EARTHQUAKE RESPONSE OF STEEL FRAME-CRACKED CONCRETE SHEAR WALL SYSTEMS

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SYNOPSIS

The results of a parametric investigation on the earthquake response of structural systems composed of steel frames and reinforced concrete shear frames are presented. The investigation included two steel frames having three bays and ten and twenty-four stories. The reinforced concrete shear walls are assumed to be intact, and during different phases of the investigation, to have various degrees of cracking due to the previous earthquake. The comparisons in terms of the changes in the natural frequencies of vibration for undamaged and damaged shear walls have been tabulated. As an indicator of the response of the structural system the lateral deflection profiles of the structural system when subjected to various ground motions are included in graphical form. The parametric investigation included the earthquake response of the frames when the structural system is subjected to three different types of ground motion spectra. It has been found that the structural response changes substantially depending upon the type of ground motion employed. It is also noted that the changes in the earthquake response of the structural system are not as extensive as would have been expected for structural systems having different structural deteriorations.

RESUME

Cet article contient les résultats d'une étude paramétrique sur la réponse aux séismes de systèmes comprenant des cadres rigides en acier et des refends en béton armé. On a étudié deux cadres rigides de trois travées, l'un ayant dix niveaux et l'autre vingt-quatre niveaux. On a d'abord supposé que les refends étaient intacts, ensuite on a admis qu'ils étaient fissurés à divers degrés à cause de tremblements de terre précédents. On a pu ainsi comparer les fréquences de vibration de refends endommagés à celles de refends qui n'ont pas subi de dommages. La réponse aux séismes des systèmes structuraux est donnée sous forme de graphiques où on trouve les déplacements latéraux en fonction des accélérations du sol. On a considéré trois différents spectres de mouvements du sol. On a constaté que la réponse aux séismes change considérablement dépendant du type de mouvement du sol utilisé dans les calculs. De plus on a noté que les changements dans la réponse sismique de systèmes ayant subi des dommages ne sont pas aussi importants qu'on l'aurait cru.

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INTRODUCTION

High-rise steel building frames subjected to lateral loads can be designed as either braced or unbraced frames. For most practical cases, especially when the magnitude of the lateral forces is more than marginal, unbraced frames are considered to be uneconomical. Consequently, most high-rise steel frames are designed as braced frames. The bracing is commonly provided by steel X- or K-bracings. Investigation of the steel braced frames after being subjected to severe earthquake loadings has indicated that the design of these types of structures still requires further refinements if they are, at least partially, to exhibit plastic hinge formations.

One of the practical methods of providing the lateral stiffness to high-rise structural steel frames is the use of reinforced concrete shear walls, instead of being concerned with other forms of bracing (10). The monolithic shear walls also have a positive contribution in the control of the lateral sway of the structure. The lateral deflection profiles of the bare frame and the bare shear wall complement each other, in the sense that through their interaction under the effects of lateral loads the overall horizontal deflection at each floor level is substantially reduced. This results in low relative floor drift index (12).

The preliminary optimal dimensioning of the reinforced concrete shear walls to stiffen the high-rise steel frames has already been tentatively defined (10). The recommendations for the dimensioning of the walls were developed for the linear elastic regime.

PROBLEM STATEMENT

The field investigations of buildings with shear walls, after having been subjected to severe ground motions, have indicated that the reinforced concrete walls tend to develop from light to severe cracking (6, 9, 11, 13). In most of these structural systems the amount of damage experienced by the structural frame has usually been negligible. This phenomenon has essentially been due to the fact that the structural frames are more flexible than the shear wall, consequently, a substantial portion of the lateral forces has been carried by the shear wall. Furthermore, if the provided ductility of the frame is higher than that of the wall, a traditional and common oversight, the reinforced concrete shear walls exhibit noticeable damage prior to the inception of any major distress in the framing system (3, 4, 7).

The critical post-earthquake decision that has to be made is the determination of the need for a major structural repair program for the damaged shear wall. If the shear wall has not appreciably lost its structural integrity, than the needed repair may be confined to a small scale retrofit program. However, if the damaged wall can not withstand future major earthquakes then the scope of the repair program will be a major one.

The paper presents some of the findings of the research program conducted on the determination of the structural response of high-rise steel frames stiffened by reinforced concrete shear wall of varying degrees of damage when subjected to severe ground motions. As a basis for comparison, the response of the structural system has also been determined for the above referred structural systems without any structural damage.

PROTOTYPE STRUCTURE

In order to arrive at conclusions that are not specifically applicable to only one structural system, it is essential that a parametric investigation be conducted. However, an all inclusive parametric investigation that will consider the important parameters, such as the frame height, frame plan layout, shear wall size, different forms of predamaging and different types of ground motions, will inevitably lead to an intractable research program. This has necessitated that certain limitations be imposed on the variability of the design parameters. These limitations prevent the attainment of general exact formulae that can be applied to all geometrical configurations. The findings can be considered as trends and "rules of thumb" to be used in the design and inspection of these types of building systems.

