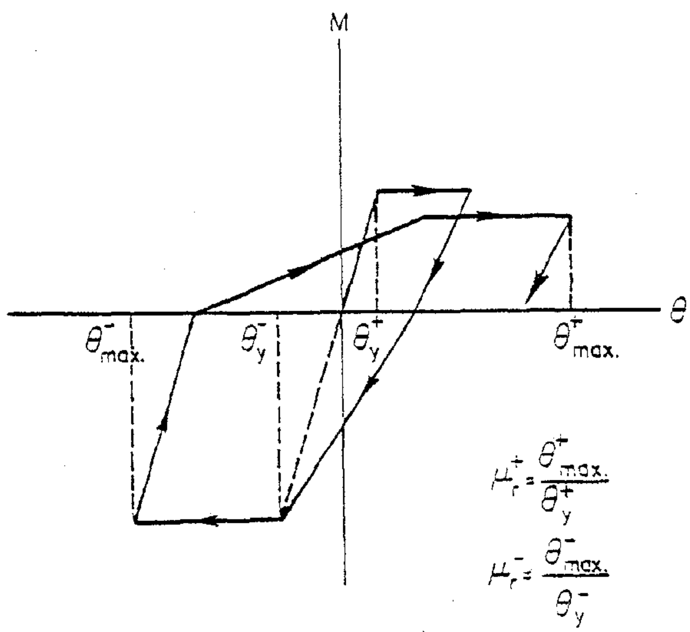

Fig. 7b - Definition of Rotational Ductility Factors ferent. In this investigation, yield rotation corresponding in sign to the maximum rotation is used in defining ductility, irrespective of corresponding axial force levels. Thus, if maximum rotation is positive μ_r^+ is used to calculate the ductility ratio. This is illustrated in Fig. 7b.

Another feature of the hysteretic loop for coupled walls is the loss of symmetry in wall behavior under reversed loading. Thus, axial force in a coupled wall can be tensile when bending in one direction. Because yield moment of a section changes with magnitude of the concurrent force, different values of yield moment and rotation generally result for each direction. In this investigation, the ductility factor is based on the maximum and yield rotations in the same direction. This is done even if first yield occurs in one direction and maximum rotation is recorded in the opposite direction, although this case rarely occurs. Usually both initial yield and maximum rotation occur while loading in the same direction. Maximum rotation generally occurs during the "tension phase" when the flexural yield level of the wall is reduced due to tension.

EFFECTS OF STRUCTURAL PARAMETERS ON DYNAMIC RESPONSE

General

A parametric study was conducted to investigate effects of structural parameters on dynamic inelastic response of coupled walls. Structural parameters that affect the modeling of a structure for dynamic analysis, and modeling for the hysteretic force deformation relationship were investigated in the initial phase of this project.[1] The following structural parameters are considered in this report:

1. Initial fundamental period of structure
2. Beam-to-wall stiffness ratio
3. Beam-to-wall strength ratio
4. Wall flexural strength
5. Coupling arm between the walls
6. Clear beam span
7. Mass for inertia forces
8. Effect of initial gravity loads

A 20-story coupled wall structure with initial fundamental period 1.8 sec was selected for the parametric investigation. This height of structure was considered reasonably representative of a majority of multistory structures. The floor plan was chosen to be symmetric in both direction as shown in Fig. 8. Symmetry is generally desirable to minimize torsional effects, which are beyond the scope of this investigation.

Columns that were placed at alternate bays in the long direction were assumed to carry vertical loads only. Resistance

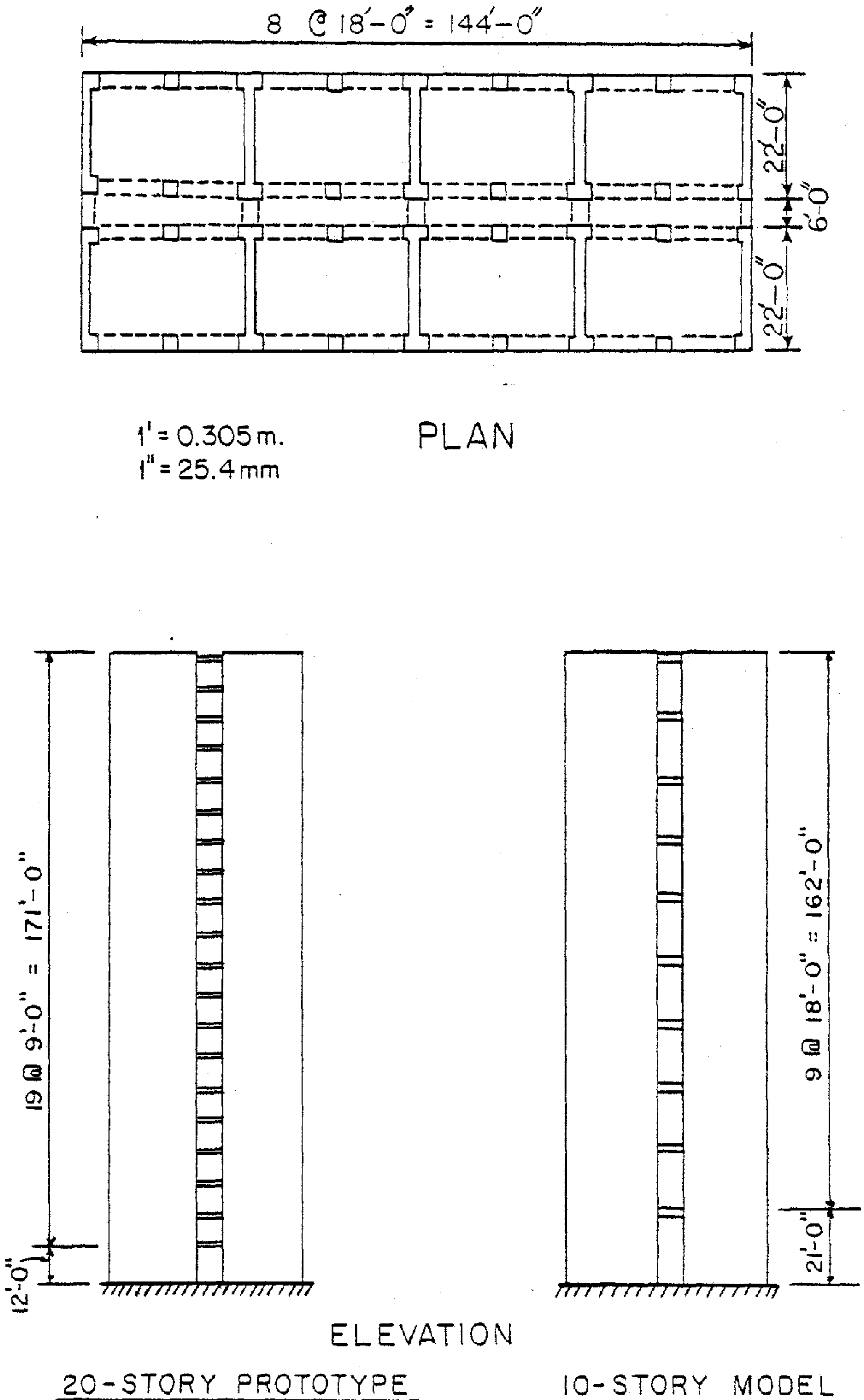

Fig. 8 Coupled Wall Structure Selected for Investigation

to lateral loads was assumed to be provided entirely by walls. Floor slabs were considered sufficiently stiff to cause all points on the same floor level to deflect by equal amounts horizontally. Ground motion and structural properties of members are listed in Table 1. Those properties that were varied for investigation of each parameter are discussed separately in respective sections.

The 20-story prototype structure was lumped vertically to be reduced to a 10-story model in order to reduce the computer time required for analysis. Properties of the 10-story model can be obtained from Table 1. Response envelopes for the 20-story prototype and the 10-story model under dynamic forces are compared in Figs. 9 and 10. Results show good agreement. Therefore, the reduced model was used throughout the dynamic analysis.

Three potentially critical input motions were chosen for the particular structure considered on the basis of their velocity response spectra. Dynamic analyses were conducted using these three input motions. Response envelopes are compared in Figs. 11 and 12. The 1940 El Centro, E-W record was selected for use with structure having fundamental period of 1.8 sec. Earthquake duration used was 10 sec except when the investigated parameter was earthquake duration.

The integration time step used in dynamic analyses was 0.01 sec. This time step was found sufficiently short to produce accurate results for the structure period considered.

TABLE 1 - PROPERTIES OF STRUCTURES SELECTED FOR PARAMETRIC INVESTIGATION

Fundamental Period	1.8 sec
Number of Stories	20
Height	183 ft
Wall Width	22 ft
Clear Beam Span	6 ft
Wall Stiffness Parameters:	
EI (million k-in.^{2})	38,600
GA (million kips)	1.13
EA (million kips)	6.70
Stiffness Taper[1]	1.00 EI at base
(Step Variation)	0.80 EI at 6th floor
	0.65 EI at 12th floor
Beam Stiffness Parameters[2]:	
EI (million k-in.^{2})	26.0
GA (million kips)	0.091
EA (million kips)	0.85
Wall Yield Moment, M_y	400,000 k-in.
Strength Taper[3]	1.00 M_y at base
(Step Variation)	0.75 M_y at 6th Floor
	0.50 M_y at 12th Floor
Beam Yield Moment[2]	4,000 k-in.
Post-Yield Stiffness on	5% of elastic for walls
Primary curve	7% of elastic for beams
Weight	2533 k/wall
Weight for Inertia Forces	4565 k/wall
Damping	5% of critical
Base Fixity Condition	fully fixed
Ground Motion	El Centro 1940, E-W
Intensity of Ground Motion[4]	1.5 El Centro 1940, N-S

Notes:
1 The same taper also applies to "GA" and "EA"
2 Stiffness parameters and beam yield moment must be multiplied by 2.0 to obtain values for 10-story model
3 Yield moments are also adjusted at every floor on the basis of weight
4 Based on spectrum intensity

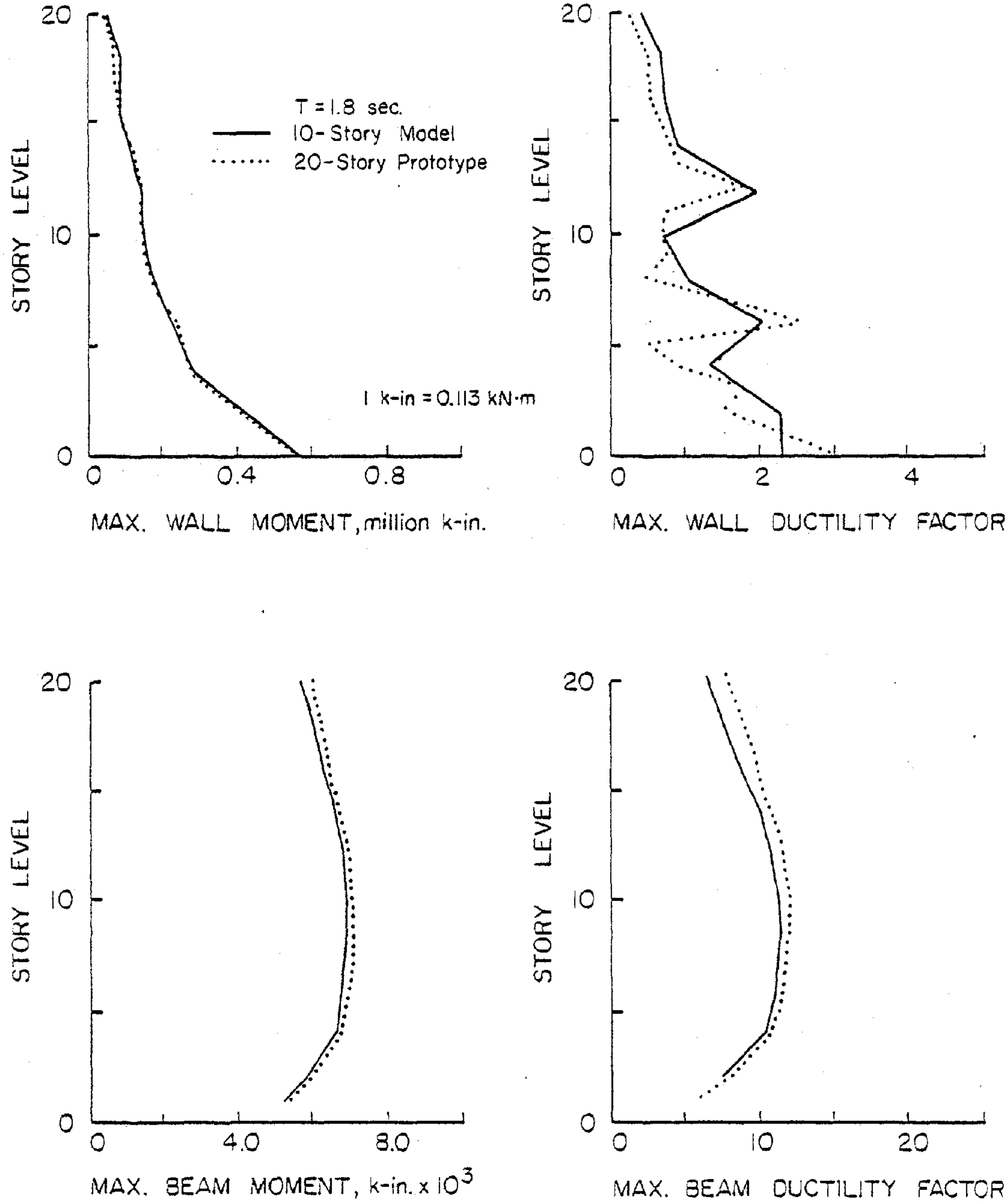

Fig. 9 Comparison of Moment and Ductility Envelopes for 20-Story Prototype and 10-Story Model Under Dynamic Excitation

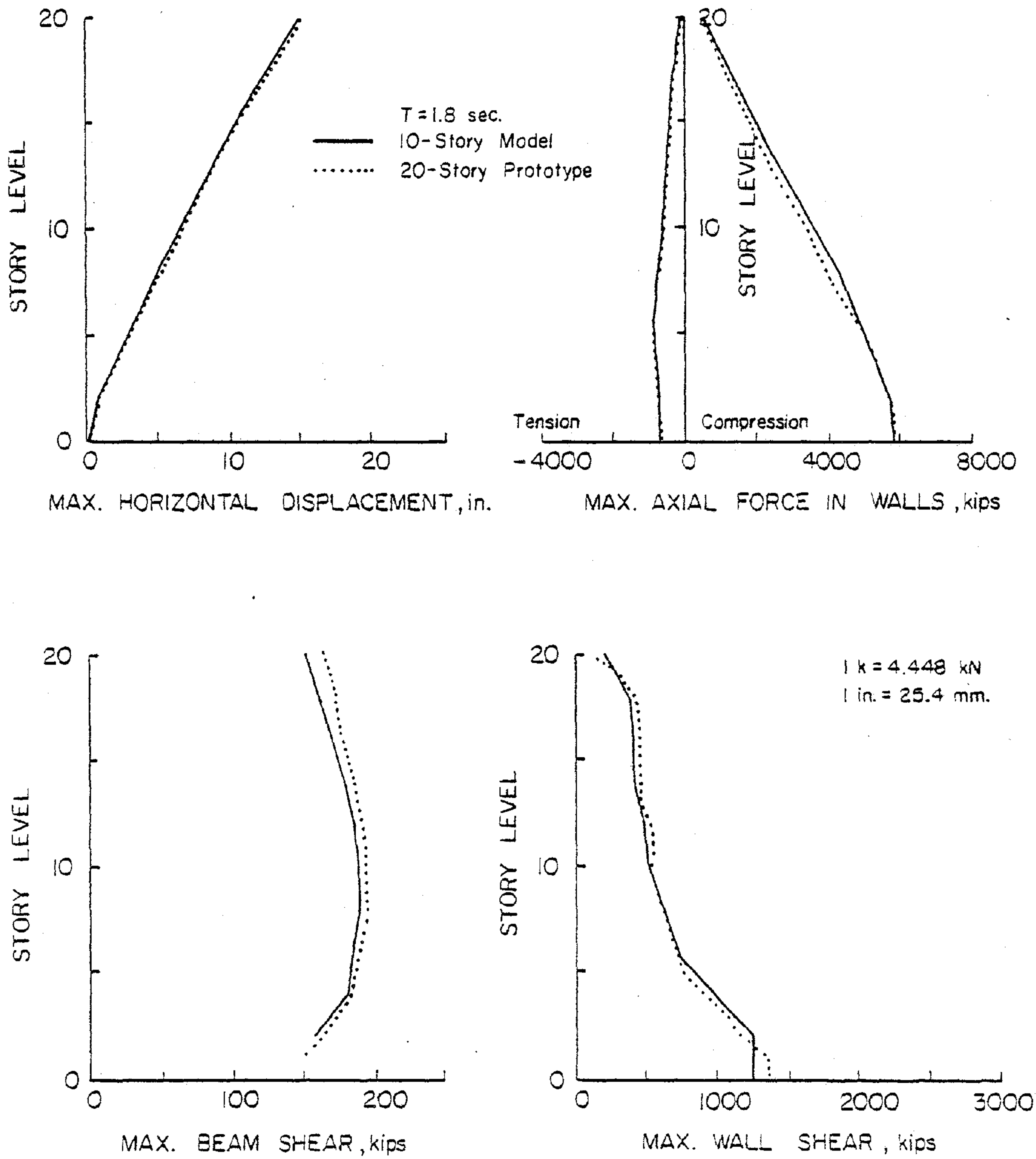

Fig. 10 Comparison of Force and Displacement Envelopes for 20-Story Prototype and 10-Story Model Under Dynamic Excitation

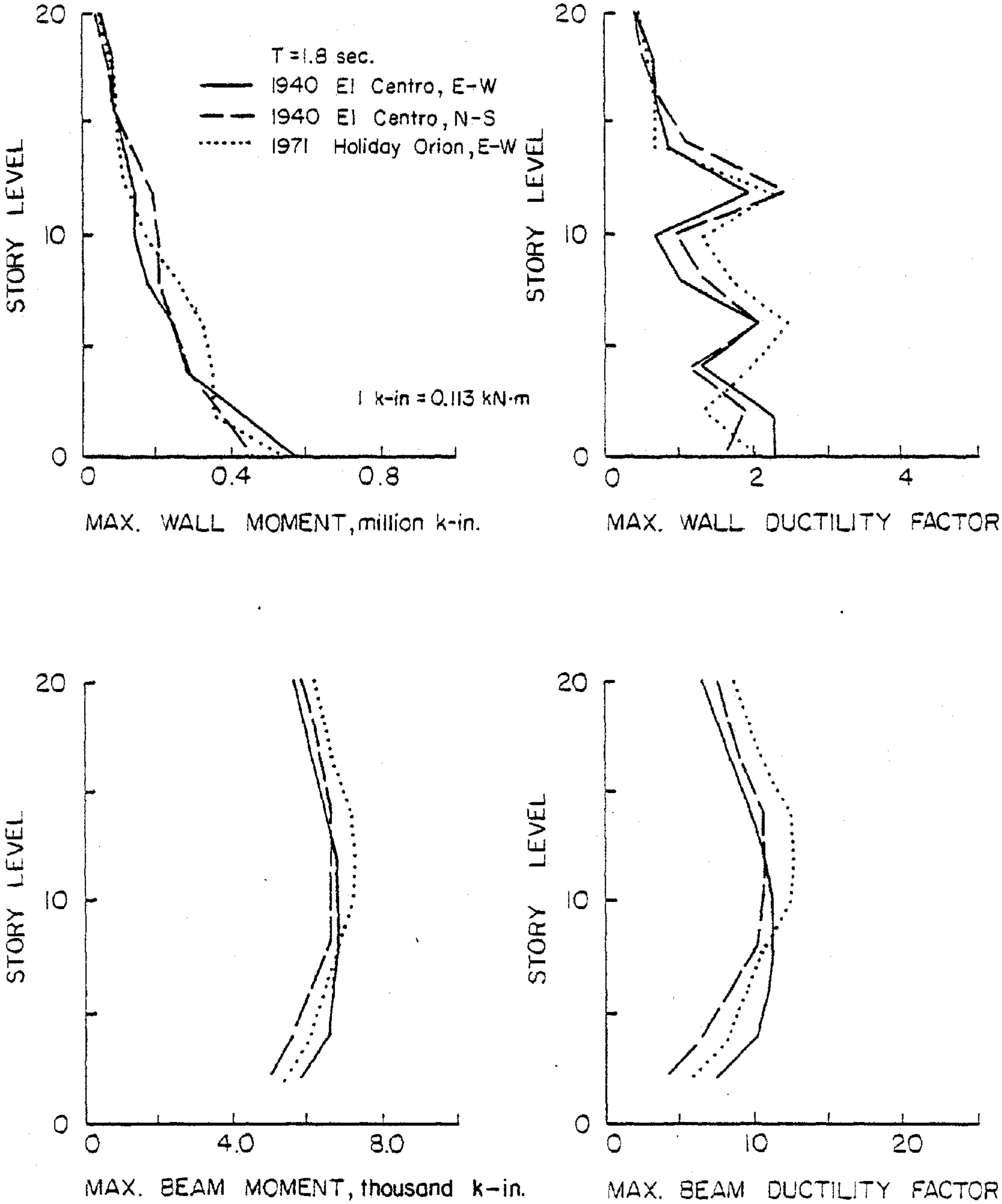

Fig. 11 Moment and Ductility Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.8 sec)

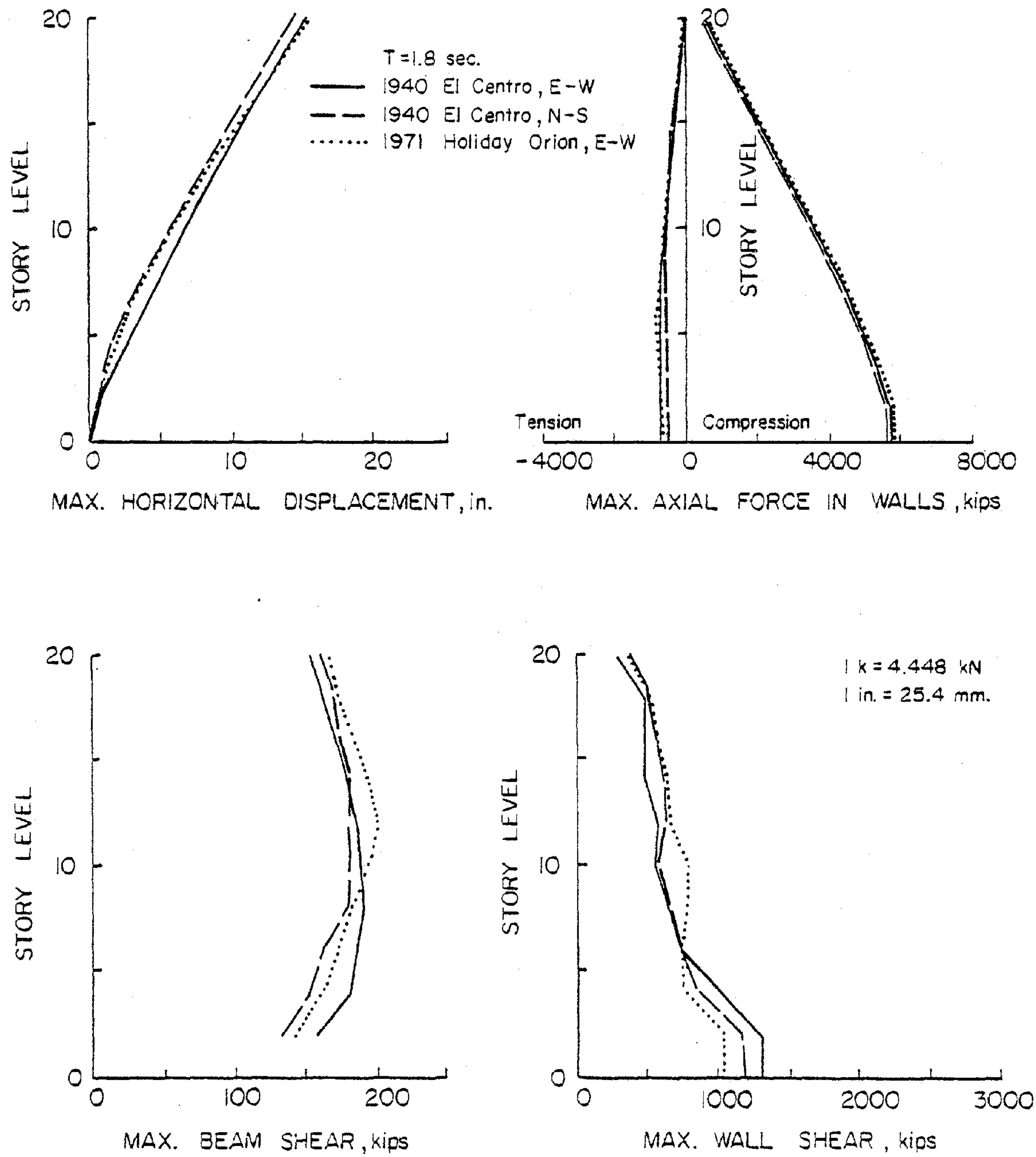

Fig. 12 Force and Displacement Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.8 sec)

Effect of Fundamental Period

In this investigation, a bilinear relationship was used to define the primary force-deformation relationship. This implies that constant stiffness was assigned to each member up to yielding. The elastic stiffness specified includes the effect of softening in members due to cracking.

The effect of initial fundamental period on dynamic response of coupled walls was investigated. Initial fundamental period decreases as structure stiffness increases. The previously selected 20-story structure was analyzed with three different fundamental periods. Particular values covered are tabulated in Table 2. Stiffness properties of each structure were selected to provide the same beam-to-wall stiffness ratio for each structure.

Response envelopes for the three structures considered are compared in Figs. 13 and 14. Results indicate a consistant increase in maximum horizontal displacement with increasing fundamental period. Rotational ductility factors increase with decreasing fundamental period. This can be explained by a decrease in yield rotation caused by increase in stiffness while yield moment remains the same, and a consequent increase in the ratio of maximum to yield rotations. Absolute values of maximum rotation were observed to be higher for structures with higher fundamental period (flexible structure). Shear force and bending moment envelopes, on the other hand, are not significantly affected by changes in fundamental period. This can be attributed to a relatively flat post yield slope of

TABLE 2 - PROPERTIES OF STRUCTURES WITH DIFFERENT FUNDAMENTAL PERIODS

Properties[1]	Structure		
	I	II	III
Fundamental Period (sec)	1.2	1.8	2.4
Wall Stiffness Parameters[2]:			
EI (million k-in.^{2})	77,200	38,600	19.800
GA (million kips)	2.26	1.13	0.58
EA (million kips)	13.40	6.70	3.44
Beam Stiffness Parameters:			
EI (million k-in.^{2})	52.0	26.0	13.4
GA (million kips)	0.182	0.091	0.047
EA (million kips)	1.70	0.85	0.435
Beam-to-wall			
Stiffness Ratio,[3] R_{SF}	0.0168	0.0168	0.0168
Wall Yield Moment[2] (k-in.)	600,000	600,000	600,000
Beam Yield Moment (k-in.)	6,000	6,000	6,000

Note:

1. For all other properties see Table 1.
2. Wall stiffness parameters and yield moment given are at structure base. For stiffness and strength taper see Table 1.
3. $R_{SF} = (\frac{kEI}{L})_{BM}/(\frac{kEI}{L})_{W} = 25\ (EI)_{BM}/(EI)_{W}$

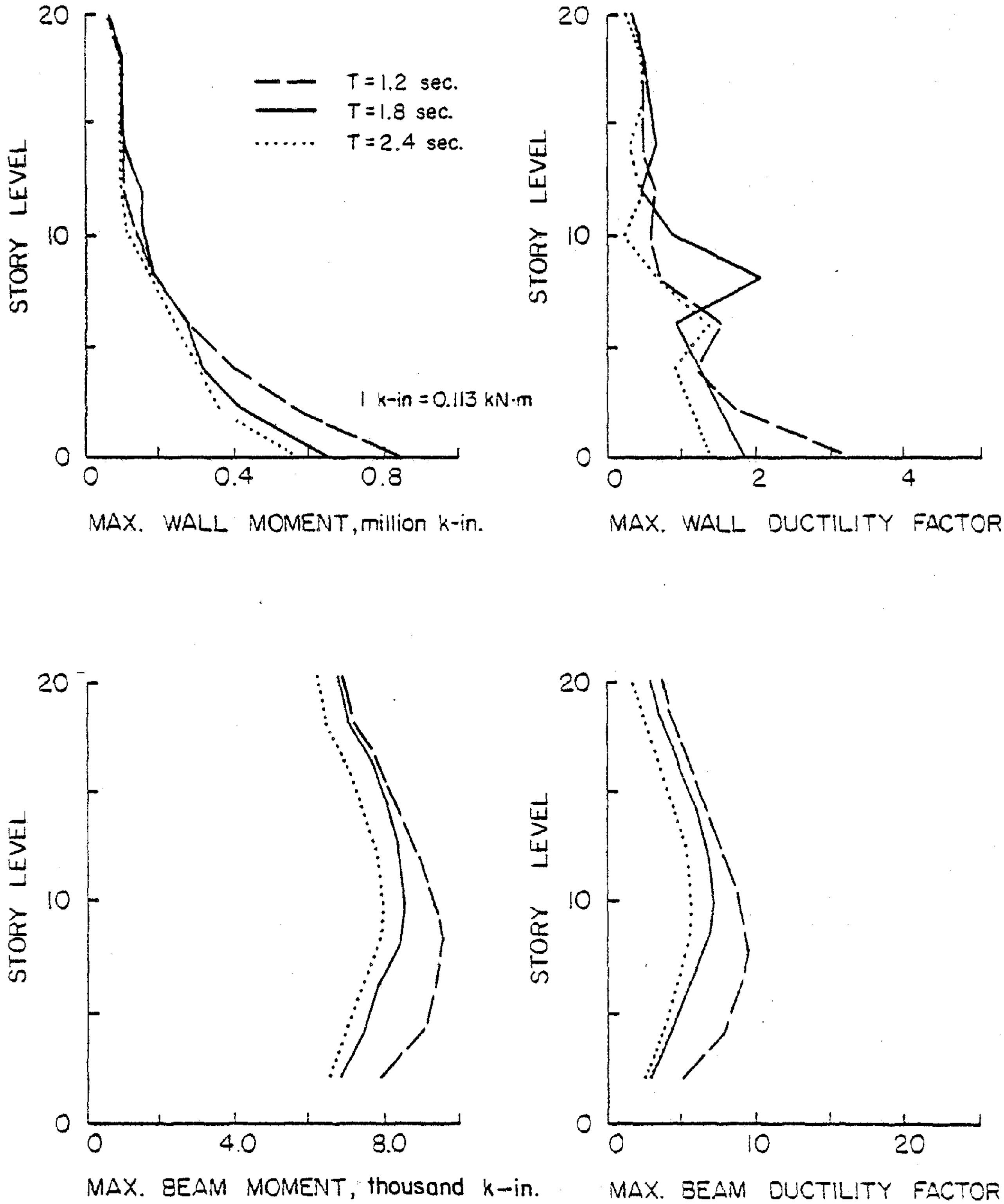

Fig. 13 Moment and Ductility Envelopes Showing Effect of Fundamental Period

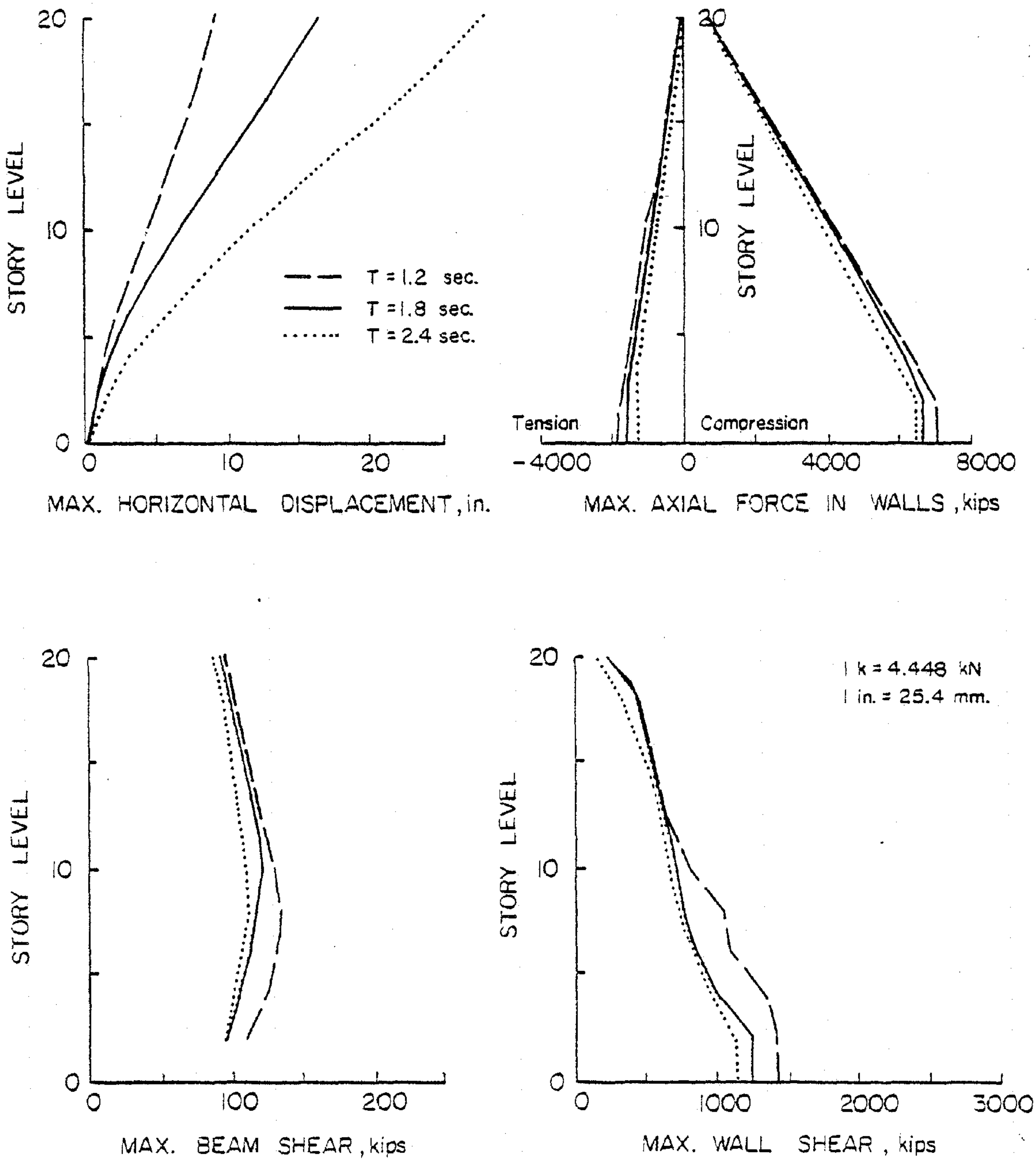

Fig. 14 Force and Displacement Envelopes Showing Effect of Fundamental Period

force-deformation relationship. Slightly higher response is obtained with increasing stiffness.

