EFFECT OF WALL STRENGTH ON THE DYNAMIC INELASTIC SEISMIC RESPONSE OF YIELDING WALL-ELASTIC FRAME INTERACTIVE SYSTEMS

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SYNOPSIS

In current practice, shear wall-frame interactive systems subjected to earthquakes are considered to dissipate energy primarily through yielding in the frame joints, while the walls are considered to remain elastic throughout their seismic response. This paper shows that keeping the frames elastic and designing the shear walls for yielding makes it possible to avoid difficult ductility details in the frame, while incorporating ductility details in the walls where required. An analytical investigation using inelastic response history analysis has been carried out to study the earthquake response of yielding wall-elastic frame structures for a limited range of varying structural parameters. This paper presents partial results of this investigation concerning the effect of wall strength on the response of yielding wall-elastic frame systems.

RESUME

En pratique, pour les systèmes de résistance aux forces latérales comprenant des refends et des cadres rigides en béton armé, on admet que, lors d'un séisme, il y a dissipation d'énergie surtout parce que les joints des cadres atteignent la plasticité alors que les refends restent élastiques. Dans cet article on démontre que, si on dimensionne le système pour que les cadres restent élastiques et que les refends atteignent la plasticité, on peut alors éliminer dans les cadres les arrangements d'armature complexes assurant la ductilité. Dans ce cas les arrangements d'armature pour la ductilité sont placés dans les refends où ils sont requis. On présente également dans cet article les résultats de l'analyse de la réponse aux séismes d'un tel système et on étudie l'effet de la résistance du mur sur le comportement sismique.

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INTRODUCTION

The current approach to earthquake resistant design of multistory reinforced concrete buildings is based on providing adequate strength for reduced static forces, as specified in various codes of practice. The reduced forces are permissible because, in response to severe earthquakes, structures yield at their nodal points where the moments reach their maximum values. Such local inelastic behavior dissipates the earthquake energy.

To assure energy dissipation through hinging at the nodes, the structural frame members and their connections must be detailed for ductility. Providing ductility in beams and columns does not pose serious problems in design or construction. However, at the beam-column joints, it becomes difficult to assure that the joint has enough shear strength to resist the forces due to yielding of the beam reinforcement. Also, the extreme congestion of reinforcement in the joint often causes serious construction difficulties.

Observation in earthquakes of the last two decades has shown that structures containing walls with or without frames exhibit superior behavior, as compared with structures consisting of frames only(1). In current practice, wall-frame systems are considered to dissipate energy primarily through yielding in the frames, while the walls are considered to remain elastic because of their brittleness.

Since 1974, extensive testing of some types of shear walls at various institutions, including the PCA, has demonstrated that such walls can be made ductile. Reinforcement details have also been developed to assure their ductility.

Dynamic response history analyses of wall-frame systems in which the walls alone are allowed to yield in the lower stories show substantially lower wall moments and shears than in corresponding structures where both the frame and the walls remain elastic. When the beams of the frames are also allowed to yield, a reduction in the beam and column moments and shears occurs, while the wall moments and shears show little change.

Considering the comparative results of the dynamic response studies of elastic and inelastic combination systems, and considering the construction difficulties in adhering to stringent frame ductility requirements, it becomes evident that utilizing elastic frames and yielding walls may eliminate many of the drawbacks of the current elastic wall-ductile frame systems. Consequently, an investigation has been carried out to study the earthquake response of yielding wall-elastic frame structures for a limited range of varying structural parameters. Keeping the frames elastic and designing the shear walls for yielding makes it possible to avoid the difficult ductility details in the frame while incorporating ductility details in the walls where required.

This paper presents partial results of the above investigation, concerning the effect of yield level of the wall on the response of yielding wall-elastic frame systems. The level of the moment at which the wall starts yielding is perhaps the most important parameter affecting the response of such systems.

The results are presented in a form which leads to definite recommendations concerning the earthquake resistant design of yielding wall-elastic frame interactive systems.

MODELLING AND ANALYSIS OF STRUCTURE CONSIDERED

Structure Considered and Analytical Model

A specific hypothetical building configuration with a rectangular core, two peripheral 7-bay frames in the longitudinal direction, and two peripheral 6-bay frames in the transverse direction is considered (Fig. 1). The member sizes and material strengths used are given in Fig. 1. The slab system consists of joists and supporting beams spanning between the core and the periphery. The function of the floor slabs within the lateral load resisting system is solely to serve as horizontal diaphragms, and to distribute the lateral loads between the individual resisting elements.

The analysis of the building in the transverse (short) direction is considered. Because of the symmetry of the structural system, only half the building on one side of the transverse axis is analyzed. One of the two 6-bay peripheral frames is connected through flexible links with a shear wall representing half the central core. The shear wall has the same area and stiffness about the longitudinal axis as that portion of the central core which lies to one side of the transverse axis. The flexible links connecting the peripheral frame with the shear wall simulate the coupling through slabs which transmit little bending.

The modelling so far is standard, but does not necessarily take one to the stage of computer analysis. The "prototype" structure obtained as a result of the above operation (Fig. 2) will, in the case of large buildings, be too large for inexpensive computer analysis, particularly where dynamic loads are involved.

Reduction of the Model

In view of the above, the 6-bay frame interconnected with the shear wall is further reduced to a single-bay frame, as shown in Fig. 2b. The span of the single-bay frame assumes particular importance in this reduction, if the action of the axial forces on the columns is to be modelled realistically.

Referring to Fig. 2a, the middle column of the 6-bay frame does not carry any axial load due to lateral forces. The three columns on one side of the central column carry tension, while the three columns on the other side carry compression. The amount of tension or compression carried by a column increases in proportion with its distance from the central column. Thus the three columns on each side of the central column are lumped at the location of the middle (second from the central) column on that side. The area of each lumped column is made equal to the combined area of three original columns (since the central column does not carry any axial load). The moment of inertia(I) of each lumped column, however, is made equal to 3.5 times that of a column in the prototype frame, since the central column, like the others, carries bending moments.

The flexural rigidity (EI/L) of each lumped beam in the model is made equal to the sum of the rigidities (EI/L) of the six beams on the corresponding floor of the prototype frame.

The validity of the above modelling technique has been demonstrated through comparative static and dynamic analyses of "model" and "prototype" structures in Ref. 2. The model of Fig. 2b has been shown to be a near-exact representation of the prototype structure of Fig. 2a under static lateral loads, as well as under seismic excitation.