The investigation employed two steel frames having three bays. Both frames were designed in accordance with the 1963 AISC specifications and checked for compliance with the current Specifications (1). The frames were designed with X-bracings, and assumed to have moment resistant connections (5). The frames are referred to as Frame-B, having 10 stories, as shown in Figure 1, and Frame-C, having 24 stories, as shown in Figure 2. Figures 1 and 2 also provide the assumed design load levels. The substitution of the reinforced concrete shear wall for the X-bracing was undertaken by removing the X-bracing and the rightmost line of columns and placing the shear wall starting at the removed column line. The shear walls have widths of 16 ft. (4.88 m) and 20 ft. (6.10 m) for Frames-B and -C respectively. The thickness of the walls was taken as 12 inches (0.305 m) for Frame-B arrangement, and 16 inches (0.41 m) for Frame-C. The dimensions of the shear wall correspond to the optimal dimensions as had been determined in a previous research program (10). The investigations have also revealed that, if the dimensions of the shear wall can be considered as "optimal," than the earthquake response of the structural system will not be greatly altered regardless of the type of connections, i.e. shear or moment (3, 4).

Initially the cracking of the shear walls was carried out for two types of crack patterns (4). Visualizing the shear wall segment defined by the vertical edges of the wall and horizontal lines that can be considered as the continuation of two consecutive beam axes, the cracks can be from the lower left to the upper right corner of this hypothetical wall segment. Similarly the cracks can be extended from the lower right to the upper left corner. These types of cracks are termed X-cracks. This is the most commonly observed crack pattern in the shear wall systems. To determine the orientation of the cracks on the response of the structural system, another pattern was also considered. In this arrangement it was assumed that the cracks intersect at the center of this hypothetical wall segment and they make 45° and 135° angles with respect to the horizontal line at the center of the wall segment. This type of cracking is referred to as 45° cracks (4).

ANALYTICAL MODELING

The analysis of the structural system was undertaken through the use of finite element displacement method, and computer program SAPIV (2). It was assumed that the shear wall and the structural frame will have planar motion. Beams and columns were idealized via beam-column finite elements, and the shear wall was discretized through the use of the plane stress finite elements. It was further assumed that throughout the analysis the structural system would respond to the earthquake excitation in a linear elastic manner. This stipulation is in agreement with most analytical schemes that are employed for similar problems (8).

The cracking of the shear wall in the predetermined pattern was accomplished through the biased alteration of the Young's Moduli of the appropriate plane stress finite elements. The parametric study on the simulation of the cracked shear walls is summarized in Table 1. Type I wall stiffness corresponds to undamaged shear wall, whereas Type IV simulates extensively damaged shear wall. Throughout the study it was assumed that the crack pattern and intensity are constant regardless of the floor height.

The dynamic response of the structural system was determined through the use of the modal superposition technique (2). Throughout the study at least five predominant frequencies of vibration and the corresponding modal shapes were considered (3, 4). The ground motion was input into the analysis scheme via the response spectrum of the previously recorded earthquakes. The preliminary studies have indicated that use of one response spectrum corresponding to a given earthquake may lead to wrong conclusions (3, 4, 6). Different types of ground motion spectrum may, and do, lead to the excitation of different frequency contents. Therefore, the reported investigation used at least three different ground motion spectra: 1940 El Centro

Earthquake (NS component), and those based on the recordings at Pacoima Dam (S71°W component) and the basement of the Kajima International Building (N54°W component) during the 1971 San Fernando Earthquake (8, 9). It should be noted that if the response spectra derived from the 1977 Bucharest Earthquake had been used, the results would have been noticeably different. Nevertheless, the use of multi-response spectra provides a major step forward from the traditional approach of using only one response spectrum, usually 1940 El Centro recordings.

In the analytical simulation it had been assumed that only the horizontal component of the earthquake would act on the structure, and it would be parallel to the plane defined by the shear wall and the steel frame. The inclusion of the vertical components of the earthquake, either acting in-phase or out-of-phase with the horizontal motion, would not have noticeably affected the findings. The structural system in question is sensitive to the horizontal ground motions; the vertical component of the motion will not excite the shear wall sufficiently enough to alter the findings.

The reported results are given in terms of "square root of the sum of the squares" (SRSS) approach. This formulation tends to give upper bound prediction to the deformation characteristics of the structural system. As had been mentioned, only the selected predominant modes of vibration were included in the SRSS formulation.

RESULTS

Any kind of parametric investigation that is based on the finite element method tends to yield a massive amount of information. The presentation of all findings in a paper form is not possible. Therefore, for thematic content only two major areas of the findings have been included in this paper. They are: 1) Changes in the predominant frequencies of vibration for the structural system having various degrees of cracking, and 2) horizontal deflection profiles of the structural system.

Natural Frequencies of Vibration

The analyses for the structural systems without any damage, i.e. Type I according to Table 1, have indicated that Frame-B has natural frequencies of vibration for the first through fifth modes as 0.53, 0.77, 1.56, 2.13, and 4 Hertz. The corresponding values for Frame-C are 0.24, 1.23, 3.13, 3.13, and 3.45 Hertz. The first five predominant frequencies of vibration of the structural system with the damaged shear walls have been normalized with respect to the corresponding frequencies of the corresponding structural system where the shear wall had not exhibited any damage. The results for Frames-B and C are presented in Tables 2 through 5. Noting that Type IV damage corresponds to severe deterioration of the shear wall, and also noting the fact that the shear wall is the essential structural component that provides the lateral stiffness to the structural systems, it is interesting to note that the changes in the natural frequencies of vibration for the structural systems considered are rather small.

Furthermore, the changes in some of the frequencies are less than ½% (indicated in the tables with values of 1.00). The change in the first frequency is about 30% for extreme damage (Type IV), regardless of the orientation of the cracking; and about 20% at most, when the damage is not that severe (Type III).

