Structural System—Getty Plaza Tower

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In areas of strong seismic activity, the lateral load resisting structural systems for buildings need to be designed with appropriate stiffness and ductility, which are often counter objectives. Adequate stiffness is needed to resist building displacement during wind and moderate earthquake loadings, primarily in order to control damage to non-structural elements—and for taller buildings to mitigate perception of building sway during frequent wind storms. Secondly, adequate ductility is needed in the structural elements and connections as well as in the overall structural system to provide sufficient reserve of strength at large displacement to absorb severe earthquakes without building collapse.

The need to provide overall building stiffness and strength at least cost has lead to development of various structural framing systems. In particular, the braced-frame and tubular framing systems have gained a wide acceptance with structural steel application. A framed-tube structure typically has closely spaced perimeter columns that are interconnected at floor levels with deep spandrel beams, thereby creating a relatively rigid peripheral grid. The shear lag along the flange frames is greatly reduced, thus improving the effective cantilever stiffness of the overall structure.

One of the important factors which influences the stiffness of a braced frame is its height-to-width ratio, the efficiency of a framed tube system decreases as this ratio increases. To minimize conflicts with architectural planning, the braced frame is typically restricted to the elevator core or other service areas. Since the sizes of these areas are usually small compared to the overall dimensions of the building, the width of the braced frame is limited, which increases its height-to-width ratio. In high-rise buildings, stiffness requirements usually cannot be met with the slender braced frame alone, and supplemental framing such as a framed tube is necessary. This type of mixed system has been used effectively for many high-rise buildings.

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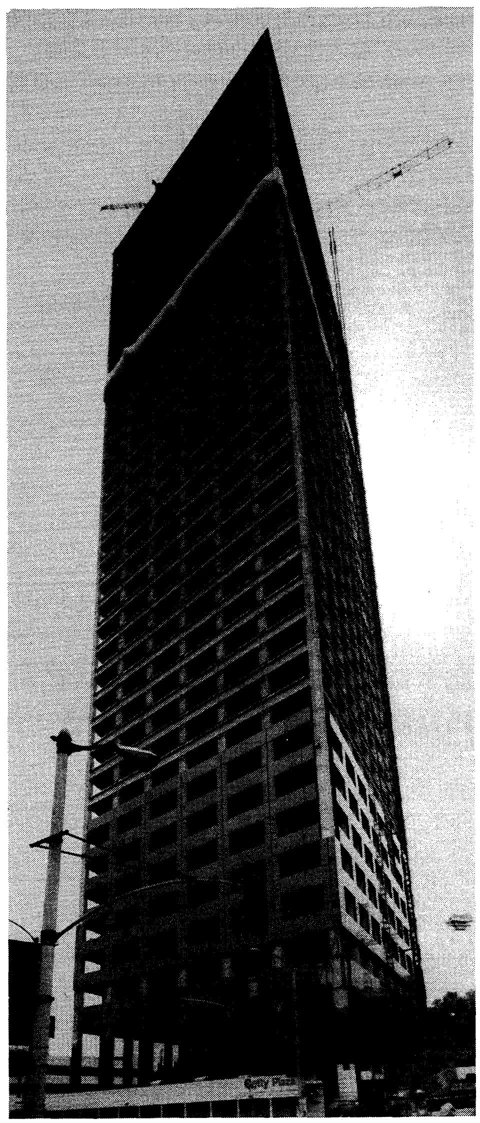


Fig. 1. Building elevation

Current building code provisions tend to reward the structural systems with demonstrated ductility such as ductile moment-resisting space frames or framed tubes, and tend to penalize the stiffer concentrically braced frames (CBF). Typically, a braced frame must be designed for the total seismic shear. A secondary backup ductile moment-resisting frame system must be provided for at least 25% of the base shear to develop a minimum overall structural ductility and resistance during a severe earthquake. Consequently, few structures are framed only with braced frames.

To improve ductility of braced frames, a continuing research effort has been in progress at the University of California-Berkeley on structural steel connections and eccentrically braced frames[2] (EBF) where the brace connection is moved out onto the beam away from the beam-column intersection. Consequently, the shear behavior of the beam controls the response characteristics of the frame, rather than the brace. As a result of this research, the ductility of the eccentrically braced frame system was confirmed and has been utilized recently in a number of structures.[3,4]

The following example provides a background for discussion of the advantages and behavior of the mixed structural system of perimeter framed tube with interior EBF. Furthermore, the stiffness and dynamic behavior characterized by the unusual parallelogram-shaped structure are examined.

The Getty Plaza project (Fig. 1), currently under construction, is a 36-story structural steel high-rise office tower (Fig. 2) with a 2,100 car parking structure and other ancillary facilities. The project is located in Universal City, near downtown Los Angeles, Cal. The site is in the proximity of the San Andreas Fault, about 31 miles to the northeast of the project. Also, the potentially active Santa Monica-Malibu Coast and Verdugo Faults are three miles and five miles northeast. The scope of this paper is limited to high-rise office building structures.

STRUCTURAL SYSTEM

Economic Evaluation of Alternate Systems—As building height increases, the forces of nature—gravity, earthquake, wind, etc. dominate the selection of the type of lateral load resisting system. The cost of controlling building drift under lateral loadings imposes a premium for high-rise construction. This is reflected in the accelerated increase in structural cost, compared to other costs for a high-rise building. Recognizing the relative efficiencies of alternate systems,[1] and the functional requirements of the architecture, two lateral load-resisting systems were considered feasible for the Getty Plaza Tower framing. Other gravity framings,