It can be concluded from the above comparison that the fundamental period of a structure plays a significant role in nonlinear response to seismic excitation. Therefore, design procedures for earthquake-resistant design should include fundamental period as a design variable.

Beam-to-Wall Stiffness Ratio

For a given story height, the stiffness of coupling beams depends primarily on the size of openings between the walls. Relative stiffness of members affects distribution of forces at each joint and response of the structure.

A series of dynamic analyses was carried out to investigate the effect of beam-to-wall stiffness ratio on structural response. Particular cases considered are listed in Table 3. All structures had the same fundamental period but different ratios of beam-to-wall stiffness.

Variation in beam stiffness while leaving wall stiffness constant, to achieve different beam-to-wall stiffness ratio, leads to variation in fundamental period. Therefore, it was necessary to vary both the beam and the wall stiffnesses to arrive at different stiffness ratios while maintaining the same fundamental period. Consequently, the results of the analyses indicate not only the effect of stiffness ratio but also the effect of the overall stiffness of the walls.

Figure 15 shows combinations of beam and wall stiffnesses that lead to the same fundamental period, $T = 1.8$ sec. In this

TABLE 3 - PROPERTIES OF STRUCTURES WITH DIFFERENT BEAM-TO-WALL STIFFNESS RATIO

Properties[1]	Structures					
	I	II	III	IV	V	VI
Fundamental Period (sec)	1.8	1.8	1.8	1.8	1.8	1.8
Wall Stiffness Parameters[2]:						
EI (million k-in.^{2})	56,200	38,600	35,000	34,600	33,100	33,100
GA (million kips)	1.64	1.13	1.03	1.01	0.97	0.97
EA (million kips)	9.68	6.70	6.08	6.00	5.74	5.74
Beam Stiffness Parameters:						
EI (million k-in.^{2})	6.3	26.0	43.5	54.0	90.0	134.1
GA (million kips)	0.022	0.091	0.152	0.188	0.314	0.467
EA (million kips)	0.213	0.85	1.42	1.755	2.925	4.355
Beam-to-Wall						
Stiffness Ratio,[3] R_{SF}	0.0028	0.0168	0.030	0.039	0.068	0.100
Wall Yield Moment[2] (k-in.)	400,000	400,000	400,000	600,000	600,000	600,000
Beam Yield Moment (k-in.)	4,000	4,000	4,000	6,000	6,000	6,000

Note:

1. For all other properties see Table 1
2. Wall stiffness and yield moment given are at structure base. For stiffness and strength taper along structure height see Table 1.
3. $R_{SF} = (\frac{kEI}{L})_{BM}/(\frac{kEI}{L})_W = 25(EI)_{BM}/(EI)_W$

figure it can be seen that in the higher range of wall stiffness, any increase in beam stiffness has to be compensated by a significant reduction in wall stiffness to keep the fundamental period constant. In the lower range of wall stiffness, change in beam stiffness does not appear to have an appreciable effect on fundamental period. This implies that below a certain value of wall stiffness, increase in beam stiffness does not affect the initial fundamental period of a coupled wall structure.

Structures I, II, and III in Table 3 are in the higher range of wall stiffness. Their response envelopes are compared in Figs. 16 and 17. Generally, stiff members show high force and ductility response. This can be observed in beam moment, shear and ductility envelopes. Structures with high beam-to-wall stiffness ratio, K_{SF} have high beam stiffness and therefore show high force and ductility response. Wall response envelopes are affected by change in wall stiffness as well as variation in axial forces in walls. Axial forces increase with increase in relative beam stiffness. This follows from the greater coupling present in structures with higher beam-to-wall stiffness ratio. Reduction in wall moment and shear that might be expected due to reduced wall stiffness is compensated to some degree by the increase in coupling due to high beam-to-wall stiffness ratio. As a result, wall moment and shear envelopes shown in Figs. 16 and 17 do not differ significantly. Wall ductilities also reflect the two simultaneous effects. The first effect is the decrease in ductility with reduced wall stiffness. The second effect is the increase in ductility due

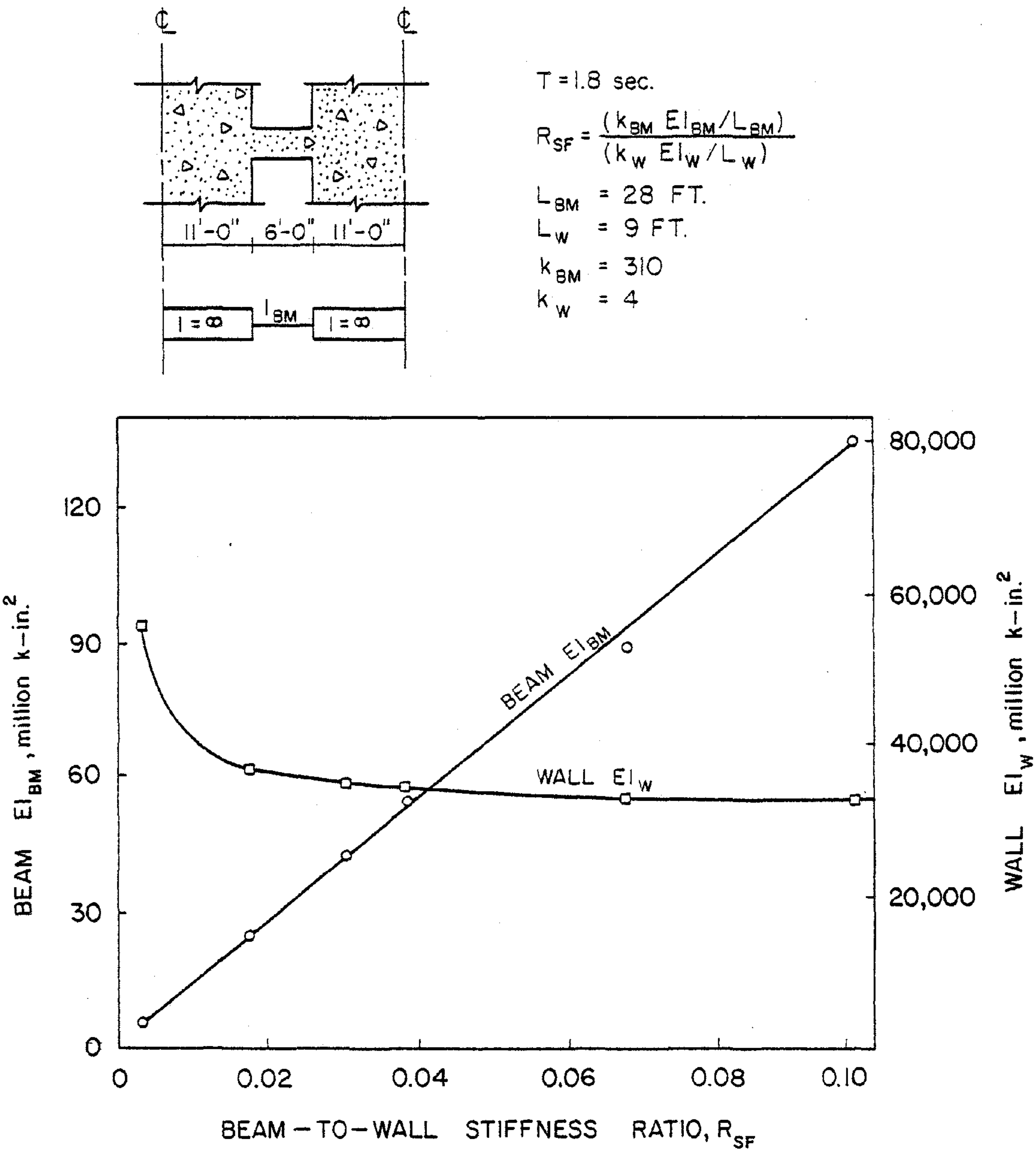

Fig. 15 Combination of Wall and Beam Stiffness Leading to Fundamental Period, T = 1.8 sec

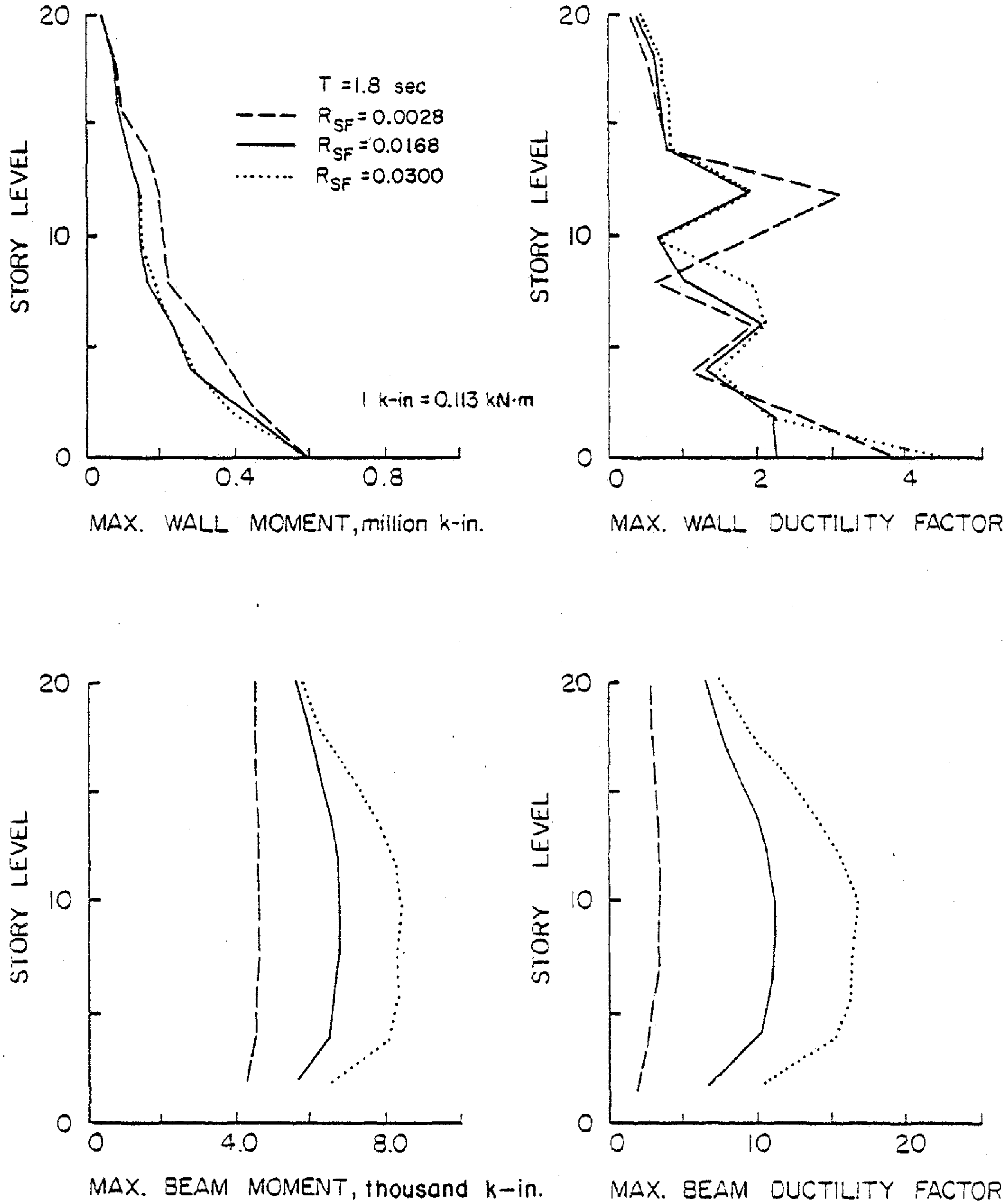

Fig. 16 Moment and Ductility Envelopes Showing Effect of Beam-to-Wall Stiffness Ratio (High Wall Stiffness Range)

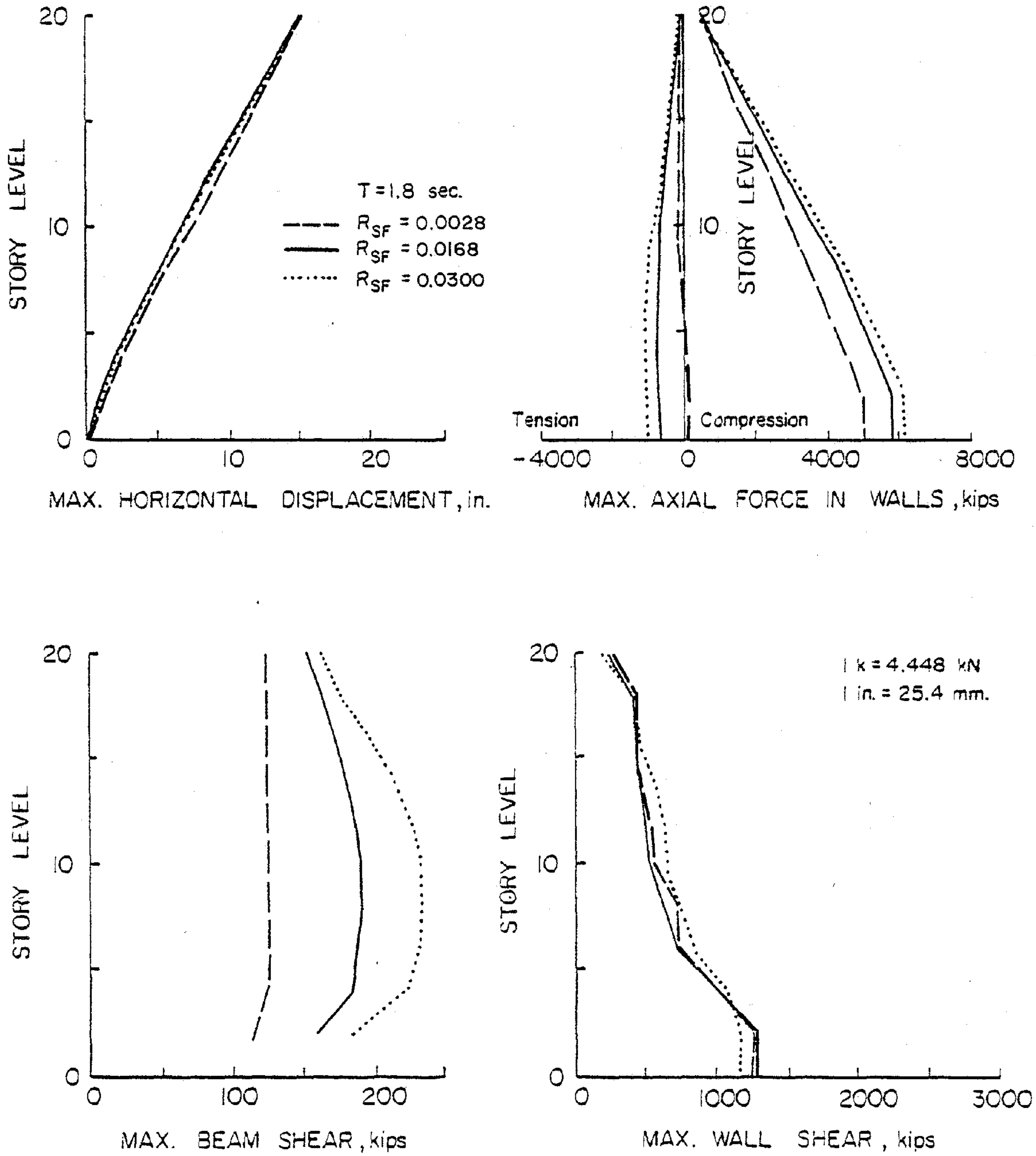

Fig. 17 Force and Displacement Envelopes Showing Effect of Beam-to-Wall Stiffness Ratio (High Wall Stiffness Range)

to high axial tension that results from strong coupling caused by stiff beams.

In Fig. 16, the structure with beam-to-wall stiffness ratio, $R_{SF} = 0.030$, shows the highest wall ductility at the base due to the increased axial tension caused by strong coupling of stiff beams. On the other hand, the structure with $R_{SF} = 0.0168$ also shows high ductility which can be explained by its relatively high wall stiffness. Higher member stiffness under constant yield moment leads to lower yield rotation, and hence higher ductility ratio.

Structures IV, V, and VI in Table 3 are in the low wall stiffness and high beam stiffness range. These structures were first analyzed with the same beam and wall yield moments as the previous set. Results showed excessive coupling. Variation in the level of axial force was high enough to cause either yielding in walls due to pure axial tension or concrete crushing due to high axial compression. Therefore, the same three structures were analyzed with increased yield moments as indicated by Table 3.

Response envelopes for the second set of analyses are compared in Figs. 18 and 19. The same general trend observed in the first set can also be seen in these figures. Structures with stiff beams showed high beam force and beam ductility response. Wall response was not affected signicantly by variation in beam-to-wall stiffness ratio. Wall ductilities were expected to increase with increased axial tension due to high coupling

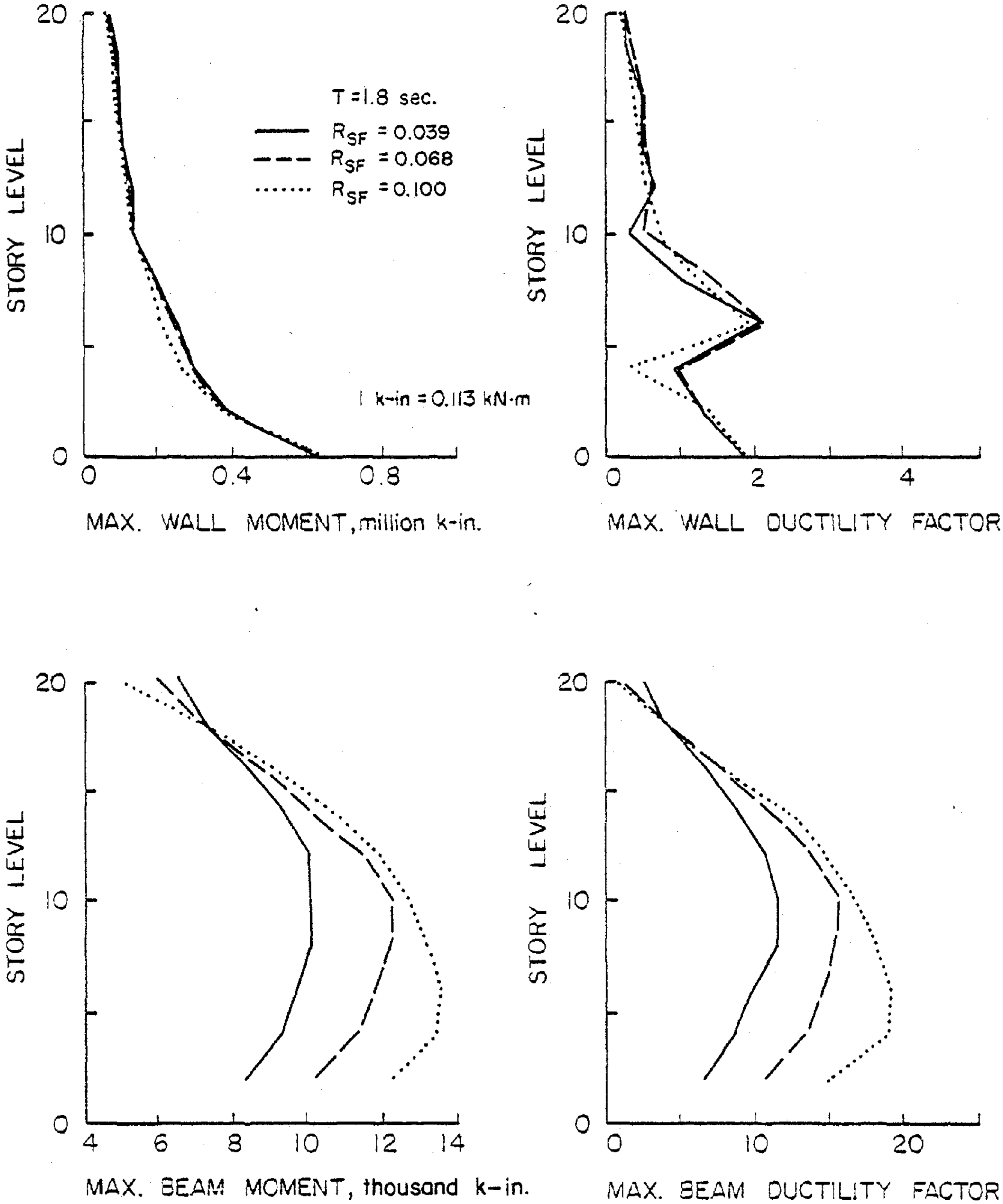

Fig. 18 Moment and Ductility Envelopes Showing Effect of Beam-to-Wall Stiffness Ratio (Low Wall Stiffness Range)

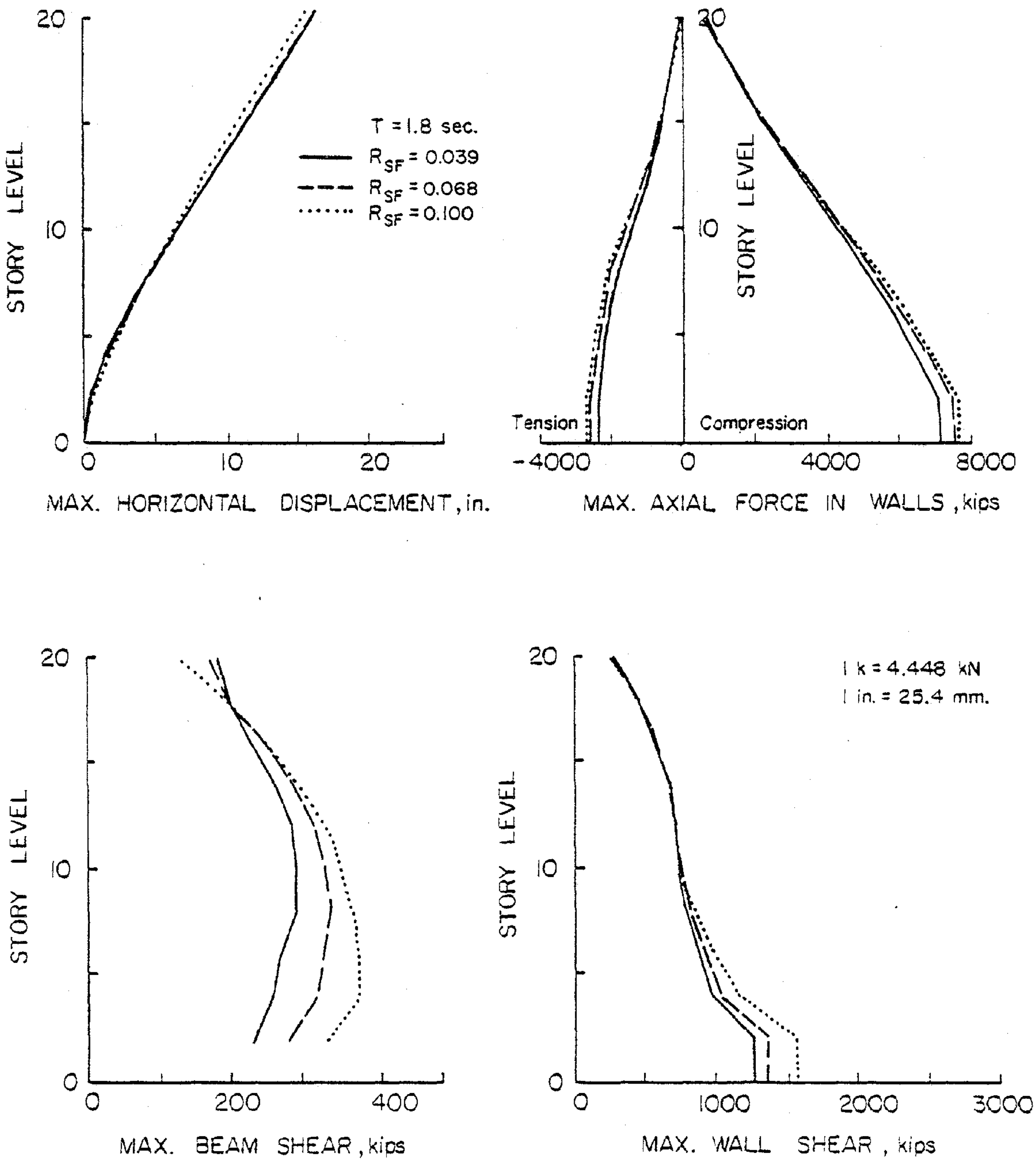

Fig. 19 Force and Displacement Envelopes Showing Effect of Beam-to-Wall Stiffness Ratio (Low Wall Stiffness Range)

caused by a high beam-to-wall stiffness ratio. However, ductility response envelopes did not show any appreciable difference at the wall base. This can be explained by reduced inelasticity that resulted from the high level of yield moment used in this set of analyses.

Results indicate that the beam-to-wall stiffness ratio has a substantial effect on nonlinear dynamic response of coupled wall structures. The magnitude of beam stiffness relative to wall stiffness is an important parameter that controls the degree of coupling. Beam moment and shear response is increased with increasing beam-to-wall stiffness ratio. This increase is also accompanied by an increase in beam ductility. Wall moment and shear envelopes are not significantly affected.
Wall ductilities can be affected through axial forces in walls depending on the yield level. Maximum horizontal displacements depend mainly on the fundamental period and therefore do not show any appreciable variation with beam-to-wall stiffness ratio.

Beam-to-Wall Strength Ratio

In the inelastic design of structures the sequence of plastification among members is extremely important. In earthquake resistant coupled walls it is generally desirable to dissipate most of the energy by yielding in beams while keeping the walls essentially elastic. Therefore, beam strength relative to wall strength plays an important role in achieving the required yielding sequence within a coupled wall structure.