In view of the crude methods of prediction of the natural frequencies of vibration of the structural systems, possible errors in the prediction of the frequencies in the order of 20-30% are fully acceptable for any preliminary, or even final design processes (6, 7, 8, 10, 12). Therefore, with the current "engineering" methods of analysis, the changes in the vibrational characteristics of the frame-shear wall system with or without any structural damage to the shear wall are barely noticeable. However, it would be observed that the changes in the vibrational characteristics of the structural systems, depending upon the orientation of the cracking in the shear walls, are more than what would have been normally assumed by earthquake engineers. A comparison of Table 2 with Table 4, and Table 3 with Table 5 reveals this fact.

The above observations in conjunction with the field assessment of the damage to the structural system, usually by cursory visual inspection, results in the observation that in the assessment of the vibrational characteristics of structural systems, especially if they have suffered any structural damage, the prediction of the structural strength of the structure and/or the definition of the earthquake susceptibility of these structures can not be stated with any exactitude. Any attempts to have precise answers may be obfuscated by the contribution of many inexact parameters.

Horizontal Deflection Profiles

The most common means of defining the interaction between the frame and the shear wall has been the determination of the horizontal deflection profile of the structural system when subjected to the lateral loadings (10). The analyses, the details of which have not been included herein, have indicated that the axial shortening of the beams is quite negligible as compared to the corresponding lateral movement of the respective story levels. Therefore the deflection of the shear wall can easily be taken with the introduction of error less than ½%, as the lateral deflection of either joints or shear wall at a given story height.

For the sake of brevity, only the deflection profiles corresponding to X-cracking have been included in the paper. The deflection profiles for 45° cracking are slightly different. Figures 3, 4 and 5 present the deflection profiles of the structural system for various degrees of shear wall deterioration, when the system is subjected to the previously defined El Centro, Pacoima Dam and Kajima Building type ground motions. It is interesting to note that for a ground motion spectrum similar to that of Pacoima Dam, the structural system behaves essentially in the same manner regardless of the type of damage; so much so that it is impossible to differentiate the deflection profiles corresponding to different earthquakes, as shown in

Figure 4. The variation in the deflection profiles of structural systems is quite noticeable when buildings are subjected to different earthquakes. For example, a comparison of Figures 3 and 5 indicates that for the El Centro type ground motion the structural system deflects most when there is extreme damage in the shear wall, whereas, if the ground motion is similar to that of the recording at the Kajima Building in 1971, the extreme deflection profile corresponds to a structure with undamaged shear wall.

Another important observation that can be made in comparing Figures 3 through 5 indicates that the structure laterally deforms in an extreme fashion when it is subjected to El Centro type ground motion, the top deflection of the structure being in the neighborhood of 21 inches (53 cm); whereas, the Kajima gound motion results in top deflection of about 2½ inches (6.3 cm). The resultant stresses in the structural system, as well as the amount of the spread of the damage, will increase with the increased deflection. A comparison of the three different deflection profiles for the three different ground motions once again illustrates that the use of one single assumed ground motion spectrum may lead to grossly erroneous results.

Figures 6-8 illustrate the deflection profiles of Frame-C. In the case of El Centro type earthquake motion, regardless of the extent of the damage in the shear wall, the deflection profiles are essentially the same, as can be seen in Figure 6. Still, the extreme deflection profile is obtained when the Frame is subjected to El Centro type earthquake. Frame-C deflects less for Pacoima Dam type ground motion as compared to Kajima type motion. This is in contradiction to the observation that had been made for Frame-B, where the structure deflected less when subjected to Kajima type motion.

Probably the most interesting observation will be the comparison of the deflection profiles of all structures. The only deflection profile with a so called "inflection point" is the one that corresponds to Frame-C, when subjected to Pacoima Dam type ground motion. It has been "intuitively" assumed by most of the researchers who are involved in the area of interaction of the structural frames with shear walls, that there is usually a point of inflection in the deformation profile for the structural system. This assumption can not be validated, with the exception of one case. It is further observed that in the deformation profiles of the structural systems in question the first mode usually provides a reliable approximation in the prediction of the earthquake response of the structural system.

Depending upon the amount of damage sustained by the shear wall, the changes in the response of the structural system do not drastically vary regardless of the types of ground motions that have been considered. In view of the gross approximations involved in the post-earthquake evaluation of the structural integrity of the buildings, it is quite improbable to come up with numerical results that can have the precise discrimination that engineers would like to have. It is therefore recommended that the post-earthquake strength assessments should be undertaken more in terms of the definition of the extreme bounds for the structural integrity of the existing system, rather than a precise prediction.

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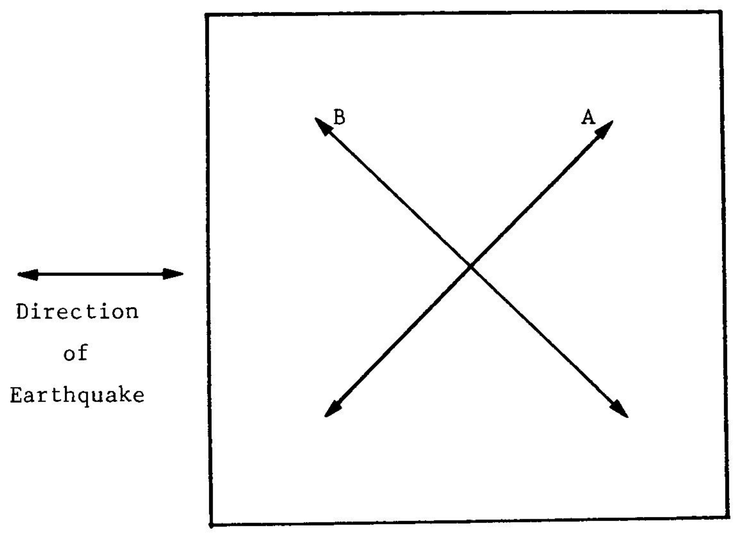
Table 1