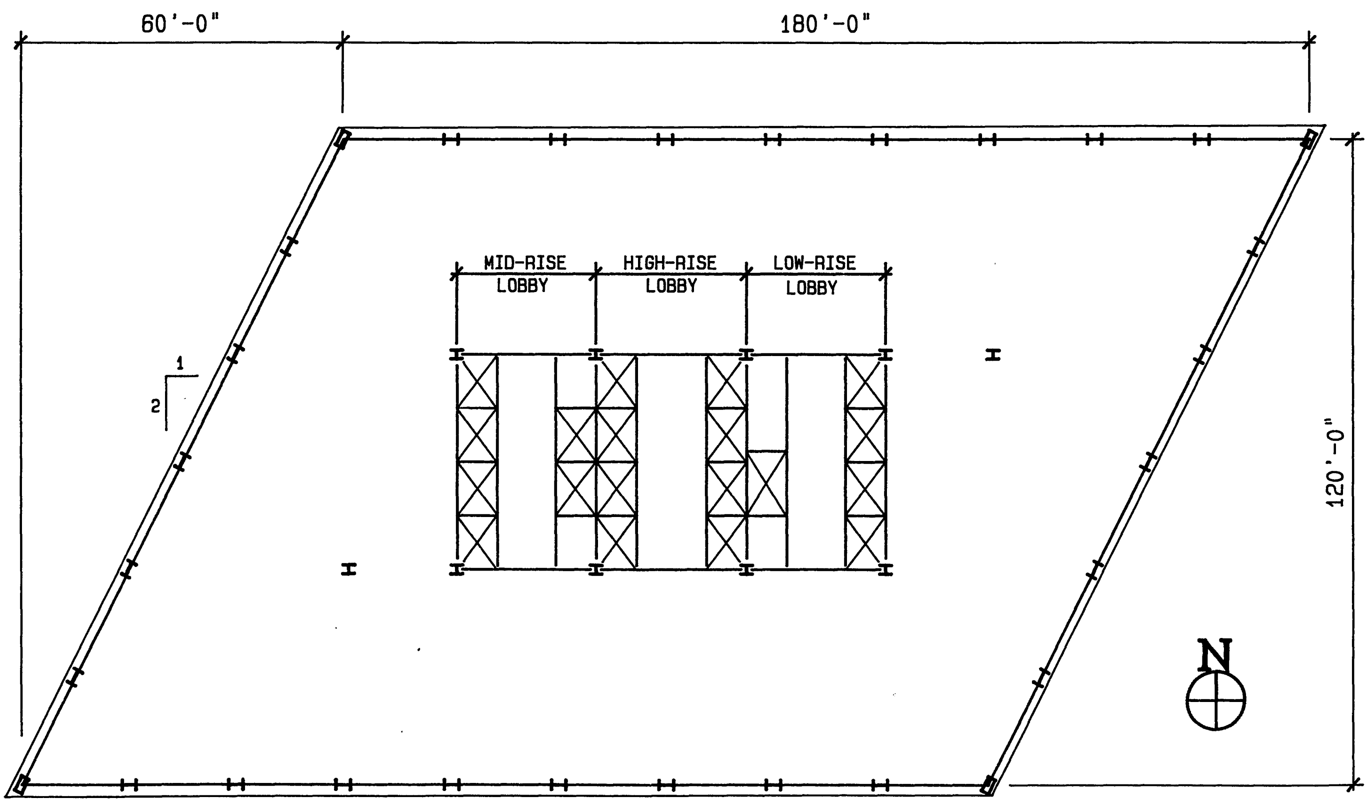


Fig. 2. Typical floor plan of office tower

such as floor systems, are essentially identical. A perimeter framed-tube structure (Scheme A) was first proposed. This scheme allows maximum freedom in interior architectural planning, since no interior frames are utilized. Second alternative (Scheme B) employs the mixed system as discussed earlier, consisting of a perimeter framed tube supplemented with an EBF. This type of framing is commonly known as the tube-in-tube or dual system.

The relative efficiency of these two systems had to be determined from the standpoint of economy. In the early stages of design, the required member sizes for stiffness and strength can be calculated by approximate methods,[5] and provide a reasonable estimate of the material quantities required. Approximate methods of analysis for framed-tube[5,6] and tube-in-tube[7] structures are well established for rectangular buildings. Additional parameters which affect the application of these methods to the building displacement due to lateral loading will be examined here. The lateral displacement of a building Δ_t, can be expressed as:

$$\Delta_t = \Delta_f + \Delta_c \tag{1}$$

in which, Δ_c is the displacement component due to overall bending of the frame as a vertical cantilever, and Δ_f is the racking component due to bending and shear deformations of the columns and beams. A simple approach to assess the cantilever deflection is based on the assumption that the entire structure acts as a vertical beam, to which elementary beam theory applies. Thus, axial stress in each column is proportional to its distance from the centroidal axis of the frame. In a typical rectangular structure with frames at right angles to each other, the centroidal axes coincide with the principal planes of the frames. On the other hand, for other structures with arbitrary configurations, the centroidal axes can be anticipated to lie approximately along the major and minor axes of the plan section. For this example structure, when subjected to wind loads along the direction of the braced frame, the axial force distribution and the centroidal axis are shown in Fig. 3. In this case, the axis is skewed with respect to the direction of applied load.

As with cantilever deflection, the racking component is also influenced by the particular plan configuration of the frames. With proper choice of stiffness combinations be-