Dynamic Analysis

A PCA modified version of the computer program DRAIN-2D (3), originally developed at the University of California, Berkeley, was selected for use in inelastic response history analyses of structures subjected to earthquake input motion. Response history analysis was found to be necessary because simplified dynamic analyses by the modal superposition method cannot provide the needed information on the amount and distribution of inelastic deformations in the various structural members. DRAIN-2D is a general purpose program for the dynamic analysis of plane inelastic structures. The mass of a structure is assumed to be concentrated at nodal points. The structural stiffness matrix is formulated by the direct stiffness method, with the nodal displacements as unknowns. The dynamic response is determined by using step-by-step integration based on the assumption of a constant response acceleration during each time step.

Program DRAIN-2D accounts for inelastic effects by allowing the formation of concentrated "point hinges" at the ends of elements when the moments at these points equal or exceed the specified yield moment. The moment-rotation characteristics of these point hinges can be defined in terms of a basic bilinear relationship which develops into a hysteretic loop exhibiting a decrease in reloading stiffness with loading cycles subsequent to yield. The modified Takeda model(4) (Fig. 3), developed for reinforced concrete, has been utilized in the program to represent the above characteristics.

The periods of the first two modes of vibration of a structure constitute an input to DRAIN-2D. Such periods were computed in this investigation using the PCA computer program DYFRQ(5).

INPUT MOTIONS

Variability in the character of the ground motion at a site makes it desirable to consider a number of representative input motions when determining the likely maximum response of a particular structure. However, because of cost, only a limited number of response analyses are usually possible. Because of this limitation, it was felt desirable in an earlier study(6) to develop a means of classifying accelerograms into fairly broad categories according to certain basic properties. Such a classification allows reasonably good estimates to be made of the maximum response of structures to potential earthquakes on the basis of a small number of analyses.

The principal ground motion characteristics affecting dynamic structural response are intensity, duration and frequency content. Intensity is used as a characteristic measure of the amplitude of the acceleration pulses in a record. Duration refers to the length of the record during which relatively large amplitude pulses occur, with due allowance made for a reasonable build-up time. The frequency characteristics of a given ground motion have to do with the relative energy content of the different component waves (having different frequencies) which make up the motion.

Duration of Ground Motion

Since most recorded earthquake motions have their strong phases--with acceleration amplitudes comparable to the maximum--lasting between 10 and 15 seconds, it is believed that a 20-second duration of strong ground motion is long enough to serve as a basis for design. In this study, the duration of acceleration records used in the dynamic analyses is limited to 10 seconds of the most intense motion. This is deemed to be generally sufficient to determine the important features of structural response. The major effect of a longer duration is on cumulative deformations, with the maximum response values remaining largely unaffected(7).

Intensity of Ground Motion

"Spectrum intensity", characterized according to Honsner as the area under the relative velocity response spectrum curve between period values bounding the range of interest, is adopted as a characteristic measure of the intensity of an accelerogram, i.e. of the amplitude of the acceleration pulses within the period range under consideration. Relative velocity response spectrum is a plot showing the variation of the absolute maximum value of the relative velocity of linear single-degree-of-freedom systems having a particular damping coefficient and different natural frequencies, when subjected to a particular input motion. Specifically, it is decided to adopt the 5%-damped spectrum intensity between periods of 0.1 and 3.0 seconds as the measure of ground motion intensity.

Each accelerogram used in the dynamic analyses of this study is normalized to a reference intensity. Such normalization is effected by scaling the amplitude of the acceleration records so that the spectrum intensity for 10 seconds of the record, at 5% of critical damping (between period values of 0.1 and 3.0 seconds), matches a specified proportion of a similarly defined spectrum intensity for the first 10 seconds of the N-S component of the 1940 El Centro record.

Frequency Characteristics

The importance of knowing the frequency characteristics of a given input motion lies in the phenomenon of resonance or quasi-resonance, which occurs when the frequency of the exciting motion approaches the natural frequency of the structure experiencing the motion.

A commonly used measure of the frequency content of an accelerogram is the relative velocity response spectrum. In this study, where a viscous damping coefficient of 0.05 of critical for the first mode is used as the basic value in dynamic analyses, the 5%-damped velocity response spectra corresponding to the first 10 seconds of a number of representative records are examined for their frequency content, as in Ref. 6. On the basis of this examination, accelerograms can be classified into two categories, depending on whether the spectrum exhibits dominant frequencies over a well-defined period range ("peaking" accelerogram - Fig. 4 a and c), or whether it remains more or less flat ("broad-band" accelerogram - Fig. 4 b and d) within the period range of interest. A sub-class of the broad-band category is a record with a spectrum which rises with increasing period within the period range of interest. This may be referred to as an "ascending" accelerogram. (Fig. 4d).

For a linear structure, or a structure that experiences only limited yielding under ground excitation, a peaking accelerogram is likely to produce stronger response than a broad-band motion of the same intensity and duration. In this context, a peaking accelerogram is one with a velocity spectrum that has its peak approximately centered about the initial fundamental period of the structure considered.

In structures where yielding significantly increases the effective period of vibration, the effect of the dominant frequency components in a peaking accelerogram diminishes as the effective period of the structure moves beyond the peaking range. For such a structure, a broad-band accelerogram of the same intensity is more likely to produce the critical response.

Choice of Input Accelerogram

Fig. 5 shows a comparison of six 5%-damped velocity response spectra for one artificial and five natural accelerograms. The 16-story structures considered in this study have initial fundamental periods of 1.38 or 1.50 seconds. For structures in this period range, three of the six input motions considered in Fig. 5 appear likely to be critical. The first of these accelerograms, 1971 Pacoima Dam S16E component, has its velocity response spectrum peaking within the period range of interest (1.4-1.5 sec.). The spectrum of the second accelerogram, 1971 Holiday Inn Orion E-W component, peaks beyond the above period range. The third accelerogram, 1940 El Centro E-W component, has a broad-band, ascending spectrum. Since extensive yielding with a consequent significant lengthening of period is not expected in a frame-wall structure, the 1971 Pacoima Dam S16E component appears likely to be the most critical for the structures considered.

A series of analyses was run on a representative 16-story wall-frame system using all three of the above 10-second accelerograms. Each was normalized so that its 5%-damped spectrum intensity between period values of 0.1 and 3.0 seconds equalled that of the first 10 seconds of the N-S component of the 1940 El Centro record. These analyses confirmed (8) the inference drawn above that the 1971 Pacoima Dam S16E component is the most critical with respect to the structures considered. This input motion is accordingly used in all analyses reported herein.