Analysis Scheme to Simulate Crack Pattern in Concrete Shear Wall



Shear Wall Stiffness Type	A	B	Simulation
I	E_c	E_c	Uncracked
II	E_c	$0.75E_c$	One Diagonal Crack
III	E_c	$0.50E_c$	One Diagonal Crack
IV	$0.75E_c$	$0.25E_c$	Double Cracks

(Note: E_c = Young's Modulus of Concrete)

Table 2

Nondimensionalized Natural Frequencies
Frame-B, 45° Cracking

MODE SHAPE	SHEAR WALL (Damage Types)			
	I	II	III	IV
1	1.0	0.93	0.78	0.67
2	1.0	0.90	0.76	0.57
3	1.0	1.00	1.00	1.00
4	1.0	1.00	1.00	0.95
5	1.0	1.00	0.94	0.89

Table 3

Nondimensionalized Natural Frequencies
Frame-C, 45° Cracking

MODE SHAPE	SHEAR WALL (Damage Types)			
	I	II	III	IV
1	1.0	0.90	0.82	0.71
2	1.0	0.84	0.77	0.58
3	1.0	0.86	0.73	0.54
4	1.0	1.00	1.00	0.84
5	1.0	1.00	1.00	0.83

Table 4

Nondimensionalized Natural Frequencies
Frame-B, X-Cracking

MODE SHAPE	SHEAR WALL (Damage Types)			
	I	II	III	IV
1	1.0	1.00	0.95	0.83
2	1.0	1.00	1.00	1.00
3	1.0	1.00	1.00	1.00
4	1.0	1.00	1.00	1.00
5	1.0	0.96	0.89	0.78

Table 5

Nondimensionalized Natural Frequencies
Frame-C, X-Cracking

MODE SHAPE	SHEAR WALL (Damage Types)			
	I	II	III	IV
1	1.0	0.95	0.91	0.82
2	1.0	0.95	0.88	0.74
3	1.0	0.94	0.89	0.73
4	1.0	1.00	1.00	1.00
5	1.0	1.00	1.00	1.00

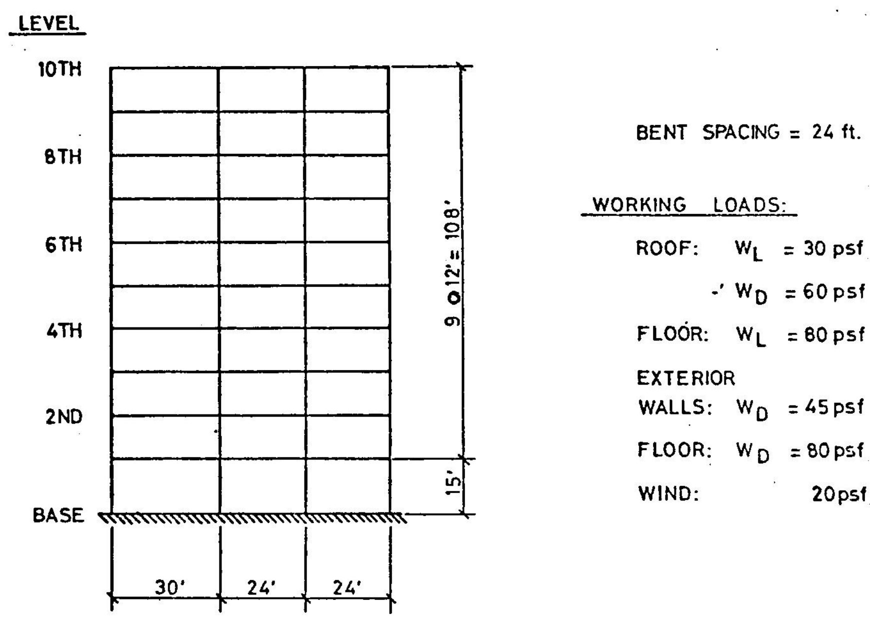


Figure 1 Frame-B Dimensions and Working Loads

(1 ft.=0.305 m, 1 psf=47.9 N/m^2)