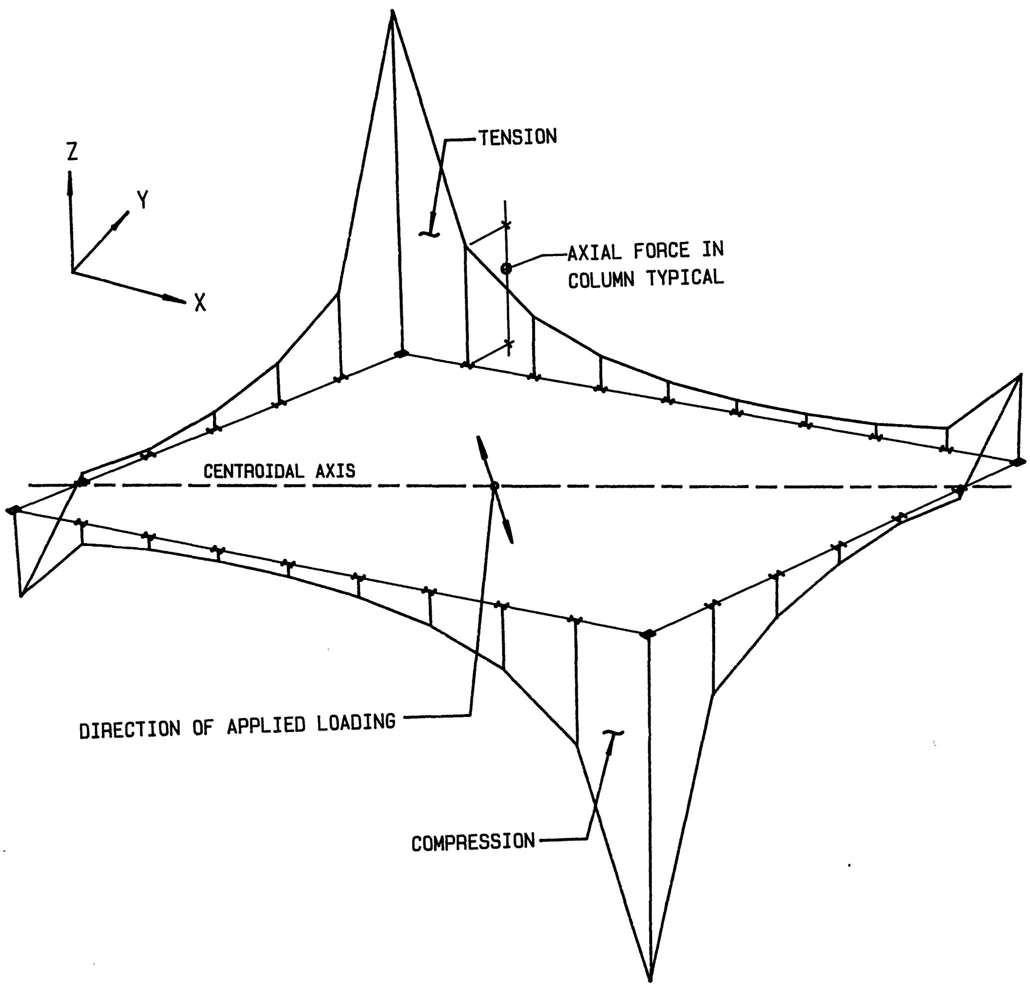


Fig. 3. Axial force distributions under lateral load

tween the longitudinal and transverse frames (see Fig. 4), identical displacement magnitude can be obtained for a given lateral loading condition, although in different directions. Taking these factors into account, the frame stiffness requirements for drift control under wind loadings are shown in Fig. 5. The stiffness requirements on the perimeter frames for Scheme B are substantially reduced because of the contribution of the EBF. As a result, smaller member sizes can be used. Next, member sizings based on the stiffness requirements are conducted, using methods as presented in Refs. 5 and 7 and are verified by computer analyses. These members are also checked to insure that strength criteria are met.

From the preliminary analysis procedures as outlined above, the material quantities for the two alternate schemes are established. It was found that Scheme B provides a 10% material saving compared with Scheme A. However, the additional fabrication costs for the EBF connections offset the saving to 5% of the total structural cost. From economic consideration, Scheme B thus was adopted for the Getty Plaza structure.

Behavior of Dual System—One important consideration in the design of a dual system is its interaction behavior between the component elements when subjected to lateral

