COMPUTER-AIDED STRUCTURAL ANALYSIS AND DESIGN OF THE 37-STOREY LOS ANGELES BONAVENTURE HOTEL

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SYNOPSIS

The paper briefly describes and summarizes the computed-aided analyses and design for a complex convention hotel structure in Los Angeles. Seismic analyses, upon which structural steel sizing was based, were both static and dynamic. The latter were carried out for two postulated levels of seismic ground motion, and included both response-spectrum and time-history methods.

Columns, column splices and joint details at beam to column connections were designed using a computer program.

THE ARCHITECT-ENGINEER TEAM

The designers of the project were John Portman & Associates (Portman), Architect-Engineers of Atlanta, Georgia. The analyses (1) were performed by URS/John A Blume & Associates (URS/Blume), Engineers of San Francisco. Concurrently, a column design program was developed by L Z Emkin in accordance with the established design criteria to ensure ductile response of the principal frame elements.

DESCRIPTION OF THE STRUCTURE

The Bonaventure Hotel, opened in March, 1977, is located in the Bunker Hill area of Los Angeles. Fig. 1 is a general plan of the building, which consists of a square 7-storey podium supporting a 37-storey (total) central tower and four 30-storey peripheral towers (Photograph Fig. 2). All five towers are circular in plan. The four peripheral towers are located symmetrically along the diagonals of the square podium and are connected to the central tower by bridges at every level above the seventh storey and at most of the lower levels. Fig. 3 shows the plan of the structural steel system at a level above the podium.

The entire structure is supported primarily by steel framing under both vertical and lateral loads. The towers consist of two concentric circular frames composed of vertical columns and horizontal beams, as shown in Fig. 3. Core walls around the inner circular frames below the third guest level have been included in the mathematical model. Except for a few girders with moment connections along the diagonal direction, the two concentric frames are connected primarily by rigid diaphragms. The floor slab diaphragms were of precast concrete planks with in-situ topping. Shear studs were provided on beams. Lateral loads are therefore resisted mainly by the tube actions of these two concentric frames.

Gravity loads are transferred to the main framing either directly or by a system of simply supported beams and girders not shown in the framing plan. Below the seventh storey there are a series of frames with moment-resisting connections in the two directions parallel to the sides of the podium (Fig. 5). The bridges connecting the central tower with the peripheral towers are assumed to behave as elastic diaphragms in the horizontal plane.

The columns were assumed to be fixed at the base in the static analyses. However, to model the structure more accurately in the dynamic analysis, the base was assumed to be hinged, and the concrete walls along the perimeter below the second storey were also included.

ANALYSIS PROCEDURE

Phase 1. Preliminary Selection of Member Sizes. Portman did the initial sizing, analysis and resizing of the structural elements. Computer programs were used in both analysis and resizing of columns.

Phase 2. Mathematical Model. A mathematical model was developed for the static and dynamic analyses of the structural system on the basis of the proposed preliminary design, including the selection and implementation of necessary modifications in the computer programs used for these analyses. The unusual structural system in the hotel design made it impossible to use an available computer program without several minor modifications. Even with these modifications, certain model idealizations were required because of program limitations and cost considerations.

Phase 3. Static Analyses. Initial static analysis of the structure was performed using preliminary design section properties subjected to the gravity and lateral loads specified by the 'Los Angeles City Building Code'(2). A second static analysis was

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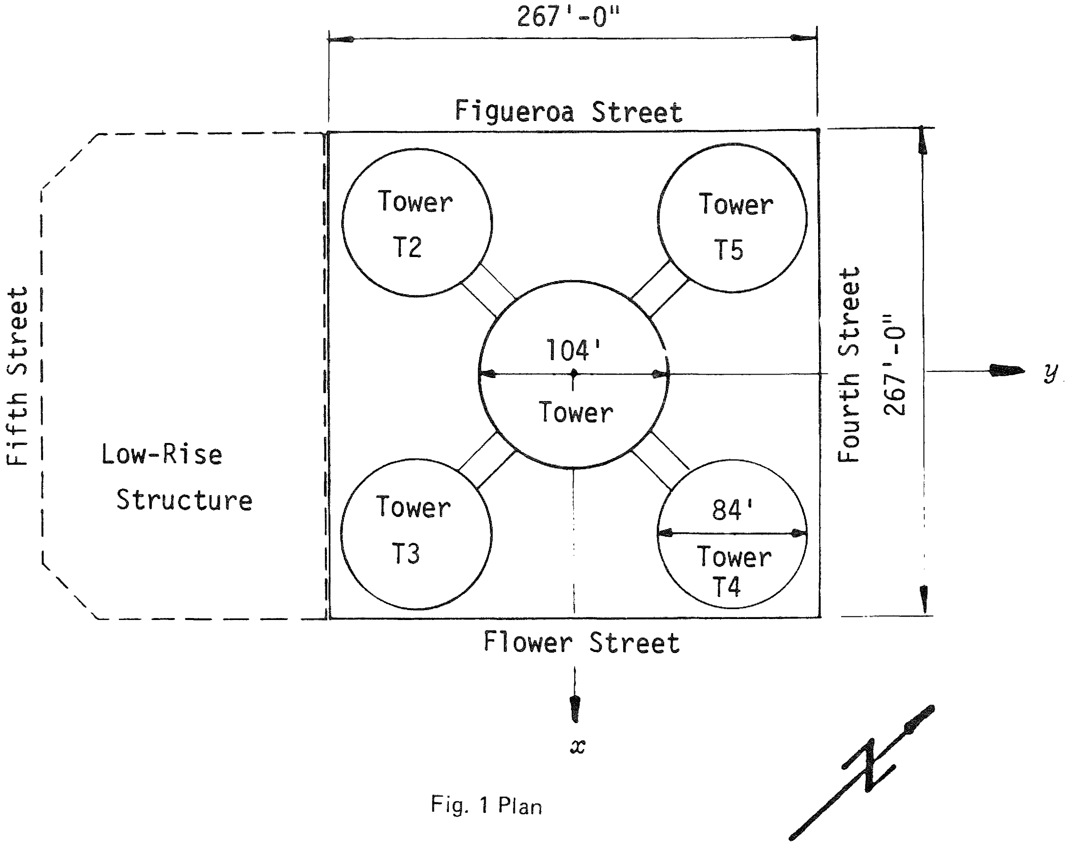