RESULTS OF DYNAMIC ANALYSES

The parameter investigated in this study, as mentioned earlier, is the yield level of the wall, or the value of the bending moment at which the wall begins to yield. Two sets of analyses are carried out for structures containing walls of different thicknesses. In the first set, the walls are 12 in. thick in the bottom 8 stories and 8 in. thick in the top 8 stories; in the second set, they are 16 in. thick in the bottom 8 stories and 12 in. thick along the rest of the height. In both sets the frames are kept elastic throughout their dynamic response. The column and beam sizes are 22 in. x 22 in. and 18 in. x 24.5 in., respectively. For structures containing the thin as well as the thick walls, elastic static analyses under Uniform Building Code (9) Zone 4 equivalent seismic forces, with K=1, were carried out using the PCA computer program STMFR-60 (10). In analyses with yielding walls, the wall yield levels were selected at values carrying certain proportions to the maximum wall moments computed in the corresponding static analyses, as shown in Table 1.

The envelope values of (a) beam moments, (b) column moments, (c) wall moments, (d) beam shears, (e) column shears, (f) wall shears, (g) lateral displacements, and (h) interstory displacements for the 12 in./8 in. wall-frame structures are presented in Figs. 6 (a) to (h), respectively. The corresponding quantities for the 16 in./12 in. wall-frame structure are presented in Figs. 7 (a) to (h), respectively. Figure 6 is first discussed below.

The key to an explanation of the structural response depicted by Fig. 6 lies in Figs. 6 (g) and (h), the first of which shows that the lateral displacements throughout the height are very comparable for the five structures and are much larger than the elastic deflection under the equivalent static code seismic forces. The structure with the elastic wall exhibits the smallest deflection in the bottom stories, and the largest deflection at the top. The fact that the structures with the yielding walls exhibit more displacements in the lower stories than the structure with the elastic wall is understandable, in view of the rotations near the bases of the yielding walls. However, it may not be readily apparent as to why the structures with progressively higher wall yield levels exhibit progressively higher deflections. The explanation lies in the phenomenon that the higher the wall yield level, the more concentrated is the inelastic rotation near its base. This is the reason why the inelastic structure with the highest yield level (3,600,000/2,400,000 in-k) exhibits the largest interstory displacements in the bottom stories (Fig. 6h). As the wall yield levels progressively decrease, inelasticity spreads further up the wall, and the interstory displacements progressively decrease. The elastic structure with no hinge rotation at the bottom, of course, exhibits the smallest interstory displacements in the lower portions, and the largest interstory displacements at the top, in keeping with Fig. 6(g). The interstory displacement diagram for the elastic structure is also free of the kink which is noticeable in the case of all structures with yielding walls. The interstory displacements computed from static elastic analysis under code seismic forces are much smaller than the actual interstory displacements.

Disregarding the completely elastic structure for the moment, it should be noted from Figs. 6(g) and (h) that a progressively decreasing wall yield level means a progressively decreasing structural response, overall. The lateral displacements as well as the interstory displacements throughout the height are lower for structures with the lower wall yield levels.

Since the beam and column moments are largely related to the interstory displacements, the beam and column moments are lower in the bottom stories in the structure with the elastic wall than in those with yielding walls (Fig. 6a,b). However, this trend reverses itself along the height of the structure, with the elastic structure showing the largest beam and column moments in the upper portions. Also, in keeping with observations made on the basis of Figs. 6(g), (h), the beam and column moments along the heights of the yielding structures are progressively lower, as the wall yield levels are made progressively lower. The moments indicated by inelastic dynamic analyses are significantly higher than the corresponding static elastic moments in the beams and column induced by the code seismic forces.

The shear forces in beams (Fig. 6d) are solely determined by the beam end moments, and thus exhibit the same trends as the beam moments. The shear forces in the columns (Fig. 6e) likewise are determined by the column moments, and exhibit the same distribution as in Fig. 6(b).

The wall moments (Fig. 6c) follow a trend that is only to be expected. The elastic wall carries much larger moments throughout its height than the yielding walls. The yielding walls with high yield levels, of course, carry more moments than those with lower yield levels. It may be noted that the dynamic moments in the yielding walls are comparable with the static wall moments induced by UBC Zone 4 seismic forces in elastic wall-elastic frame systems.

The shear forces in the walls (Fig. 6f) largely follow the same trends as the wall moments. The elastic wall carries much larger shear forces throughout its height than the yielding walls. Among yielding walls, those with higher yield levels usually carry larger shear forces, although this trend is not all that distinct near the two ends of the walls. The dynamic shear forces in the yielding walls are again comparable with the static wall shears induced by UBC Zone 4 seismic forces in elastic wall-elastic frame systems.

Figure 7 (a) to (h) qualitatively agree in all respects with Figs. 6 (a) to (h), respectively, and indicate that the structural behavior of the 12 in./8 in. wall-frame interactive system is similar to that of the 16 in./12 in. wall-frame system.

The shear capacity and ductility requirements in the walls of the ten systems analyzed in Fig. 6 and 7 are presented in Table 1. The thicker 16 in./12. walls, of course, require less shear stress capacity, and are preferable in that respect. For the same wall thickness, there is, in general, a decrease in the demand for shear stress capacity as the wall yield level decreases. There is a simultaneous increase in the demand for ductility which is understandable in view of the fact that lower yield levels usually mean excursions further into the inelastic range.

From Table 1 it is apparent that WF209 with 16 in./12 in. walls, and yield levels equal to about 70% of the maximum elastic moments induced in such walls by UBC Zone 4 equivalent static seismic forces, represents the most favorable combination of shear capacity and ductility requirements, among the cases considered. While 70% may appear to be low, it must be remembered that the maximum moments from dynamic analyses are considerably higher than the yield moments. The increase in moment from the yield level to the maximum value, along the strain-hardening branch of the moment-rotation characteristics, utilizes ductility, and also causes an increase in the corresponding shear stress levels.

The interdependence noted above among flexural strength, shear capacity and ductility requirements is of prime importance in the design of a structural system against dynamic excitations. Several analyses may be required to establish the most advantageous balance between the strength and ductility requirements, since a structure may be designed to resist the same earthquake at various levels of strength with corresponding variable ductilities.

DISTORTIONS OF COLUMNS AND BEAMS

In the presently available mathematical models of the inelastic behavior of reinforced concrete elements (i.e. in models of their moment-rotation characteristics), the flexural and shear deformations are lumped together. As additional analytical studies are carried out, and as laboratory testing procedures are improved, it is hoped that mathematical models separating flexural and shear deformations will become available, and that their verification will be based on test results.

At this stage, the overall computed dynamic response of reinforced concrete buildings may not reflect the effects of large inelastic shear distortions on frame behavior.

In general, the presence of substantial structural walls in wall-frame interactive systems restricts the interstory drift and thus limits the possibility of significant lateral translation of columns within a story. However, when grinding occurs along large shear cracks in walls subjected to reversing load cycles, large interstory drifts can take place within the region of "hinging" of the walls. Within this critical region, if large wall distortions should occur, they may be forced on the columns and beams. Under such circumstances the columns and beams interacting with the "hinging region" of the walls may become inelastic and may require ductility.