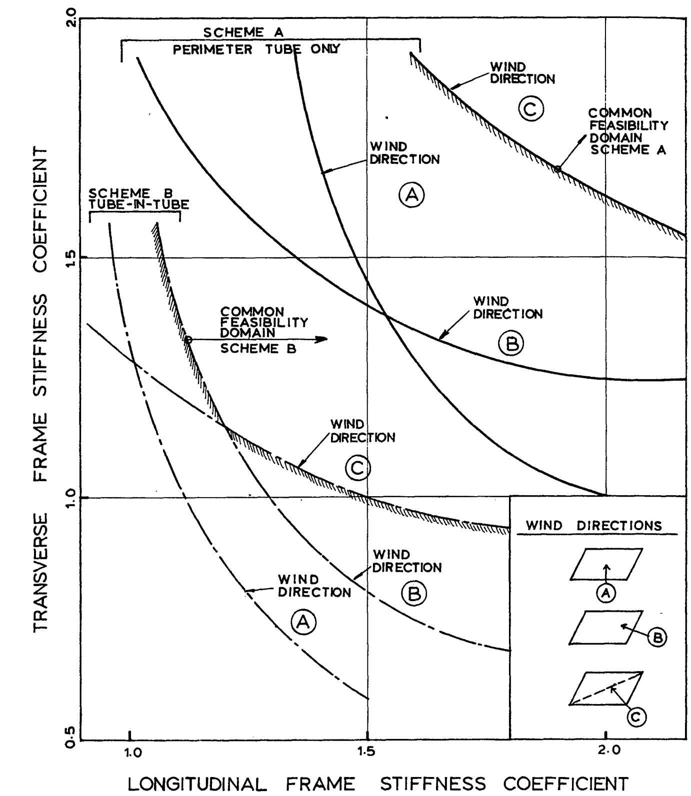


Fig. 5. Deflection contours for preliminary design

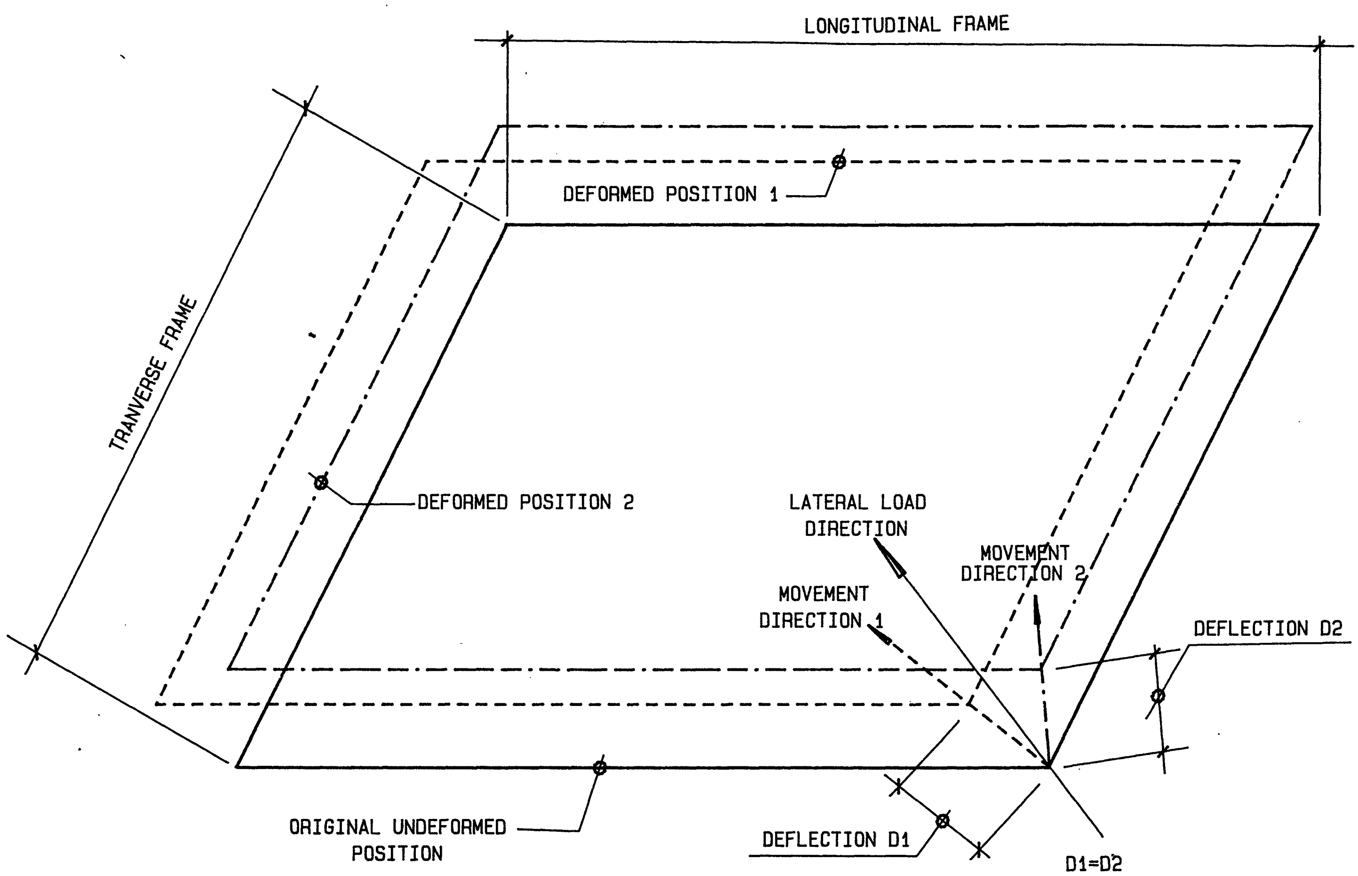


Fig. 4. Building movements under lateral loading

loads. The braced frame and framed tube exhibit different deformation patterns when subjected separately to lateral loads. A framed tube deforms predominately in a shear mode (Fig. 6a). On the other hand, a braced frame deforms primarily in bending mode (Fig. 6b). When both elements are connected by floor slabs, normally considered rigid in their own planes, identical displacement is enforced. The interaction between the two elements is such that the frame reduces the lateral deflection of the braced frame at the top. But near the base of the structure, the braced frame tends to dominate the deformed configuration. Interaction forces resulting from the different deformation patterns are shown in Fig. 6c. Obviously, the distribution of the applied shear to the resisting element in proportion to their relative stiffnesses can lead to erroneous results.

A typical variation of the interaction forces is shown in Fig. 6d for an applied static earthquake direction along the plane of the braced frame. The interaction forces are expressed as a ratio of the shear force as carried by the braced frame to that of the applied shear at the building height under consideration. It is important to note that at top stories, the total shear carried by the frame can exceed the applied story shear. This condition is evident from the observation of the shear forces in the braced frame, which act in the same direction as the applied loads. Fig. 6d also shows the increase in horizontal shear in the braced frame at the lower stories, where the braced frame is relatively stiffer than the framed tube. During a major earthquake, the yielding of the frame tube columns will impose additional strength requirements in the braced frame. The character of shear distribution, as discussed, is typical of a dual system.