To investigate the significance of beam-to-wall strength ratio, five structures were analyzed. Except for variations in beam strength the structures were identical. The particular values considered are listed in Table 4. Response envelopes for three cases are shown in Figs. 20 and 21. Structures with

TABLE 4 - PROPERTIES OF STRUCTURES WITH DIFFERENT BEAM-TO-WALL STRENGTH RATIOS

Structure	Properties [1]		
	Wall Yield[2] Moment (k-in.)	Beam Yield Moment (k-in.)	Beam-to-Wall Strength Ratio
I	400,000	2,000	0.0050
II	400,000	3,000	0.0075
III	400,000	4,000	0.0100
IV	400,000	5,000	0.0125
V	400,000	6,000	0.0150

Notes:
1 For all other properties, see Table 1
2 Yield moment is at structure base. For strength taper along the structure height, see Table 1

strong coupling beams showed lower beam ductility demands and higher wall ductility requirements. This was due mainly to strong coupling action leading to high tension in walls. In structures with strong coupling, first yielding occurred in the tension wall prior to yielding of beams because of the significant decrease in wall capacity. This early yielding in walls was accompanied by higher ductility demands. Because excessive inelastic action at the base of walls can undermine

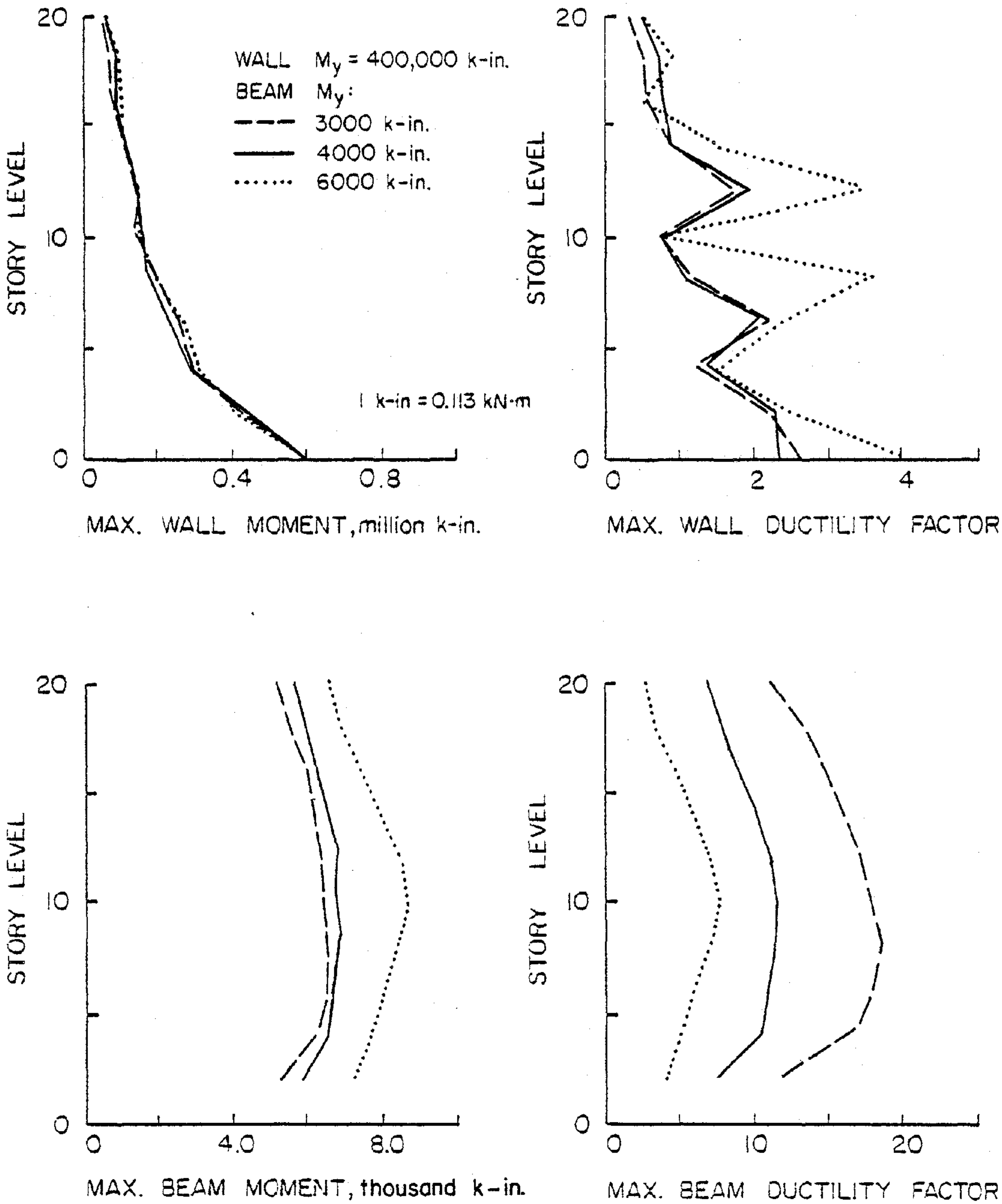

Fig. 20 Moment and Ductility Envelopes Showing Effect of Beam-to-Wall Strength Ratio

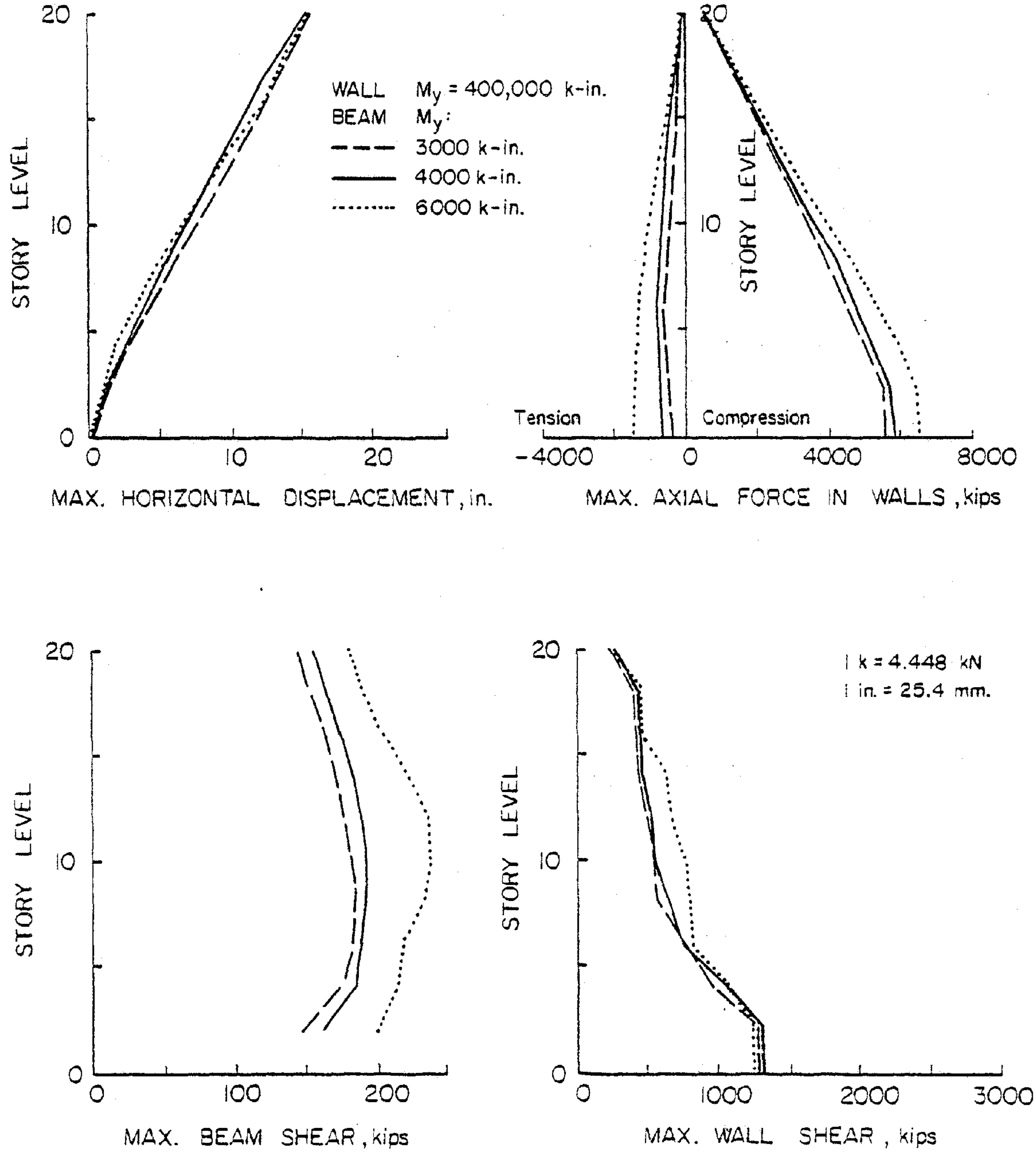

Fig. 21 Force and Displacement Envelopes Showing Effects of Beam-to-Wall Strength Ratio

their vertical load carrying capacity, this type of behavior should be avoided.

Structures with strong coupling beams exhibited undesirable behavior, with yielding occurring first in the walls. Structures with weak beams also showed undesirable behavior. In this case, although all beams yielded prior to yielding in walls, beam ductility demands were large enough to be outside the practical range.

The effect of beam strength on wall behavior can be viewed in two ways. First, the individual wall moments are reduced with increased coupling to achieve the same over-turning moment. Second, the maximum wall moments are increased due to increased compressive axial force and accompanied increase in flexural capacity.

Wall ductility demands are increased at high and low extremes of beam strength. When coupling beams are strong, increased tension and accompanying reduction in flexural capacity of walls results in high wall ductility demands. As beam strength is reduced more yielding takes place in beams and wall ductility demands are reduced. However, as beam strength is reduced further, inelastic action in beams becomes excessive and beams become ineffective as coupling members. Beyond this point, walls start acting as isolated walls. This means that overall stiffness of the system is substantially reduced. Wall displacements and ductility demands start increasing as walls continue to be exposed to ground shaking.

The analyses were expanded to cover walls with different yield moments to compile design data. Variation of maximum wall ductility at the base is shown in Fig. 22 as a function of beam-to-wall strength ratio. It should be noted in Fig. 22 that the rate of increase or decrease in ductility due to variation in beam strength changes with wall yield level. The relationship between beam strength and beam ductility, for walls having different yield levels, is shown under "Sample Design Aids" as part of the design charts.

Results of dynamic analysis indicate that for a given coupled wall structure it is possible to determine a range of beam-to-wall strength ratio within which the desired pattern of plastification occurs.

Wall Strength

Member strengths are specified in terms of yield moment levels for the purpose of dynamic analysis. Member strength has a significant effect on inelastic structural response. Elastic behavior takes place if the yield level is higher than the maximum force level that the structure can experience during response. As the yield level is reduced, increasing level of inelasticity results. The structure becomes softer following yielding. Lengthening of the fundamental period takes place. Deformations and hence ductility requirements increase.

A series of dynamic analyses was conducted on the 20-story coupled wall structure with different levels of wall yield

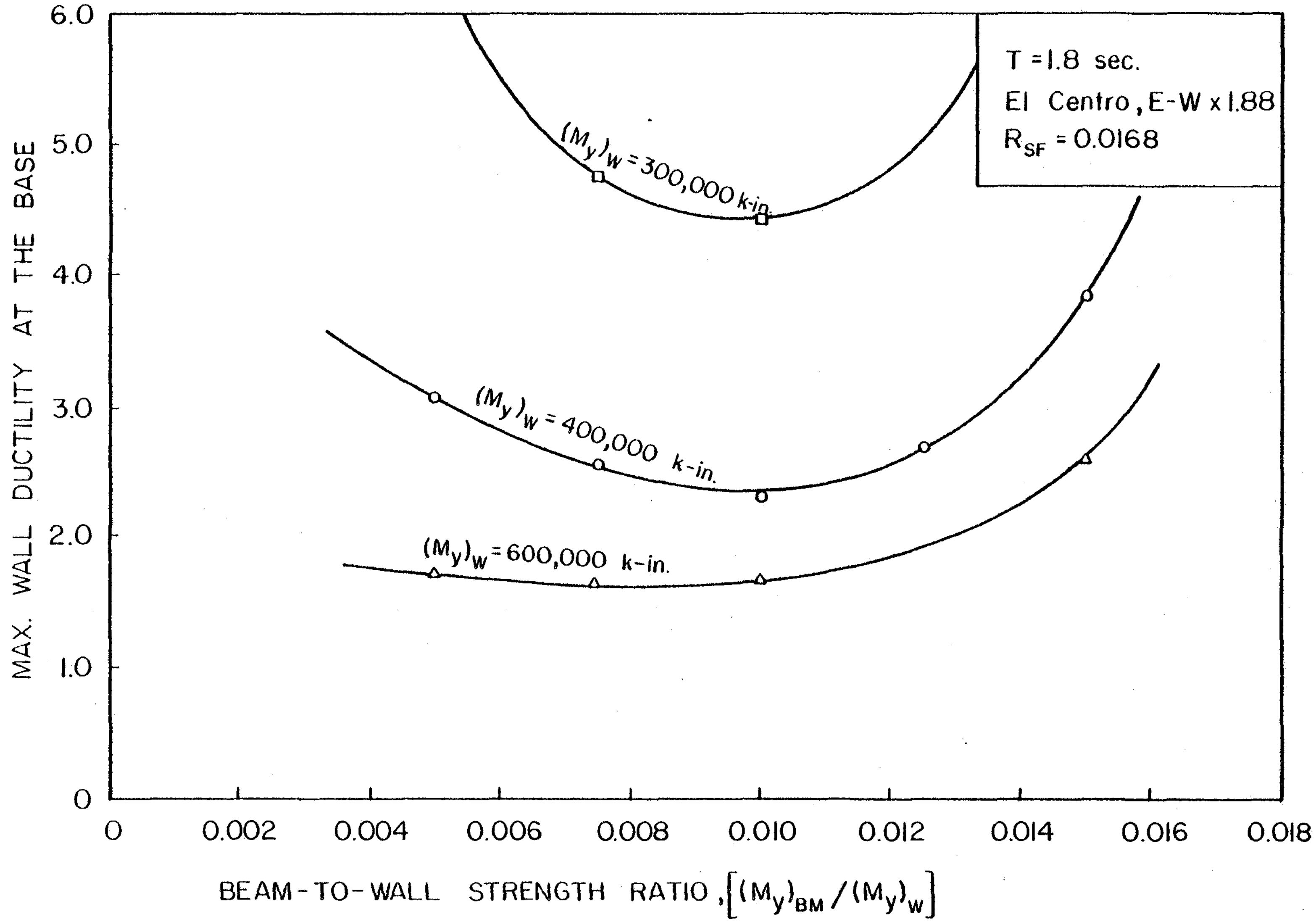

Fig. 22 Variation of Wall Ductility with Beam-to-Wall Strength Ratio

moment. In each analysis, beam yield level was adjusted so that the beam-to-wall strength ratio remained constant. Table 5 lists properties of the structures analyzed. Maximum response envelopes are compared in Figs. 23 and 24.

TABLE 5 - PROPERTIES OF STRUCTURES WITH DIFFERENT WALL STRENGTH

Structure	Properties[1]		
	Wall Yield[2] Moment (k-in.)	Beam Yield Moment (k-in.)	Beam to Wall Strength Ratio
I	300,000	3,000	0.010
II	400,000	4,000	0.010
III	500,000	5,000	0.010
IV	600,000	6,000	0.010

Notes:
1 For all other properties, see Table 1
2 Yield moment is at structure base. For strength taper along the structure height, see Table 1

Results of the analyses confirm the expected behavior. Early yielding in walls and beams increases ductility requirements. Maximum moments and shears in walls do not show signicant variation with yield level. However, beam force envelopes show significant variation with yield level. Although early yielding in the structure increases the period of the structure, maximum horizontal displacements appear to be dependent on the initial fundamental period and do not vary significantly with yield level.

Interaction of wall yield level with other design parameters is discussed under "Design Information."

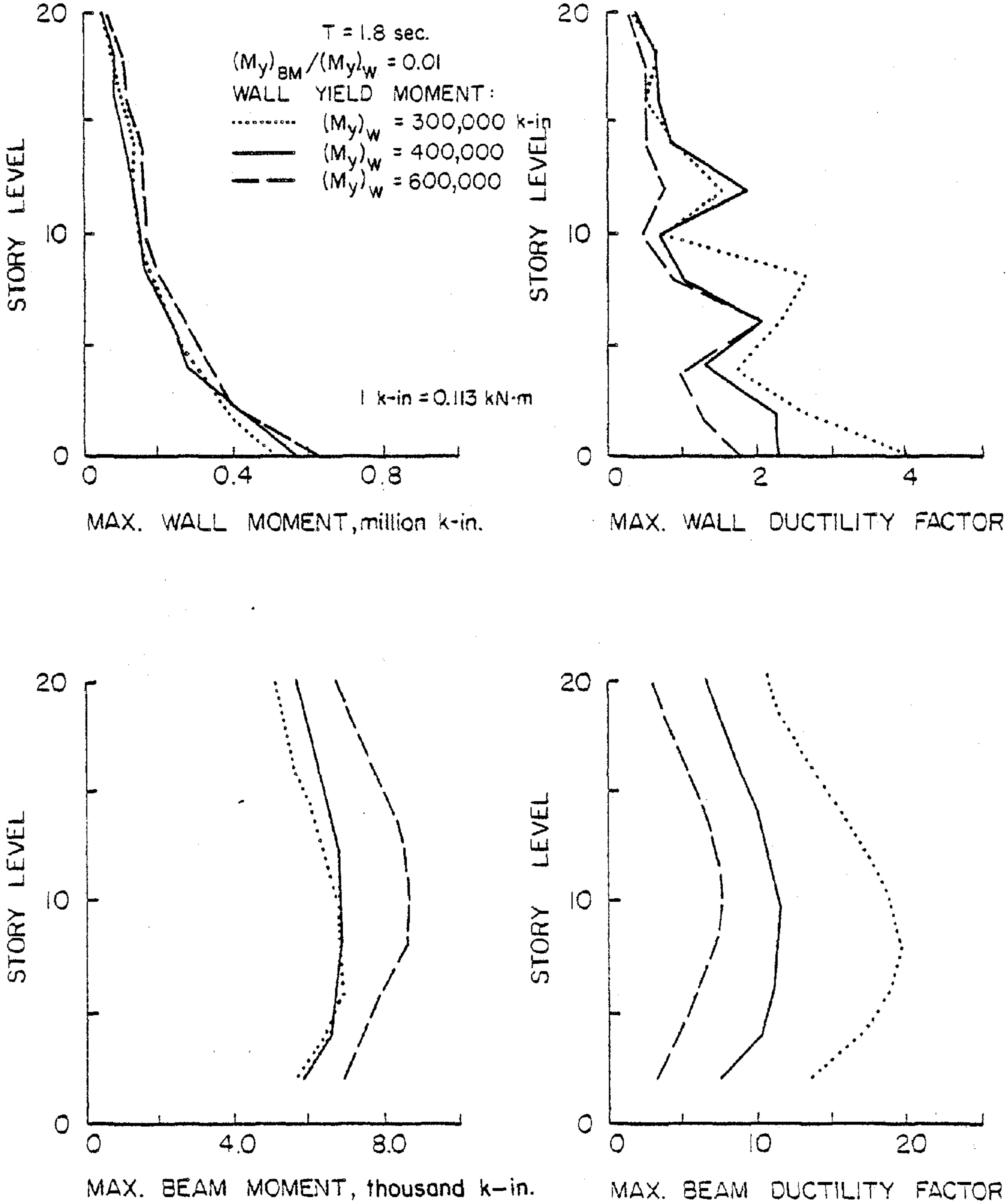

Fig. 23 Moment and Ductility Envelopes Showing Effect of Wall Yield Moment

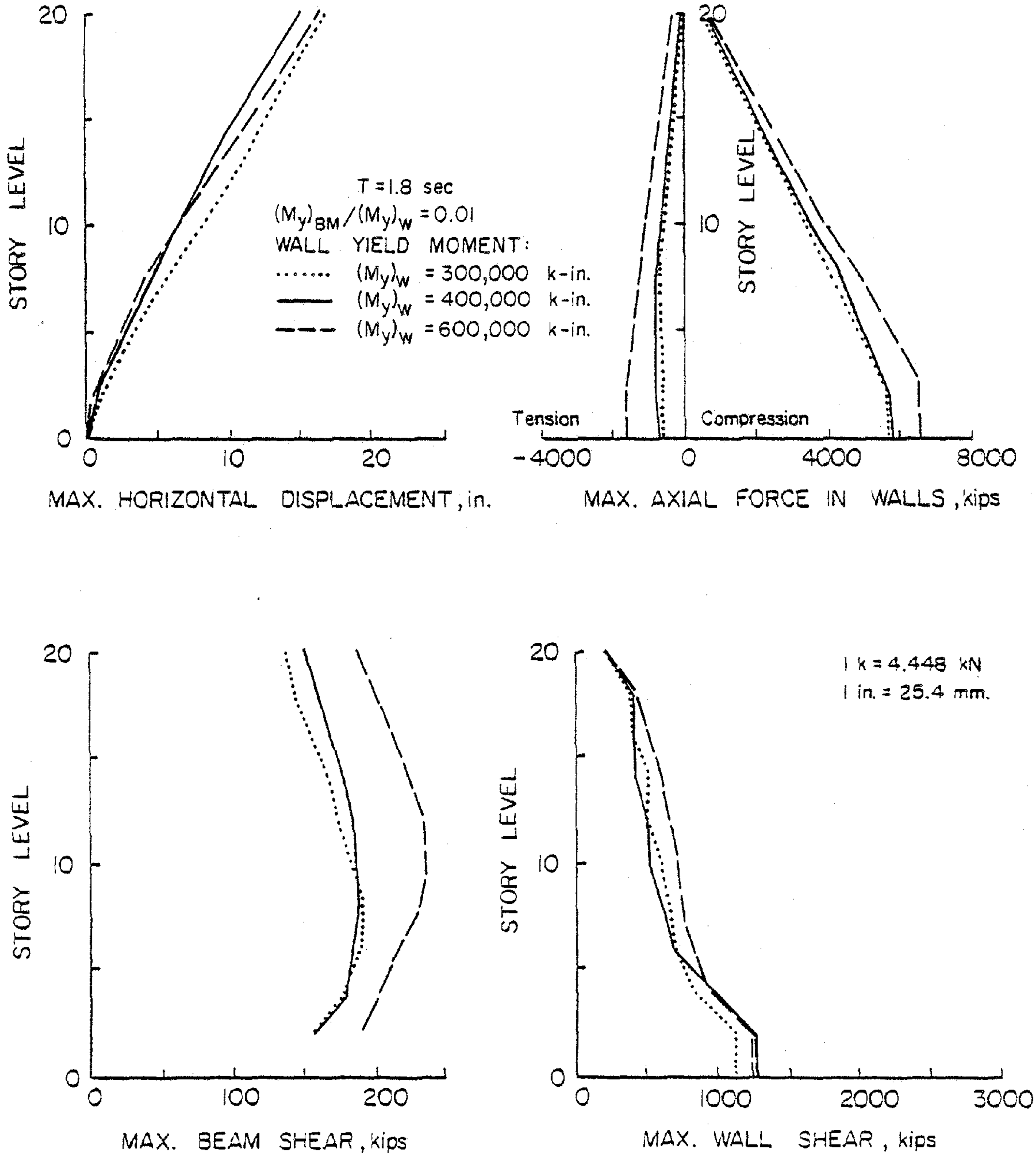

Fig. 24 Force and Displacement Envelopes Showing Effect of Wall Yield Moment

Coupling Arm

The coupling arm of a coupled wall system is defined as the center-to-center distance between the two walls. Variation in coupling arm affects structural properties and hence response of the structure.

Other parameters being constant, if the coupling arm of a coupled wall system is changed then the beam stiffness is also changed. This results in changes in overall structural stiffness and period of the structure. Furthermore, stiffness ratio and distribution of forces between walls and beams are affected. This leads to a variation in the yielding pattern and consequently produces a different response.

Although the coupling arm plays an important role in the response of a structure, for design purposes it is possible to eliminate coupling arm as an independent parameter. Results of dynamic inelastic analyses indicate that if two structures have the same fundamental period and beam-to-wall stiffness ratio, and if yielding occurs at the same deformation level*, then their displacement and ductility response is similar irrespective of the length of the coupling arm. Force response envelopes are also comparable when divided by their respective yield moments. Figure 25 illustrates two structures analyzed with different lengths of coupling arm. Both structures have the same fundamental period. The structure with the longer coupling arm has a higher value of beam moment of inertia to compensate for the loss of beam stiffness due to increased

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* Same deformation level implies equal yield rotation.

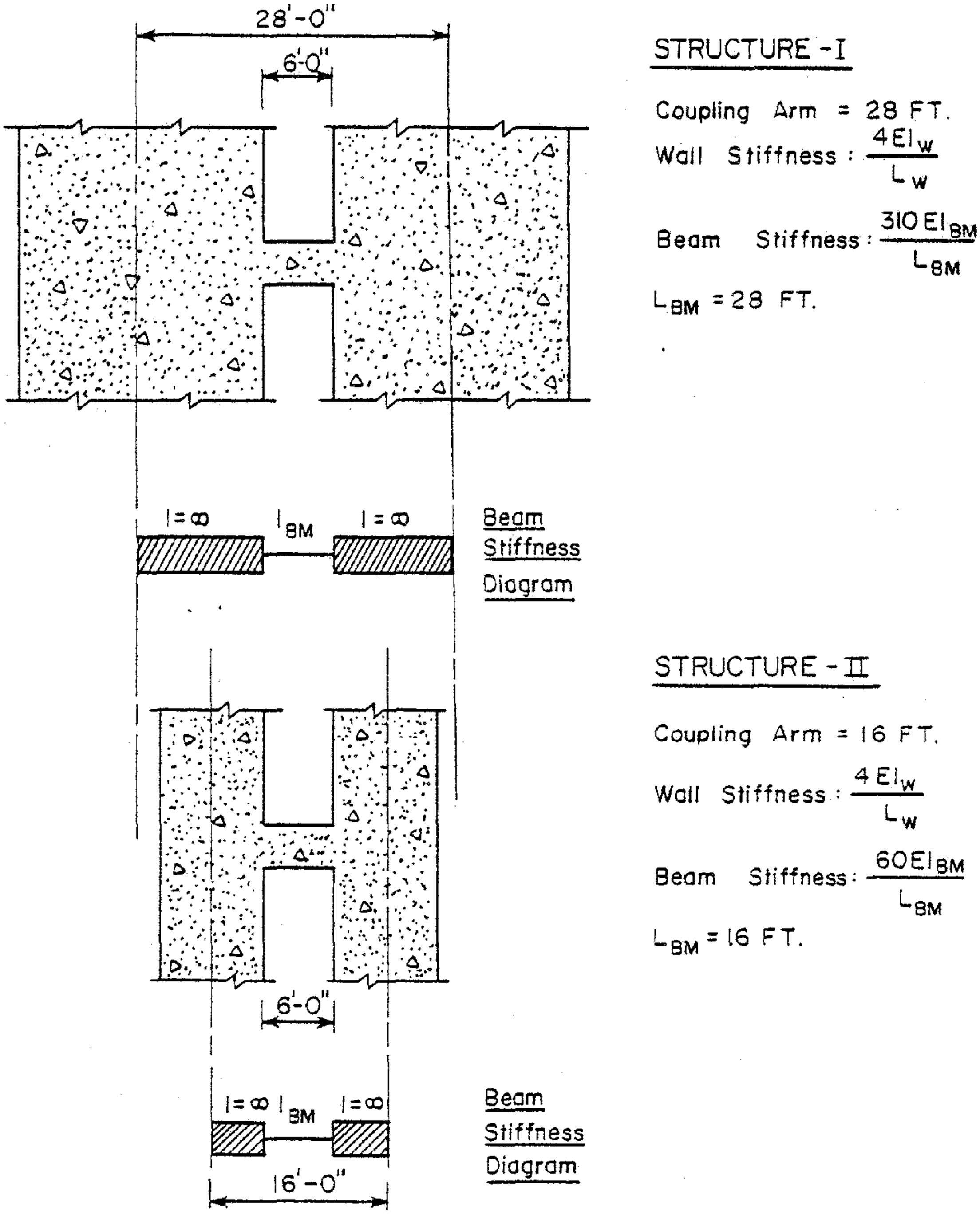


Fig. 25 Structures with Different Coupling Arms

member length. Wall width is different in each structure while beam clear span is identical.

Yield moments were assigned to each member such that yielding would start at the same deformation level* in both structures. This implies that stiff members were assigned higher yield moments to ensure that yielding started in both structures at the same time during the response. Structural properties used for the two analyses are given in Table 6. Response envelopes are compared in Figs. 26 and 27. The results indicate that the coupling arm alone cannot affect displacement and ductility response if the conditions of equal period, beam-to-wall stiffness ratio, and yielding deformation are satisfied. Bending moment, shear force, and axial force envelopes for the two structures compared are proportional to their stiffnesses.

Clear Beam Span

Clear span length of coupling beams is one of the parameters that control inelastic response of coupled wall structures. If other parameters are held constant, variation in clear beam span affects beam stiffness and therefore alters response of the structure. However, clear beam span is not an independent parameter and can be eliminated as a design variable.

If two structures have the same fundamental period, and beam-to-wall stiffness ratio and yielding occurs at the same deformation level, then their inelastic response is comparable

* Same deformation level implies equal yield rotation.

