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A REVIEW OF RECENT MATHEMATICAL MODELS MADE ON ACTUAL BUILDINGS AND THE ACCURACY OF THE PREDICTED PERIODS

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This report reviews recent analytical studies made on actual buildings for which both mathematical models have been developed and experimental measurements have been made to determine the fundamental periods of vibration. Three case studies are presented: a concrete shear wall building; a steel moment-resisting frame building; and a concrete moment-resisting frame building. These are followed by a brief review of analytical studies for which the measured period is obtained by ambient vibration tests, man-excited vibration tests, or forced vibration tests only. Each review gives a brief description of the building, the experimental tests performed, the mathematical model used, and a summary of results obtained. A review of studies is reported in which the periods computed by the mathematical model are compared to the periods actually experienced during earthquake excitation. Two tables summarize the results of the studies and list the percentage differences between the computed and observed periods for each building.

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PREFACE

The response of buildings to earthquake ground motion has been a subject of widespread concern, particularly to the engineer involved in the seismic design of structures. The current design practice has been based both on analytical studies in structural dynamics and on the actual behavior of structures during earthquakes. Mathematical models of buildings have been tested in an effort to correlate theoretical and experimental results. Ideally, the engineer would like to be able to fabricate a model which can describe a theoretical response which closely approximates what has been actually observed. Moreover, in the design phases, the engineer would like to be able to select a model which can reliably *predict* the dynamic response of the building to a given ground motion. However, at present there is no well defined modeling scheme which is known to give consistently accurate results. A number of detailed studies have been made of actual buildings, resulting in varied degrees of accuracy. The inaccuracies of the models arise from the fact that in order to represent the actual building in analytical terms, a large number of assumptions and approximations must be admitted; otherwise the problem of modeling the building would be too expensive to be resolved efficiently by current methods of analysis. Therefore, at present we wish to focus upon mathematical models which have been made for specific cases and consider the calculated periods of the building as an indicator of the model's accuracy. Possibly, by reviewing the set of all existing models, several "state-of-the-art" modeling schemes may be identified. It is expected that each of these selected models would involve various compromise decisions which facilitate computation. Comparison of the various "state-of-the-art" models may give an indication of the potential error implicit in a given modeling technique and suggest areas in which more work need to be done in order to devise more reliable models in the future.

This report summarizes some of the recent models made. The intent is to begin assembling a set of models upon which to base further work along the lines discussed above.

INTRODUCTION

This report reviews recent analytical studies made on actual buildings for which both mathematical models have been developed and experimental measurements have been made to determine the fundamental periods of vibration.

First, three case studies are presented: a concrete shear wall building, a steel moment-resisting frame building, and a concrete moment-resisting frame building.

The case studies are followed by a brief review of analytical studies for which the measured period was obtained by ambient vibration tests, man-excited vibration tests, or forced vibration tests only. Each review gives a brief description of the building, the experimental tests performed, the mathematical model used, and a summary of results obtained.

Next, there is a review of studies made where the periods computed by the mathematical model are compared to the periods actually experiences during earthquake excitation.

Finally, two tables are presented to summarize the results of the studies herein reviewed; they present the percentage differences between the computed periods and observed periods for each building.

CASE I. THE CERTIFIED LIFE BUILDING

The Certified Life Building (1) at 14724 Ventura Blvd., Los Angeles, is a 14-story office tower with an adjacent three-story parking structure (separated by a 3-inch seismic joint). The structure is of reinforced concrete shear wall construction with a typical plan of 76' x 156'. The foundation consists of Raymond step-tapered, cast-in-place piles with 12-inch diameter corrugated steel shells. The average length of the piles is 41 feet, providing a design load-carrying capacity of 90 tons per pile. One hundred eighty-five piles are vertical and the remaining 45 are battered at an inclination of 1 to 4. Continuous lateral ties are provided between pile caps by reinforced concrete tie beams.

The vertical load-carrying system consists of lightweight concrete flat slabs supported by columns and bearing walls (which also serve as shear walls). The two exterior spandrel frames in the E-W direction (from the second floor level to the roof and from the third floor to the roof) were designed only to carry vertical loads.

The lateral force resisting system in the tower consists primarily of 12-inch thick reinforced concrete shear walls with some lateral support coming from the exterior spandrel frames and flat slab-column frames. In the first three stories, part of the lateral resistance is provided by 12-inch thich, grouted concrete block walls. The N-S (transverse) shear walls are at the ends of the building and around the elevator core. The E-W (longitudinal) shear walls are entirely around the elevator core.