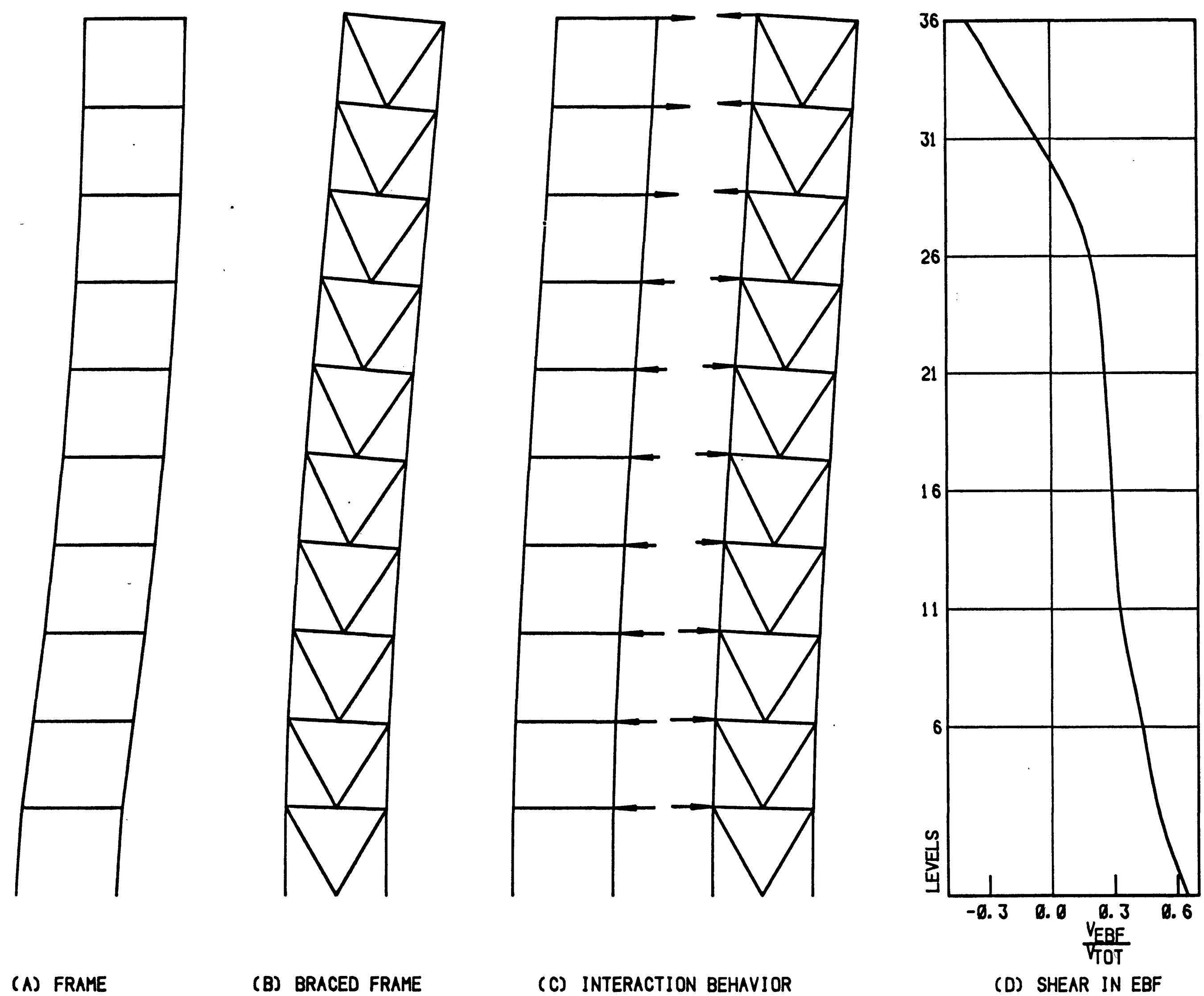


Fig. 6. Frame-braced frame interaction

Seismic Application of Dual System—Current building codes'[8,9] provisions have discouraged the application of dual systems in seismic zones. In conventional dual system ($K=0.8$), where a concentrically braced frame (CBF) is used, from experimental studies[19] it was found that CBF exhibits pinched and deteriorating hysteretic behavior as a result of buckling of the brace under repeated and reversed loading. This indicates poor energy dissipation capability as well as stiffness degradation. Such behavior can be eliminated if the braced frame remains elastic during severe earthquake excitation. Present codes[8,9] stipulate that 100% of the total base shear is to be resisted by the braced frames, for the purpose of substituting elastic behavior for ductility demands. In high-rise buildings, this criterion cannot be easily satisified, primarily because the stiffness contribution of the braced frame is comparable, or in some cases, smaller than that of the moment frames (see Fig. 6d). The foregoing criterion will generally require members of large proportion as well as an additional cost premium to the project. From these considerations, a dual system with CBF is seldom used.

On the other hand, experimental studies[2] on EBF confirm the favorable energy dissipation and stiffness behavior during cyclic tests. Such behavior is essential to the satisfactory performance of a structure during a severe earthquake. Recognizing the ductile behavior of the EBF, the requirement that the braced frame resist 100% of the base shear was waived by the building authority for this project. Therefore, in seismic regions, the EBF would be preferred over a CBF in a dual system.

PLANNING REQUIREMENTS OF ECCENTRICALLY BRACED FRAMES (EBF)

The design procedures and behavior of an EBF have been well-documented.[16,17] Planning requirements and functional considerations during design phase will be considered here.

To minimize interference with architectural functions, the EBF can be located ideally between adjacent elevator cores. Due to the wide spacing between columns, and also

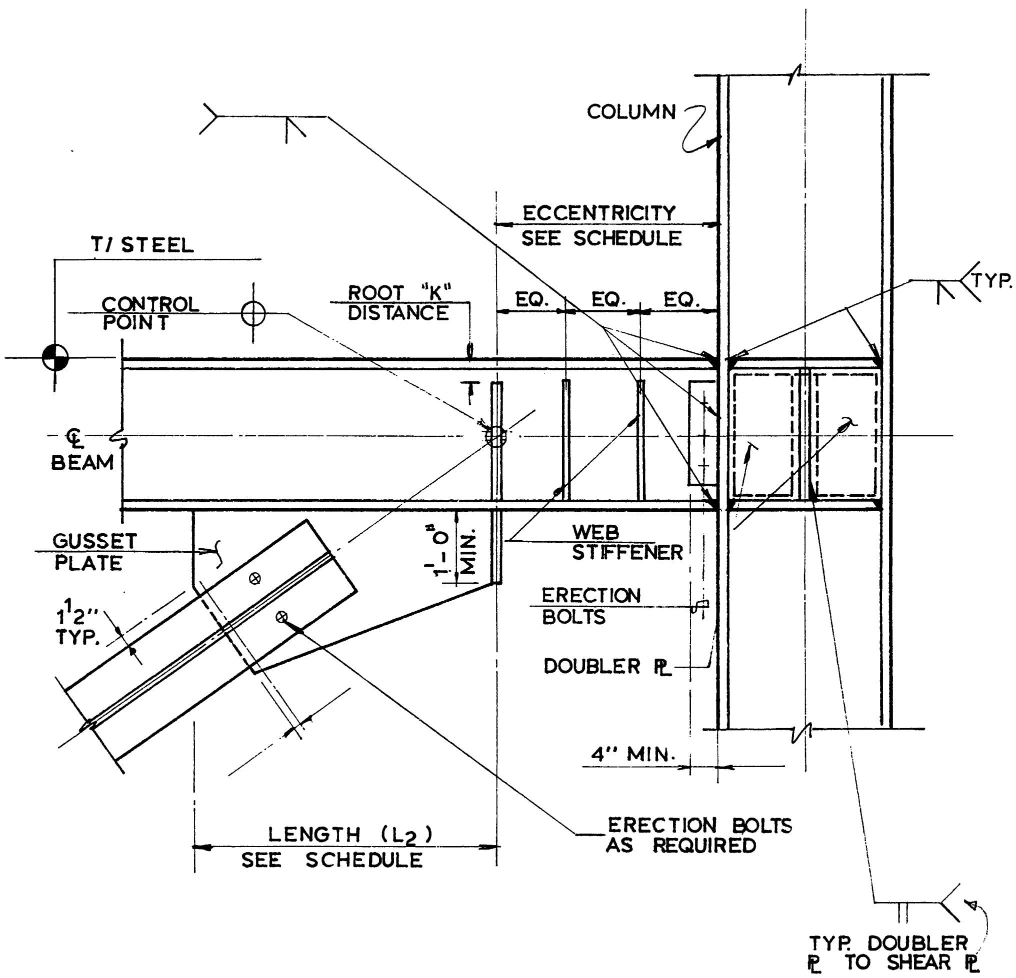


Fig. 7. Eccentrically braced frame detail at column end

for accessibility purposes, the inverted K configuration is used. Active shear links (Fig. 7) are provided at both ends of the beam adjacent to the column. It is important to note that the active shear link has to be braced at the work point of the beam—brace intersection, to inhibit lateral torsional buckling under large deformations. However, such beam braces cannot be accommodated without interfering with the elevator operations. During the early development phase of the project, coordination with the architect warranted a layout of the elevators to allow for the beam brace. The brace elements are comprised of two WT sections laced together with intermediate spacer plates to form an integral unit. The active link beam elements are designed to withstand the stress induced by the maximum probable earthquake (to be defined later) according to the interaction formula:

$$f_a/F_a + f_b/F_b = 1.0 \tag{2}$$

in which f_a is the computed axial stress resulting from the horizontal components of the brace forces. The eccentricity in the brace offset is proportioned to insure shear yielding in the beam elements such that :

$$Vp = 2.0{*}M_p/L \tag{3}$$

Due to the large magnitude of earthquake forces, the beam members used in the project range from W27 to W36's, which far exceed the test specimens as reported by Roeder and Popov in the previous test. To insure the stability of the beam web during inelastic yielding, additional web stiffeners were added. Upon Popov's recommendation, web stiffeners were spaced such that:

$$S/t \leq 30 \tag{4}$$

where S is the spacing of stiffeners and t is the web thickness.