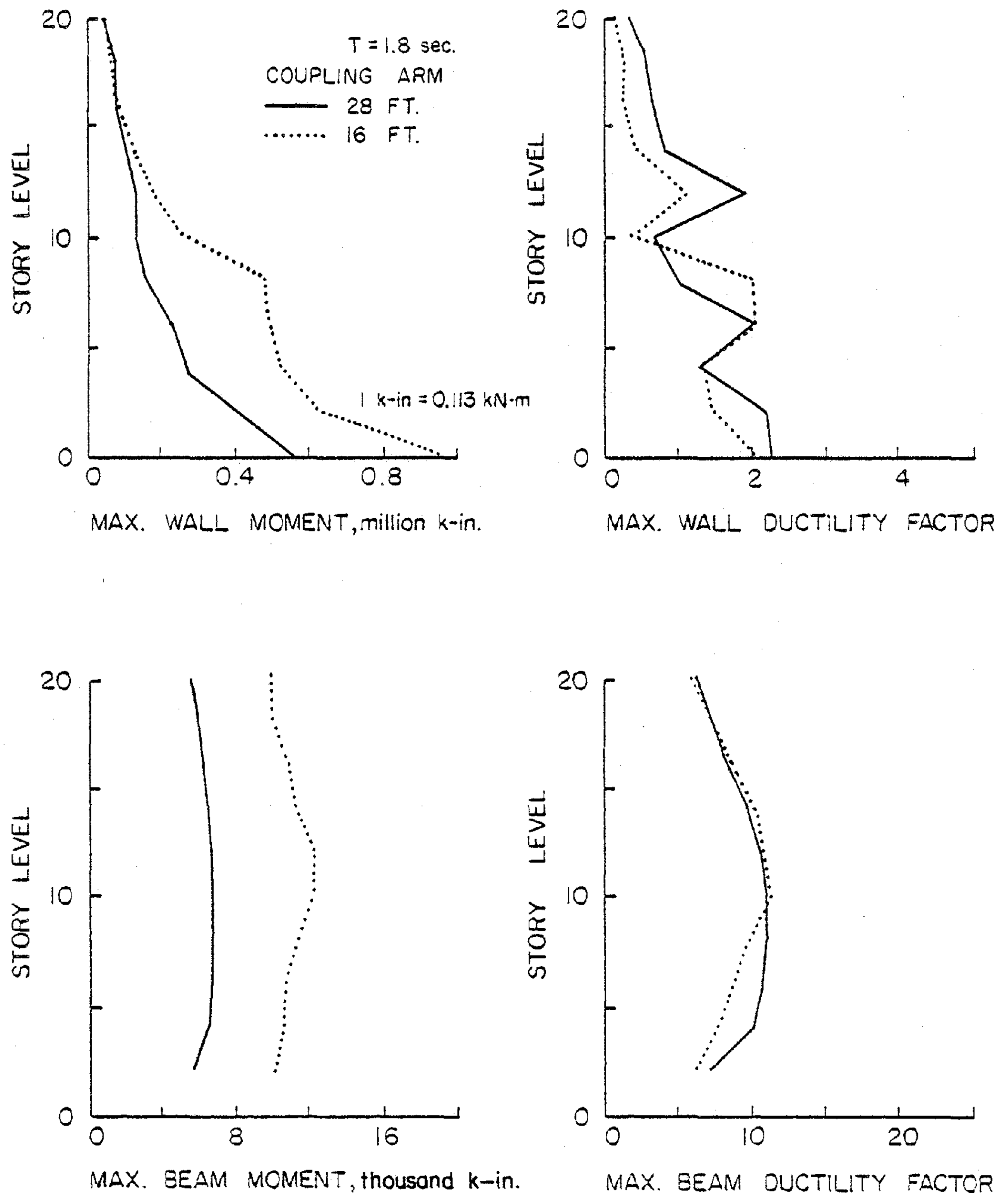


Fig. 26 Moment and Ductility Envelopes Showing Effect of Coupling Arm

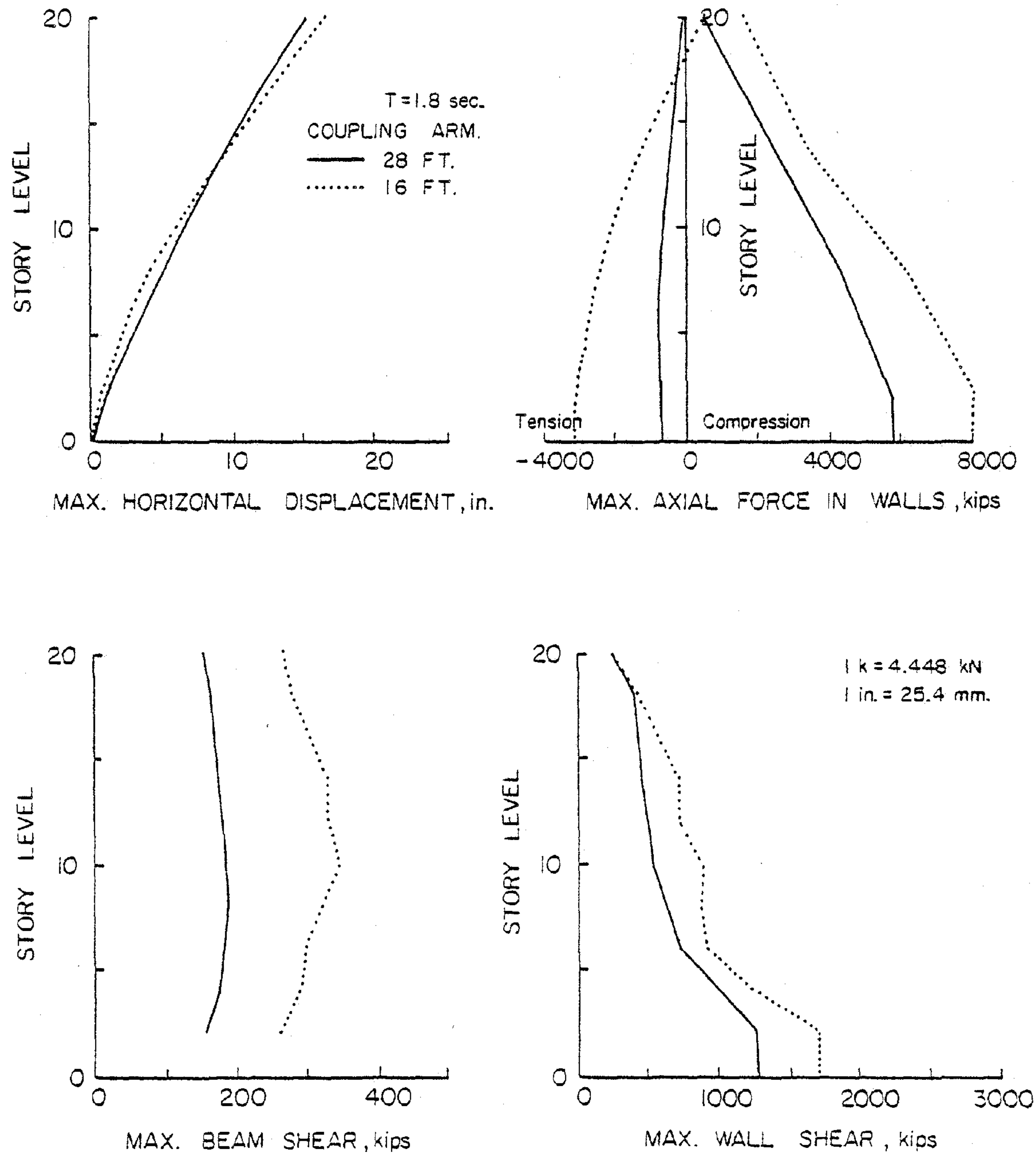


Fig. 27 Force and Displacement Envelopes Showing Effect of Coupling Arm

TABLE 6 - PROPERTIES OF STRUCTURES WITH DIFFERENT COUPLING ARMS

Properties[1]	Structure	
	I	II
Fundamental Period (sec)	1.8	1.8
Coupling Arm (ft)	28	16
Clear Beam Span (ft)	6	6
Wall Width (ft)	22	10
Wall Stiffness Parameters[2]		
EI (million k-in.^{2})	38,600	69,500
GA (million kips)	1.13	2.03
EA (million kips)	6.70	12.03
Beam Stiffness Parameters		
EI (million k-in.$_2$)	26.0	139.1
GA (million kips)	0.091	0.164
EA (million kips)	0.850	1.530
Wall Yield Moment[2] (k-in.)	400,000	720,000
Beam Yield Moment (k-in.)	4,800	7,200
Beam-to-Wall Stiffness Ratio	0.0168	0.0168
Beam-to-Wall Strength Ratio	0.01	0.01

Notes:

1 For all other properties, see Table 1

2 Wall stiffness and yield moment are given at structure base. For strength and stiffness taper along the structure height, see Table 1

irrespective of clear beam spans. Figure 28 and Table 7 give the details of two coupled wall structures with two different clear beam spans. These two structures were analyzed using the same ground motion. Response envelopes are compared in Figs. 29 and 30. The results indicate that beam span length alone cannot

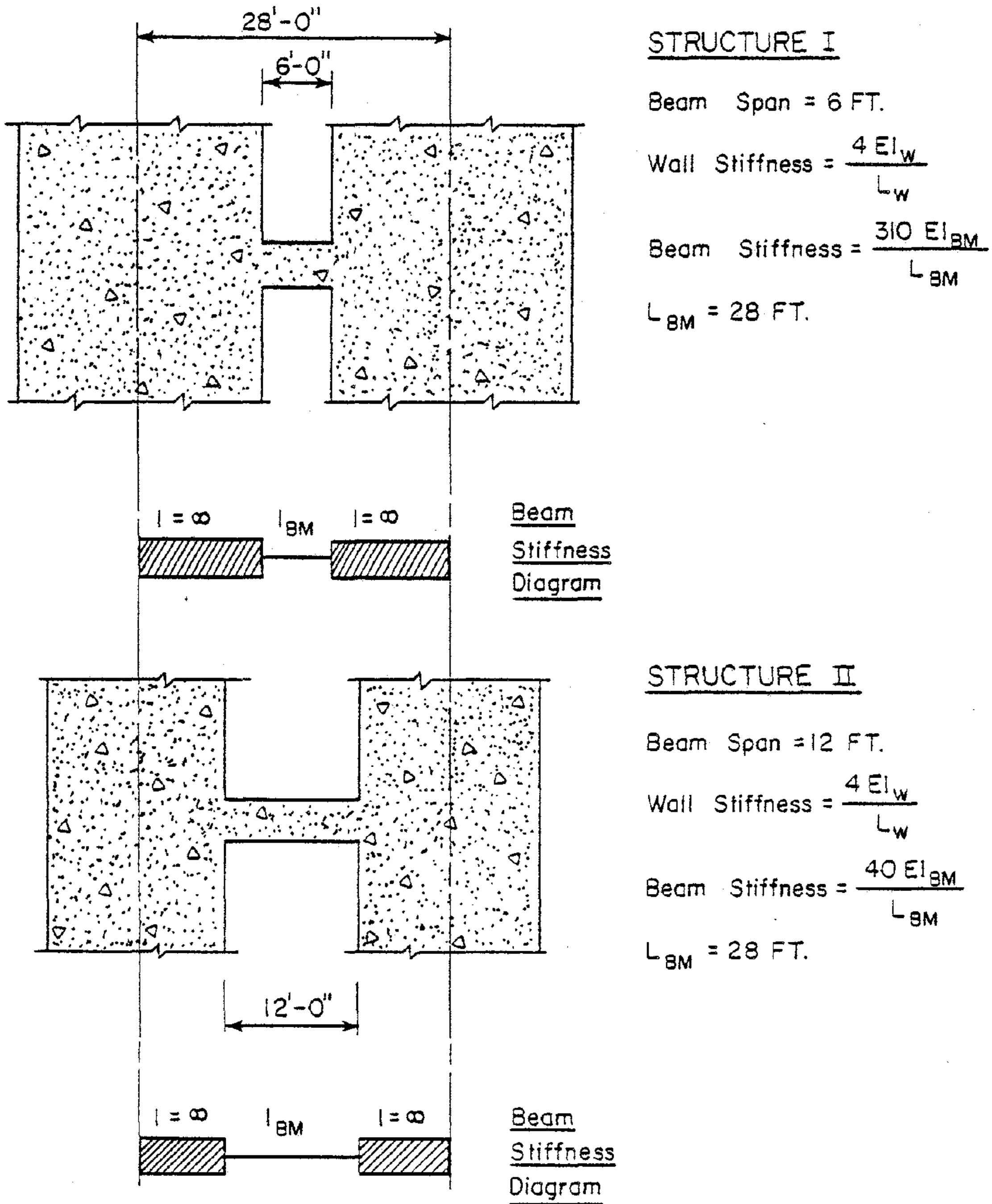


Fig. 28 Structures With Different Clear Beam Spans

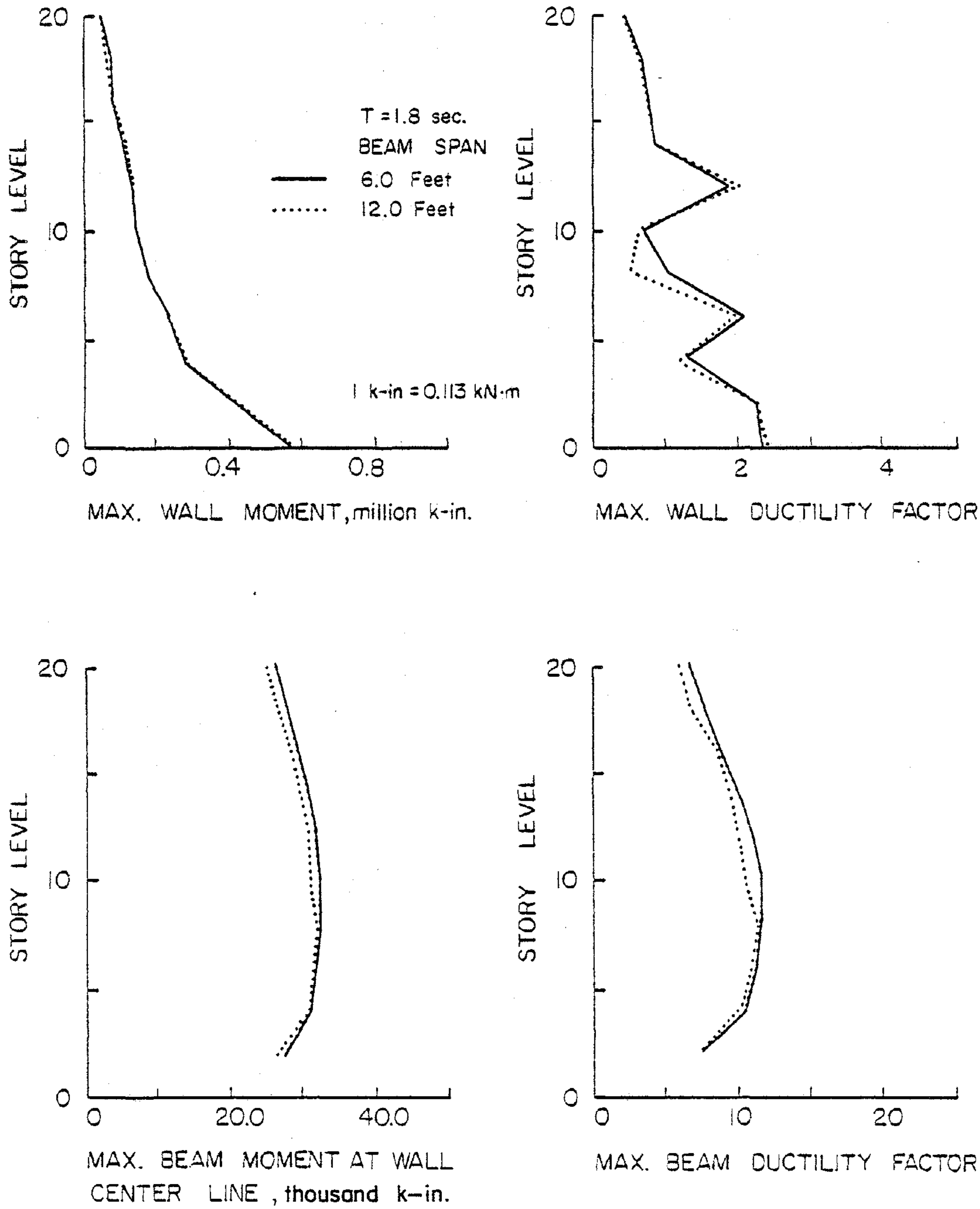


Fig. 29 Moment Displacement Envelopes Showing Effect of Clear Beam Span

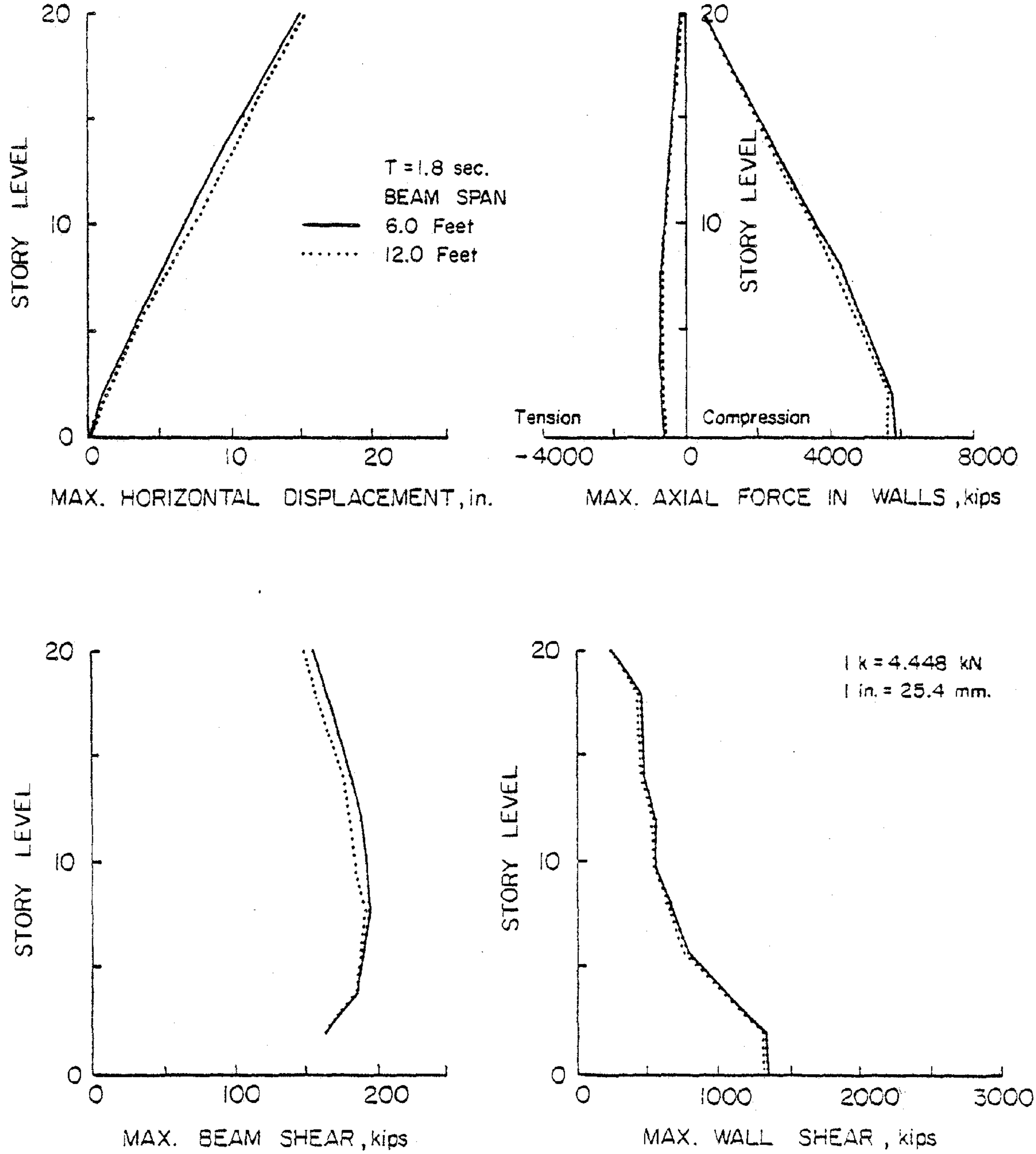


Fig. 30 Force and Displacement Envelopes Showing Effect of Clear Beam Span

TABLE 7 - PROPERTIES OF STRUCTURES WITH DIFFERENT CLEAR BEAM SPANS

Properties[1]	Structure	
	I	II
Fundamental Period (sec)	1.8	1.8
Wall Width (ft)	22	16
Clear Beam Span (ft)	6	12
Coupling Arm (ft)	28	28
Wall Stiffness Parameters[2]		
EI (million k-in.^{2})	38,600	38,600
GA (million kips)	1.13	1.13
EA (million kips)	6.70	6.70
Beam Stiffness Parameters		
EI (million k-in.^{2})	26.0	199.0
GA (million kips)	0.091	0.091
EA (million kips)	0.850	0.850
Wall Yield Moment[2] (k-in.)	400,000	400,000
Beam Yield Moment (k-in.)	4,000	8,000
Beam Moment at Wall Centerline When Beam Yields (k-in.)	18,677	18,677
Beam-to-Wall Stiffness Ratio	0.168	0.168

Notes:

1 For all other properties, see Table 1

2 Wall stiffness and yield moment are given at structure base. For strength and stiffness taper along the structure height, see Table 1

affect the inelastic response if the conditions of equal period, beam-to-wall stiffness ratio and yielding deformation are satisfied.

Mass for Inertia Forces

Vibration periods of structures are affected by mass as well as stiffness. Dynamic analyses were conducted to explore the possiblity of expressing mass and stiffness in terms of a single parameter such as fundamental period for design purposes. Two coupled wall structures with the same fundamental period but different mass and stiffness properties were analyzed. Yield moments for each structure were adjusted on the basis of their respective stiffnesses so that yielding occurred at the same deformation level in both structures. The same vertical distribution of mass was used in both structures. Table 8 lists the properties of the two structures analyzed. Response envelopes are compared in Figs. 31 and 32.

Results indicate that variations in mass and stiffness do not significantly affect horizontal displacements provided that the fundamental period remains constant. Inelastic deformations and rotational ductilities are not affected if yield moments are specified such that yielding starts at the same rotation in members of both structures. The stiffer structure shows a higher moment, shear, and axial force response as expected. Force response between the two structures is proportional to their stiffnesses. Therefore, in formulating a design procedure, mass need not be considered as a separate variable and its effect is reflected in fundamental period.

Effect of Initial Gravity Loads

For the analysis, structural mass is specified at nodal points. The specified mass is used to solve the equations of dynamic equilibrium and does not have any contribution to initial static vertical forces due to gravity loads.

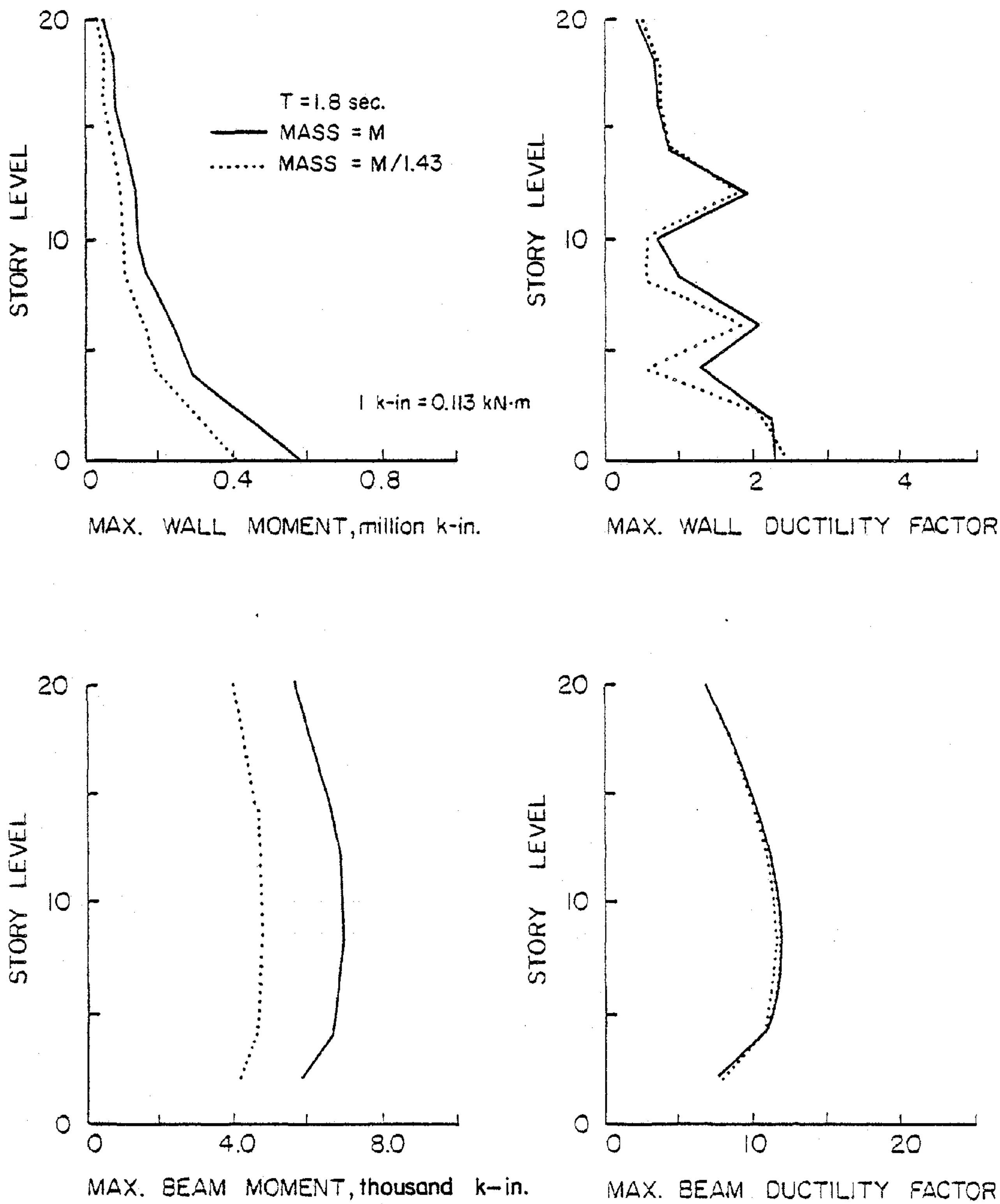


Fig. 31 Moment and Ductility Envelopes Showing Effect of Mass for Inertia

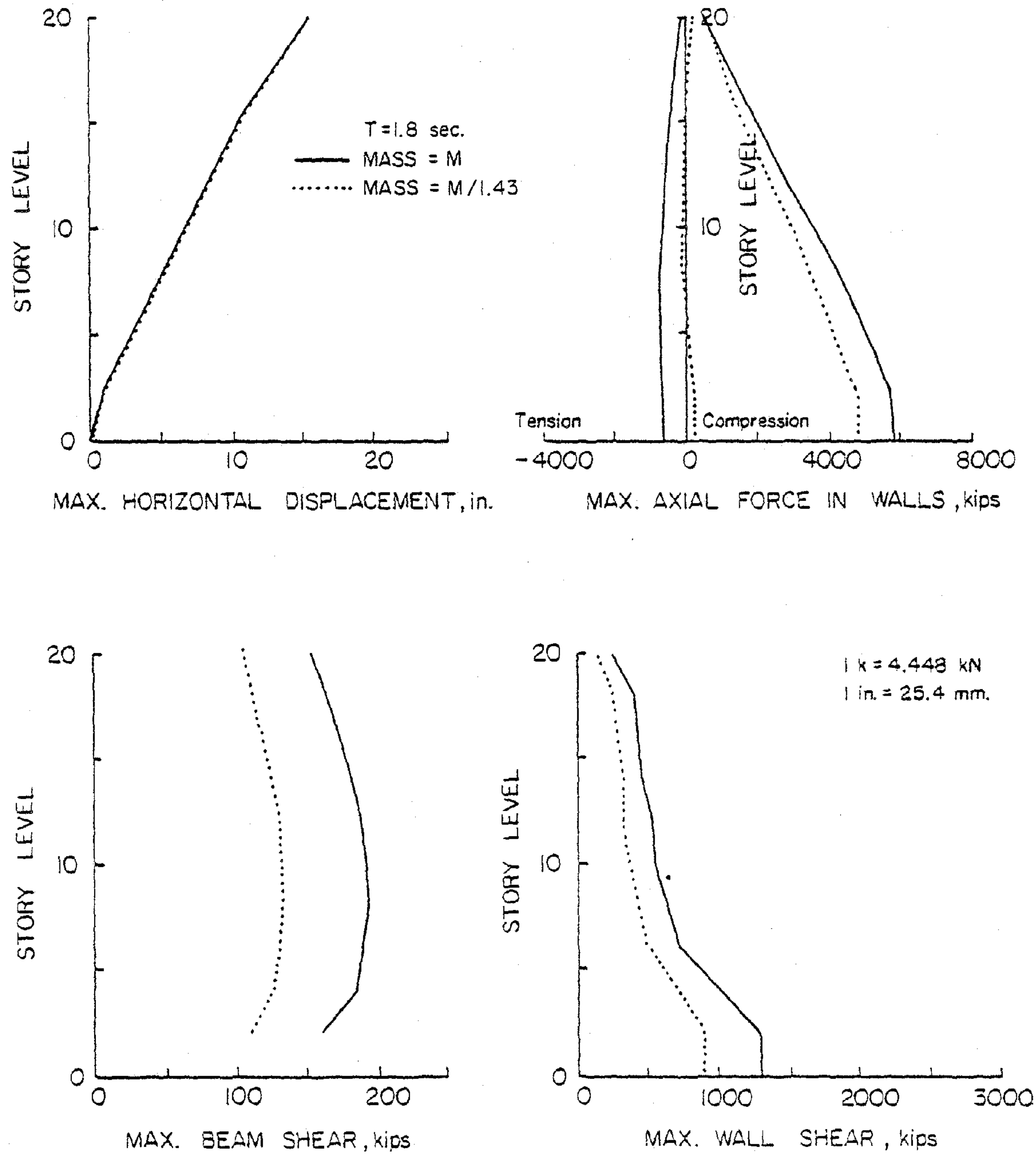


Fig. 32 Force and Displacement Envelopes Showing Effect of Mass for Inertia

TABLE 8 - PROPERTIES OF STRUCTURES WITH DIFFERENT MASS FOR INERTIA

Properties[1]	Structure	
	I	II
Fundamental Period (sec)	1.8	1.8
Wall Stiffness Parameters[2]		
EI (million k-in.^{2})	38,600	38,600
GA (million kips)	1.13	0.79
EA (million kips)	6.70	4.70
Beam Stiffness Parameters		
EI (million k-in.^{2})	26.0	18.3
GA (million kips)	0.091	0.064
EA (million kips)	0.850	0.595
Mass for Inertia (k-in./sec^2)		
Stories 20-13	2.367	1.630
" 12-7	2.432	1.692
" 6-1	2.528	1.775
Wall Yield Moment[2] (k-in.)	400,000	280,000
Beam Yield Moment (k-in.)	4,000	2,800

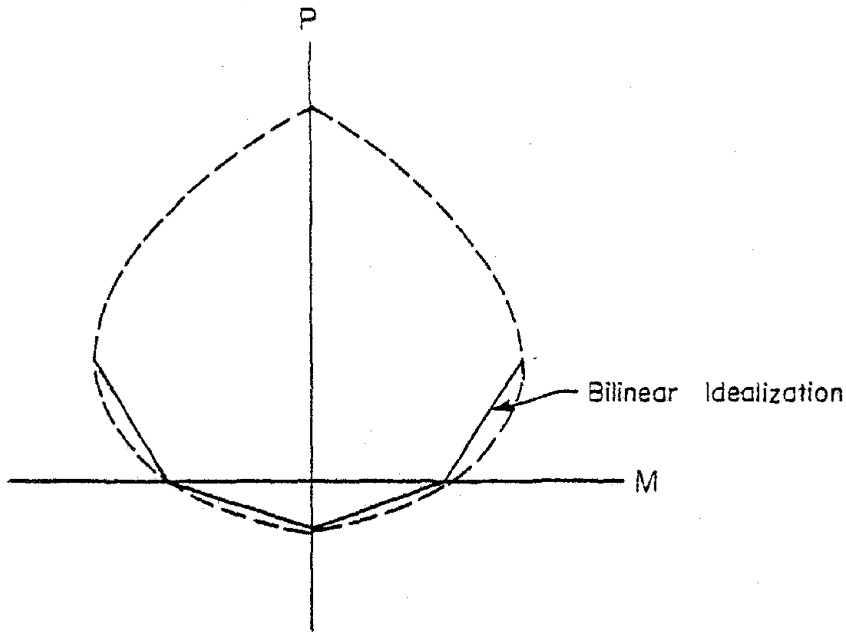
Notes:
1 For all other properties, see Table 1
2 Wall stiffness and yield moment are given at structure base. For strength and stiffness taper along the structure height, see Table 1

The primary effect of gravity loads is to change the capacity of vertical members. Flexural capacity of a wall section depends on the level of axial force. Axial forces develop in walls due to static gravity loads prior to dynamic response. Their effect on dynamic response is to increase or decrease the initial yield level. Initial yield level (strength) of a member is an important parameter and therefore is handled separately in this investigation. However, two walls may be subjected to different levels of initial gravity loads and yet their initial yield moments may be equal if their sectional characteristics are different. It is the objective of this section to investigate the significance of variation in initial gravity loads on dynamic response when structures have the same yield moment.

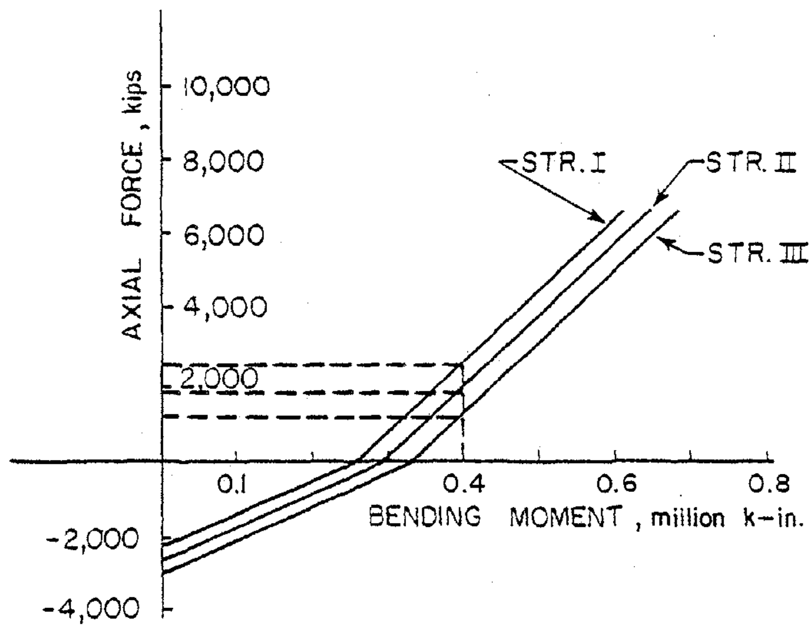
The effect of variation in initial gravity loads for structures having the same initial yield level is to shift the moment-axial force (M-P) interaction relationship so that response starts at different levels of axial load while the yield moment remains constant. This is shown in Fig. 33 for three structures having yield moment of 0.4 million k-in. under different axial loads.

Dynamic inelastic analyses were carried out on three coupled wall structures with the same initial yield moments but different levels of initial axial load. M-P interaction characteristics for the base walls of the three structures are shown in Fig. 33. All other properties are given in Table 1. Response envelopes are compared in Figs. 34 and 35.