267'-0"
Figueroa Street
Tower
T2
Tower
T5
104'
Tower
Fourth Street
267'-0"
y
Fifth Street
Low-Rise
Structure
Tower
T3
84'
Tower
T4
Flower Street
x
N

Fig. 1 Plan

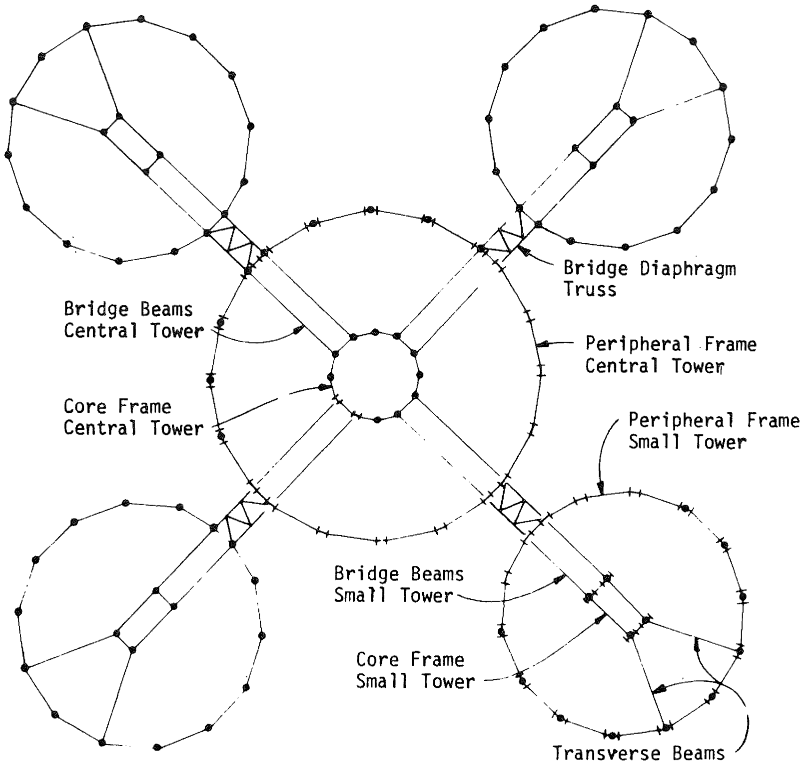


Fig. 3 Structural Framing Plan, Levels 11-31

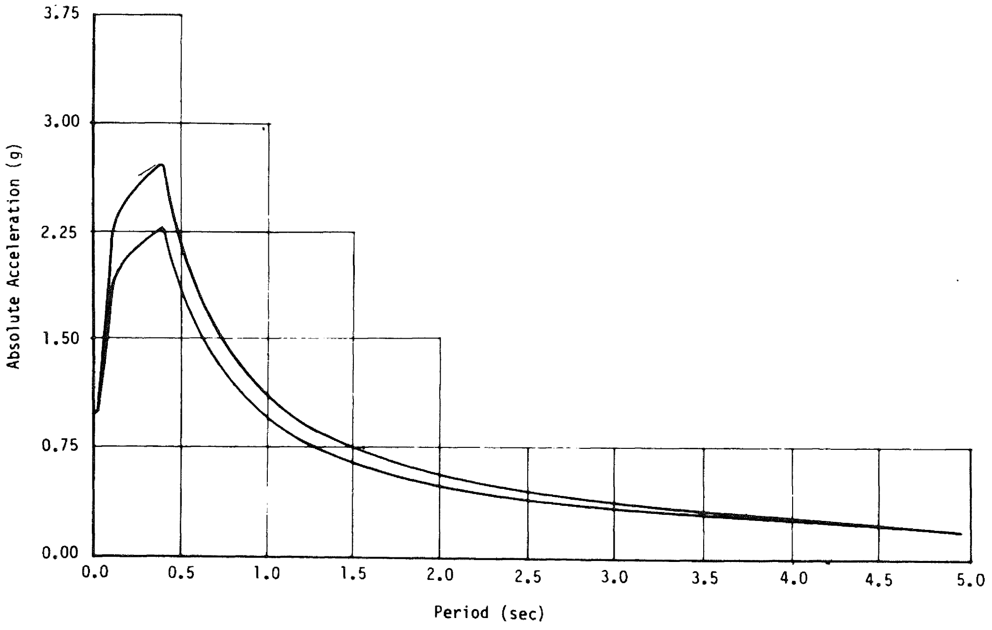


Fig. 4 Design Spectrum Shape Normalized to 1.0g (Damping = 7%, 10%)

performed with the revised structural member sizes.

Phase 4. Redesign of Members. Redesign was based on the first static analysis results, and partial redesign was based on results of the second static analysis. This phase was performed by the Architect-Engineers.