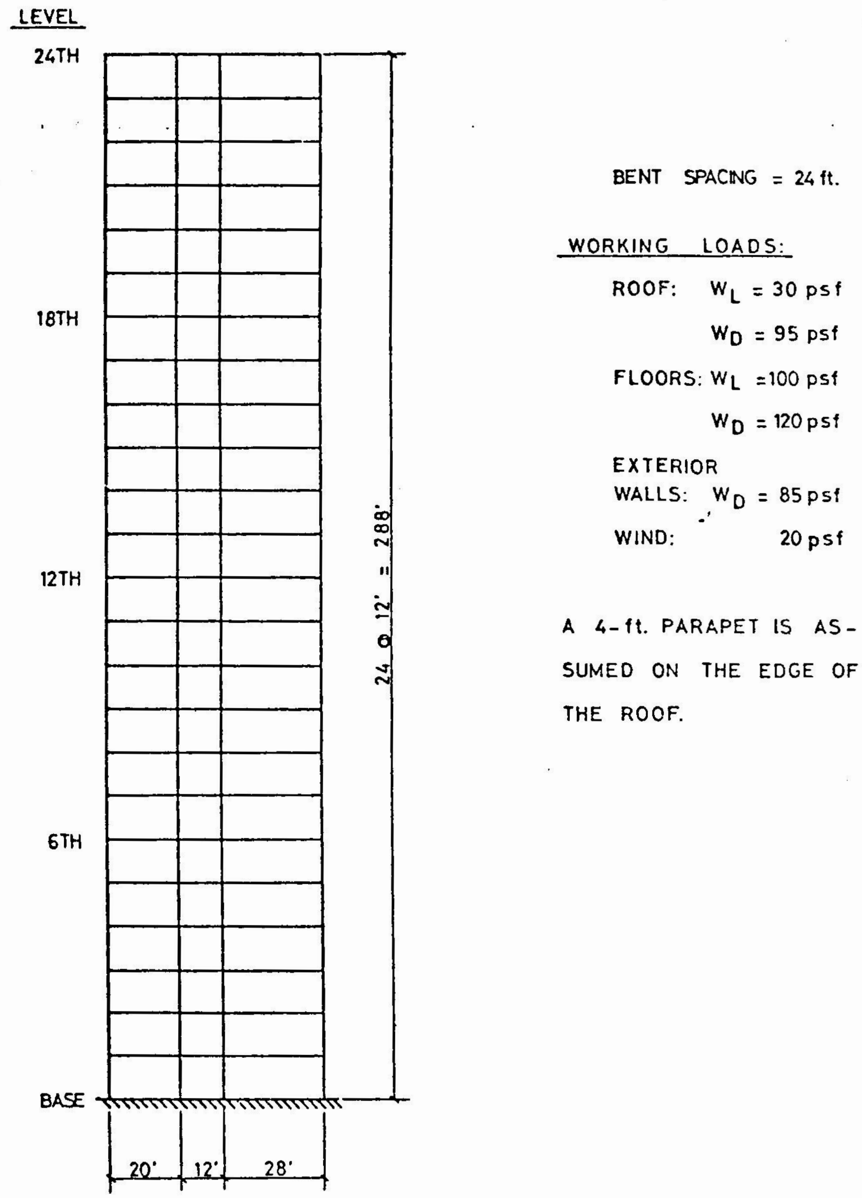


Figure 2 Frame-C Dimensions and Working Loads

(1 ft.=0.305 m, 1 psf=47.9 N/m^2)

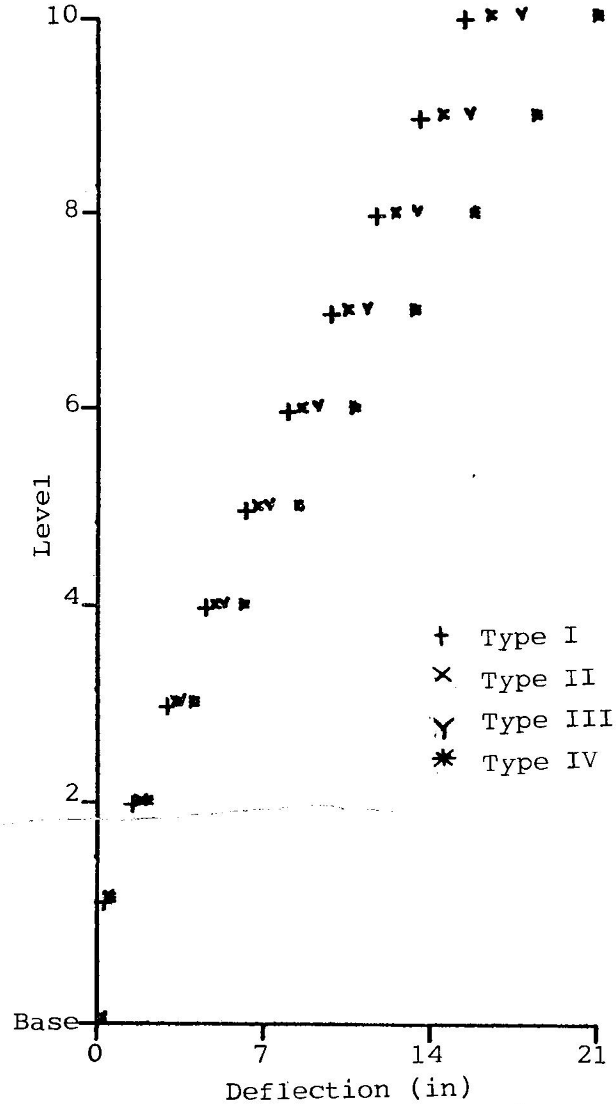


Fig. 3 Deflection Profile of Frame-B
El Centro Ground Motion
(1 in = 25.4 mm)