Large distortions may also occur in the linkage beams (if such exist) connecting the walls with the frames of a wall-frame system, or in the coupling beams linking the walls of a coupled wall system. In these cases, the cumulative fiber elongation or shortening of the walls causes both a vertical displacement and a rotation of the beam ends. Such beams require ductility.

In view of the above, it would seem prudent to supply the columns and beams within the region of potential wall hinging (which may be subjected to substantial shear distortions) with a measure of ductility as a second line of defence.

CONCLUDING REMARKS

This study of the seismic response of structural wall-frame interactive systems points out the important inter-dependence among the flexural strength, shear capacity and ductility requirements of structural members. It is in general possible, through several exploratory analyses, to design into a structure the most desirable balance between strength and ductility. A structure may be designed with elastic members without ductility for high forces, or it may be designed for ductile behavior with reduced forces and a definite amount of inelastic deformation. The designer can also select a structural system in which only some of the elements become inelastic and dissipate energy, while others remain elastic throughout seismic response.

Keeping the frames of a frame-wall system elastic throughout seismic response, and designing the walls for yielding, makes it possible to avoid difficult ductility details in the joints of the frame, while incorporating ductility details in the walls where required. However, in view of the limitations of current analytical models which may not adequately reflect the effects of large inelastic shear distortions on frame behavior, it is recommended that the columns and beams within the region of potential wall hinging be supplied with a measure of ductility as a second line of defense.

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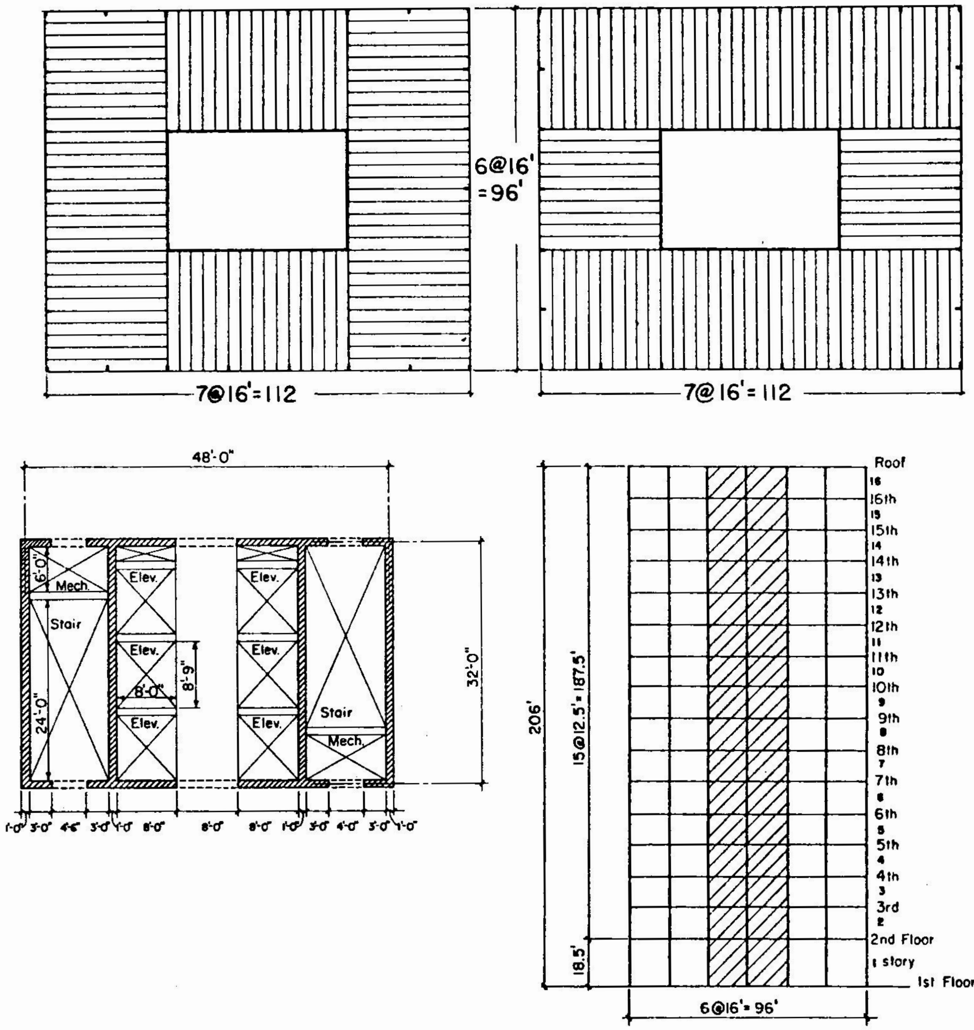
Table 1: Ductility and Shear Capacity Demands from Dynamic Analyses

Structure	Analysis No.	Yield Level Bottom 8 Stories/Top 8 Stories ($x10^6$ in.-k)	$\frac{M_y}{M^e_{max}}$ at wall base (%)	T_1 T_2 (sec)	μ Based on Rotations Within 1st and 2nd Stories	MAX SHEAR V_{max} (k)	$v'_u = \frac{V_{max}}{b_w\ 0.8\ l_w}$ (psi)	$\frac{v'_u}{\sqrt{f'_c}}$	$\frac{v'_u}{\varphi\sqrt{f'_c}}$
12in./8in. wall-light frame	FW205	Elastic	-	1.496 0.311	-	6308	830	13.12	15.44
	FW203	3.6/2.4	93.70	1.496 0.311	2.73	4281	563	8.90	10.47
	FW201	3.1/2.1	80.69	1.496 0.311	3.11	4303	566	8.95	10.53
	FW214	2.6/1.7	67.67	1.496 0.311	3.49	4240	558	8.82	10.37
	FW206	2.1/1.4	54.66	1.496 0.311	3.63	3960	520	8.22	9.67
16in./12in. wall-light frame	FW208	Elastic	-	1.377 0.274	-	6489	634	10.02	11.79
	FW207	4.5/3.0	105.73	1.377 0.274	2.65	5247	512	8.10	9.52
	FW210	4.0/2.7	93.98	1.377 0.274	2.94	4950	483	7.64	8.98
	FW202	3.5/2.3	82.24	1.377 0.274	4.00	4329	423	6.69	7.87
	FW209	3.0/2.0	70.49	1.377 0.274	3.66	3791	370	5.85	6.88

M^e_{max} is the factored maximum moment from static analysis under UBC Zone 4 equivalent seismic forces (K=1, load factor = 1.4)



SIZES

Walls - 12" bottom 8 stories
8" top 8 stories
or 16" bottom 8 stories
12" top 8 stories
Columns - 22" x 22"
Beams - 18" x 24.5"