Phase 5. Development of Special Seismic Design Criteria. Seismic design criteria were developed separately, concurrently with Phases 1 through 4. Phase 5 consisted of postulating design earthquakes, specifying the criteria that the structure should satisfy during those earthquakes, and adopting a design procedure based on these criteria.

Phase 6. Dynamic Analysis. This phase consisted of dynamic analysis of the structure subjected to the ground motions postulated in Phase 5 and evaluation of the structure's responses considering the criteria established in phase 5.

Phase 7. Redesign. Redesign was based on the dynamic analysis results that followed from the suggested design procedure adopted in Phase 6. This phase was performed by the Architect-Engineers.

DESIGN CRITERIA

Static Analysis

The 'Los Angeles City Building Code' requirements for seismic loads were similar to the corresponding 'Uniform Building Code (UBC)' requirements. However, the Los Angeles code was in the process of being modified, requiring a dynamic analysis for all structures taller than two or three storeys. The modification also specified the maximum probable ground motion (0.5g) and the input spectrum to be used as input for analysis. Because a dynamic analysis was planned for the structure and because initial evaluation and sizing were needed for the structure before dynamic analysis, the 'UBC' seismic force requirement was used for static analysis. The 'Los Angeles City Building Code' was followed for gravity and wind loads.

Dynamic Analysis

Design Earthquakes. Based on a study of seismic history of the area and the local soil conditions(3,4) a set of response spectra (called earthquake risk spectra) were developed for various assumed useful lives of structure (L) and probability of non-occurrence (P) values. They also provided their interpretation of SEAOC criteria for minor, major and moderate earthquakes in terms of L and P values. In consultation with Wiggins, it was decided to use earthquake spectra based on SEAOC performance criteria for moderate and major earthquakes(5). Furthermore, it was decided to use a proposed response spectrum shape developed with the help of a URS/Blume study for the Atomic Energy Commission. The proposed response spectrum shape, shown in Fig. 4, was based on statistical studies of response spectra for 33 accelerograms from 12 major earthquakes(6). The spectra were normalized as follows for the major and moderate earthquakes:

Earthquake	Maximum Ground Acceleration (g = acceleration due to gravity)
Major (B)	0.54g
Moderate (A)	0.38g

These levels of ground acceleration may be interpreted as a magnitude 7+ earthquake (e.g. the 1940 El Centro earthquake) at 10 miles epicentral distance for the moderate earthquake (A) and a magnitude 8+ (e.g. the 1906 San Francisco earthquake) at 15 miles epicentral distance for the major earthquake (B).

Damping. Damping ratios of 0.07 and 0.10 were assumed to be appropriate for the mostly welded steel construction subjected to the moderate and major earthquakes, respectively.

Allowable Ductility. The following damage criteria were considered reasonable. For moderate earthquakes, non-structural damage and minor-to-moderate structural damage is acceptable. For a major earthquake, non-structural damage as well as moderate-to-major structural damage without collapse is acceptable. Based on these criteria, the following allowable ductility factors were recommended: girders - less than 3.0 for earthquake A and less than 6.0 for earthquake B; columns - less than 1.5 for earthquake A, and less than 2.0 for earthquake B.

The following equations may be used to compute the reduced plastic moment capacity, M_1, of columns that are supported (i.e. have no buckling problem) or unsupported, respectively:

$$M_1 = 1.18\left(1 - \frac{P}{P_y}\right) M_p$$

$$M_1 = \left(1 - \frac{P}{P_o}\right)\left(1 - \frac{P}{P_e}\right)\frac{M_p}{C_m}$$

where

M_p = moment capacity of the section if there were no axial force

= fy Z, where fy = yield stress and Z = plastic section modulus

P_o = axial load that would cause failure if no moment were present

P_e = elastic buckling load in the plane of bending

$C_m = 0.6 - 0.4q \leqslant 0.4$ (side sway prevented)

$= 0.85$ (side sway not prevented)

q = ratio of moments at the two ends $(1.0 \geqslant q \geqslant -1.0)$

P = computed axial load

P_y = axial load at yield stress

The reduction factors used in the second equation above are the basis for those specified by the American Institute of Steel Construction (AISCE. see Refs. 8 and 9) for working stress design of columns.

The ductility factor cited above refers to curvature ductility. Because an elastic dynamic analysis is contemplated, the approach used must obtain ductility factors from the elastic analysis forces that are greater than the corresponding yield values. To do this, it is assumed that the displacements of the actual structure, which behaves inelastically, and those of the elastic model are essentially the same. The ductility factor may then be estimated simply as the ratio of computed (elastic) force to the yield level force. Although somewhat crude, this approach is considered a reasonable alternative to the as yet impractical approach of computing inelastic dynamic response of large structures.

Connection Design. Although the structural framing is designed to remain elastic under code gravity and lateral loads, it is recognized that plastic hinges may form in the rigid frames under response to severe seismic motion in excess of code requirements. With only a few limited exceptions, the frames for this structure were designed under the weak beams/strong columns criterion, which implies that plastic hinges would form at the beam ends and thereby limit the forces to be resisted by the beam-column panel zone, column splices, and column-base connection. The connection design criteria developed for the structure required that all elements of the above connections would be designed to resist, at or below yield stress, the forces associated with formation of plastic hinges in the beams or in the columns for those few exceptions where column yielding would occur before yielding of the beams.