His recommendation is based on recent research findings on tests of wide flange sections of 18 in. and larger depth. Typical details of EBF connections at mid-span and active link ends at the column are shown in Figs. 7 and 8.

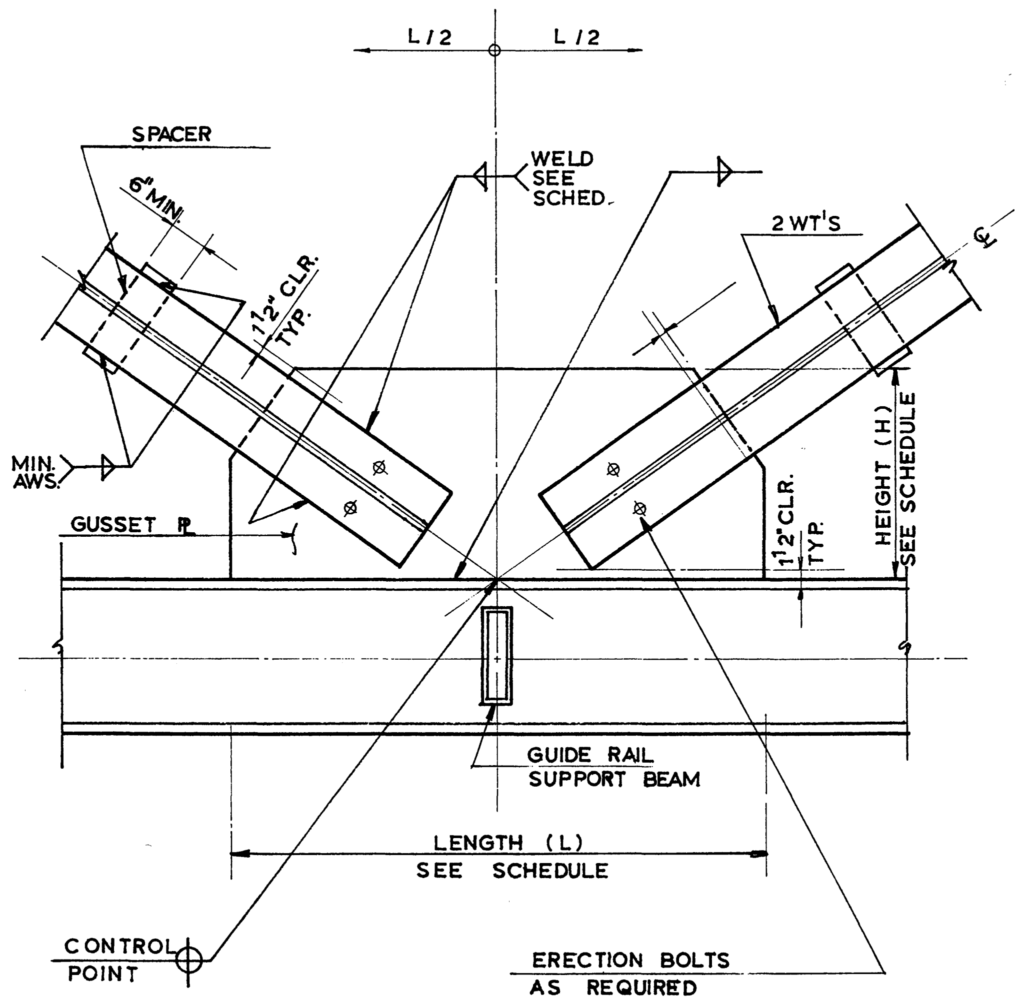


Fig. 8. Eccentrically braced frame detail at mid-point

DYNAMIC ANALYSES

Performance of a complex structure during earthquake excitation cannot be accurately predicated by the code prescribed "Equivalent Static Earthquake" approach. For a building with a complex structural system and shape such as the Getty Plaza, a detailed dynamic analysis is warranted. Linear elastic dynamic analysis employing the response spectrum method was carried out with a three-dimensional mathematical model using the program ETABS.[10]

For dynamic analysis, the seismic design criteria as set forth by the Los Angeles City Building Department[11] require that the structure meet two different levels of earthquake performance, based on the magnitude of two different earthquake events. First, for the maximum probable earthquake, defined as an event with 50% probability of being exceeded in a 50-year period, the structure has to remain essentially elastic. Second, from the considerations of the local geological conditions, the most extreme earthquake that can be anticipated, defined as the maximum credible earthquake, the structure shall not collapse, and controlled inelastic action is anticipated. Seismic studies were undertaken to determine the magnitude of such events. Fig. 9 shows the response spectra[13] with proper modification for the regional and local site conditions.

Of great importance in dynamic analysis are the structural stiffness properties. The contribution of the panel zone stiffness in a framed-tube structure will be reviewed in this section. Furthermore, problems associated with the unusual geometry of the example building will be examined.

Stiffness Formulations—The dynamic properties of the structure are directly related to the overall building stiffness. For a framed-tube structure, the finite member joint size (panel zone) has a significant contribution to the overall building stiffness, and therefore directly affects the building response to earthquake excitation. If rigid behavior of the panel zone is assumed, then the stiffness of the frame will be overestimated. The story deflection resulting from the panel zone deformation d_{zone} can be calculated approximately from the portal method. This assumes that points of