MATERIALS

Concrete - Normal wt,
w = 150 pcf
f'_c = 4 ksi

Steel - f_y = 60 ksi

Fig. 1: Building Configuration with Sizes and Material Strengths

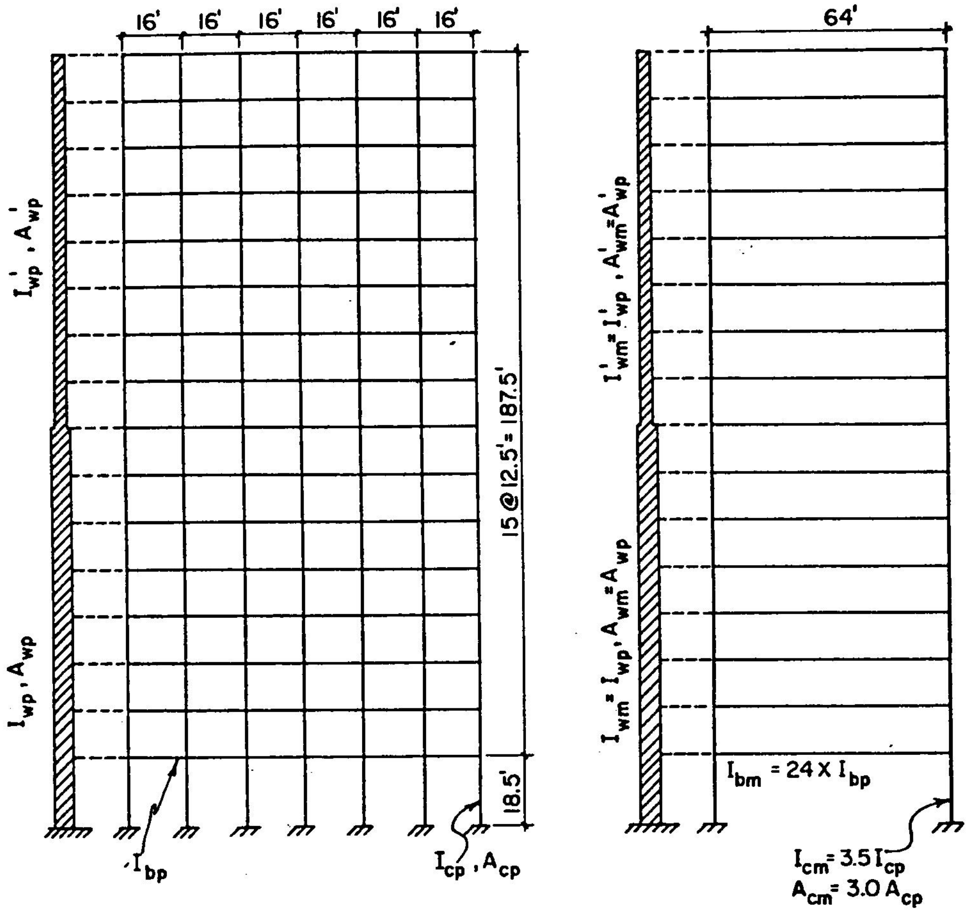


Fig. 2: Prototype and Model Structures

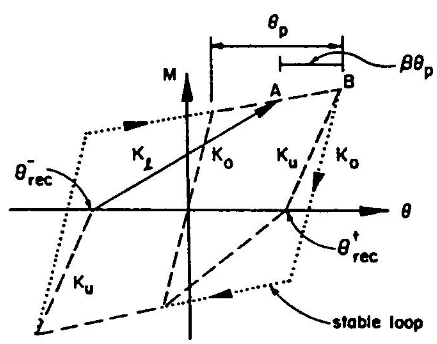