MATHEMATICAL MODEL

A modified version of the computer program SAP (Structural Analysis Program) was used in the static and dynamic analysis (7). It can accommodate static and dynamic analyses of general threedimensional elastic structures that may be modeled by beam elements, various two- and three-dimensional finite elements, or a combination of both. The program has been modified by URS/Blume to improve its input/output capabilities; it includes some added capabilities, especially with respect to the beam elements. For the analyses described here, a restart capability for dynamic analysis and additional binary outputs for use as input to other post-processor programs were added. Several other computer programs, some developed specifically for this project, were used to generate input data for SAP and to combine the results of various SAP runs.

Because of the complexity of the Bonaventure Hotel structural system, many simplifying assumptions were necessary to make the analysis feasible. The structure is essentially doubly symmetric about two pairs of mutually perpendicular vertical planes. For the purpose of analysis it was assumed that the structure is doubly symmetric about vertical planes xz and yz (Fig. 1). Therefore, modeling of only one-quarter of the structure is sufficient for analysis purposes if proper boundary conditions are applied at the nodes on the planes of symmetry.

All loadings were assumed to be either symmetrical or antisymmetrical about the xz and yz planes. An additional simplification was the use of a part of the structure symmetrical about a plane at 45° to both the xz and yz planes. The lobby level was assumed to be entirely above ground in order to retain this symmetry.

The structural system is essentially modeled as a framework of beam elements having the same structural properties in shear, tension and bending as the corresponding real beams and columns. The core walls along the perimeter of the inner circular frames (Static Analysis No. 2 and Dynamic Analysis) and the walls along the perimeter below the second storey (Dynamic Analysis only) were also included in the model.

The floor slabs were accounted for by restricting the movement of nodal points included in or bordering on these slabs in accordance with the assumption that they are rigid in their own plane except for bridge slabs, which were modeled for stiffness in their own planes and represented by beam elements.

The weights of structural elements were applied as uniformly distributed loads. The weights of non-structural elements were applied either as distributed loads or as concentrated loads and moments. All gravity loads (dead and live), with the exception of the weights of columns, were assumed to be located at the floor levels.

A node and element numbering system was adopted that permits use of the data generation option in the SAP program. The structure was divided into three groups of floor levels, and within each level the nodal points were numbered sequentially. The nodal points of Level 1 were numbered first, then those of the next floor above, and so on. (Fig 5.) There are a total of 1398 nodal points.

The nodes at the centroid of the towers are master nodes where the horizontal degrees of freedom (x, y, zz) of a particular floor are concentrated to reflect the rigid floor-diaphragm assumption. Corresponding degrees of freedom of other nodes at a floor were slaved to those of a master node.

All elements are the beam element-type defined in the SAP computer program manual. A simple numbering scheme is also used for the elements. All the beams in a given bay or all the column in a given column line are numbered sequentially from top to bottom.

The material properties assigned to beams included those of 'air' (virtually zero density and strength to represent numbered but non-existent elements), steel and concrete. Structural steel cross-section properties were for standard wide flange (I) sections and certain selected box sections.

STATIC ANALYSES

The purpose of these analyses was to obtain the displacements and member forces generated in the structure under vertical dead and live loads and lateral wind and seismic loads. The member forces were used to check the structural member sizes and modify them if necessary. The loads and design of structural members were based on the 'Los Angeles City Building Code' requirements.

Two separate static analyses were made for the subject structure. Preliminary design member sizes were used in the first analysis as input to the computer program. Extensive modifications in member sizes were found to be necessary. It was decided to make another static analysis run using the new sizes. Although some modifications to the member sizes were required to satisfy the results of the second analysis, it was judged that these were not extensive enough to justify an additional static analysis.

All the lateral loads were applied at the nodal points only, while the gravity loads were applied both as member and nodal loads. Two types of lateral loads were applied to the structure: wind and seismic. In the static analyses, both were commuted and applied in the manner outlined by the 'Los Angeles City Building Code'. The seismic forces were computed by treating the entire structure as an assemblage of four substructures (see Table 1); (1) the podium, (2) the central tower above the podium, (3) a small tower above the podium, and (4) the helistop on top of the central tower.

The use of the quarter structure required that the loads be symmetrical or antisymmetrical about the two planes of symmetry. The applied forces must thus fall into one of four categories. Because each category required different boundary conditions at the cut points on the plane of symmetry, a separate model and a separate SAP run were required for each category. These four categories of applied loads and corresponding SAP run names are (1) symmetrical about xz plane, symmetrical about yz plane (gravity loads); (2) symmetrical about xz plane, antisymmetrical about yz plane (lateral loads, x-direction); (3) antisymmetrical about xz plane, symmetrical about yz plane (lateral loads, y-direction); and (4) antisymmetrical about xz plane, antisymmetrical about yz plane (horizontal torsion loads).

The wind and seismic forces were therefore separated, where necessary, into components that would fall into one of the above categories for analysis in different SAP runs. The resulting displacements and member forces for the components were later combined by separate programs to obtain the resulting values for each load case.

The results of the static analyses were supplied to the Architect-Engineers in the form of computer print-outs listing the displacements at each nodal point for each load case and member forces in each element for the appropriate combination of load cases. A total of ten load cases and eight load case combinations were used in the analyses. The member forces in each element for each individual load case on a magnetic tape (in binary form) were also supplied. The latter could then be directly used as input to the design computer program to check the adequacy of sizes of all the elements for all desired load combinations.

Lateral displacements under seismic forces computed in Static Analysis No. 2 are shown in Figs. 6 through 8. These displacements are in the direction of the applied forces and are of the same magnitude for the three in-phase seismic load cases. Fig. 8 gives the displacements of the two towers for the out-of-phase seismic load case. The central tower is displaced in a direction opposite to the applied forces acting on it up to the roof of the small tower and then reverses its displacement pattern. This shows that the forces on the four small towers govern the direction of the central tower displacements, and that the bridge, as modeled, is an effective medium of horizontal shear transfer between the two towers.