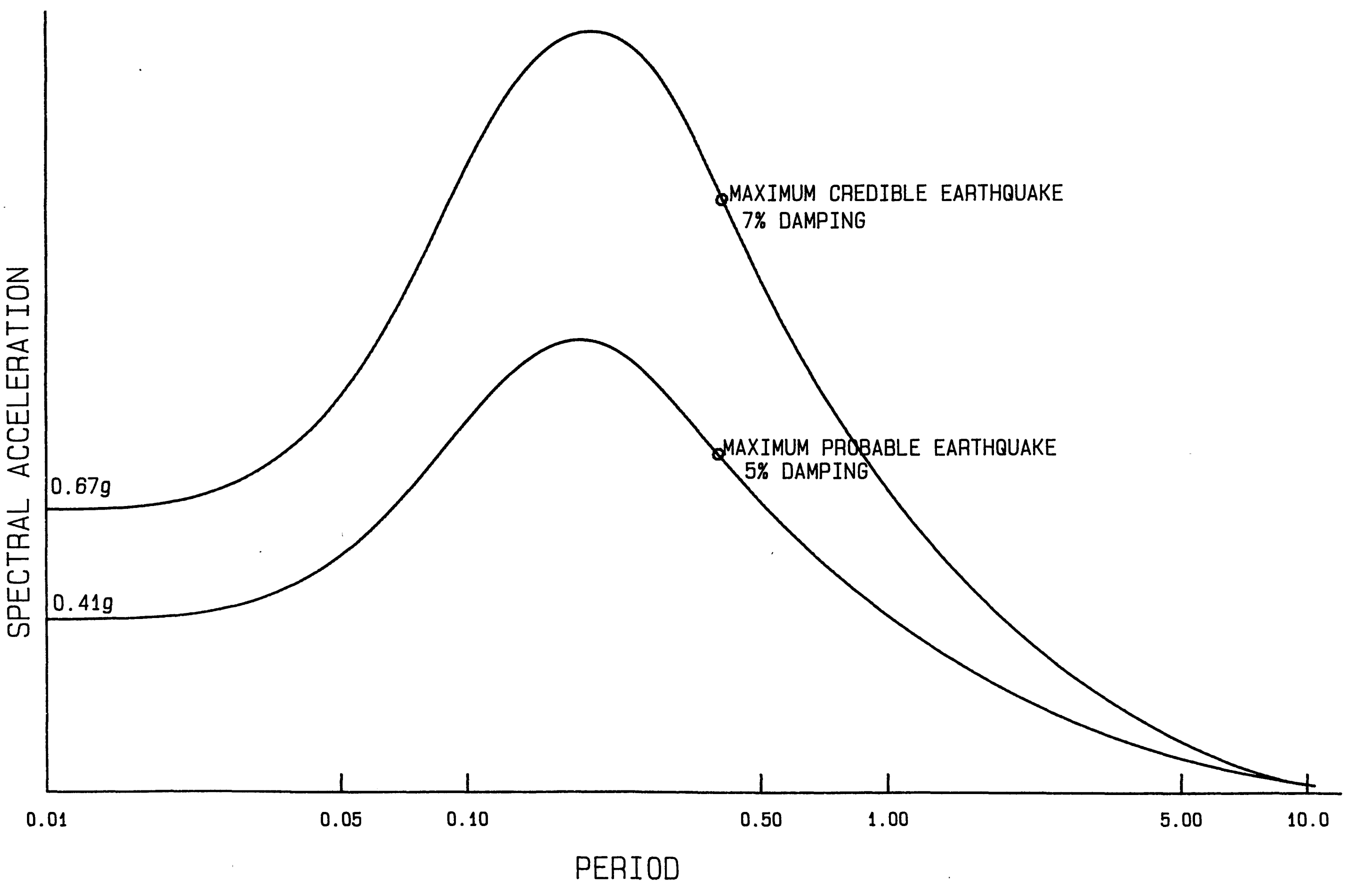


Fig. 9. Response spectra

inflection occur at mid-heights of columns and at mid-spans of beams. Krawinkler[12] reports such deflection under unit lateral load to be:

$$d_{zone} = \frac{h - d_b}{d_c t G} \quad (5)$$

where d_b,d_c are the finite beam and column depths, t is the web thickness, including doubler plate if any, H is the story height and G is the shear modulus.

It is common practice to represent the mathematical model based on the center line dimensions of the physical structure, in which case, the stiffness of the panel zone should be modelled with appropriate methods.

Critical Directions of Earthquake—Once the stiffness and mass properties of the structure are defined, the mode shapes and frequencies can be determined from the classical free vibration problem. The periods, as well as the directions of the mode shapes, are shown in Table 1 and in Fig. 10 for this example building. Figure 11 shows the configurations

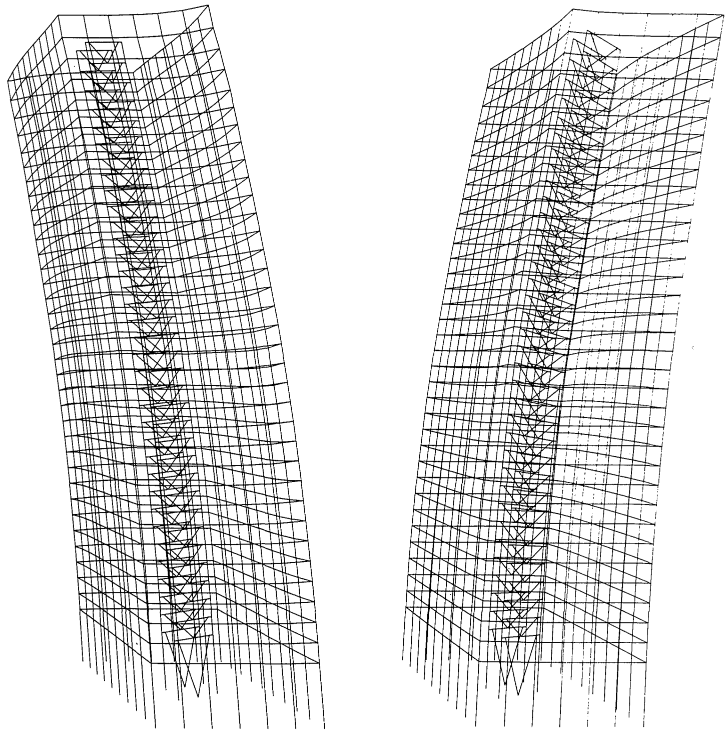
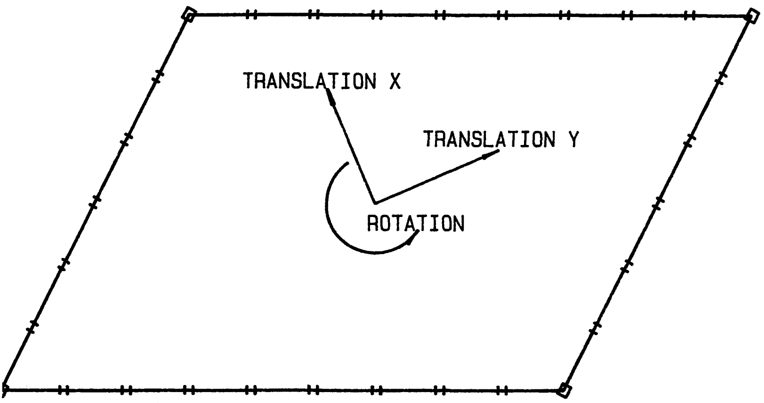