Fig. 3: Modified Takeda Model

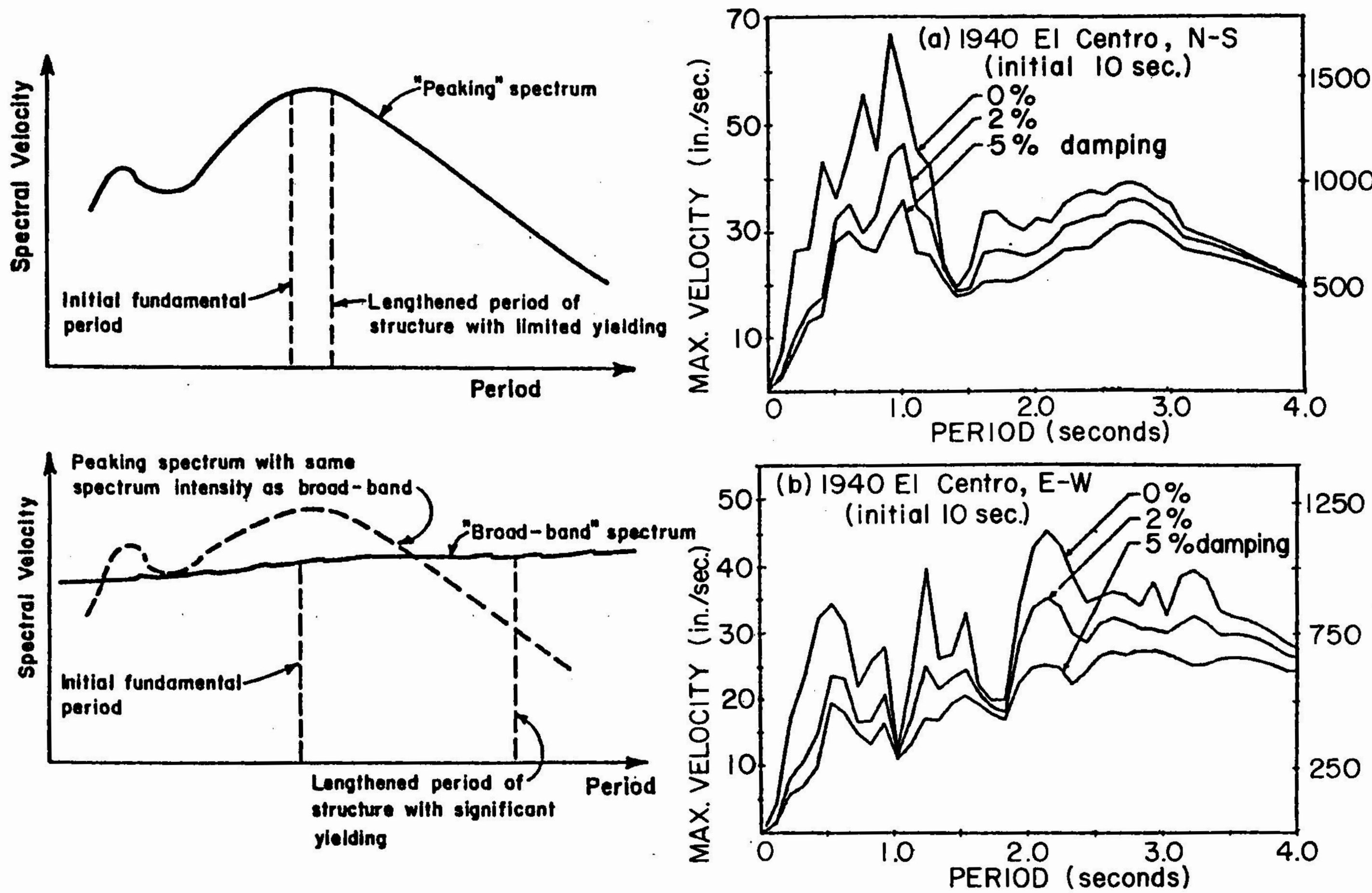


Fig. 4: Typical Shapes of Damped Velocity Response Spectra

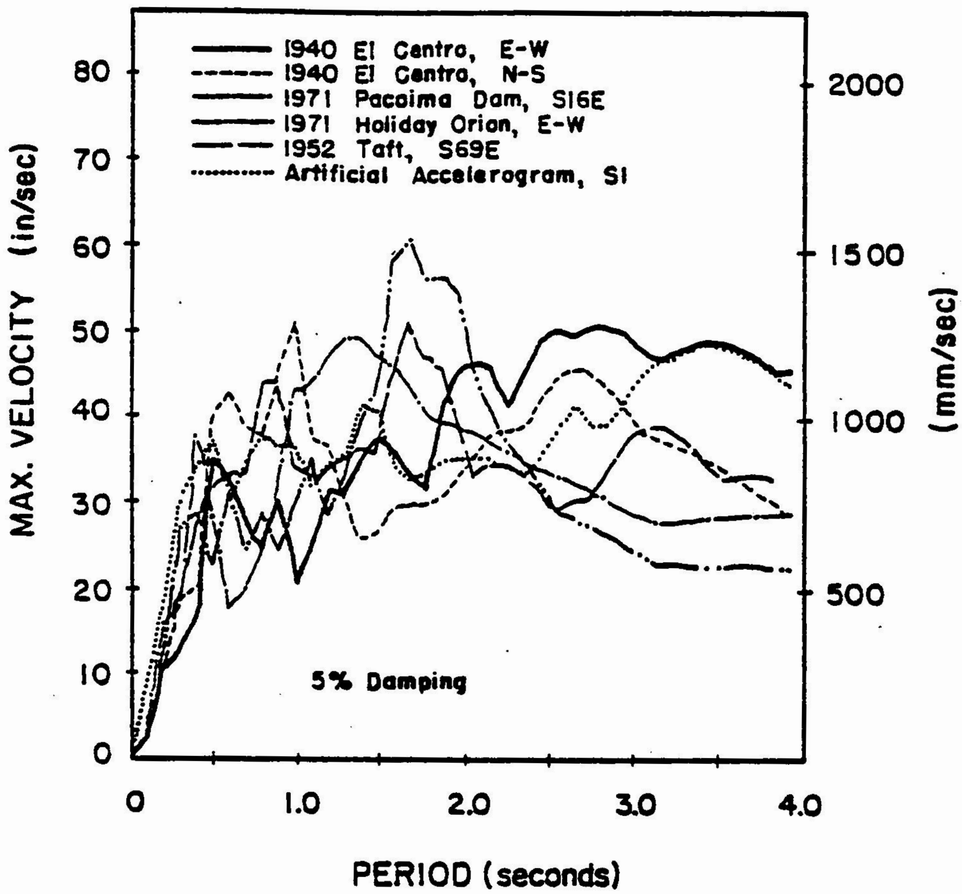


Fig. 5: Relative Velocity Response Spectra for the First Ten Seconds of Normalized Input Motions