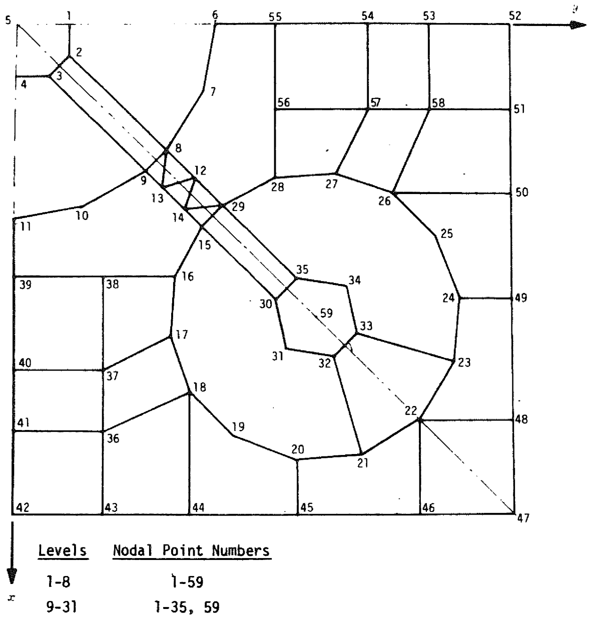
Lateral displacements under wind loads are much less than those under seismic loads and are slightly different in the two directions along which the forces are applied. The comparison in Table 2 of lateral displacements obtained for the two static analyses for some of the load cases shows that the structure has been made much stiffer in the second analysis. This is especially so in the podium levels (below the 4th Retail Roof level) and is caused by the addition of the cylindrical concrete wall in the core region of the two towers. For Static Analysis No. 2, the average drift index under seismic loads is 0.0035 in the tower region and slightly less in the podium levels. Although slightly high, this is acceptable. The average drift index under wind loads is less than .0009. The modification of design (i.e. change in member sizes) that was performed based on the results of the second static analysis was expected to stiffen the structure further.

DYNAMIC ANALYSIS

The purpose of the dynamic analysis of the structure was to obtain its dynamic response to design earthquakes A and B, defined earlier under 'Design Criteria' as moderate and major earthquakes, respectively, and to evaluate the structure's capacity to withstand these earthquakes in accordance with the established criteria. The mathematical model was essentially the same as described under 'Static Analyses', with similar input except that the columns were assumed hinged at the base, and the walls along the perimeter below the second storey were included. The dynamic analysis was performed for the two earthquakes in directions, x, y, x(+45)y (i.e. east-west), and x(-45)y (i.e. north-south).

The SAP computer program was used in the analysis. As stated earlier, several modifications were made in this program to accommodate the present dynamic analysis, including introduction of a capability so that a modal analysis performed first in the x and y directions for the two design earthquakes yielded natural frequencies and mode shapes that were stored on a magnetic tape for later use in the earthquake response analysis. The natural periods and mode shapes were then reviewed critically. The fundamental periods of the structure in x and y directions are quite long, 4.48 and 4.45 sec, respectively.

The first 20 natural mode periods in the x direction are listed in Table 3. The corresponding periods in the y direction are almost the same, as expected. The requirements of symmetry restrict the central tower to movement in the direction of applied load. However, the small tower is free to move in x and y directions as