Fig. 11. Principal translation mode shape configuration



DIRECTION / MODES	TRANSLATION X	TRANSLATION Y	ROTATION θ
1st Mode	4.900	3.240	2.585
2nd Mode	1.880	1.396	1.180
3rd Mode	1.071	0.869	0.758
4th Mode	0.709	0.592	0.523
5th Mode	0.524	0.460	0.400
6th Mode	0.408	0.379	0.326

Fig. 10. Directions of fundamental mode shapes and building period

Table 1

Responses / Components	Responses in X & Y			Critical Directions		
	X	Y	$\Sigma f_{xi} f_{yi}$	P $\theta_{cr} = -69.4°$	M_x $\theta_{cr} = 23.1°$	M_y $\theta_{cr} = 22.9°$
P Kips	939.0	2013.0	−1384604	2138.2	608.7	607.6
M_x ft-kips	914.4	590.4	254442	491.3	972.0	972.0
M_y ft-kips	562.0	311.8	112409	224.2	602.7	602.8

of the two principal fundamental translation modes. In the response spectrum method of analysis, the response of an individual mode to a particular earthquake motion can be obtained from the spectral values of a single degree of freedom system (Fig. 9). In view of the fact these individual maximum modal responses do not occur at the same time, a better estimate of the maximum overall system response is obtained from statistical combination of the modal maxima. Different combination methods[14,18] have been proposed, of which the square root of the sum of the squares combination has particular interest. In this method, the overall response is expressed as:

$$f_{max} = \sqrt{\Sigma f_i^2} \qquad (6)$$

where f_{max} denotes the estimated maximum response and f_i is the modal maximum of the ith mode.

In most cases, a simplification of the true three-dimensional earthquake response of structure is made by assuming the design horizontal acceleration components to act nonconcurrently in the direction of the main axes of a building. It is generally accepted that a building designed by this approach will have adequate resistance against the acceleration acting in any direction. However, it is doubtful such assumption is valid for this example structure. In particular, the main axes of this building are not well defined. As clearly illustrated in Fig. 10, the alignments of the mode shape axes are skewed with respect to the planes of the frames. Therefore, the critical directions of earthquake must be quantitatively evaluated to determine what will produce maximum forces and deformations.

Consider a three dimensional structure, subjected to earthquake excitation along two arbitrary horizontal axes X and Y, which are mutually perpendicular to each other. The modal responses, as well as the overall response, can be readily calculated along each axis. For an earthquake excitation along an axis, inclined at an angle θ to the x axis, the overall response (F_θ) can be represented as:

$$F_\theta = \sqrt{(f_{xi}\cos\theta + f_{yi}\sin\theta)^2} \qquad (7)$$

where f_{xi}, f_{yi} denotes the responses along the X and Y axes respectively for the ith mode.

This equation can be simplified as:

$$f_\theta = \sqrt{F_x^2\cos^2\theta + F_y^2\sin^2\theta + \sin2\theta\Sigma f_{xi}f_{yi}} \qquad (8)$$

where F_x,F_y represents the overall response along the X and Y axis.

The critical angle (θ_{cr}) for maximum response can be then obtained by:

$$\frac{d(F_\theta)}{d\theta} = 0.0 \qquad (9)$$

thus, the critical angle can be shown to be:

$$\tan(2\theta, 180+2\theta) = \frac{2\Sigma f_{xi}f_{yi}}{F_x^2 - F_y^2} \quad \text{where } 90 \geq \theta \geq -90 \qquad (10)$$

These formulations can be readily incorporated into existing analysis programs, and enable the calculations of critical angles from the responses along two orthogonal axes. As can be seen from Equation 10, for a column member, the critical angles for axial force, major and minor axis bending moments can have different values. However in the design of this member in accordance with the interaction formulas of the AISC Specification,[15] these variables are evaluated along the same angle of earthquake input. Table I shows the critical angles for a typical corner frame column. This column is of rectangular shape box section. In this particular case the critical angles for the major and minor axis bending are essentially identical and the critical angle for maximum axial load is along one of the principal mode shape directions.

The foundation system for the perimeter-tube structure consists of a continuous strip footing, with depth varying from six to eight ft. This footing is analysed as a beam on an elastic grade. Since the equations previously developed are valid only for individual members, for a continuum such as the strip footing, the corresponding forces and bending moments on all columns resulting from the same direction of earthquake excitation have to be evaluated globally. Five earthquake directions are examined along the following directions: (a) the principal mode shape directions (3 directions) (b) the direction along the plane of each frame (2 directions). The earthquake along the direction of the braced frames was found to produce the maximum bearing pressure underneath the footing for this structure.

CONCLUSIONS

In the design of building structures located in areas of high seismic risk, the proper balance between stiffness and ductility must be maintained. The dual system, as used in Getty Plaza Tower, fulfills both of these objectives. With proper detailing both of the components in this system, the perimeter framed tube and the interior EBF, exhibit excellent energy dissipation capability, essential to the satisfactory performance of the building during a major earthquake. In addition, the advantages of this system in meeting the stiffness requirements at relatively low cost provide an economical solution for highrise framing.

Due to the non-ductile behavior of CBF, current building codes stipulate the braced frame component in a dual system has to be designed for the total seismic base shear. In contrast, the EBF demonstrates excellent energy dissipation capability. This code requirement should be revised to recognize such behavior, and to establish proper classification for such systems in a seismic application.

The deflection and dynamic characteristics of the Getty Plaza Tower have been identified. To a large extent this behavior is typical for buildings with general configurations. For such general shaped structures the prediction of the critical angles of earthquake excitations that produces the maximum force or deformation in a particular member is not readily apparent. Formulations are presented which enable the calculations of such angles from the responses of two orthogonal earthquake directions.

To provide an economical framing system for the Getty Plaza Tower, a unique dual structural system was developed. The unusual configuration of the structure required close examination of its stiffness and dynamic properties, and led to a better understanding of the general behavior of a three-dimensional structure.

ACKNOWLEDGEMENTS

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NOMENCLATURE

f_a = computed axial stress
F_a = allowable axial stress
f_b = computed bending stress
F_b = allowable bending stress
F_x,F_y = axial or bending stress
G = shear modulus
h = story height
K = system factor for equivalent static earthquake method
L = span for beam
M_p = plastic moment capacity
V_p = modified yield shear capacity
X,Y = principal directions of mode shapes

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