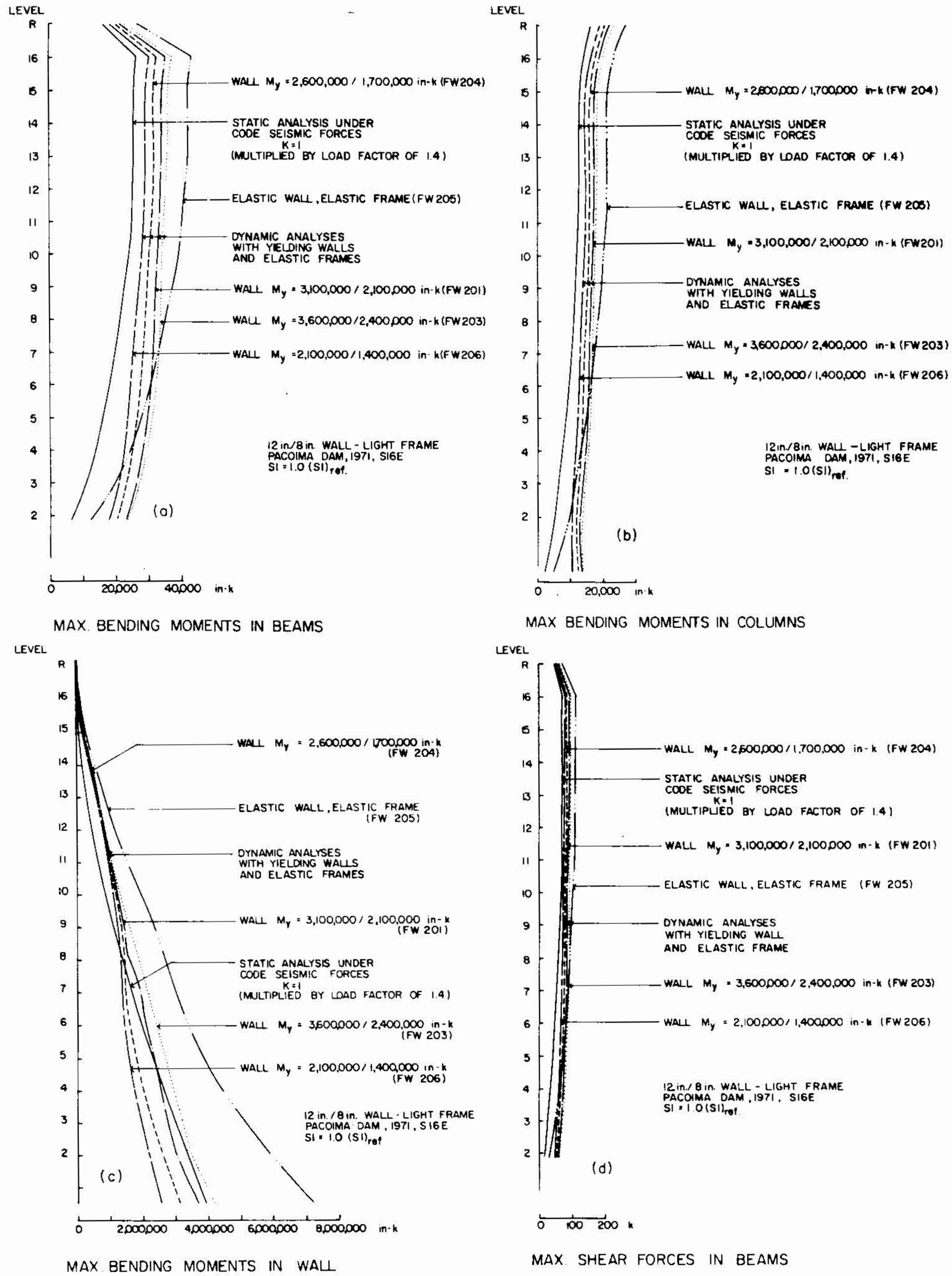


Fig. 6: Results of Dynamic Analysis - Wall-Frame Interactive Systems
(a)-(d) with 12 in./8 in. Walls

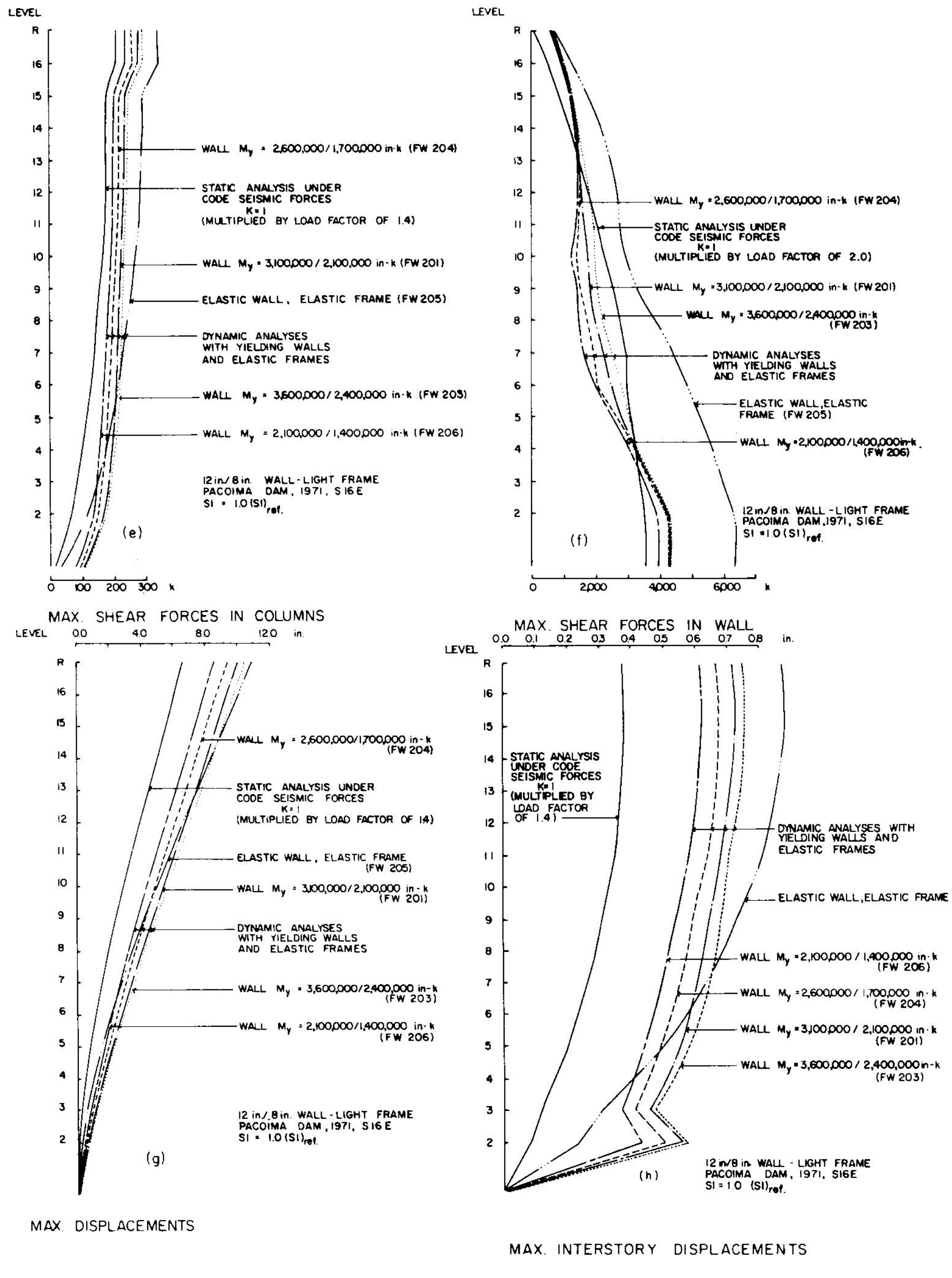


Fig. 6: Results of Dynamic Analysis - Wall-Frame Interactive Systems
(e)-(h) Systems with 12 in./8 in. Walls