Levels	Nodal Point Numbers
1-8	1-59
9-31	1-35, 59
32-37	1-14

Fig. 5 Nodal Point Numbering

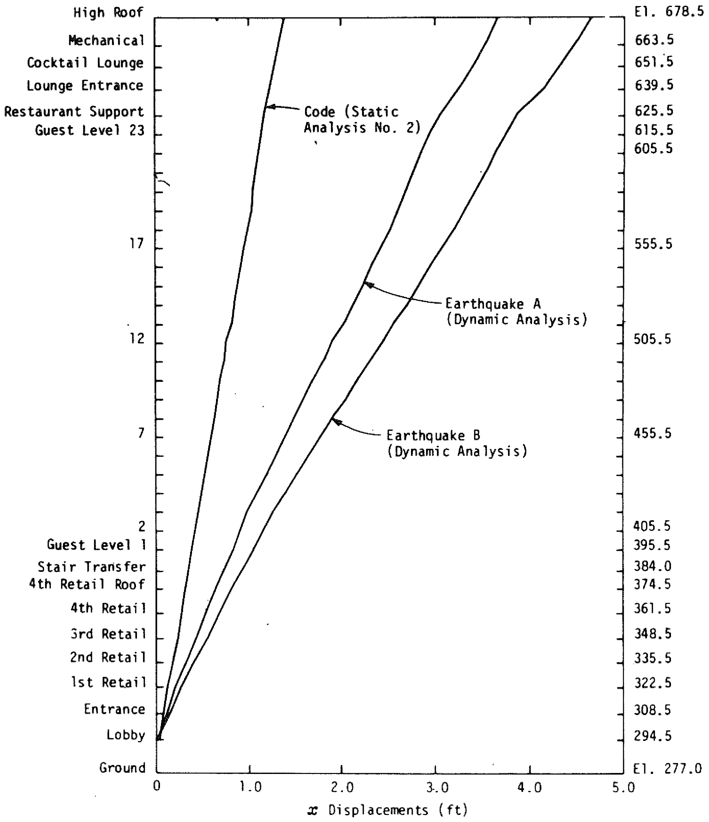


Fig. 6 Displacements, Central Tower

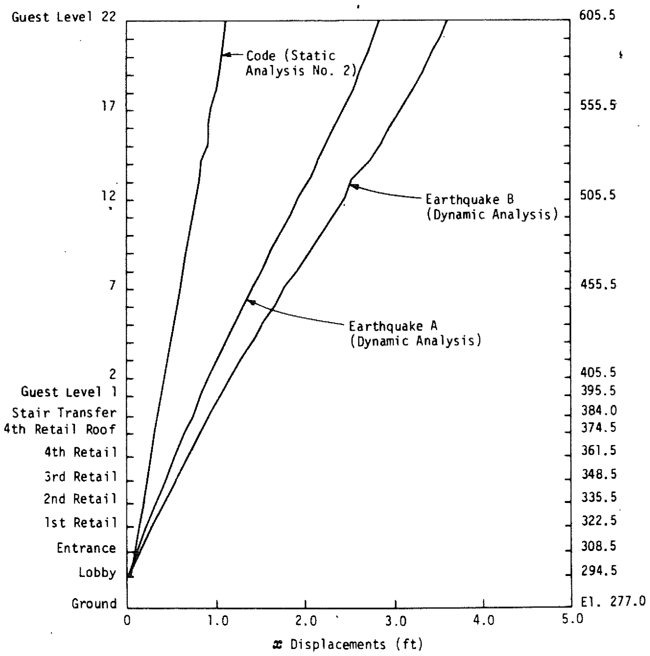


Fig. 7 Displacements, Small Tower

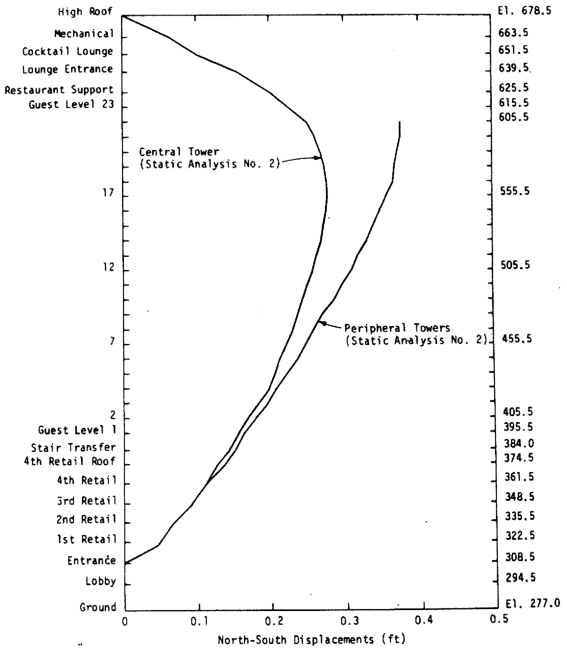


Fig. 8 Displacements, Central and Small Towers

well as rotate about the vertical (z) axis. The natural mode shapes reflect this fact by showing coupled motions of the central tower in one direction and of the small tower in all three directions.

Using the above mode shapes and the natural frequencies, response-spectrum analyses were performed in x and y directions for both design earthquakes. Damping of 0.07 and 0.10 of critical was used with moderate and major earthquakes, respectively. The modal responses (member forces and node displacements) were obtained and stored on magnetic tape. Separate computer programs were then used, as in the static analysis, to compute the modal member forces and mode displacements in the x(+45)y and x(-45)y directions from the corresponding values in the x and y directions stored on tape. The modal responses in the x(+45)y and x(-45)y directions were also stored on another magnetic tape. These modal member forces and displacements for the x, y, x(+45)y, and x(-45)y directions were combined by the square-root-of-the-sum-of-the-squares method to obtain the maximum member forces and the nodal displacements in those directions.

The results obtained from the response spectrum analysis were verified by running a time-history analysis for one of the earthquakes. The time-history results were within 10 per cent of the response-spectrum results.

Another computer program was used to compute the maximum building or storey responses, i.e. storey displacements, storey accelerations, storey shears, and storey overturning moments in the x, y, x(+45)y, and x(-45)y directions from the modal master node displacements and column shear forces in the corresponding directions. Some of these results, are shown in Figs. 6 to 11, and describe the overall dynamic behaviour of the structure.

Figs. 6 and 7 show the maximum floor displacements in the central tower and small tower, respectively, under the code seismic forces (Static Analysis No. 2) and the two earthquakes. The dynamic displacements are approximately 2.7 (Earthquake A) to 3.4 (Earthquake B) times higher than the code displacements, reflecting that much larger ground motions are hypothesized under the two earthquakes. The average storey drift index under Earthquake B is approximately 0.012 (1.4 in. in 10-ft. or 36 mm in 3 m storey height), and is considered acceptable for the magnitude of the earthquake considered.

Maximum floor accelerations are generally between 0.2 and 0.4 g. That is, accelerations at almost all levels except the top are smaller than the maximum ground accelerations, due primarily to long natural periods of the structure: the eleventh or twelfth mode periods are close to the peak period of the ground motion response spectrum (Fig. 4).

Figs. 9 and 10 show the storey shears in the two towers and the podium under the two earthquakes and the code forces. These are based on applied forces and are not the actual resisting shears. They are obtained by the summation (for each mode separately, in the case of earthquakes A and B) of computed inertia forces at the floor levels. The podium shears shown are computed inertia forces at the floor levels. The podium shears shown are computed by treating the podium as an independent structure. The total shears in the podium levels of the building as a whole would be obtained by adding the base shears in the central tower and small towers to be podium shears shown in the figures. The applied and resisting shears in the central and small towers under Earthquake A compared to Fig. 11 show that the shears are transferred from the central tower to the small tower by the bridge diaphragm connecting them. Note also that the applied shears are partly resisted by damping in the structure, and hence the sum of resisting shears is less than the sum of applied shears.