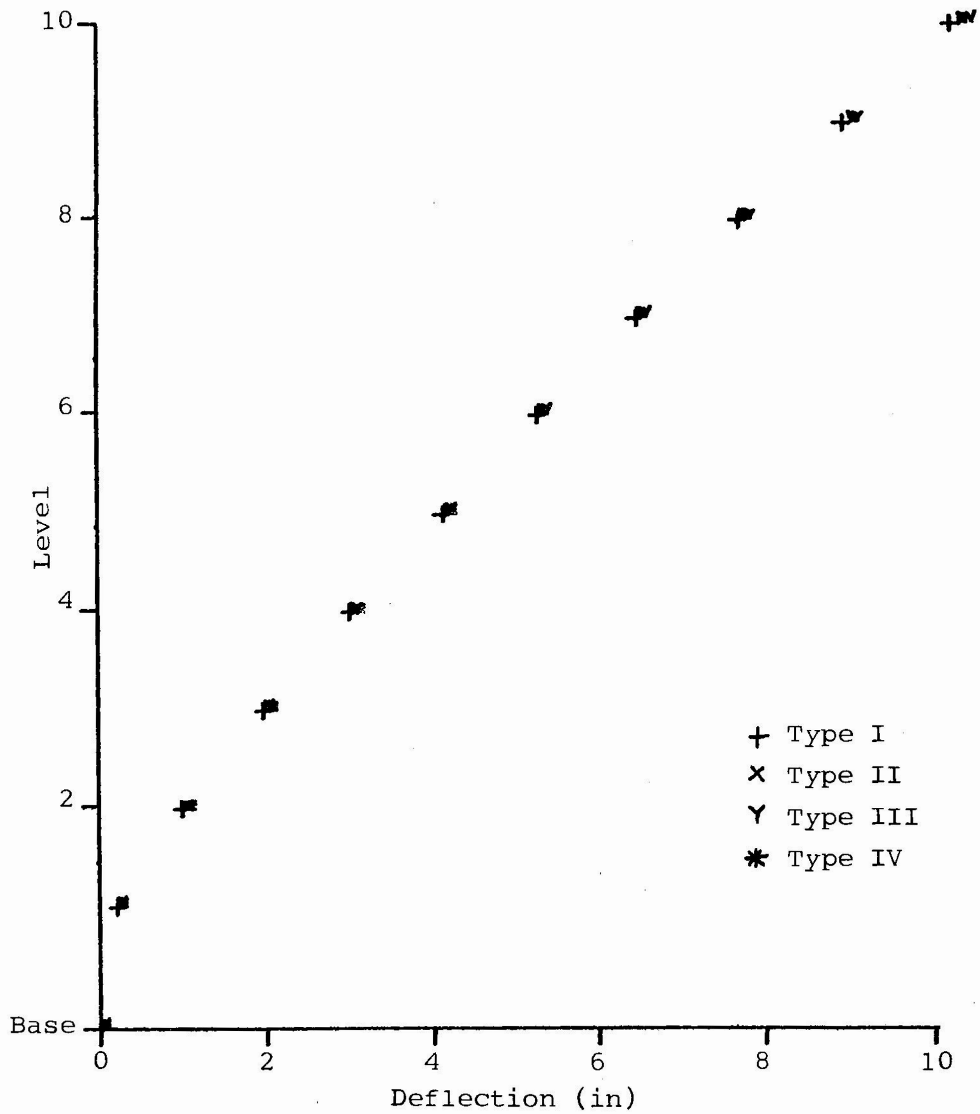


Fig. 4 Deflection Profile of Frame-B
Pacoima Dam Ground Motion
(1 in = 25.4 mm)