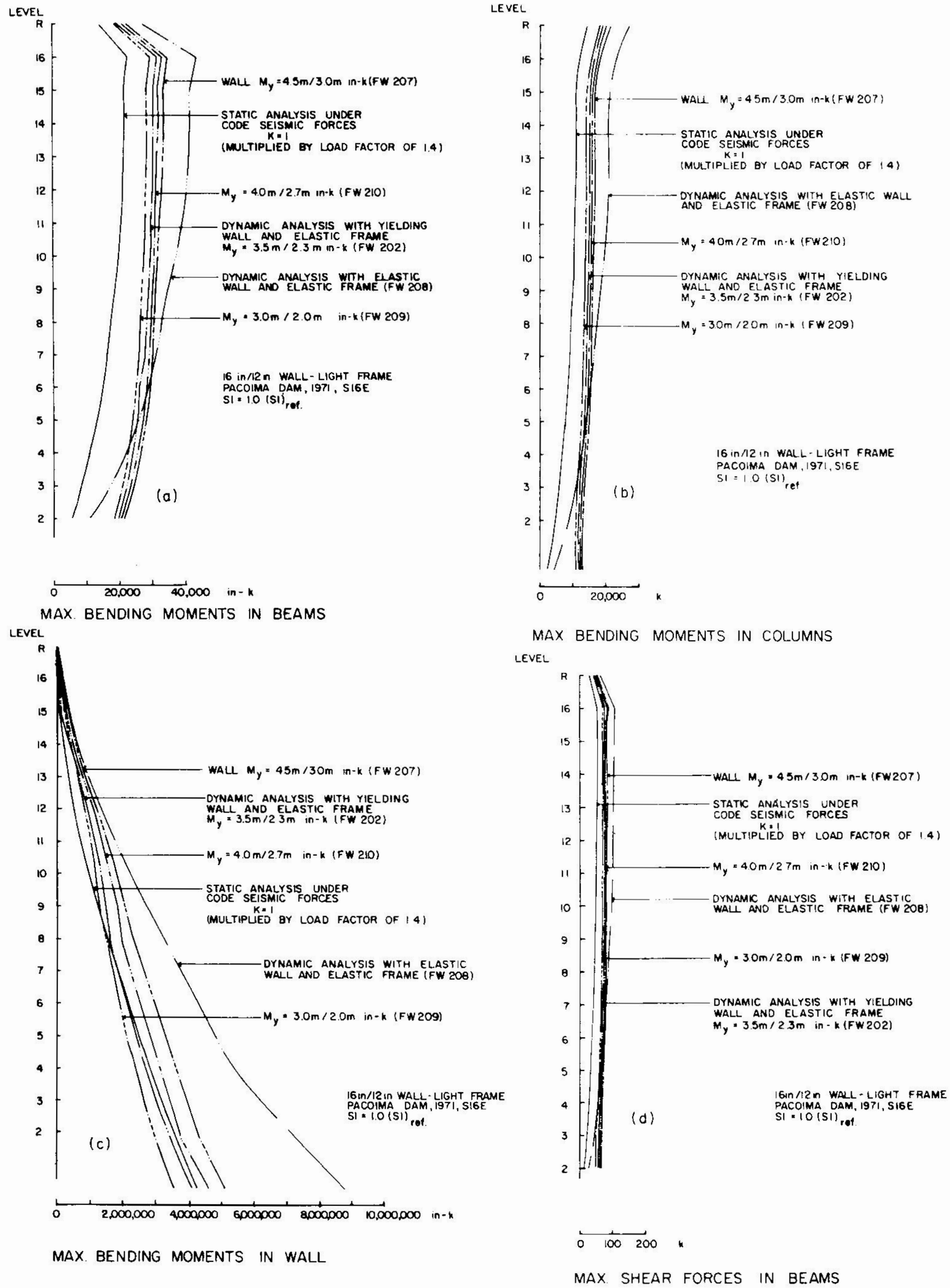


Fig. 7: Results of Dynamic Analysis - Wall-Frame Interactive Systems (a)-(d) with 16 in./12 in. Walls

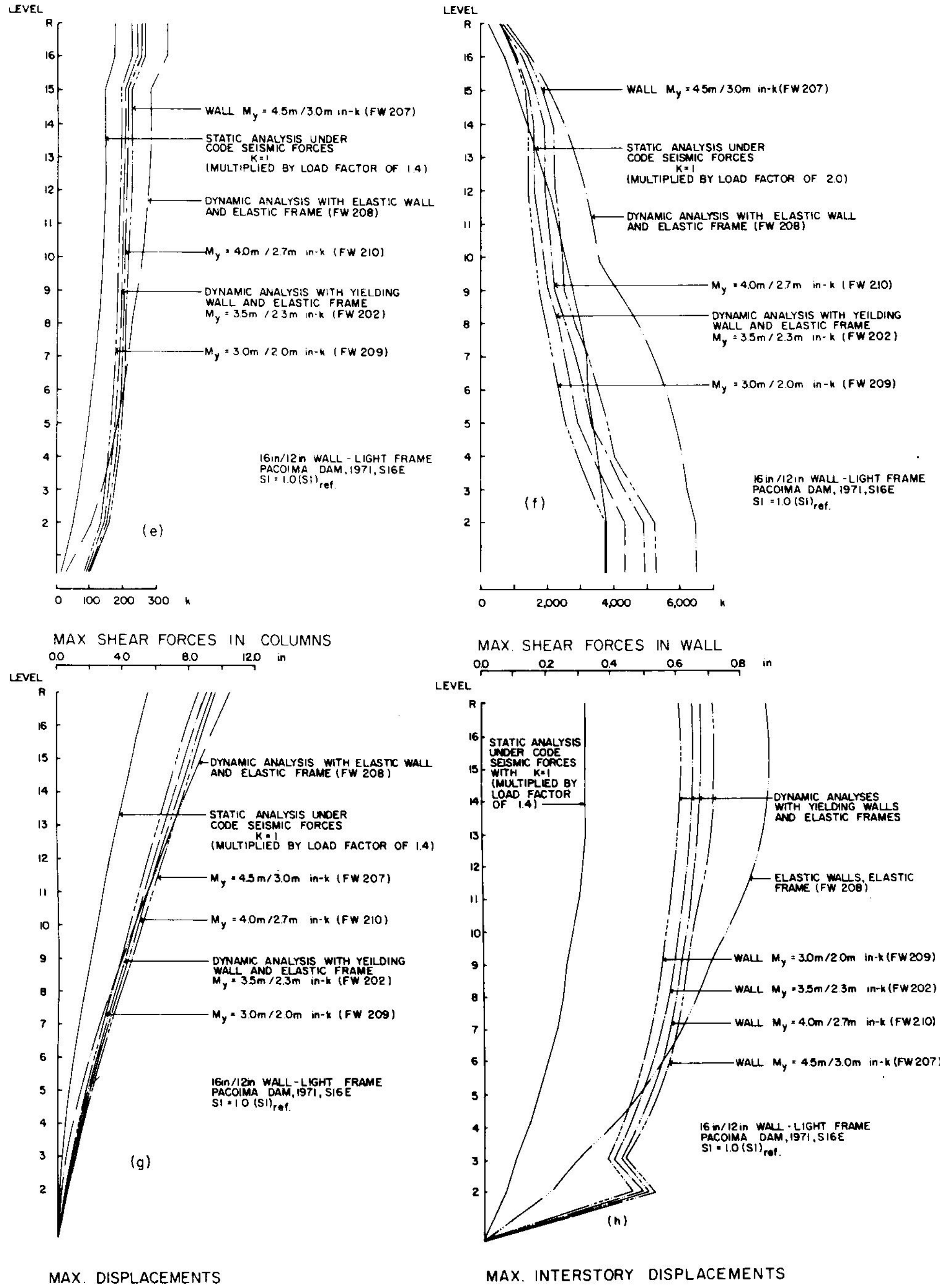


Fig. 7: Results of Dynamic Analysis - Wall-Frame Interactive Systems
(e)-(h) with 16 in./12 in. Walls