In terms of applied shears (Figs. 9 and 10), the base shear coefficient comparison between the building code and the two earthquakes is as follows:

Structure	Code	Earthquake A	Earthquake B
Central Tower	0.035	0.145	0.184
Small Tower	0.038	0.123	0.155
Podium	0.056	0.211	0.269

The base shear coefficients for the two earthquakes are approximately four (Earthquake A) to five (Earthquake B) times as large as the corresponding code value for the central tower and podium, and three to four times as large for the small tower.

The results of the dynamic analyses were supplied to the Architect-Engineers in the form of computer printouts listing the maximum displacements at each node point and maximum member forces in each element for eight load cases, i.e. for the two earthquakes, each applied in four directions. Element forces from Static Analysis No. 2 for dead and live loads were also included for convenience. The maximum member forces in each element for the resulting ten load cases were also supplied on a magnetic tape in binary form. These results were used by the Architect-Engineer to check the ductility requirements in all the elements, as outlined under 'Design Criteria'. It was found necessary to increase the column sizes throughout the structure, usually to the next larger section, to satisfy the ductility requirements for Earthquake B. Very few changes in the girders were necessary.

Forces in the bridge slab diaphragm were found to be very high, due to the large stiffness of the slab, which was modeled as a deep horizontal beam bending about the vertical axis. Because it was not possible to design the slab for the high moments and shears (in the plane of the slab) computed, it was decided to transfer the shear to the bridge horizontal truss, which was strengthened substantially for this purpose.

COLUMN DESIGN BY THE COLDES COMPUTER PROGRAM

Introduction

The design of steel column sections in large multi-storey building frames can be an enormously time-consuming and expensive task when performed by hand computation, particularly when a large number of design