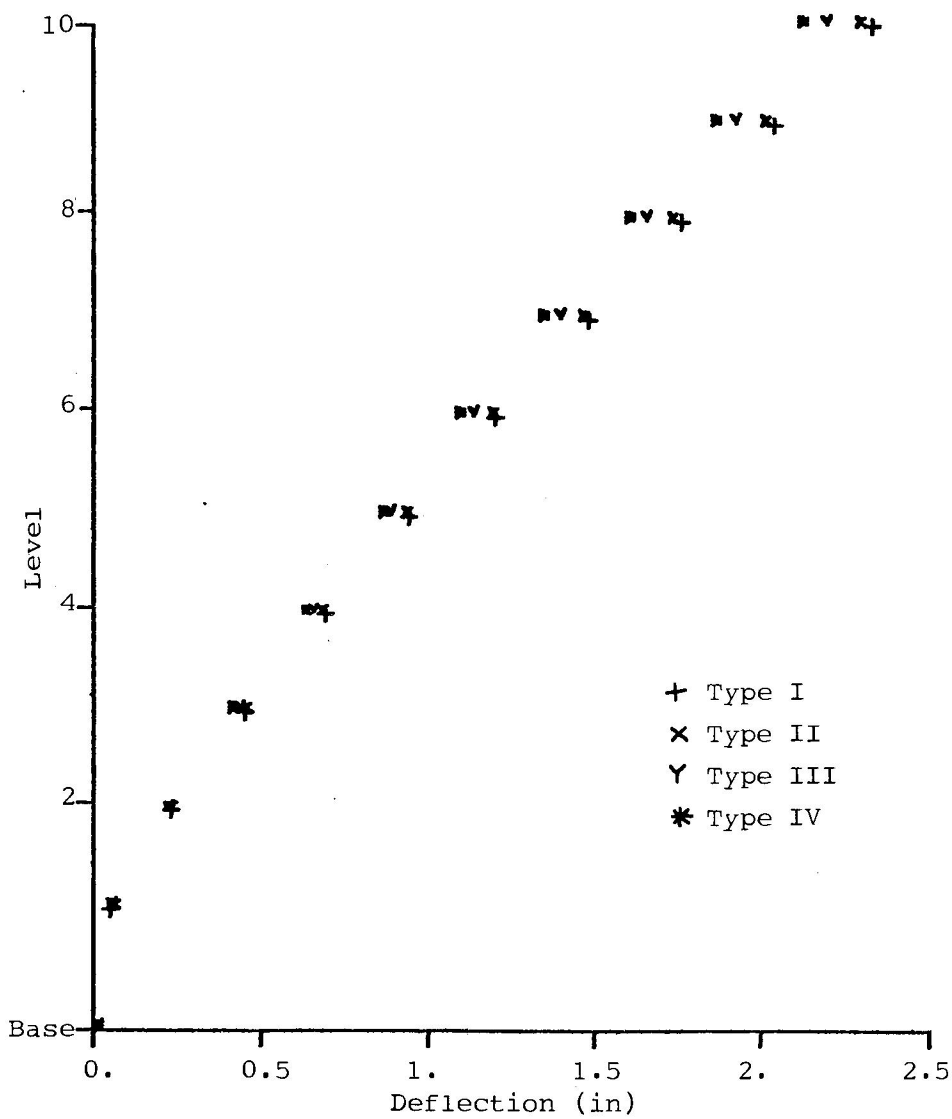


Fig. 5 Deflection Profile of Frame-B
Kajima Building Ground Motion
(1 in = 25.4 mm)

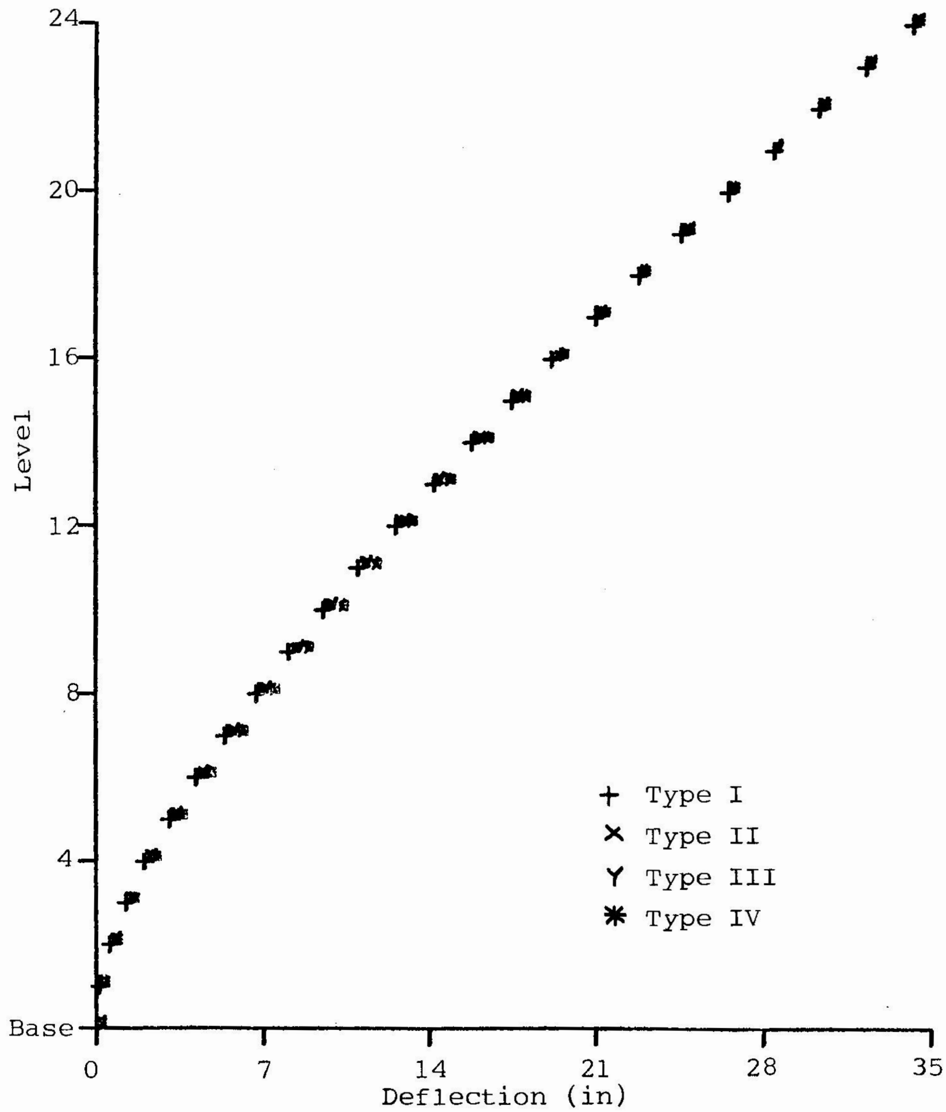


Fig. 6 Deflection Profile of Frame-C
El Centro Ground Motion
(1 in = 25.4 mm)