Results indicate that change in M-P interaction relationship due to variation in axial loads can affect some response quantities significantly. Depending on the amount of increase or decrease in axial loads due to dynamic response, wall ductility response shows considerable variation at the base. In the structures with relatively light gravity loads, axial tension associated with lateral displacement is only partially counterbalanced by axial compression due to gravity loads. This



a) Bilinear Idealization of M−P Interaction



b) M−P Characteristics of Structures Analyzed

Fig. 33 Yield Moment as Affected by Axial Loads

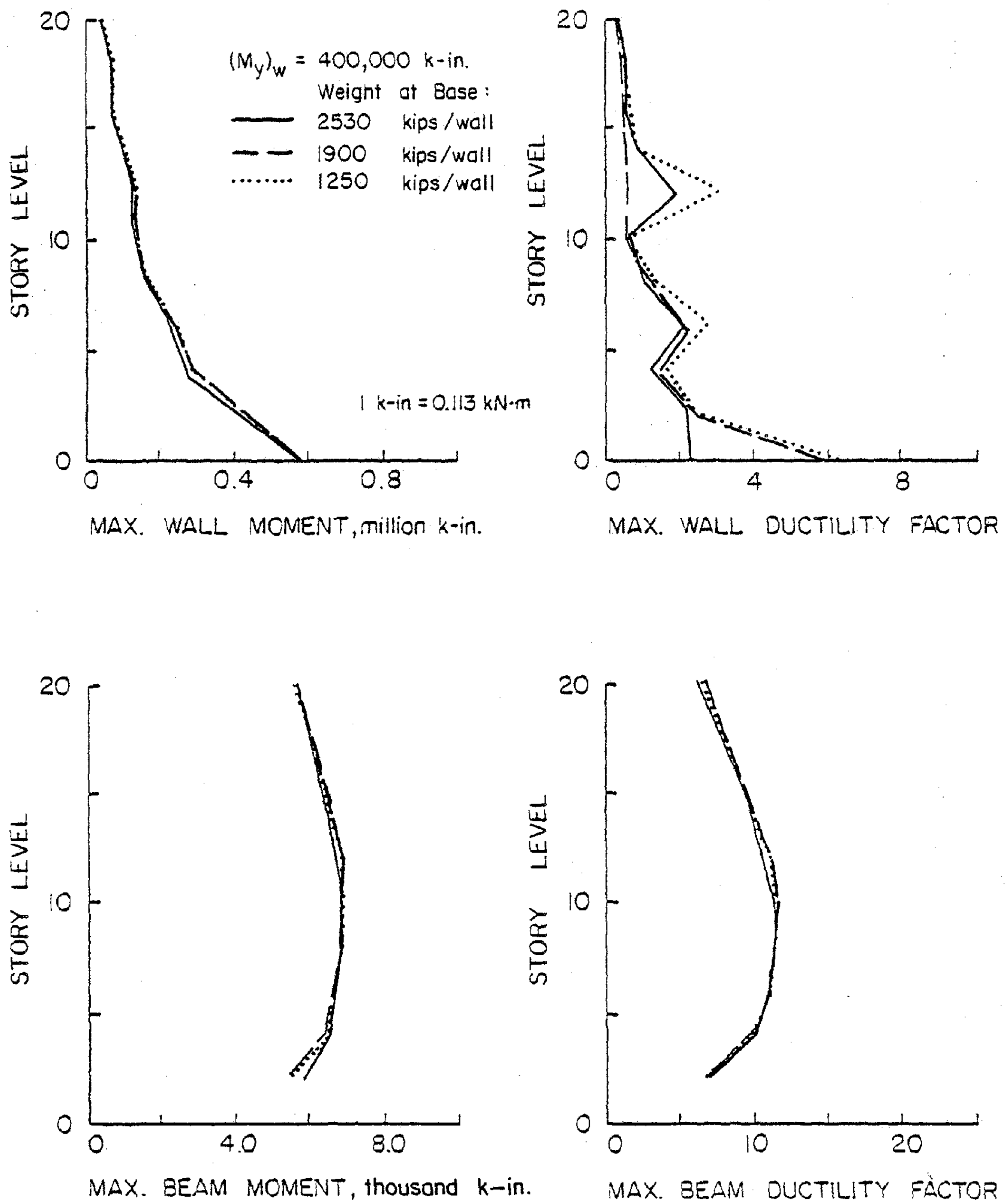


Fig. 34 Moment and Ductility Envelopes Showing Effect of Mass for Gravity Forces

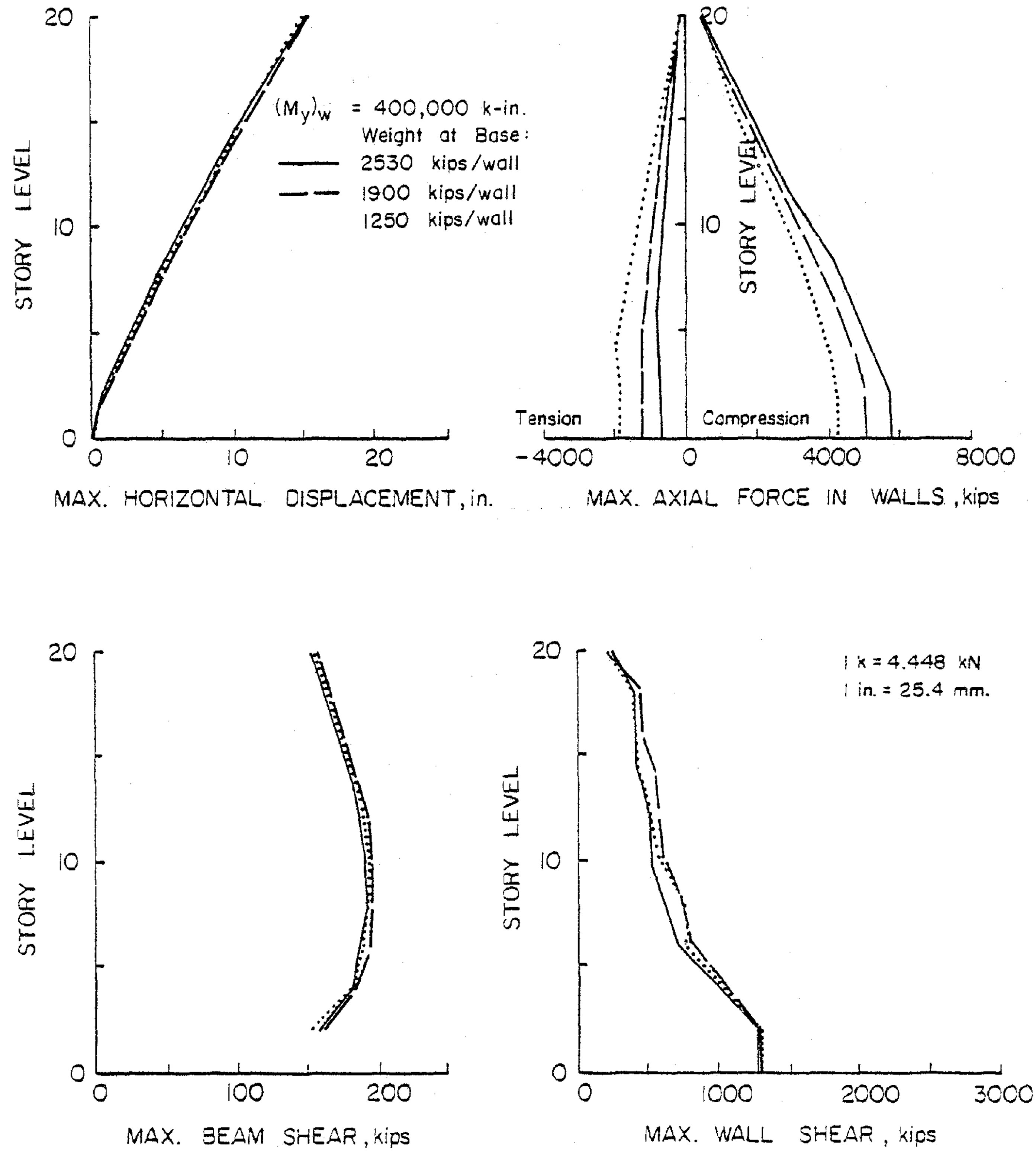


Fig. 35 Force and Displacement Envelopes Showing Effect of Mass for Gravity Loads

results in increased net tension. A high level of axial tension in walls is usually accompanied by a reduction in flexural capacity and increase in ductility ratios. However, if the change in gravity loads is small compared to axial forces due to dynamic response, then the effect on dynamic response is small. The 20-story structure considered for parametric investigation showed that a decrease of 100% in gravity loads produced an increase in base wall ductility by a factor of approxomately 2.5. Comparison of response envelopes in Figs. 34 and 35 indicates that all other response quantities remained essentially unaffected by change in gravity loads.

Significance of initial gravity load effects depends primarily on the range of variation in the axial force couple during response. High coupling usually produces a higher level of axial tension in walls and increases the importance of initial gravity loads. It is usually desirable to control the degree of coupling to avoid increased plastification in walls. If the level of axial force couple increases beyond the balanced point during response, failure is governed by concrete crushing rather than yielding of steel. At the other extreme, excessive axial tension in walls diminishes the flexural capacity such that the member fails under pure axial tension. Neither case is well defined in the analysis. Warning messages are printed under such cases.

EFFECTS OF GROUND MOTION PARAMETERS ON DYNAMIC RESPONSE

General

Input motion characteristics have a significant effect on dynamic response of structures. For design purposes, it is generally desirable to subject structures to input motions that produce maximum response quantities. However, it is not possible to predict the character of input motion that the structure may be subjected to during its lifetime. Therefore, one has to rely on previously recorded earthquake motions in understanding characteristic features of ground motions and their significance on dynamic response.

Based on a previous study conducted at the Portland Cement Association(7) three important features of ground accelerograms were selected for examination. These were:

a. frequency characteristics

b. intensity

c. duration

Effects of these parameters on nonlinear dynamic response of coupled wall structures were investigated. Six base accelerograms were considered. Velocity response spectra for these accelerograms are shown in Fig. 36.

Earthquake Frequency Characteristics

Structural response is closely related to the frequency characteristics of the exciting force. If a dominant vibration period of a structure falls within the peaking range of the input motion, strong structural response can be expected.

Stiffness and vibration periods of concrete structures generally vary during response. Cracking of concrete reduces stiffness and elongates the period of the structure. Further softening in a structure takes place when yielding occurs in one or more elements. Therefore, an input motion that is critical for an uncracked structure may not be critical after yielding occurs. It may be possible, however, to estimate the range of variation in the fundamental period. This, along with a response spectrum, provides preliminary information on selection of a base motion that can create strong response.

Two structures with different initial fundamental periods were selected for dynamic analysis under different ground excitations. Each accelerogram was normalized with respect to 1.5 times the 5%-damped spectrum intensity of the N-S component of the 1940 El Centro record. The two structures analyzed and the input motions considered for each analysis are listed in Table 9.

The first set of analyses was made for the structure with initial fundamental period, T = 1.0 sec. Figures 37 and 38 show envelopes of response quantities for this set. The results indicate that S16E component of 1971 Pacoima Dam record creates the most critical force, displacement, and ductility response. The 5%-damped velocity response spectrum for this accelerogram shows a pronounced peak at or close to the fundamental period of the structure considered, as shown in Fig. 36.

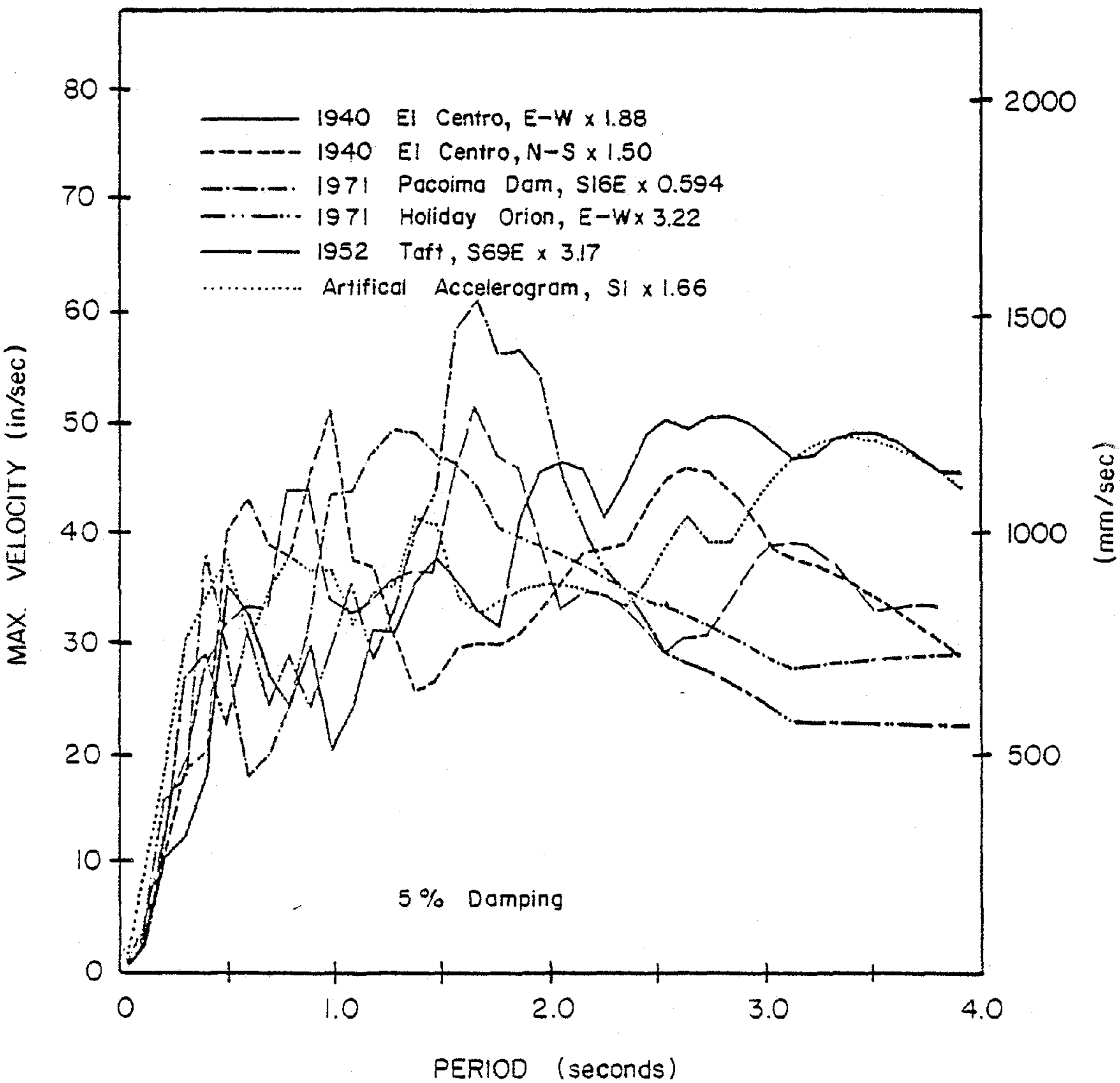


Fig. 36 Relative Velocity Response Spectra for First Ten Seconds of Normalized Input Motions

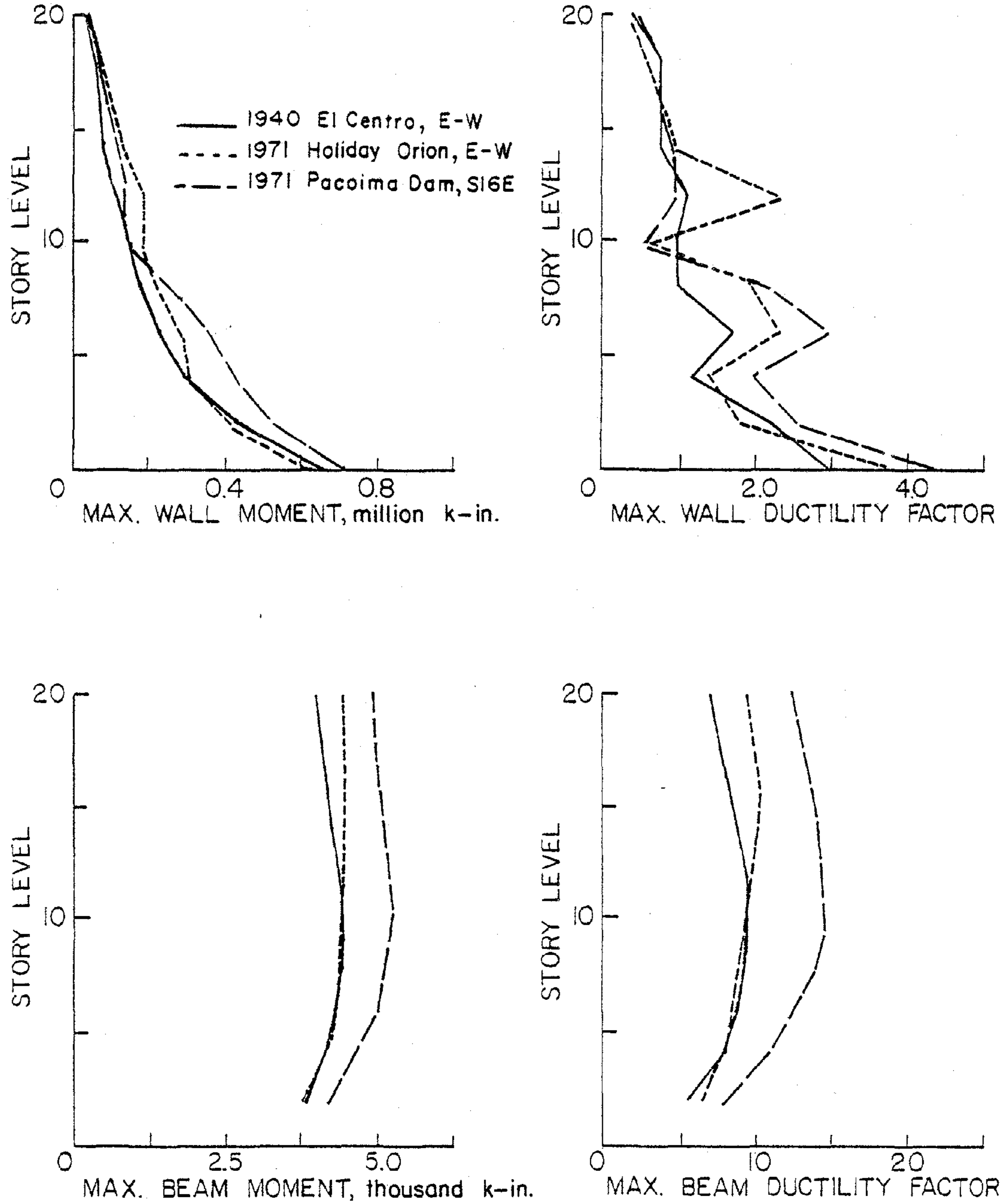


Fig. 37 Moment and Ductility Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.0 sec)

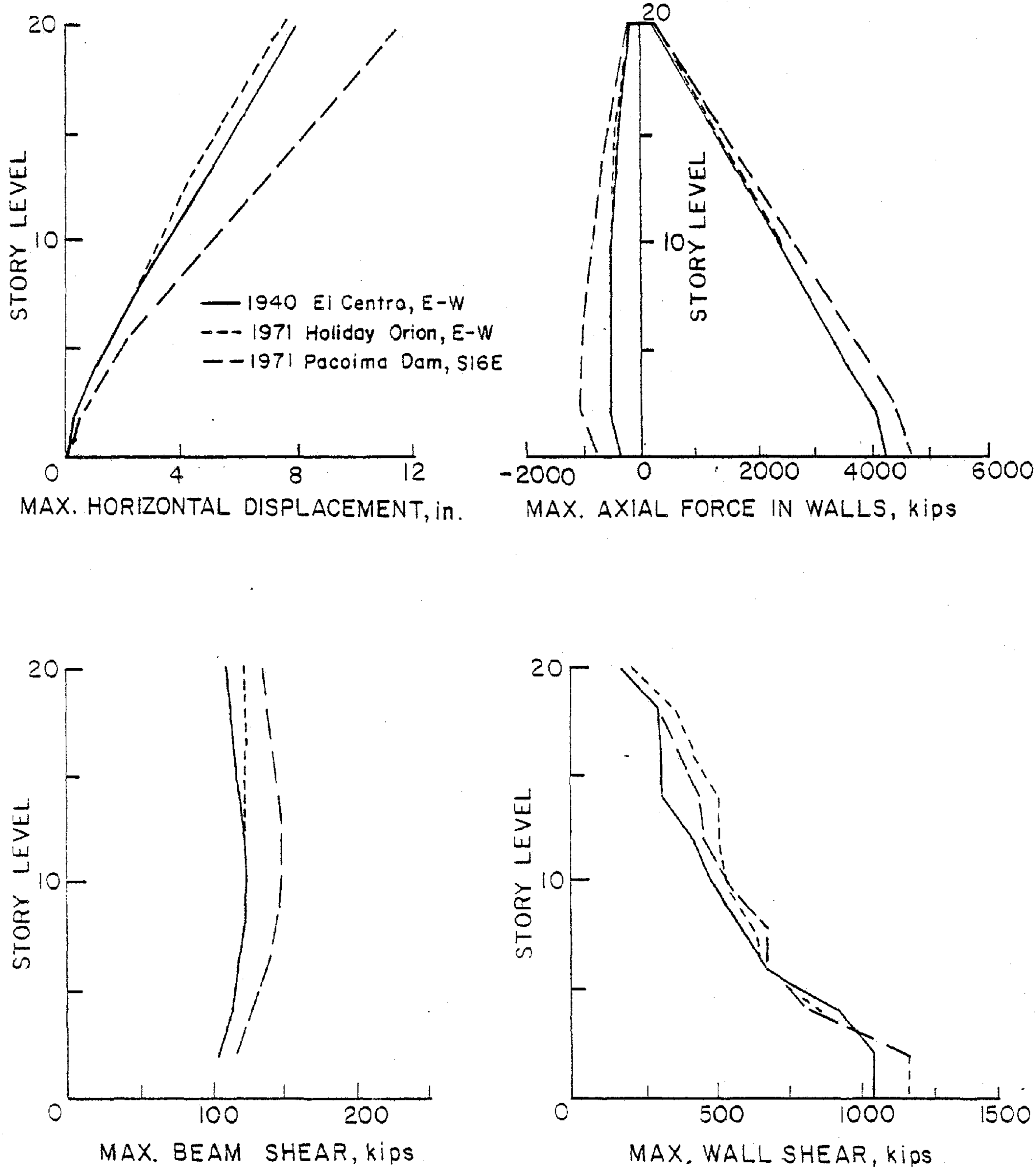


Fig. 38 Force and Displacement Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.0 sec)

TABLE 9 - STRUCTURES ANALYZED FOR EFFECT OF EARTHQUAKE FREQUENCY CHARACTERISTICS

Properties[1]	Structures	
	I	II
Fundamental Period (sec)	1.0	1.8
Input Motions	1940 El Centro, E-W 1971 Holiday Orion, E-W 1971 Pacoima Dam, S16E	1940 El Centro, E-W 1940 El Centro, N-S 1971 Holiday Orion, E-W

Note:
1. For all other properties, see Table 10 and Table 1 for structures with fundamental periods 1.0 sec and 1.8 sec, respectively.

The second set of analyses was conducted on the structure with fundamental period, T = 1.8 sec. Three potentially critical earthquake motions were selected from the 5%-damped velocity response spectrum. The input motions selected are tabulated in Table 9. Response envelopes for this set are compared in Figs. 39 and 40. Results of the second set of analyses indicate that none of the three accelerograms considered produced critical response in all the response quantities along the height of the structure. While one input motion created strong response at the base of the structure, the others produced stronger response at different levels along the height. However, as shown in Figs. 39 and 40, those input motions that did not produce critical response in a particular

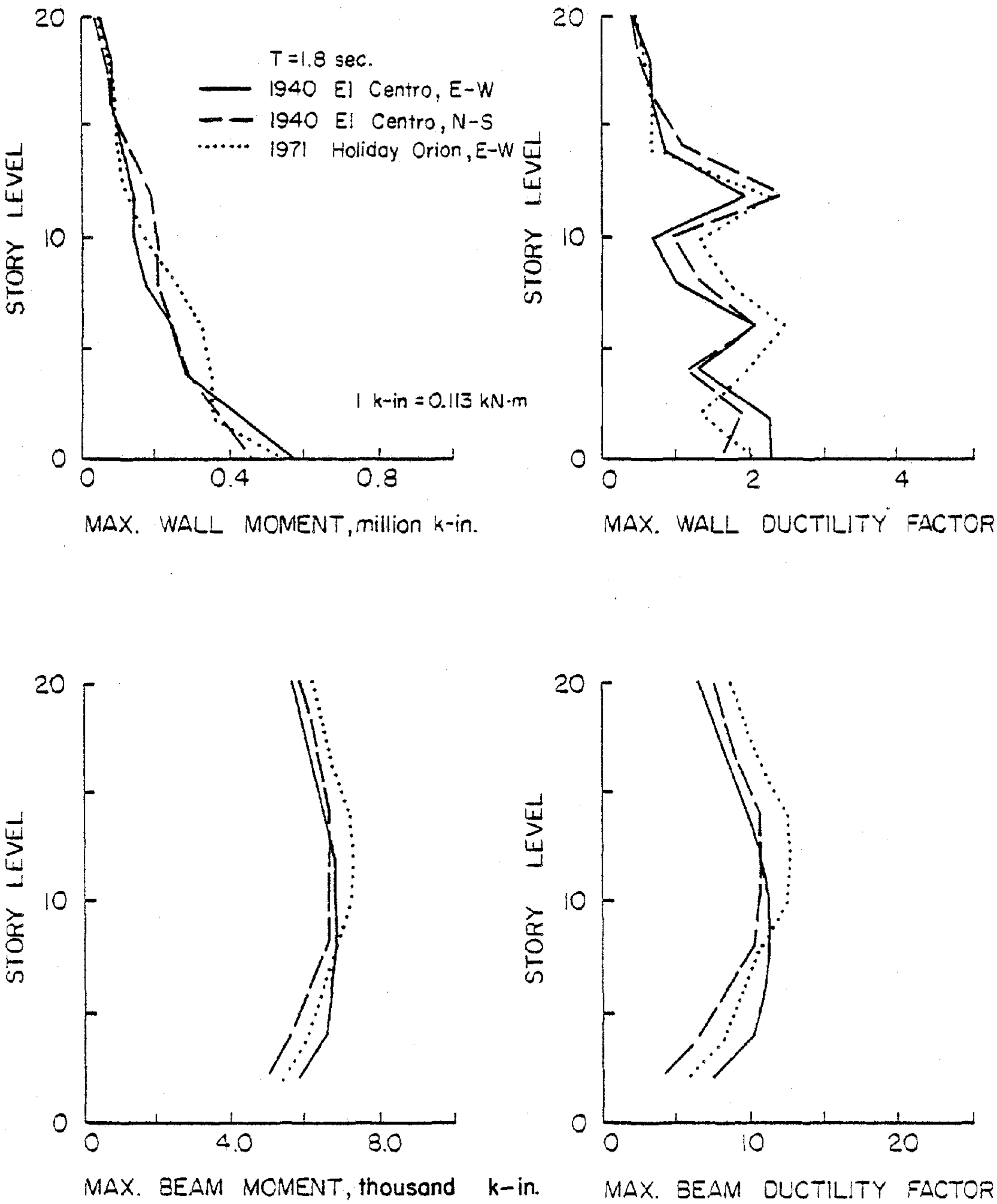


Fig. 39 Moment and Ductility Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.80 sec)

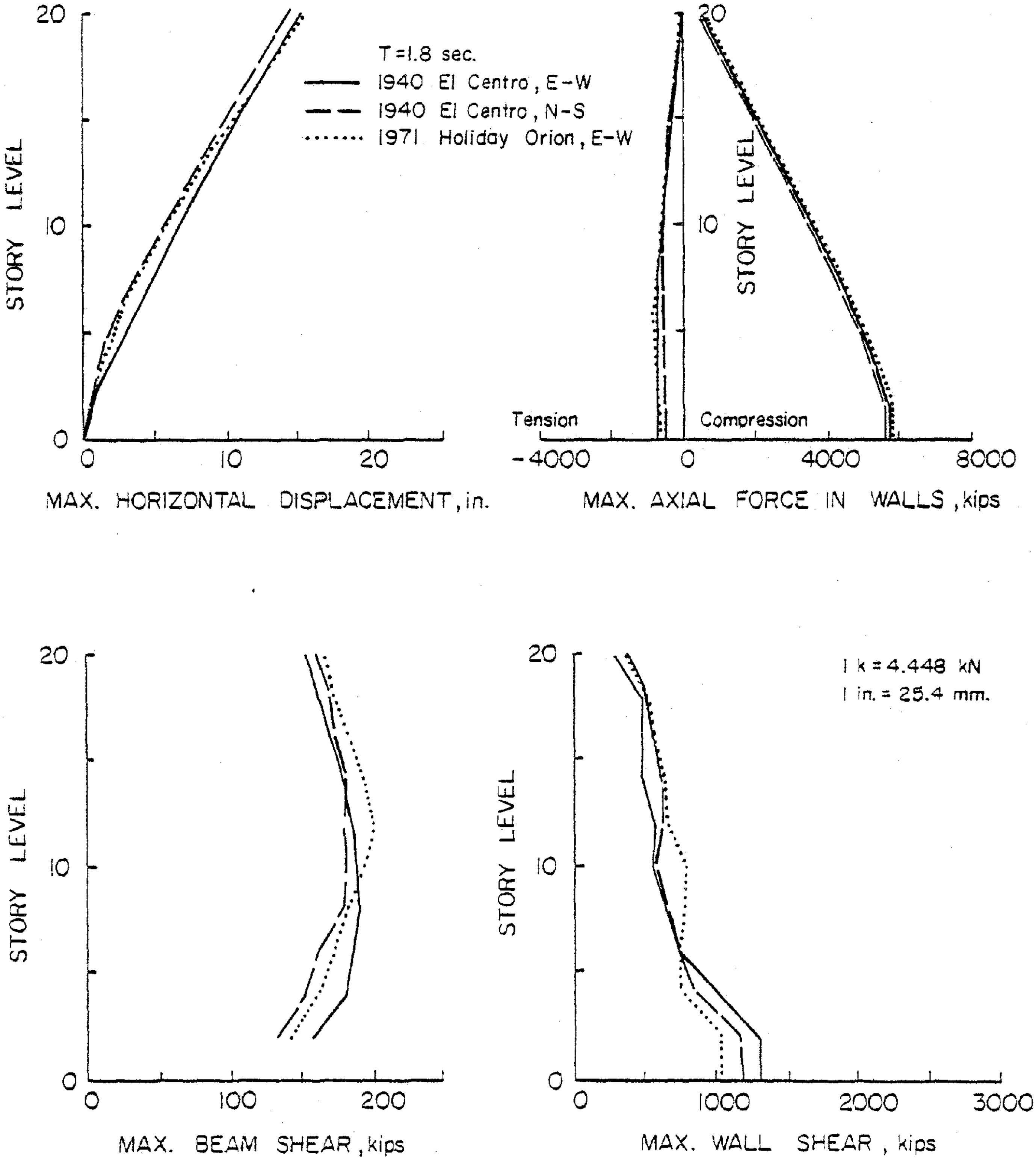


Fig. 40 Force and Displacement Envelopes Showing Effect of Earthquake Frequency Characteristics (T = 1.80 sec)

TABLE 10 - PROPERTIES OF STRUCTURE WITH T = 1.0 SEC

Fundamental Period (sec)	1.0
Number of Stories	20
Height (ft)	183
Wall Stiffness Parameters:	
EI (million k-in.^{2})	82,800
GA (million kips)	6.09
EA (million kips)	142
Stiffness Taper[1]	1.0 EI at base 0.8 EI at 6th floor 0.65 EI at 12th floor
Beam Stiffness Parameters[2]	
EI (million k-in.^{2})	22.75
GA (million kips)	0.094
EA (million kips)	0.922
Wall Yield Moment, M_y (k-in.)	400,000
Strength Taper[3]	1.00 M_y at base 0.50 M_y at 12th floor
Beam Yield Moment[2] (k-in.)	3,000
Damping	5% of critical
Post-Yield Stiffness on Primary Curve	5% of elastic for walls 6% of elastic for beams
Weight (k/wall)	1880
Weight for Inertia Forces (k/wall)	3270
Base Fixity Condition	fully fixed
Ground Motion	Pacoima Dam 1971, S16E
Intensity of Ground Motion[4]	1.5 El Centro 1940, N-S

Notes:
1. The same taper also applies for "GA" and "EA."
2. Beam stiffness parameters and beam yield moment must be multiplied by 2.0 to obtain values for 10-story model.
3. Yield moment are also adjusted at every floor based on the weight of the structure.
4. Based on spectrum intensity.

response quantity produced near critical values. In this investigation, the E-W component of the 1940 El Centro record was used for structures with period T = 1.8 sec.