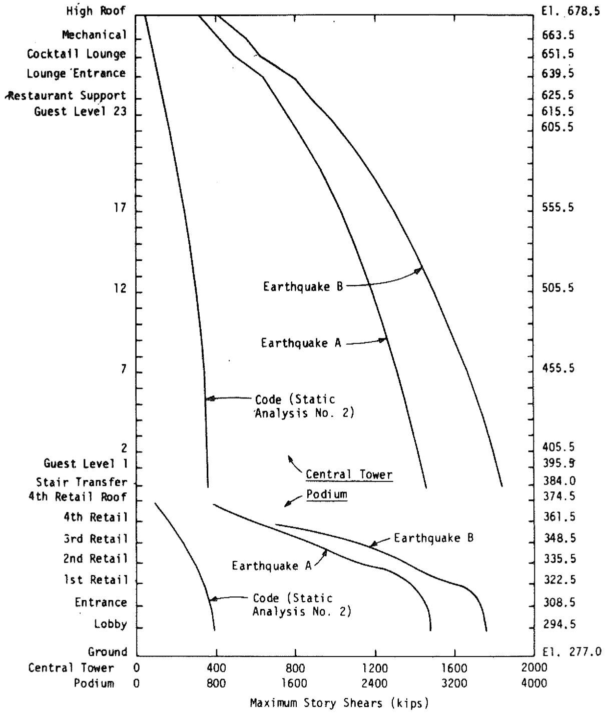


Fig. 9 Shears, Central Tower and Podium

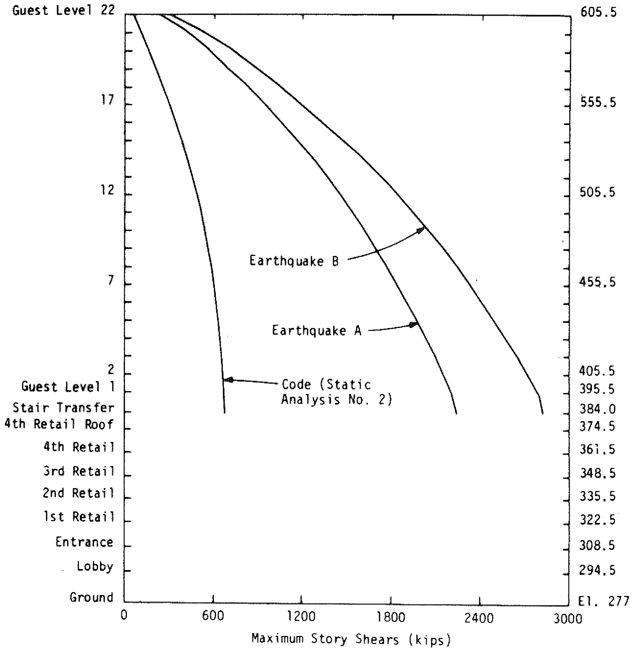


Fig. 10 Shears, Small Tower

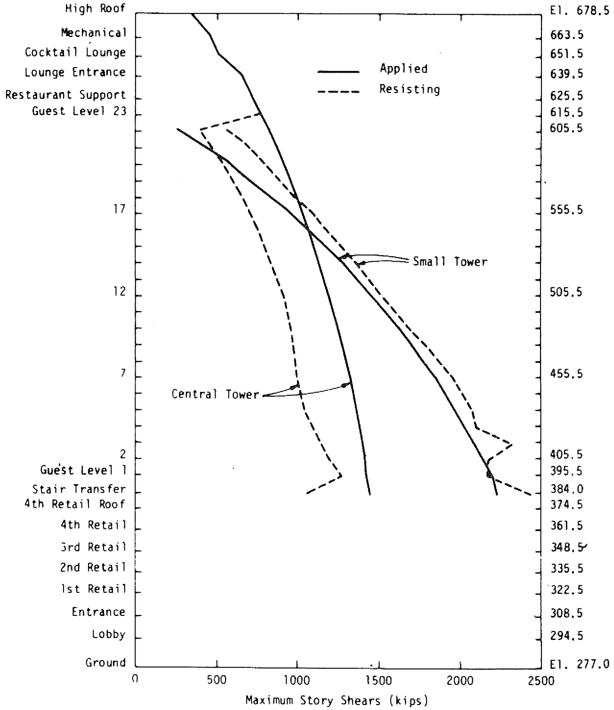


Fig. 11 Applied and Resisting Shears, Earthquake A

load combinations are involved. However, their design can be performed by applying fairly straightforward and highly repetitive design code procedures. Consequently, computer-assisted column design can reduce design costs substantially while at the same time increasing the productivity and effectiveness of the engineering designer. This section describes such a design program, developed during the Bonaventure Hotel design project$^{(10)}$.

The COLDES computer program is intended to design or check a full height of steel columns in strict accordance with a design code specification or recommended design procedure.

Summary of COLDES Capabilities

The COLDES program capabilities in the version used for this project may be briefly summarized as follows:

1. The user may request design or check of column sections and/or design of column splices, and/or design of column joint connection stiffener plates.
2. Design and checks are in accordance with the 1963 or 1970 AISC specifications or URS/Blume's earthquake design procedures.
3. Column forces may be input directly in card form or obtained automatically from computer files of SAP runs$^{(7)}$.
4. Design loading combinations are formed directly by COLDES according to user-specified combination factors. Up to 26 design load combinations can be formed from up to 15 independent loading conditions.
5. Column effective length factors may be specified by the user or automatically computed by COLDES. If computed, COLDES will take into account beams that are skewed (not parallel or orthogonal to a column principal axis) when framing into the column.
6. A facility is provided to account for live load reduction effects.
7. Beam and column section sizes are taken from internal beam and column section tables. These tables currently contain most of the 1970 AISC specification I-shaped members as well as a selected group of box sections for column design. The user may modify tables at any time.
8. COLDES automatically cycles through the column 'stack', first designing and then adjusting for splice points and column size decreases going up the column stack, if requested; recomputing effective length factors and then checking until the final column section design is consistent with the design forces, effective length factors, and other design parameters that are dependent on section size.
9. The user may request that all columns in a column height are to be rotated 90° so that the input major axis bending moments are to be resisted by the designed column's minor axis, and the minor axis bending moments are to be resisted by the designed column's major axis. This is a useful technique to investigate the effect of a principal axis direction change on the column line design. This, of course, assumes that the column forces are relatively insensitive to this axis rotation.
10. A strong column-weak beam constraint on the column design may be requested by the user in order to force plastic hinges to form in the beams rather than the columns.
11. Column splice design requires its own set of design load combinations so that the user may impose a higher safety factor against uplift and other critical loading combinations that tend to produce tensile stresses in the splice.
12. Column stiffener plate design consists of web doubler plates, flange continuity plates, and web stiffener plates at beam-column connections.
13. The user may specify a tolerance that is the amount by which a design interaction equation may be exceeded.

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TABLE 1

CODE SEISMIC BASE SHEAR

Item	Podium (use 1/4 Podium)	Central Tower (use 1/4 Central Tower + 4 x 1/2 Bridge)	Small Tower (use Small Tower + 1/2 Bridge)	Helistop (use 1/4 Helistop)
Number of Stories	7	29	23	1
Period (sec)	0.7	2.9	2.3	-
$C = \frac{.05}{\sqrt[3]{T}}$	0.056	0.035	0.038	0.2
Weight (kips)	14,053.25	10,309.75	18,081.0	38.0
Base Shear* (kips)	791.37	361.48	684.88	7.60
F_t (kips)	0.0	13.10	0.0	-
5% eccentricity (ft)	20.5	5.2	4.2	1.8

*V = ZKCW, Z and K assumed equal to 1.0

Note: 1 ft = 300 mm; 1 kip (force = 4.45 kN; 1000 lb mass = 450 kg.

TABLE 2

COMPARISON OF LATERAL DISPLACEMENT OF SELECTED NODAL POINTS FOR STATIC ANALYSIS 1 & 2

Load Case	Lateral Displacements (ft)					
	Central Tower Roof NP No. 1389		Small Tower Roof NP No. 1300		4th Retail Roof NP No. 418	
	Static Analysis No. 1	Static Analysis No. 2	Static Analysis No. 1	Static Analysis No. 2	Static Analysis No. 1	Static Analysis No. 2
Wind North	0.427	0.331	0.395	0.294	0.168	0.093
Wind Northwest	0.403	0.313	0.369	0.276	0.156	0.085
Seismic Northwest	1.877	1.429	1.574	1.125	0.600	0.319

Note: 1 ft = 300 mm

TABLE 3

NATURAL MODE PERIODS, X DIRECTION

Mode	Period (sec)	Mode	Period (sec)
1	4.48	11	0.41
2	1.89	12	0.38
3	1.40	13	0.32
4	1.21	14	0.29
5	0.84	15	0.27
6	0.78	16	0.25
7	0.57	17	0.18
8	0.55	18	0.17
9	0.47	19	0.15
10	0.47	20	0.13