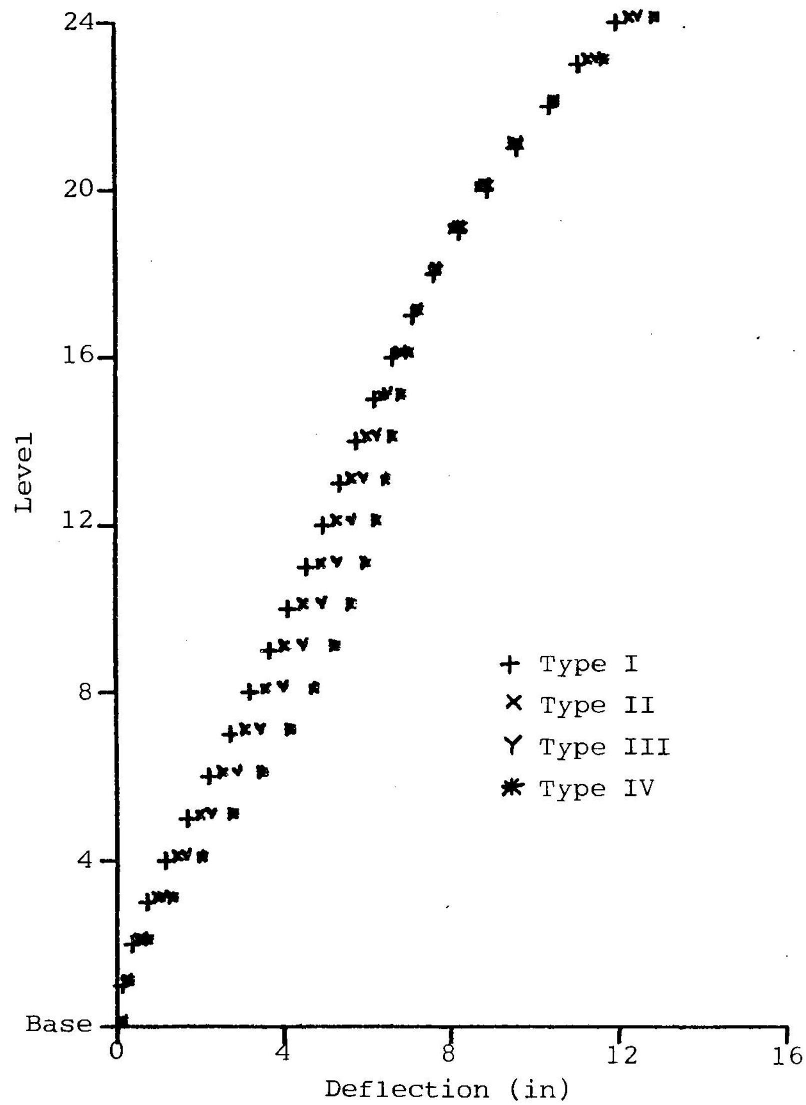


Fig. 7 Deflection Profile of Frame-C
Pacoima Dam Ground Motion
(1 in = 25.4 mm)

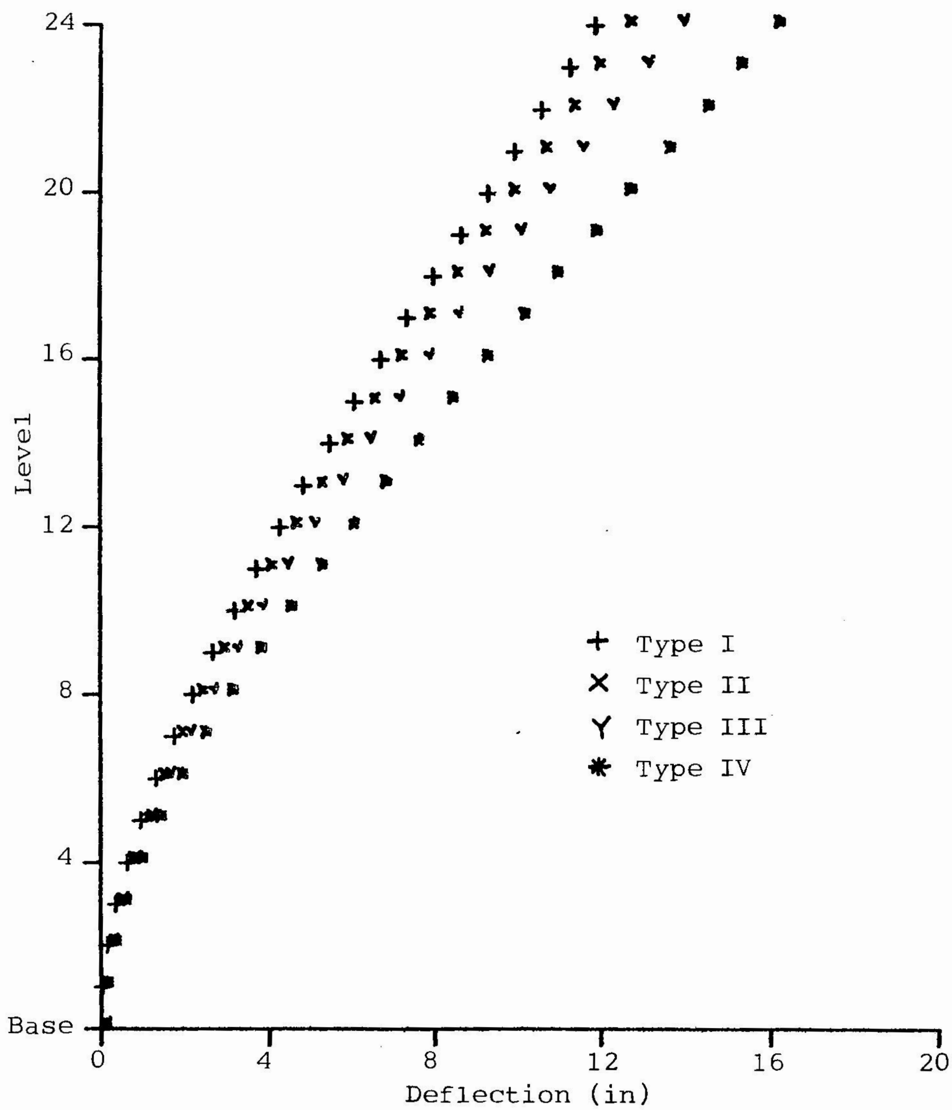


Fig. 8 Deflection Profile of Frame-C Kajima Building Ground Motion (1 in = 25.4 mm)