Both sets of analyses discussed above confirm that single-degree-of-freedom response spectra provide estimates of the critical base accelerogram as affected by frequency characteristics. In some cases, it is difficult to select the critical earthquake motion from the response spectra because of the variation in period of structures during response. Other factors, such as yield level of members, directly affect plastification of members and hence the variation in vibration periods during response. However, for a given ductility range, elongation of period is generally limited to a range that does not vary greatly. Therefore, for design purposes, it is reasonable to assume that an input motion that is critical in terms of frequency characteristics for a given structure produces critical or near critical response in other structures having the same initial fundamental period.

In formulating the design procedure discussed under "Design Information," the input motion that creates critical or near critical response in most response quantities for a particular period range is used as the design earthquake.

Intensity of Input Motions

It is fairly difficult to define intensity of an earthquake motion quantitatively. Some investigators[8,9] use peak acceleration as a measure of intensity. Others[10] use some means of averaging the amplitude of acceleration pulses.

Spectrum intensity was used in this investigation as a measure of earthquake intensity. The spectrum intensity used is defined as the area under the 5%-damped relative velocity response spectrum between periods of 0.1 and 3.0 seconds. Except when intensity was the investigated parameter, all input motions were normalized with respect to 1.5 times the 5%-damped spectrum intensity of the N-S component of the 1940 El Centro record (SI).

The 20-story coupled wall structure selected for parametric investigation was analyzed under three different intensity levels to examine the effect of earthquake intensity. The same three analyses were repeated with a different yield level. Particular cases covered in the analyses are specified in Table 11. The E-W component of the 1940 El Centro record was used in all cases.

Figures 41 and 42 illustrate response envelopes under different levels of earthquake intensity. In all cases there is a consistent increase in response with increasing intensity, as would be expected. Maximum moments, shears, displacements, and ductilities are plotted against earthquake intensity in Fig. 43. Relationships between maximum response quantities and earthquake intensity indicates that structural response increases almost linearly with increasing intensity. This information is useful for design purposes in extrapolating the results to obtain structural response at different earthquake intensities.

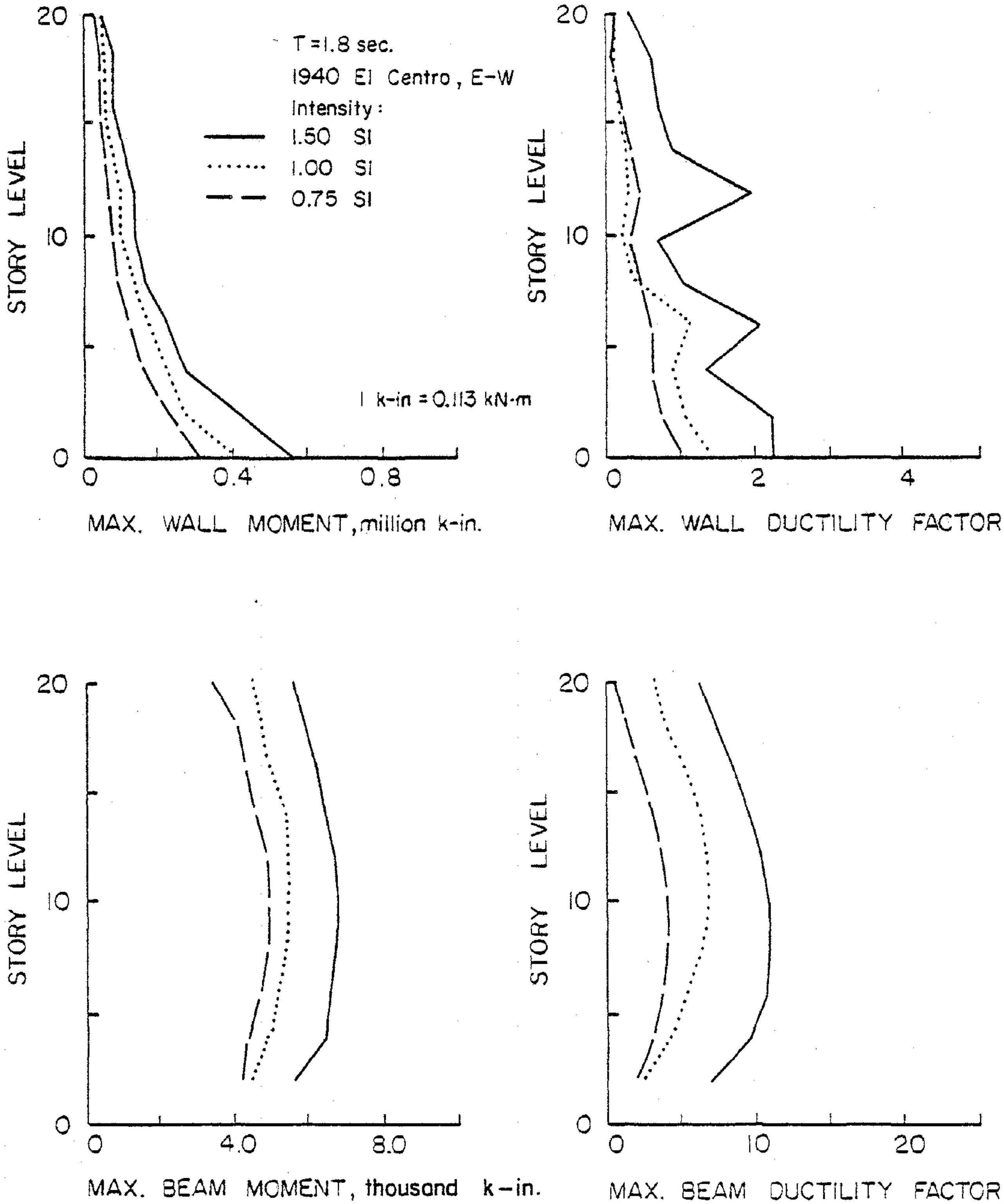


Fig. 41 Moment and Ductility Envelopes Showing Effect of Earthquake Intensity

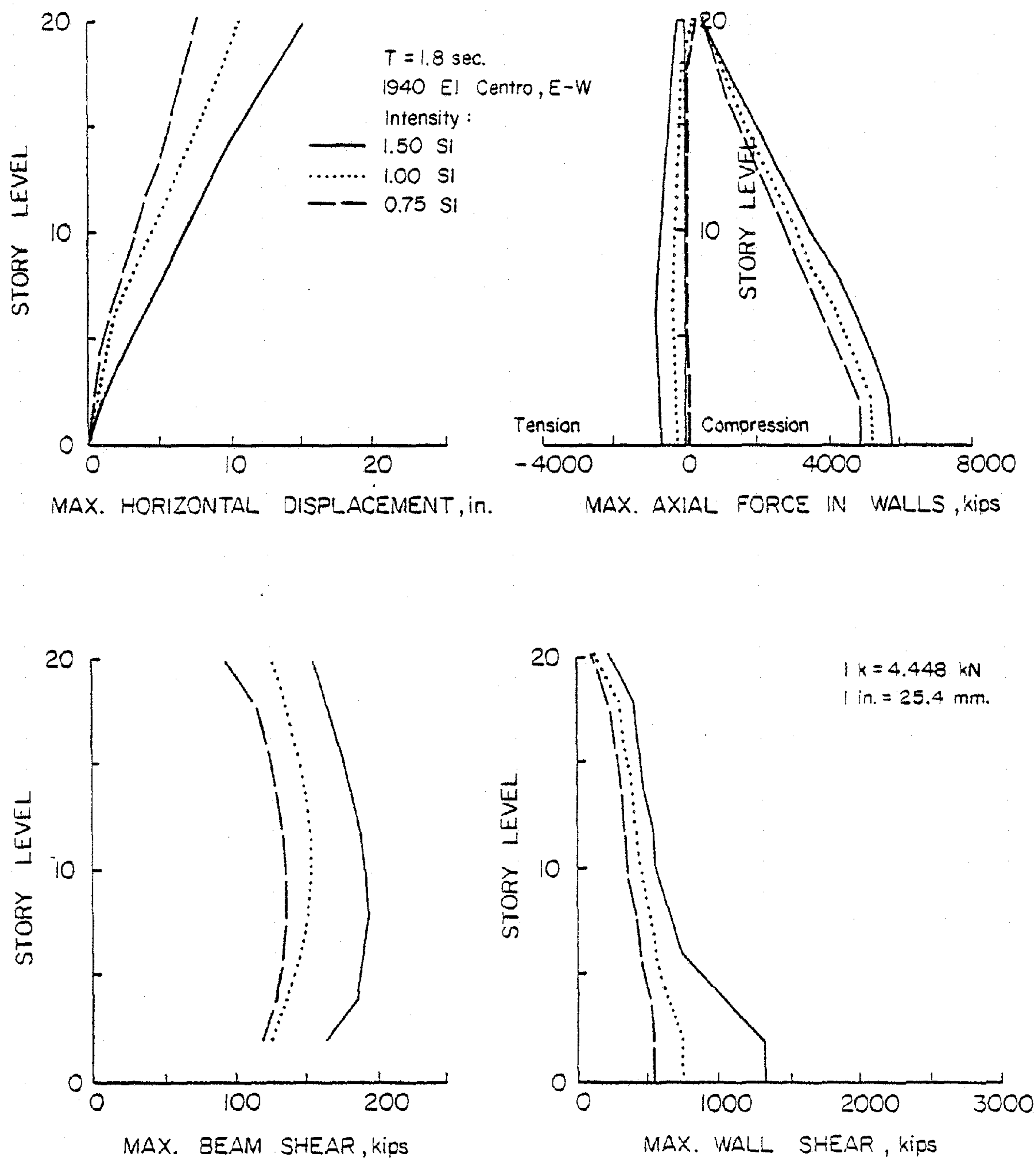


Fig. 42 Force and Displacement Envelopes Showing Effect of Earthquake Intensity

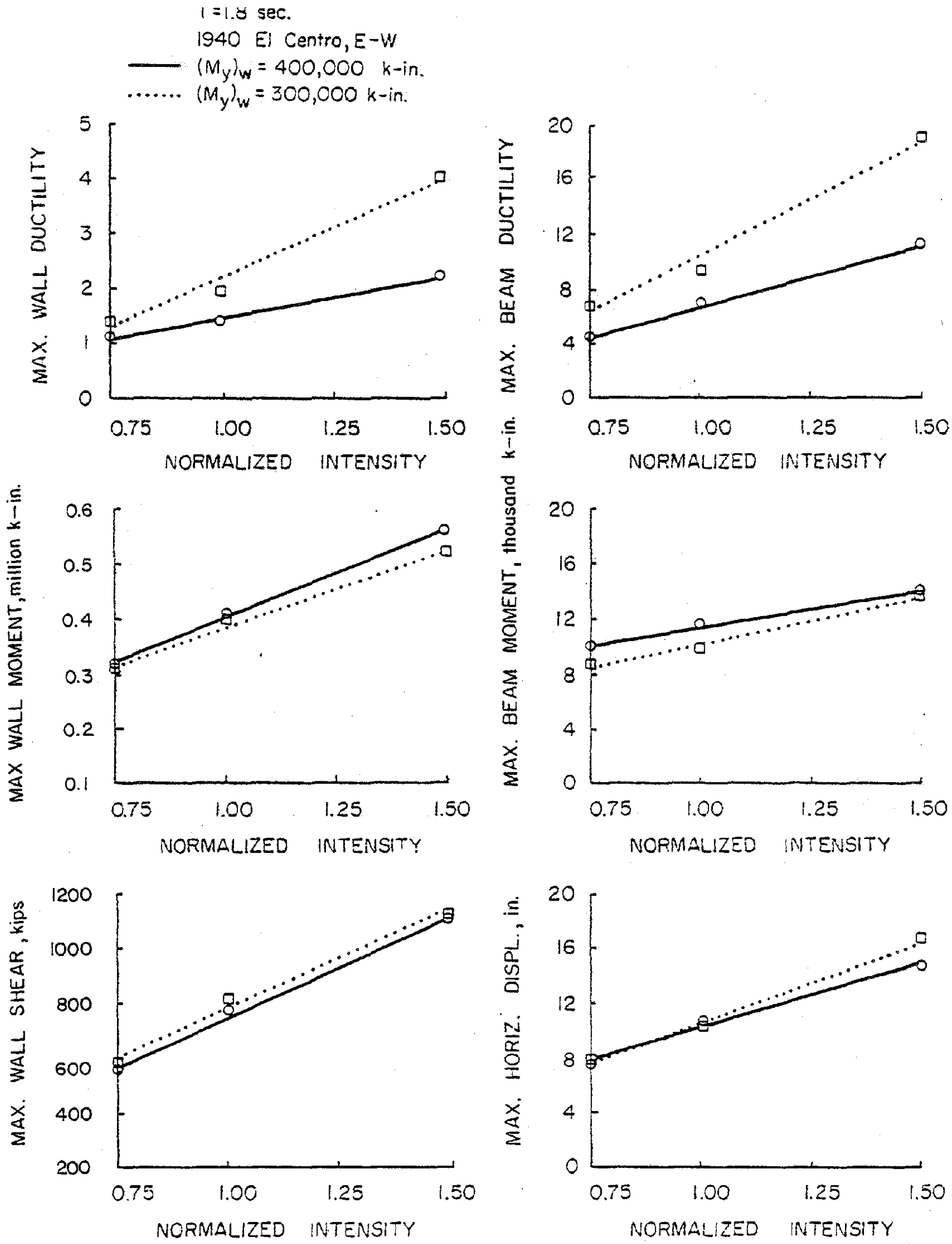


Fig. 43 Relationships Between Response Quantities and Earthquake Intensity

TABLE 11 - STRUCTURES ANALYZED FOR EFFECT OF EARTHQUAKE INTENSITY

Properties[1]	Structures	
	I	II
Fundamental Period (sec)	1.8	1.8
Wall Yield Moment[2] (k-in.)	400,000	300,000
Beam Yield Moment (k-in.)	4,000	3,000
Input Motion	1940 El Centro, E-W	1940 El Centro, E-W
Intensity:[3]		
i	1.50 SI	1.50 SI
ii	1.00 SI	1.00 SI
iii	0.75 SI	0.75 SI

Note:

1. For all other properties, see Table 1.
2. Wall yield moment given is at wall base. For strength taper along structure, see Table 1.
3. Intensity is specified in terms of spectrum intensity. SI denotes spectrum intensity of N-S component of 1940 El Centro record.

Duration of Earthquake

Most strong-motion accelerograms contain high intensity oscillations during a 5 to 15 second phase. Other researchers(11,12,13) have shown that damage to a structure is most likely to occur during this intense phase.

Effect of earthquake duration on nonlinear response should be viewed along with other characteristics of ground accelerograms. If a ground motion exhibits intense oscillations during a short period of time, then the effect of duration beyond this intense phase may not be significant. Previously recorded strong-motion accelerograms generally do not show intense pulses for an extended time period.

Another point that deserves attention is the interaction between earthquake duration, period of structure, and frequency characteristics of ground motion. If earthquake duration is such that elongation in structure period puts the dominant vibration period within the peaking range of the input motion, strong structural response can be expected. Interaction between vibration periods of structures and frequency characteristics of input motions was previously discussed under "Earthquake Frequency Characteristics."

Dynamic inelastic analyses were carried out to investigate the effect of earthquake duration on coupled wall structures. The E-W component of the 1940 El Centro record was selected as the input motion. The velocity response spectrum for this earthquake record shows high response for structures having fundamental periods in the range of 1.8 sec to 4.0 sec. The structure selected for this particular case has an initial fundamental period of 1.8 sec. Any softening in the structure due to yielding would elongate the period. However, the input motion selected would remain critical for a lengthened period within the range of 1.8 to 4.0 sec.

Two analyses were carried out with 10 sec and 20 sec duration, respectively. To minimize variation in intensities of oscillations between the two analyses, the 20-second analysis was conducted using an artificial accelerogram consisting of twice the first 10 seconds of the E-W component of the 1940 El Centro record. This implies that the 20 sec duration used in the second analysis was obtained by repeating the 10-second accelerogram used in the first analysis.

Response envelopes for the two analyses are compared in Figs. 44 and 45. Results show a considerable increase in maximum wall ductility at the base with increasing duration. In general, all other response quantities were increased by a longer duration of ground motion. It should be noted, however, that the ground motion used for the 20-second analysis was made of relatively intense oscillations for the entire earthquake duration. Therefore, the accelerogram used for this case can be considered more severe than actual earthquakes that were previously recorded. For this reason, it would be a misleading conclusion to state that increased duration always causes more structural response. Instead, one has to correlate earthquake duration with its frequency content. The comparison of response in Figs. 44 and 45 indicates that under unfavorable conditions with respect to frequency content and structure period, the earthquake duration can have a significant effect on structural response quantities, especially maximum wall ductilities.

Another response quantity that was of interest in relation to earthquake duration was the amount of plastic energy absorbed

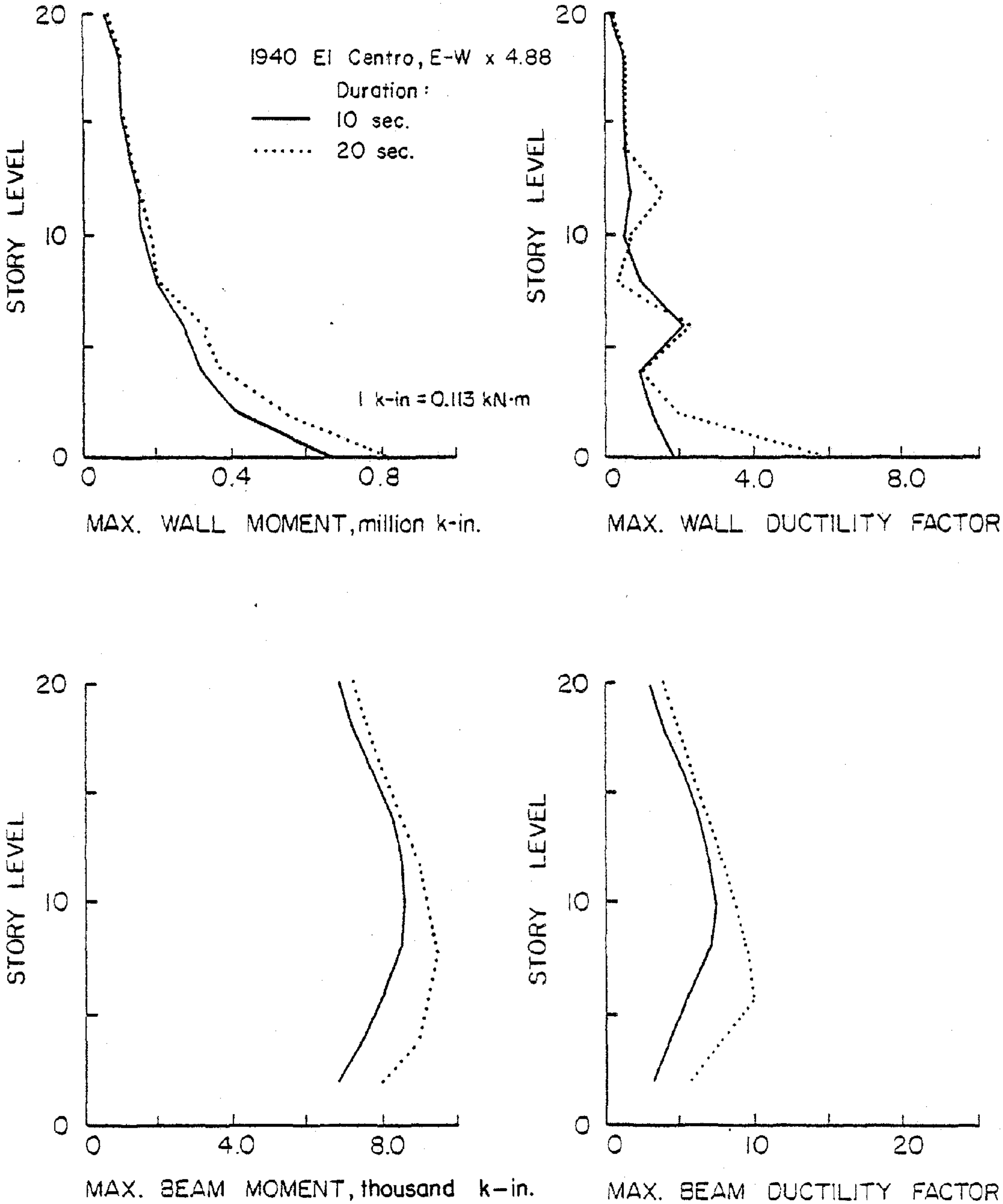


Fig. 44 Moment and Ductility Envelopes Showing Effect of Earthquake Duration

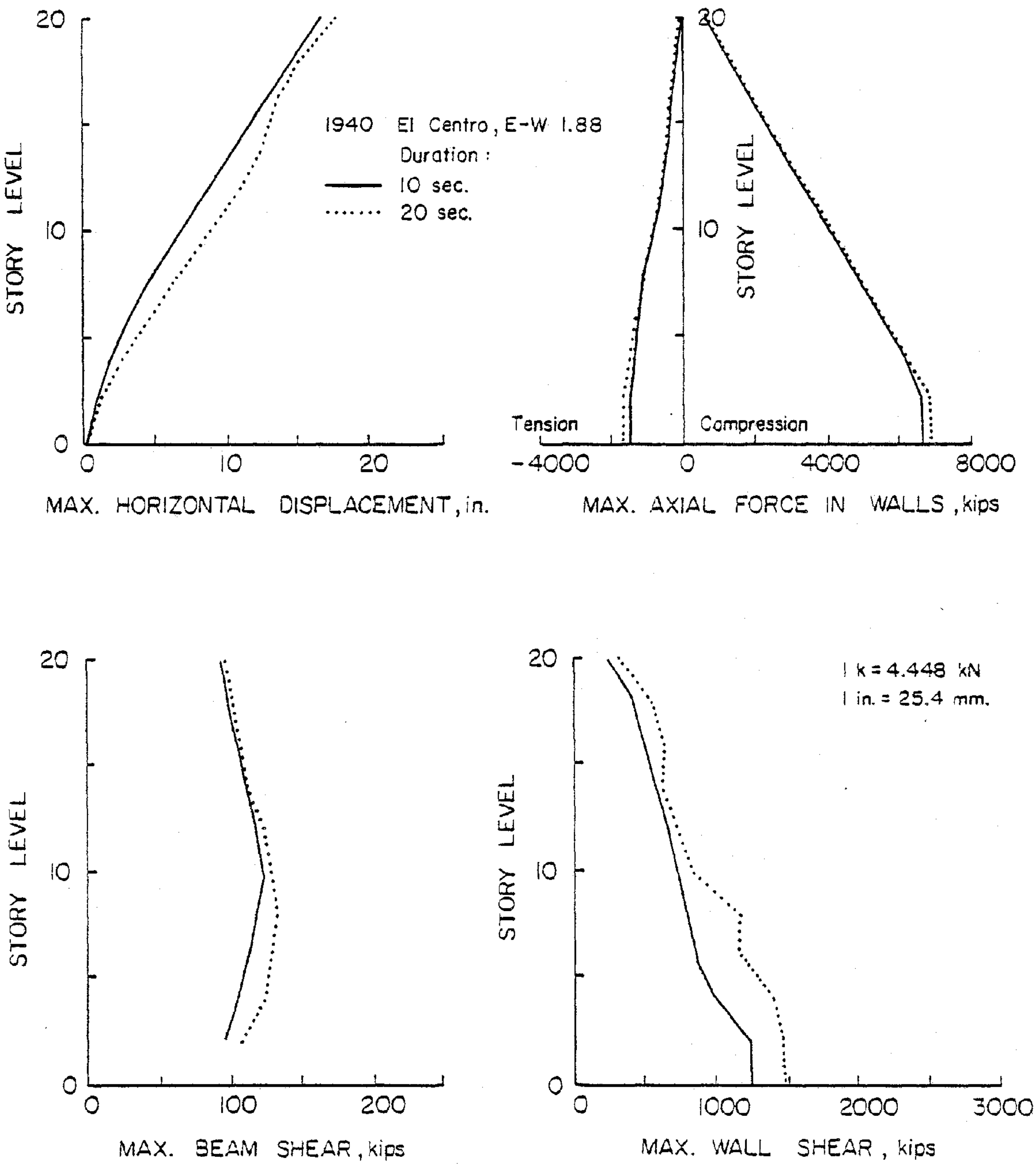


Fig. 45 Force and Displacement Envelopes Showing Effect of Earthquake Duration

by the structure. Structures in the inelastic range are required to dissipate more energy under increased earthquake duration, even if maximum response values are not necessarily increased. In the analyses conducted here, plastic deformations were accumulated during dynamic response. Cumulative plastic deformations were used as a means of representing energy dissipated during structural response. Cumulative plastic rotations along height of the structure are compared in Fig. 46 for the two cases considered here. The results indicate the expected trend. As the earthquake duration is increased from 10 sec to 20 sec, cumulative plastic rotations and hence the dissipated energy is increased.

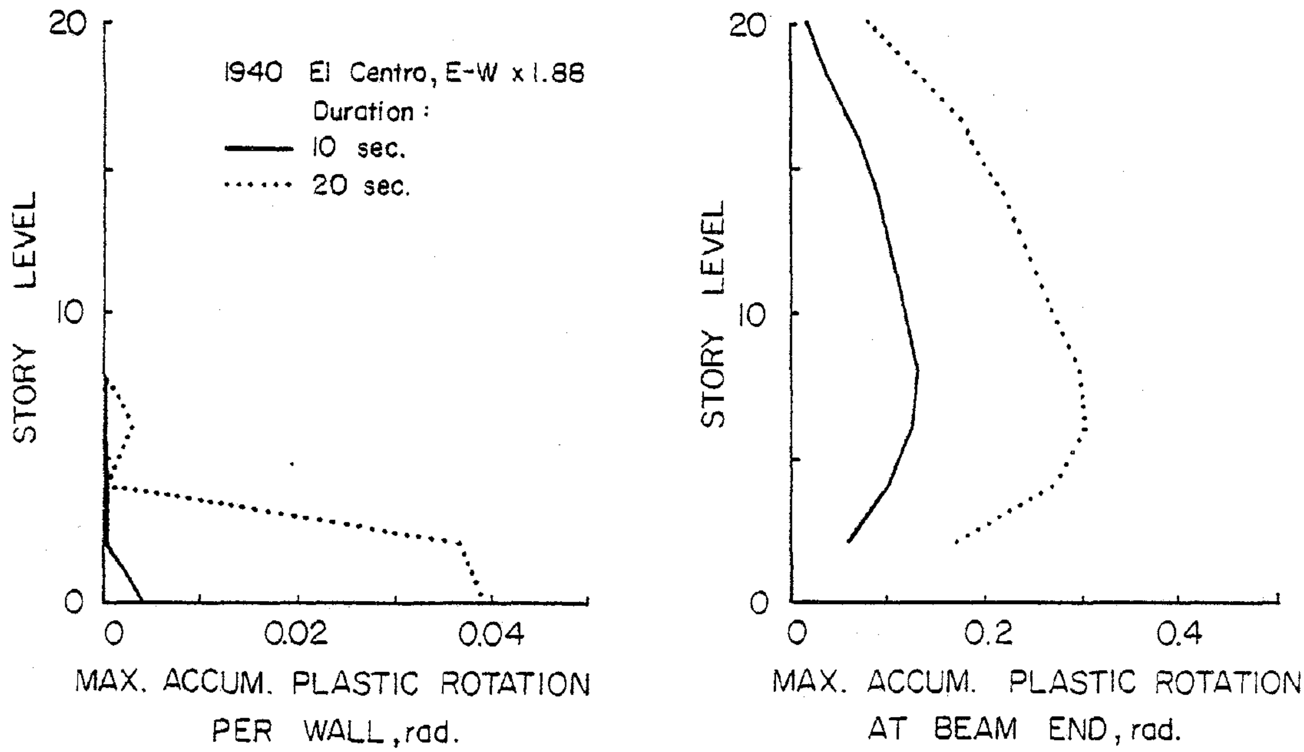


Fig. 46 Effect of Earthquake Duration on Cumulative Plastic Rotations

DESIGN INFORMATION

General

The first step in any structural design involves determination of design loads. Structures are proportioned for required strength to resist these design loads. Serviceability requirements usually set the limits for deformations, especially in critical regions of structures.

Earthquake resistant design differs from other types of structural design, mainly because of the nature of earthquake forces. In seismic design, forces are internal inertia forces rather than external loads. The magnitude and characteristics of earthquake induced inertia forces depend on the structure characteristics. Determination of structural response requires solution of differential equations expressing equilibrium of dynamic forces. Evaluation of the force history requires extensive computational effort and is normally done with the aid of a digital computer.

Due to complexities involved in evaluating dynamic structural response, engineers historically made simplifying assumptions for design. Earlier building code requirements were based on evaluation of a rigid, single degree of freedom system. Design base shear could be determined as the product of maximum ground acceleration and structure mass. However, dynamic reponse of actual structures differs significantly from rigid structure behavior. Flexibility and mass of structures produce vibration properties that directly affect structural response and hence the magnitude of design forces.

Consideration of structural stiffness and damping in formulating the equation of dynamic equilibrium results in more realistic modelling of structural response. Solution of the differential equation for a single degree of freedom system indicates that structural response depends, in a very direct fashion, on the fundamental period of vibration of the structure, which in turn depends on its stiffness and mass. Recognizing the importance of fundamental period of vibration on structural response, building codes later introduced a base shear coefficient that was based on the fundamental period.

Base shear for a single mass system can be evaluated using the Duhamel integral expression.[14] Solution of the Duhamel integral expression for a complete force history of any ground motion requires considerable computational effort. Furthermore, evaluation of structural response for a one mass system is only an approximation for multi-story structures. Solution of the equations of motion for a multi-story structure requires extensive computational effort and therefore has not been accepted as a practical design procedure for most structures.

Solution of differential equations for dynamic equilibrium of multi-mass systems appears to give the complete picture for design purposes. However, for moderately severe earthquakes this procedure results in design forces significantly higher than those given by current building code requirements. On the other hand, it has been observed that structures with considerably less strength than required by building codes have withstood rather severe earthquakes. This clearly indicates that structures continue resisting earthquake forces beyond their yield strength. Cracking and yielding allow absorption of a high percentage of total vibrational energy imposed on a structure. Energy is dissipated through the formation of plastic deformations. Development of forces indicated by elastic analysis is thereby prevented. Inelastic action results in smaller forces induced by ground excitations than predicted by elastic analysis. At the same time increased deformations are produced in critical regions.

It is not economically feasible to design building structures to remain elastic under severe earthquake excitations. Therefore, it is inevitable that inelastic action will take place in most structures under relatively severe ground excitations. Inelastic action reduces structure stiffness and elongates vibration period. These changes in structural properties affect dynamic response beyond yielding. Therefore, design procedures for earthquake resistant structures should consider inelastic behavior.

Building codes take inelasticity into account indirectly by requiring design for a lower force level than is actually expected under elastic conditions. Ductility coefficients introduced in recent building codes[15] modify the equivalent static design shear values for inelastic action. A rational approach to design requires that inelastic action be incorporated into the analysis. Use of inelastic dynamic response analysis requires appropriate modeling techniques. The amount of computational effort required cannot usually be justified for design purposes.

Dynamic inelastic response analysis was used in this research project to develop a rational and systematic design procedure for reinforced concrete coupled wall structures. A comprehensive parametric study was conducted to investigate significance of design variables. Qualitative observations were made on nonlinear behavior of coupled walls as affected by design variables. A large number of analyses was carried out to collect data for this design procedure. The proposed design procedure, design aids, and examples are discussed in subsequent sections.

Design Variables

Dynamic inelastic analyses were conducted to determine the significance of selected parameters on dynamic response of coupled walls. The following design variables, among the eighteen that were considered, were found to be significant for purposes of formulating a design procedure:

1. fundamental period
2. beam-to-wall stiffness ratio
3. strength (yield level)
4. beam-to-wall strength ratio
5. yield deformation level
6. earthquake characteristics

The above parameters can be classified as stiffness parameters, strength parameters, and ground motion parameters.

Fundamental period together with beam-to-wall stiffness ratio characterize structure stiffness. Member strengths were specified in terms of flexural yield level throughout this investigation. Wall strength and beam-to-wall strength ratio define structure strength. Yield deformation level controls member ductility. A design earthquake can be selected on the basis of earthquake frequency characteristics and intensity.

The following general observations were made concerning design variables and response quantities:

1. Fundamental period is the governing parameter in determining horizontal displacements for a given earthquake intensity.
2. Beam-to-wall stiffness ratio affects distribution of forces between members. For structures having the same strength, coupling between walls is controlled by beam-to-wall stiffness ratio. Stiff beams result in high coupling.
3. Yield deformation level is the controlling factor for rotational ductility. Structures that yield at the same rotation level produce the same ductility irrespective of their stiffness and strength.
4. Maximum force envelopes are directly related to structure strength.
5. Beam-to-wall strength ratio controls the sequence of plastification among members.
6. Horizontal displacement and ductility envelopes are not affected by structure mass, coupling arm length,

or clear beam span for structures having the same fundamental period, beam-to-wall stiffness ratio, and yielding deformation. Force envelopes are proportional to structure stiffness.

7. Ground motions that are critical in terms of their frequency characteristics show critical or near-critical response in most response quantities.

8. Response increases almost linearly with increasing earthquake intensity.

In addition to the major design variables discussed above, two other features of coupled wall structures deserve attention. These are beam strength decay, and stiffness and strength taper along the structure height. While early rapid strength decay causes increased ductility, strength and stiffnesss taper leads to yielding along the wall height, at locations of discontinuity.(1) Allowance should be made for these effects in inelastic design of coupled walls.

Proposed Design Procedure

A design procedure has been developed on the basis of nonlinear response analysis of coupled wall structures. Relationships among the significant design variables have been established. The main purpose of the design procedure is to determine design force levels corresponding to selected values of deformation capacity. Deformation capacities of members can be obtained from tests, in the form of ductility ratios. Many such test results have been reported. Tests conducted at the

Portland Cement Association[5,6,16] provide information on inelastic deformation capacities of structural walls and coupling beams. Assumptions and limitations involved in the development of design data are discussed under "Limitations and Discussion." The proposed design procedure involves the following steps:

1. Design for gravity and wind loads
2. Determine the following design variables

   Fundamental Period

   Beam-to-Wall Stiffness Ratio

   Wall Yield Moment

   Beam Yield Moment

   Wall Strength Parameter

   Beam-to-Wall Strength Ratio

   Earthquake Intensity

   Ductility Capacity
3. Determine maximum horizontal displacement
4. Determine required rotational ductilities in walls and beams
5. Revise flexural strength and/or stiffness if required ductilities exceed assumed ductility capacities
6. Determine design shear forces for walls and beams
7. Design walls and beams for shear
8. Design members along structure height

Each step in the design procedure is discussed below in detail:

1. <u>Preliminary Design</u>: Structural members are usually proportioned on the basis of gravity and wind loads

prior to seismic considerations. This preliminary design generally provides a good basis for estimating stiffness and strength of members.

2. Determination of Design Variables: The following design variables can be computed on the basis of preliminary member design:

   a) Fundamental Period, T: Initial fundamental period can be determined using preliminary member sizes. Effective stiffness is required to compute fundamental period. Concrete cracking must therefore be considered in determining member stiffness.

   b) Beam-to-Wall Stiffness Ratio, R_{SF}: Wall stiffness can be taken as $4(EI/L)_W$. Beam stiffness given by $k(EI/L)_{BM}$ requires consideration of end zones (portions of beams that are integral with walls) with infinite moments of inertia. Stiffness factors for beams with different end zone lengths are given in Table 12. "R_{SF}" is calculated as the ratio of effective beam stiffness to effective wall stiffness. Uniform wall and beam sizes were assumed in developing design charts included in this report.

   c) Wall Yield Moment, $(M_y)_W$: Wall strength is expressed in terms of wall yield moment. Wall yield moment is based on sectional characteristics of walls at the structure base. Axial force due to gravity prior to ground excitation, is included

TABLE 2 - BEAM STIFFNESS COEFFICIENTS

$$k = \left(1 + \frac{3}{(1-2a)^2}\right)\frac{1}{(1-2a)}$$

where aL = length of end zone

L = beam span length measured center to center of walls

a	k	a	k
0.00	4.00	0.24	23.20
0.02	4.43	0.26	29.20
0.04	4.94	0.28	37.50
0.06	5.54	0.30	49.40
0.08	6.25	0.32	67.10
0.10	7.11	0.34	94.70
0.12	8.15	0.36	140.00
0.14	9.43	0.38	221.00
0.16	11.00	0.40	380.00
0.18	13.00	0.42	739.00
0.20	15.60	0.44	1740.00
0.22	18.80	0.46	5870.00

in the calculation of yield capacity of a wall section. Wall sectional properties are assumed to be symmetric about the wall centerline.

d) Beam Yield Moment, $(M_y)_{BM}$: Beam strength is expressed in terms of beam yield moment at the face of the wall. Beam yield moment can be computed from beam section properties. Beam capacities at the wall face are assumed to be equal for bending in both directions.

e) Wall Strength Parameter, f_w: Wall yield moment is expressed in terms of wall strength parameter to determine the required rotational ductility from design charts. Wall strength parameter is defined as moment in the structure used to develop design charts, that produces rotation equal to yield rotation of the structure under consideration. This parameter is expressed as:

$$f_w = \frac{3.6 \times 10^{10}}{(EI_e)_w} \times (M_y)_w \qquad (19)$$

where $(EI_e)_w$ is the effective flexural rigidity for walls in k-in.^{2} and $(M_y)_w$ is wall yield moment expressed in k-in. It should be noted that when $(EI_e)_w = 3.6 \times 10^{10}$ k-in.^{2}, the structure is same as the reference structure for which the design data were developed. In this case, $f_w = (M_y)_w$.

f) <u>Beam-to-Wall Strength Ratio</u>, R_y: Relative strength between beams and walls is expressed by beam-to-wall strength ratio. Beam moment used for this ratio is measured at a distance 3.0 ft away from midspan when beam yielding takes place at the wall face. The section 3.0 ft from mid-span coincides with the location of the face of the wall of the reference structure that was used to generate design data. Therefore, beam strength at the wall face is multiplied by $6.0/\ell_c$ to find beam moment at 3 ft away from mid span. Beam-to-wall stiffness ratio is then defined as follows:

$$R_y = \frac{(M_y)_{BM}}{(M_y)_w} \times \frac{6.0}{\ell_c} \qquad (20)$$

where ℓ_c is clear beam span in feet.

g) Earthquake Intensity, Spectrum Intensity (SI) of 1940 El Centro, N-S record was used as a reference intensity. SI was defined earlier under "Intensity of Input Motions." Increments of SI were selected for different seismic risk zones. For design purposes, an increment of SI is required to characterize the seismic activity and risk in the area considered. An intensity of 1.5 SI is considered applicable for major damage zones that are on or around major fault systems. Intensity of 0.75 SI can be used for minor damage zones. Intermediate values can be used for moderate damage and major damage zones that are distant from major fault lines.

h) Ductility Capacity, Ductility capacities of members have to be assumed for comparison with ductility requirements. Ductility capacities can be determined from tests. Tests conducted at the Portland Cement Association[5, 6, 16] provide information on ductility ratios of walls and coupling beams. It is beyond the scope of this report to survey and summarize all the test results that are available. However, rotational ductility ratios of up to 3 and up to 6 for properly detailed walls and beams respectively, can be taken as initial estimates of ductility capacities.

3. <u>Design for Damage Control</u>: Damage control is associated with deformations along the height of the structure. Maximum deformations in earthquake resistant design are usually limited to control damage to nonstructural elements.

   In this design procedure, a chart similar to the one shown in Fig. 47 is provided to determine maximum horizontal displacement. This response quantity is governed by the fundamental period to a great extent. In this step, maximum horizontal displacement can be obtained from a design chart for specific values of initial fundamental period and earthquake intensity. This displacement can then be compared with maximum allowable deformation.

4. <u>Design for Inelastic Deformation Capacity</u>: An important step in inelastic design of concrete structures is determination of inelastic deformation capacity of members. Inelastic deformations are directly related to the energy dissipation capacity of a structure. Therefore, it is generally desirable to design and detail reinforced concrete members so that they possess high ductility.

   In inelastic design, it is necessary to establish ductility requirements in a structure for given ground

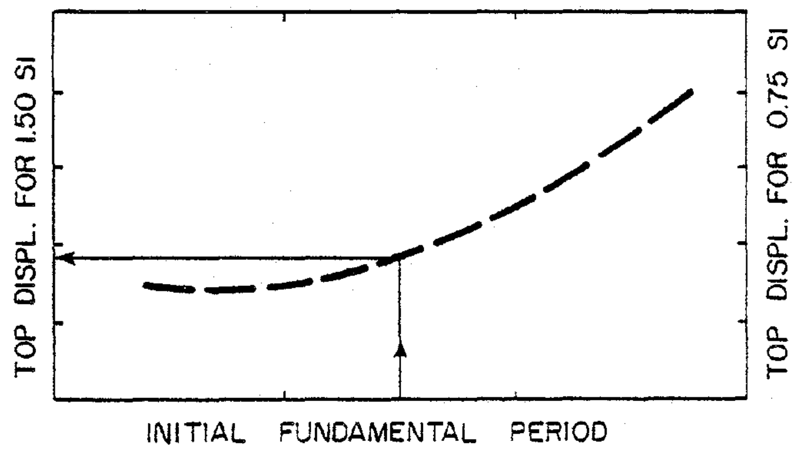


Fig. 47 Sample Design Chart for Top Displacements

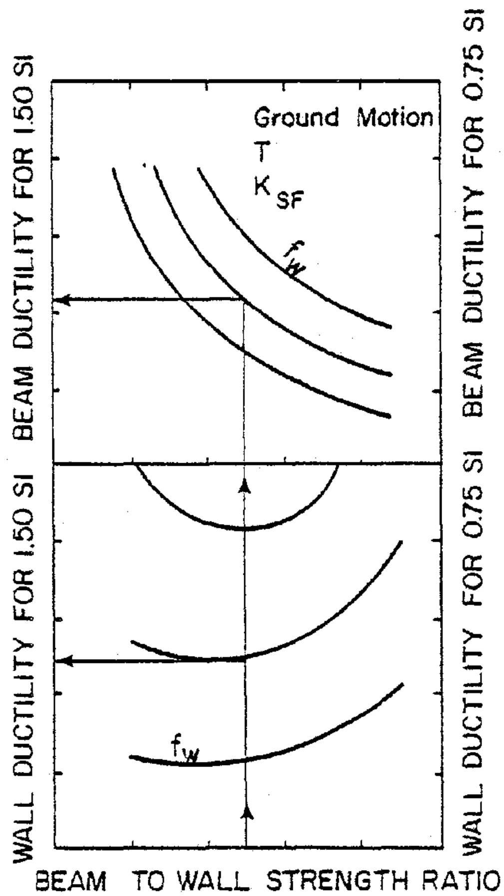


Fig. 48 Sample Design Chart for Ductility Ratios

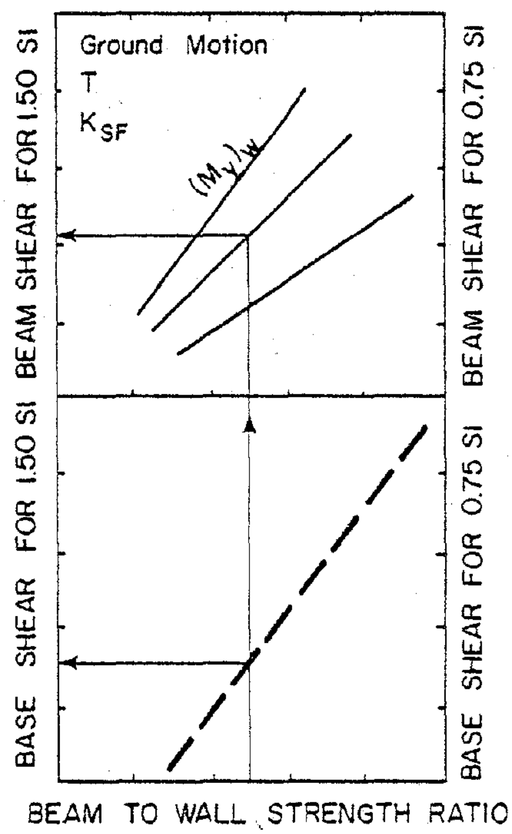


Fig. 49 Sample Design Chart for Shear

motion and intensity. Inelastic deformation capacity is commonly expressed as ductility factor or ratio as described under the heading "Rotational Ductility Factor." In this design procedure, maximum required ductility factors for walls and beams are obtained from a chart similar to the one shown in Fig. 48. Rotational ductilities under a specific earthquake motion and intensity can be determined for a given coupled wall structure with specific values of fundamental period, wall strength parameter, and beam-to-wall stiffness and strength ratios. These ductility demands are then compared with member ductility capacities. If the assumed ductility capacity is exceeded, a change in member stiffness and/or strength is required. Flexural strength (yield level) of members can be determined for assumed rotational ductility capacities from the same design charts.

5. <u>Design for Shear Capacity</u>: Structural walls are generally designed such that flexural yielding occurs prior to shear distress. Under inelastic loading conditions, structural members should maintain their shear capacity to avoid shear failure. This requirement implies that earthquake resistant coupled walls should be able to develop required ductility under calculated design shear forces.

   In the proposed design procedure, maximum design shear forces in walls and beams are found from a chart similar to the one shown in Fig. 49. This chart provides maximum dynamic shear forces for structures with specific values of fundamental period, wall flexural strength, beam-to-wall stiffness and strength ratios, and design earthquake intensity. The required shear reinforcement can then be determined to resist these shear forces.

Results of the analyses indicate that the relationship between dynamic base shear and flexural capacity of walls does not always follow a specific pattern. Design base shear shows a scatter when plotted against beam-to-wall strength ratio for different values of wall strength. Maximum base shears obtained by dynamic inelastic analysis are found to be significantly higher than base shears associated with maximum base moments. This can be attributed to the fact that maximum moments are associated primarily with first made effects while maximum dynamic shears are associated with higher mode effects. Similar response was found in analyses of isolated walls.(17) In Ref. 17 several reasons were put forward for reducing maximum dynamic shears for design purposes.

Shear strength equations in current building codes are based largely on static tests of members with shear span ratios commonly found in beams. However, maximum dynamic shears associated with higher modes occur at very high strain rates, are of extremely short duration, and are associated with short shear spans. Research is needed to establish correlation between dynamic shear demand and capacity for design purposes. In Ref. 17, a reduction factor, r_v, applied to dynamic base shears was suggested for use in shear design of isolated walls. A similar approach is suggested for coupled wall systems.

6. <u>Design of Members Along Structure Height</u>: The design procedure described in the preceding steps is based on maximum deformation and force levels. Maximum response in walls occurs at the base, in the critical region where most of the hinging takes place. Therefore, design forces and ductilities determined for walls in the above steps apply to the critical region of walls at the structure base.

Strength and deformation requirements are lower in the upper stories of multistory structures. For this reason, it is accepted practice to reduce strength and size of members in the upper stories. Dynamic response analyses of this investigation indicate that coupled walls with uniform cross-sectional dimensions perform better under earthquake excitations. By having uniform wall stiffness along the structure height, inelastic deformations in the upper stories are eliminated to a great extent. Therefore, in this design procedure, it is recommended that uniform member stiffness be provided along the structure height.

Because of the reduction in maximum moments and shears in upper stories, member strength can also be reduced with increasing height. Observations on maximum moment and shear diagrams indicate that moments and shears along structure height can be approximated by a linear variation between the maximum value at the base and the minimum value at the top. Based on this observation, the following empirical relationships are proposed to determine design moments and shears along structure height so that yielding in the upper portions of walls are either eliminated or substantially reduced.

$$M_i = 0.1\ (M_y)_w \quad 1.0 + \left(\frac{h - x_i}{h}\right) 9.0$$

$$V_i = 0.15\ V_b \quad 1.0 + \left(\frac{h - x_i}{h}\right) 5.7$$

Where "M_i" and "V_i" are design moment and shear at story level "i". Wall flexural strength (yield moment) at base and maximum base shear are denoted by "$(M_y)_w$" and "V_b", respectively. Total structure height and story height of interest are denoted by "h" and "x_i", respectively. These equations are applicable to design of walls in the upper stories. Structure base (critical region) should be designed with the full value of $(M_y)_w$ and V_b. This region usually includes more than one story level at the base. Tests indicate that the hinging region usually extends a distance equal to the wall width.[5,6]

Sample Design Aids

Three types of design chart have been provided for use with the proposed design procedure. These charts can be used to determine structure displacement, member ductility demands, and design shear force levels. Each chart has two vertical scales representing two different earthquake intensities. The intent is to allow determination of design quantities for different levels of design earthquake intensity. It was previously shown that response quantities are related to earthquake intensity in an almost linear manner. Therefore, the two scales provided in the design charts enable users to interpolate for intermediate values of earthquake intensity.

Fig. 50 shows the relationship between top displacement and fundamental period. Among all the structural parameters considered in this investigation, fundamental period appears to be the governing parameter for displacements. Therefore, Fig. 50 can be used to estimate maximum top displacement as a function of fundamental period, independent of other structural parameters.

Figs. 51 and 52 provide wall and beam ductilities corresponding to two different beam-to-wall stiffness ratios. These sample design aids are given for structures with fundamental period equal to 1.8 sec. For a coupled wall structure with a

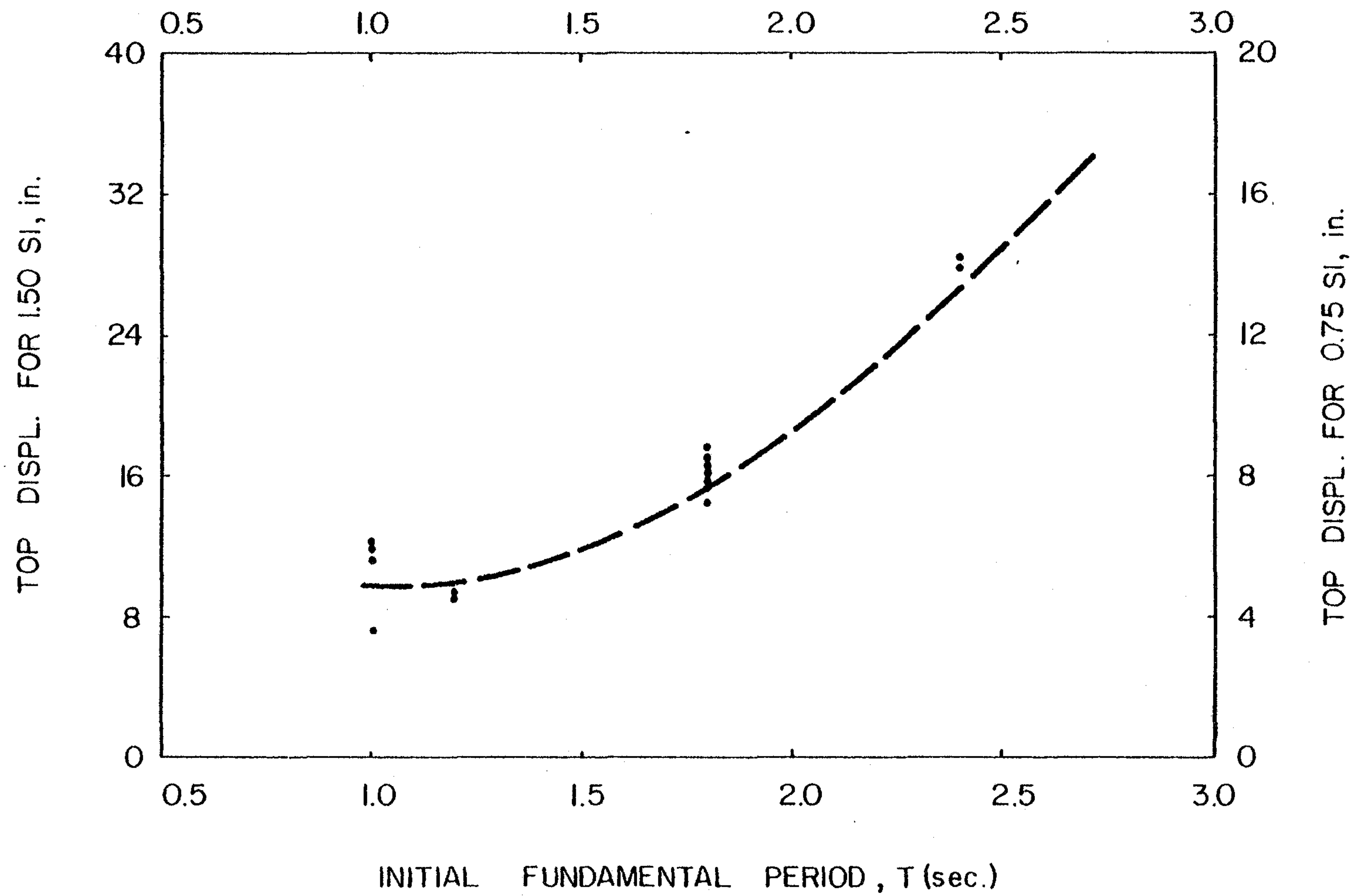


Fig. 50 Design Chart for Determining Maximum Top Displacement

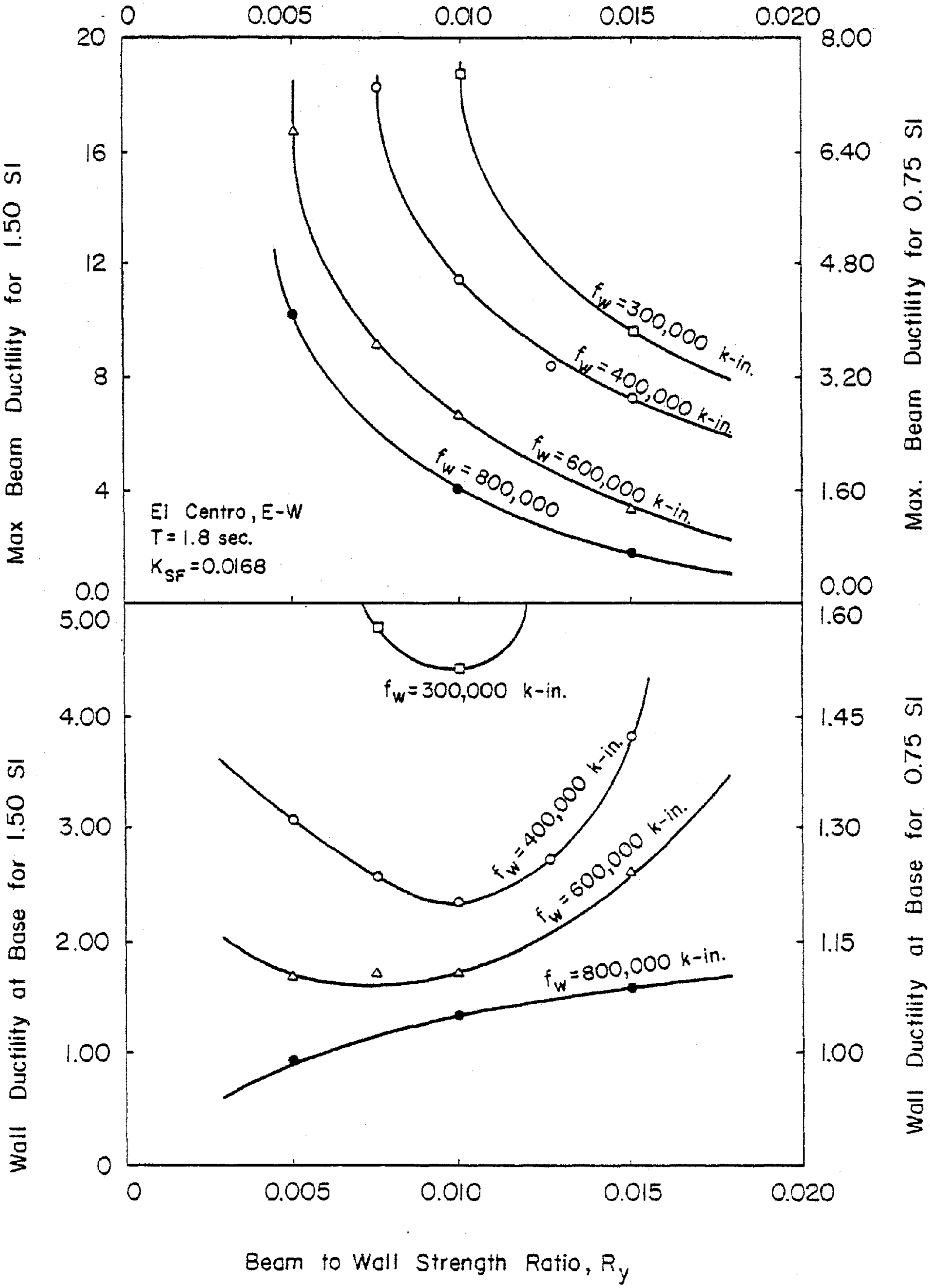


Fig. 51 Design Chart for Determining Ductility Ratios

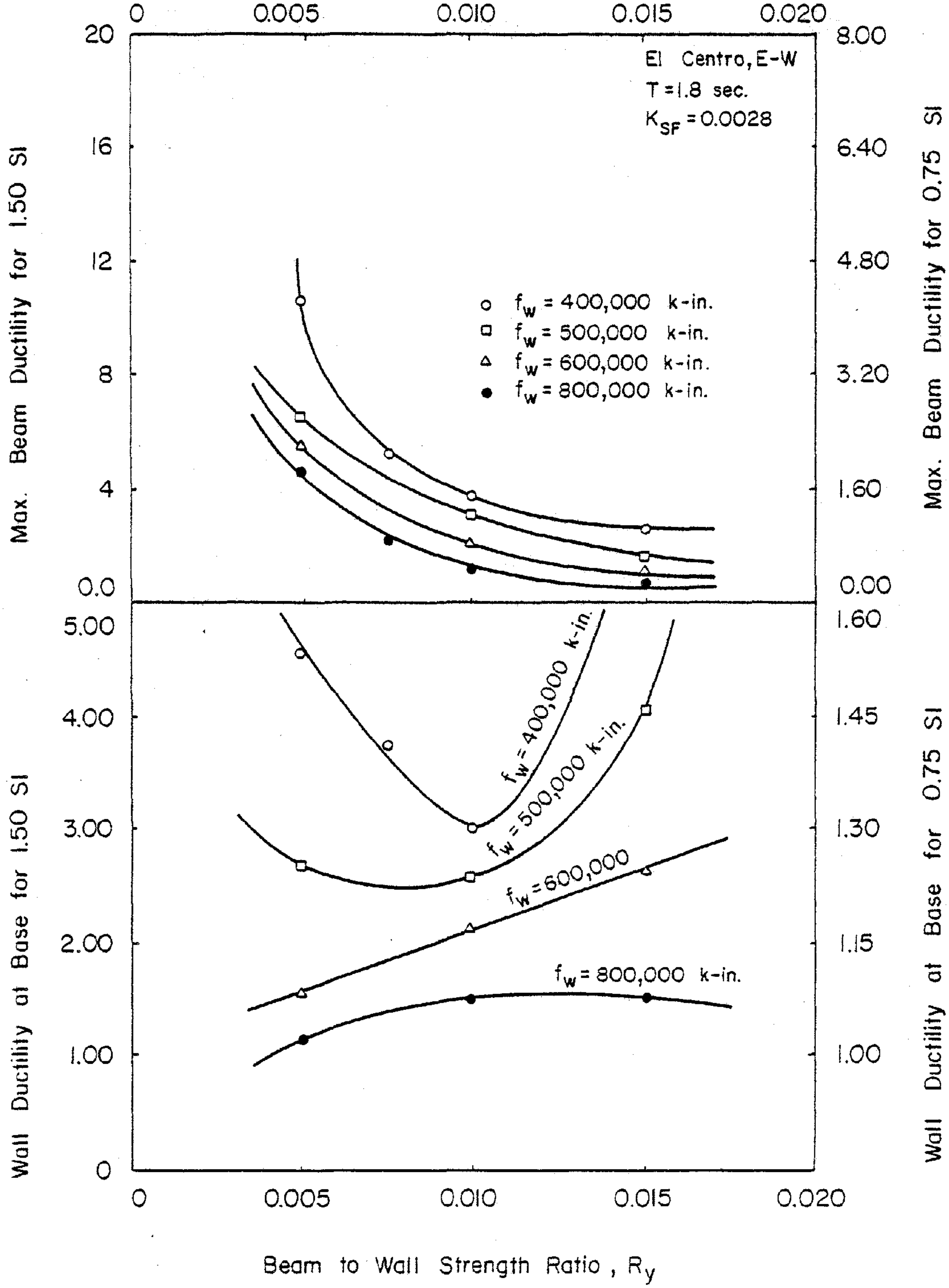


Fig. 52 Design Chart for Determining Ductility Ratios

given beam-to-wall strength ratio, R_y, maximum wall ductility at the base and maximum beam ductility can be determined from the same chart. The chart arrangement is such that users can see the relationship between wall and beam ductilities. This is important in determining the required degree of plastification among members and the related energy dissipation capacity.

Wall strength parameter, f_w, rather than the wall yield moment is used in ductility charts. This is mainly to introduce yield rotation as a parameter. Two structures with identical yield moments do not exhibit the same ductility demand if they do not yield at the same deformation level.

Figs. 53 and 54 can be used to determine design shear force levels for walls and beams. Results of dynamic analyses indicate that base shear in walls cannot be related to flexural strength in a systematic manner. Similar observations were also made by other researchers.[17] As indicated under "Proposed Design Procedure" a reduction factor, r_v, is proposed to adjust maximum calculated dynamic base shears for design use. Calculations given under "Design Example" indicate a suggested range of 0.55 to 0.80 for use with the design charts.

Beam shear forces, on the other hand, show systematic variation with flexural strength. Maximum design shear for beams can be obtained from Figs. 53 and 54.

The sample design aids given in this chapter cover a wide range of beam and wall strength and stiffness. Ductility and shear design charts are given only for structures with fundamental period of T = 1.80 sec. For a complete set of design

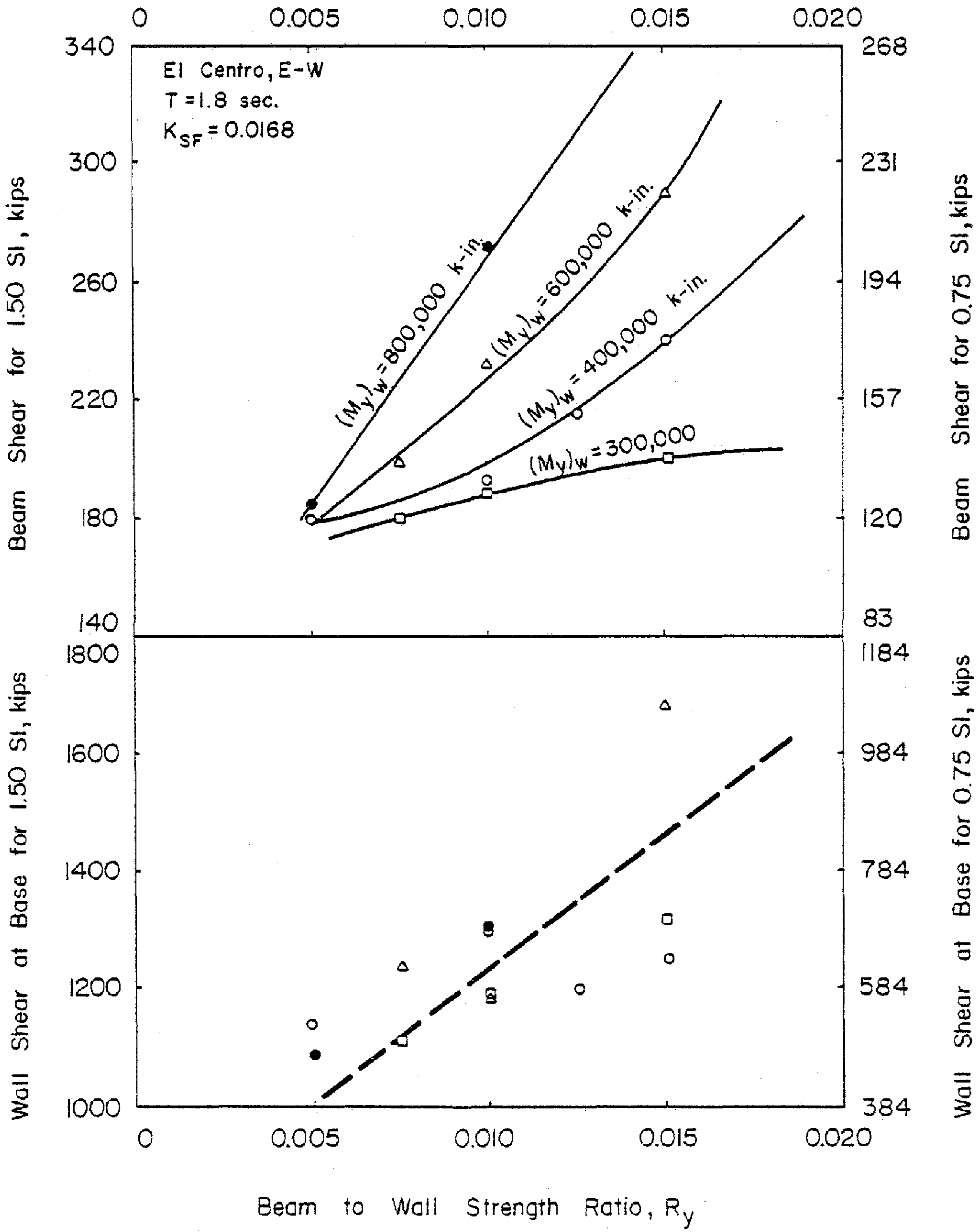


Fig. 53 Design Chart for Determining Shear Force Level

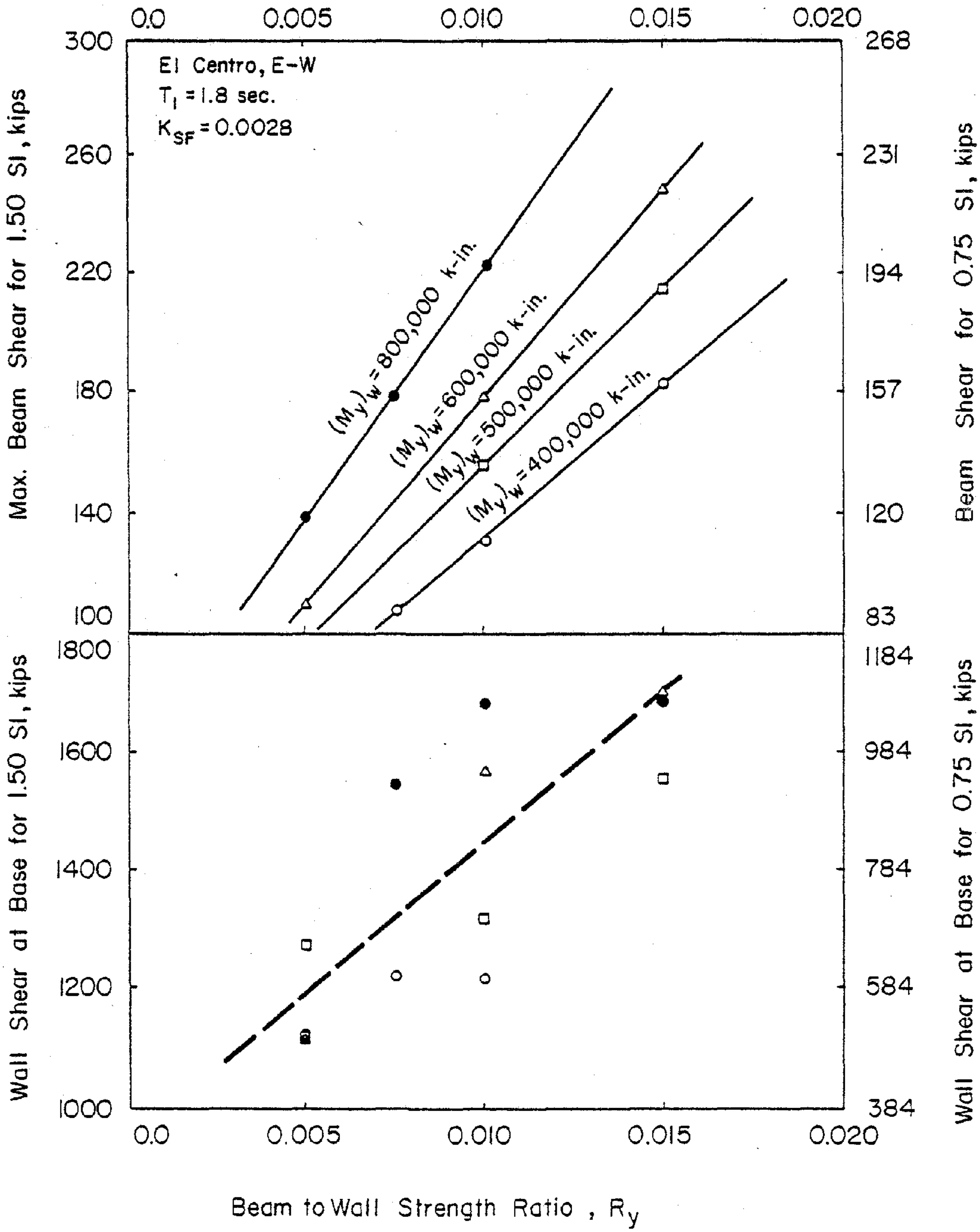


Fig. 54 Design Chart for Determining Shear Force Level

aids, similar charts can be produced for structures with different initial fundamental periods. However, the design procedure developed in this investigation can be applied to other coupled wall structures with different fundamental periods. Discussion and limitations of the design procedure and the design aids are given under "Limitations and Discussion.

Design Example

Fig. 55 illustrates a typical floor plan and elevation of a 20-story office building to be built in San Francisco, California. Floor slabs are 8-in.-thick two-way slabs spanning between 20-in. deep and 14-in. wide girders. Columns are 24-in. by 24-in. The structure has uniform walls that are 14-in. thick and 22-ft wide. Floor slabs are assumed sufficiently stiff to cause all points on the same floor level to deflect horizontally by the same amount. Normal weight concrete with $f'_c = 4$ ksi is used throughout the structure. Design live load is 100 psf. It is assumed that 25% of the live load is always present in the structure. It is assumed that the columns carry gravity loads only and do not contribute resistance to seismic forces. Walls are assumed to provide the full resistance to seismic forces. Total gravity load tributary to each wall is 134 kips per floor. This is made up of 118 kips of dead load, including superimposed dead loads, and 16 kips of live load representing 25% of total live load. Total mass effective for inertia forces is 7.55 kip-sec^2/ft per floor per wall.

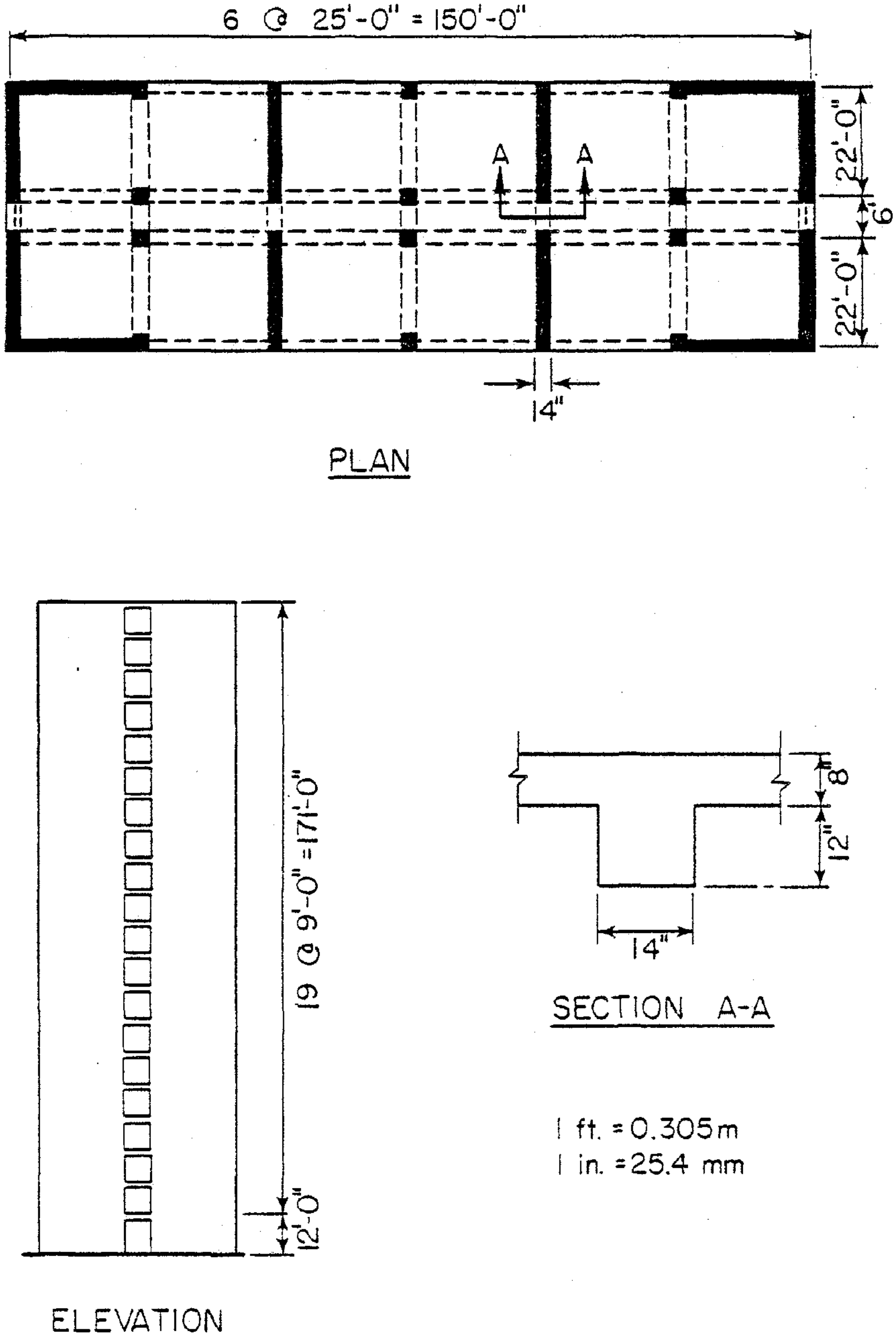


Fig. 55 Design Example

Design steps are as follows:

1. Design for Gravity and Wind Loads

Preliminary design under wind and gravity loads indicates a yield moment at the base of the wall $(M_y)_w = 250{,}000$ k-in. under an axial load of 2700 kips. Coupling beam yield moment is $(M_y)_{BM} = 3500$ k-in.

2. Determine the Following Design Variables

Fundamental Period: T = 1.8 sec. (Found be using program DYFRQ. Equations given in UBC-79(15) could also be used for finding T)

Beam to Wall Stiffness Ratio:

Assume $I_e = 0.5I$

use wall and beam sectional dimensions given in Fig. 55 to compute effective moment of inertia

$(I_e)_w = 1.07 \times 10^7$ in^4

$E = 3600$ ksi

$(I_e)_{BM} = 7228$ in^4

see table 12 for stiffness coefficients

$k_w = 4$, $k_{BM} = 310$

$$\frac{4E(I_e)_w}{L_w} = \frac{4(3600)(1.07 \times 10^7)}{(9)(12)} = 1.4 \times 10^9 \text{ k-in.}$$

$$\frac{310(I_e)_w}{L_{BM}} = \frac{310(3600)(7228)}{(28)(12)} = 24.0 \times 10^6 \text{ k-in.}$$

$$R_{SF} = \frac{24.0 \times 10^6}{1.4 \times 10^9} = 0.071$$

Wall Yield Moment:

Based on preliminary design under gravity and wind loads $(M_y)_w$ = 250,000 k-in. under P = 2700 kips

Beam Yield Moment:

Based on preliminary design under gravity and wind loads $(M_y)_{BM}$ = 3500 k-in.

Wall Strength Parameter:

$$f_w = \frac{3.6\text{x}10^{10}}{\quad} \text{ x } (M_y)_w = \frac{3.6\text{x}10^{10}}{(3600)(1.07\text{x}10^{10})} \text{ x } 250{,}000$$

$$= 234{,}000\text{k-in.}$$

Beam-to-Wall Strength Ratio:

$$R_y = \frac{(M_y)_{BM}}{(M_y)_w} = \frac{3{,}500}{250{,}000} = 0.014$$

Earthquake Intensity:

Use 1.5SI for the area on or around major fault systems.

Ductility Capacity:

Assume walls can develop rotational ductility of 3.0 and beams can develop rotational ductility of 6.0. See PCA test results.[5,6,16] It is also assumed that within this ductility range no significant strength loss occurs due to reversed load cycles.

3. Determine Maximum Horizontal Displacements

From Fig. 50 for T = 1.80 sec. and Intensity = 1.5 SI

Top displacement = 15 in.

Drift ratio = 1/146 O.K.

4. Determine Required Ductilities

From Fig. 51 for T = 1.80 sec.

R_{SF} = 0.0171 (0.0168)

R_y = 0.014

f_w = 234,000 ksi

Intensity = 1.5 SI

Read; Max. wall ductility 3.0 N.G

Max. Beam ductility 6.0 N.G

5. Revise Flexural Strength (Yield Moments)

Increase beam and wall flexural capacities.

Try $(M_y)_w$ = 500,000 k-in. and $(M_y)_{BM}$ = 7500 k-in.

Repeat Steps 2 thru 4.

T = 1.8 sec. (remains unchanged)

R_{SF} = 0.071 (remains unchanged)

$(M_y)_w$ = 500,000 k-in.

$(M_y)_{BM}$ = 7000 k-in.

f_w = 467,000 k-in.

R_y = 0.014

Top displacement = 15 in (remains unchanged)

From Fig. 51 Read;

Max. wall ductility = 3 O.K.

Max. beam ductility = 6.0 O.K.

Therefore, use $(M_y)_w$ = 500,000 k-in.

and $(M_y)_{BM}$ = 7000 k-in.

6. Determine Design Shear Force

From Fig. 53 for T = 1.80 sec.

$R_{SF} = 0.0171$ (0.0168)

$(M_y)_w = 500,000$ k-in.

$R_y = 0.014$

Intensity = 1.5 SI

Read; Max. base shear = 1420 kips/wall

Read; Max. beam shear = 250 kips/beam

7. <u>Design Walls Along Structure Height</u>

It is assumed that wall strength is tapered at 10th story level.

Use equations given under "Proposed Design Procedure"

For $i = 10$

$h = 183$ ft

$x_{10} = 93$ ft.

$M_{10} = 270,000$ k-in.

$V_{10} = 486$ kips

For comparison, the design example was repeated for an intensity of 1.25 $\overline{SI}$. This intensity is considered to be approximately equivalent to the intensity implied by UBC requirements (Zone 4). Results for 1.25 $\overline{SI}$ are summarized as follows:

Base Wall Moment	=	410,000 k-in.
Beam Moment	=	6,000 k-in.
Base Wall Shear	=	1,200 kips
Beam Shear	=	205 kips
Max. Top Displacement	=	13 in.

As discussed under "Proposed Design Procedure," it is proposed that a reduction factor r_v, be applied to maximum dynamic wall shears obtained from the design charts. In the absence of data on shear capacity under dynamic loading a value for r_v can be obtained by comparing dynamic shears for an intensity of 1.25 $\overline{SI}$ with design shears obtained from current UBC requirements. Calculations shown under "Comparison with Current Design Practice" indicate an unfactored base shear of 964 kips for the structure considered. Applying load factors of 1.4 and 2.0 and comparing with dynamic shear for 1.25 $\overline{SI}$ gives,

For Load Factor = 1.4

$$r_v = \frac{1.4 \times 964}{2 \times 1200} = 0.56$$

For Load Factor = 2.0

$$r_v = \frac{2 \times 964}{2 \times 1200} = 0.80$$

Assuming a triangular lateral force distribution along the height and yielding in both walls and all coupling beams a total base shear of 1320 kips is calculated. Yield moments are based on intensity 1.25 $\overline{SI}$. For this case reduction factor r_v is calculated as,

$$r_v = \frac{1320}{2 \times 1200} = 0.55$$

Comparison with Current Design Practice

Design forces are calculated based on requirements of the Uniform Building Code, 1979 Edition.(15) A comparison between the proposed design procedure of this investigation and the UBC-1979 requirements is given.

Design Example: Determine the required flexural and shear strength of the 20-story office building specified in the previous section. Use UBC-1979 seismic requirements.

Mass per floor = 7.55 kip-sec^2/ft

Weight per floor = (7.55)(32.2) = 243 kips/wall

Total load = (243)(20)

= 4860 kips/wall

W = (4860)(2) = 9720 kips/coupled wall

Total base shear given by Eq. (12-1) of UBC-1979 is:

$$V = ZIKCSW$$

where W = 9720 kips

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{1.8}} = 0.0497$$

$S = 1.5$

$K = 1.33$

$I = 1.0$

$Z = 1.0$

$V = (1.0)(1.0)(1.33)(0.0497)(1.5)(9720)$

$= 964$ kips

Load Factor = 1.4

Base Shear = 964 x 1.4 = 1350 kips

$F_t = 0.07\ TV$

$F_t = 170$ kips

Distribution of forces along structure height:

$$F_x = \frac{(V - F_t)\ w_x h_x}{\sum_{i=1}^{20} w_i h_i}$$

TABLE 13 - EQUIVALENT STATIC FORCES BASED ON UBC-1979

Stories	h_x (ft)	$w_x h_x$ (k-ft)	F_x (kips)
20	183	62,256	110.7 + 170
19	174	59,194	105.3
18	165	56,133	99.9
17	156	53,071	94.4
16	147	50,009	88.9
15	138	46,948	83.5
14	129	43,886	78.1
13	120	40,824	72.6
12	111	37,762	67.2
11	102	34,700	61.7
10	93	31,639	56.3
9	84	28,577	50.8
8	75	25,515	45.4
7	66	22,453	40.0
6	57	19,391	34.5
5	48	16,330	29.1
4	39	13,268	23.6
3	30	10,206	18.1
2	21	7,144	12.7
1	12	4,082	7.3
Totals		663,390	1350.0

Equivalent static force, F_x, is given in Table 13 for each story level.

For shear design of wall

Load Factor = 2.0

Design Base Shear = 964 x 2.0 = 1930 kips

A static lateral load analysis provided beam and wall design moments and shears for the UBC specified earthquake forces.

A comparison of design results obtained from the two procedures is shown in Table 14. Design wall shears for the proposed design procedure are based on a reduction factor, r_v = 0.55. The comparison indicates that maximum top displacement determined by UBC-1979 is lower than the value obtained by the proposed design procedure. The proposed procedure is based on dynamic inelastic analysis whereas the UBC procedure involves elastic analysis under equivalent static earthquake forces. Although inelasticity is implied indirectly in the UBC procedure by the use of lower force levels than would be expected under elastic dynamic conditions, displacements calculated on the basis of elastic frame analysis, do not reflect this implied inelasticity.

Perhaps the most important difference between the two approaches is the design for inelastic deformation capacity. In the proposed design procedure, designers can determine strength and stiffness of members to achieve the desired level of ductility in each member. In the UBC approach, based on elastic analysis, designers have no means of assessing the level of inelastic deformation in individual members. In this

TABLE 14 - SUMMARY OF RESULTS FOR DESIGN EXAMPLE

Design Quantities	Proposed Design Procedure		UBC - 1979
	Intensity=1.50SI	Intensity=1.25SI	
Max.Top Displ.(in)	15	13	10
Design Moment(k-in)			
Base Wall	500,000	410,000	348,000
10th Story Wall	270,000	222,000	71,000
Beams	7,000	6,000	10,000
Design Shear(kips)			
Base Wall	781	660	965
10th Story Wall	445	377	737
Beams	172	141	287
Max.Wall Ductility	3	3	- -
Max. Beam Ductility	6	6	- -

respect, the proposed design procedure provides a more rational design approach that reflects inelastic behavior of members. The proposed procedure also provides a clear picture of member behavior. This enables designers to adjust stiffness and strength of individual members so that the required sequence and degrees of plastification take place.

Comparison of wall shears at the structure base indicates that unadjusted shears obtained by the proposed procedure are higher than those obtained from the existing UBC procedure. In the proposed design procedure it is suggested that a reduction factor be applied to calculated dynamic shears to produce design shears approximately equal to those obtained from current code requirements. The reduction factor is justified on the basis of lack of data correlating calculated dynamic shear demand with shear capacity.

It should also be pointed out however that the UBC design approach assigns half of the total base shear to each wall. Dynamic analysis of coupled walls indicates that distribution of forces between walls results in a higher shear force in the compression wall. While the shear force is higher, the capacity of walls in compression is improved by the presence of compressive forces. However, there is no assurance that under dynamic conditions, maximum shear force and maximum compression always occur simultaneously. A conservative approach may be to consider axial forces due to gravity loads only, when computing the nominal shear capacity with the proposed design shear values given in this investigation.

Limitations and Discussion

The design procedure outlined in this chapter was developed on the basis of nonlinear dynamic analysis of a 20-story coupled wall structure. Although the general procedure is applicable to any plane multistory coupled wall structure, the design data and the sample design aids developed are limited to 20-story structures. Design charts could be produced for structures in different height ranges.

The structure used in the analyses was a symmetric coupled wall structure with equal capacities in each wall. Strength and stiffness of individual walls may vary in practice for architectural reasons. Results of this investigation are not applicable to structures with significant differences in strength and stiffness between individual walls.

Soil-structure interaction was not considered in the investigation. The structure base was assumed to be in fully fixed condition.

Rotational ductility was used to express inelastic deformation requirements of members. Rotational ductility for walls and beams was previously discussed under the heading "Rotational Ductility Factor." Ductility ratios used in the design procedure refer to ductility requirements under a specific earthquake excitation. Ductility capacity can only be determined by tests and is beyond the scope of this investigation.

Maximum axial forces in walls are not given in the design charts. However, effects of axial force on design force levels and ductilities are included. Maximum rotational ductility factor in walls is governed by the wall in tension. Increase in ductility due to reduced wall strength under increased tension is included in design charts. Excessive tension is indicated by increased wall ductilities. Similarly, increase in shear due to increased compression is included in maximum shear values given in design charts.

It is recommended that both flexural and shear capacities be determined under axial loads due to gravity loads. Flexural yield level used in design is based on sectional characteristics, including effects of axial stress due to gravity loads.

Design earthquakes used in developing design data are listed in Fig. 36.

SUMMARY AND CONCLUSIONS

Nonlinear behavior of reinforced concrete coupled wall structures was investigated under earthquake induced inertia forces. A 20-story coupled wall structure was selected for dynamic inelastic analysis.

Effects of selected structural parameters on nonlinear response of coupled walls was investigated. Structural parameters characterizing stiffness and strength properties of structures were selected for examination. Wall strength, fundamental period, beam-to-wall stiffness, and strength ratios were among the parameters considered. These parameters play significant roles in structural behavior. Inelastic deformation and degree of plastification among members and amount of coupling are controlled by these parameters.

Emphasis was placed on generalization of results for development of a general design procedure. Therefore, other structural parameters based on structure geometry and loading were also considered. Different values of coupling arm, clear beam span, tributary mass for inertia, and gravity forces were considered to examine their effects on response. Significance of each structural parameter was assessed. A procedure was established to predict response of structures having different structural layouts based on their stiffness and strength characteristics.

Six ground accelerograms were selected to examine the general character of earthquake excitations. Significance of earthquake frequency characteristics, intensity, and duration on structural response was investigated.

Dynamic analyses were expanded to cover a wide range of stiffness and strength parameters. Results were used to formulate a general design procedure. The significant structural and ground motion parameters were considered as design variables. A step-by-step design procedure was outlined. Sample design charts were provided for 20-story symmetric coupled walls. A design example was solved to illustrate the use of design aids. A comparison was made between existing and proposed design procedures.

Based on the results of this investigation, the following conclusions can be made:

1. Fundamental period plays an important role in dynamic response. Fundamental periods of coupled wall struc-

tures reflect overall structural stiffness and directly control horizontal displacements. Structures with short initial fundamental periods (stiff structures) show considerably less horizontal displacement than structures with long periods. For all practical purposes, it is sufficient to know the initial fundamental period to estimate maximum horizontal displacements.

2. Ductility demand as expressed by rotational ductility ratio increases with decreasing fundamental period. Occurence of higher ductility in stiff structures indicates a reduction in yield rotation rather than an increase in maximum inelastic rotation. Flexible structures with longer periods show higher values of maximum rotation.

3. Beam-to-wall stiffness ratio is an important parameter that controls the degree of coupling between the walls. Structures with high beam-to-wall stiffness ratio show a higher degree of coupling accompanied by increased axial tension and compression in the walls.

4. The degree of inelasticity in a structure is directly related to flexural yield level of members. A structure with a high yield level may behave elastically under a given earthquake excitation. As flexural yield level is reduced, yielding of members takes place. Further reduction in flexural yield level increases inelastic deformations and the amount of vibrational

energy dissipated by the structure. Reduction of 50% in wall yield level of the structure considered produces an increase of 100% in maximum wall ductility. In the same structure, with constant beam-to-wall strength ratio, the same decrease in wall yield level produces an increase of 250% in maximum beam ductility. Although early yielding in structures elongates the period, maximum horizontal displacements do not appear to be significantly affected by yield level provided that maximum wall ductilities are in the range of 2.0 to 6.0.

5. Beam-to-wall strength ratio is an important design variable that controls sequence of plastification among members. In inelastic design of coupled walls, it is generally desirable to dissipate most of the earthquake induced energy by yielding in beams while walls continue providing overall strength and stiffness. Structures with strong coupling beams exhibit undesirable behavior, with yielding first occurring in walls. Structures with weak beams yield early in response and show excessive ductility demands. An optimum beam strength relative to wall strength can be achieved through the use of design charts developed in this investigation.

6. Two structures having the same fundamental period, beam-to-wall stiffness ratio, and yield rotation show

the same displacement and ductility response irrespective of differences in their mass, coupling arm, and clear beam span. Having the same yield rotation implies that member strength is adjusted on the basis of member stiffness so that yielding occurs at the same deformation level. Moment and shear envelopes for the same two structures are related by the ratio of their respective yield levels.

7. Two structures with equal wall strength under different levels of initial axial load do not show significant variation in response when subjected to ground excitation. However, if the maximum tension in walls is excessive, increased gravity loads in walls reduce ductility requirements and improve behavior.

8. Earthquake frequency characteristics have a substantial influence on the dynamic response of coupled walls. If a dominant vibration period of a structure falls within the peaking range of the input motion, strong structural response can be expected. Results of this investigation show that variation in earthquake frequency characteristics can affect displacement and ductility response by as much as 50%.

9. Structural response increases almost linearly with increasing earthquake intensity.

10. The primary effect of earthquake duration on structural response is to increase cumulative plastic deformations.

11. Earthquake resistant coupled wall structures can be designed by the procedure developed in this investigation. The systematic approach employed in the design procedure allows for determination of ductility ratios for given structural and ground motion parameters. Design force levels in critical regions of members can be determined under a specific combination of structural and ground motion parameters.

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NOTATIONS

A	=	Cross sectional area
a	=	Parameter defining that portion of coupling beam which is integral with wall.
C	=	Earthquake design coefficient used in UBC-1979
E	=	Elastic modulus of concrete.
EA	=	Axial rigidity
EI	=	Flexural rigidity
F_t	=	Additional seismic force at the top of the structure
F_x	=	Total story shear at level "x"
f_w	=	Wall strength parameter = $\frac{3.6 \times 10^{10}}{(EI_e)_w} \times (M_y)_w$
G	=	Shear modulus
GA	=	Shear rigidity
h	=	Total structure height
h_x	=	Structure height up to level "x."
I	=	Moment of inertia of a section
I	=	Coefficient used for seismic design in UBC-1979
I_e	=	Effective moment of inertia
K	=	Stiffness
K	=	Coefficient used for seismic design in UBC-1979
k	=	Stiffness coefficient
L	=	Member length
ℓ_c	=	Clear beam span in feet
M	=	Bending moment
M_i	=	Design moment at story level "i"
M_y	=	Bending moment at flexural yield
P	=	Axial force

R_y = Beam-to-wall strength ratio = $6(M_y)_{BM}/\ell_c(M_y)_w$

R_{SF} = Beam-to-wall stiffness ratio =

$(\frac{kEI}{L})_{BM}/(\frac{kEI}{L})_w$

S = Coefficient used for seismic design in UBC-1979

SI = Spectrum intensity of N-S component of 1940 El Centro record.

T = Fundamental period of vibration

V = Shear force

V_b = Design shear for base wall

V_i = Design shear for walls at story level "i."

X_i = Structure height up to story level "i."

W = Weight of a structure

$(\ )_{BM}$ = For beams

$(\ )_W$ = For walls

θ_{max} = Maximum rotation

θ_y = Rotation at yeild

μ_r = Rotational ductility factor, $\mu_r = \theta_{max}/\theta_y$