The moduli of elasticity used were as given in the construction specifications.

Three strong motion accelerographs (MO-2 35 mm film recorders) were mounted to the floor slab at the ground floor level, sixth floor level, and roof. During the 1971 San Fernando earthquake all three instruments recorded the building accelerations. From the time histories, the natural periods of vibration of the building were estimated as the average from the records.

A mathematical model was developed for each of the two principal building axes. Two-dimensional, lumped mass, distributed stiffness models were used. The horizontal translational stiffness of each frame and shear wall in the actual structure was combined to produce the total building stiffness. The masses were taken as the sum of the actual dead weights tributary to each floor level including the structural floor system, columns, partitions, exterior walls, precast spandrels, windows, mechanical equipment, electrical equipment, and code live loads. The base of each model was considered to be rigidly attached to an infinitely stiff ground. Any possible effects of soil-structure interaction in the horizontal directions were assumed to be included in the recorded accelerations at the ground level. Building rocking on the soil was not considered. In stiffness calculations, the gross moment of inertia for all members and gross cross-sectional area for columns and shear walls were used without consideration for the transformed area of steel. Beam stiffness calculations were based on clear spans, and column stiffness calculations were based on full story heights. All joints between beams and columns were considered to be rigid and moment-resisting. Floor slabs were assumed to act as rigid horizontal diaphragms (including the full plan area of the second floor which is actually a mezzanine). The interior N-S shear walls from the ground to second floor levels were treated as equivalent rigid frames. The main vertical shear walls in the N-S direction were represented as single columns possessing the shear and flexural characteristics of the gross concrete section of the wall.

For the core walls, flange action of perpendicular walls was considered in computing the gross moment of inertia; the shear area was based on the gross web area of the wall in the strong direction, neglecting flange areas. The heavily perforated E-W shear walls were modeled as frames with each doorway representing a bay and each shear wall section between doorways as a column.

Because the computer program used in the analysis considered all members in a frame to have the same modulus of elasticity, member sections were adjusted proportionately to account for the variations in the actual moduli. The analysis considered only linear-elastic, horizontal translational response.

The acceleration time-history records indicated that the building went through three distinct phases of response, with lengthening of the building period due to micro-cracking in the concrete shear walls.

In addition, pre-earthquake ambient vibration tests provided data on the natural periods of vibration.

Because the computed periods are shorter than those measured (even the ambient vibration periods), it is believed that soil-structure interaction may have been the main cause (in addition to the errors

COMPONENT	MODE	COMPUTED PERIOD	AMBIENT PERIOD	MEASURED PERIOD 0-5 SEC. OF EQ	MEASURED PERIOD 5-14 SEC. OF EQ	MEASURED PERIOD AFTER 14 SEC. OF EQ
Transverse (N-S)	1	0.682	0.81	-	-	1.08
	2	0.154	-	0.186	0.200	0.215
	3	0.073	-	0.076	0.093	0.097
Longitudinal (E-W)	1	0.784	0.88	-	-	1.127
	2	0.195	-	0.218	0.256	0.260
	3	0.096	-	0.100	-	-

introduced in making the assumptions), since this is a stiff shear wall building supported on soft soils.

CASE II. BUILDING 180 AT THE JET PROPULSION LAB

Building 180 at the Jet Propulsion Lab (2), Pasadena, California, is a 9-story steel frame structure (plus one basement) with a typical plan of 40' x 220'. The foundation consists of 7-foot wide reinforced concrete spread footings around the periphery of the building. Lateral forces in the transverse direction are resisted by the rigid steel frame action of the partly concrete-encased steel columns and the welded steel trussed floor girders. Lateral forces in the longitudinal direction are resisted by the action of the columns and trussed spandrel girders. Earth retaining walls in the basement and first story add appreciable stiffness to the frames in their lower sections. The 5-inch lightweight concrete floor slabs are supported on E-W directed 12WF27 steel beams and the N-5 trusses. The north and south faces of the building are of glass curtain wall construction. The east and west end walls consist of precast concrete panels supported by the steel frame.

The natural periods of the building were measured during the 1971 San Fernando earthquake as well as by other vibration tests. From the time histories of acceleration recorded during the earthquake, the Fourier amplitude spectra for the roof and basement were computed for the first 40 seconds of shaking. The ratios of roof spectra values to basement spectra values were plotted and a smoothing operation applied. The natural frequencies were obtained from the smoothed ratio of Fourier spectra.

In addition, Nielsen performed man-excited tests and forced vibration tests during the construction of the building in 1963, Teledyne performed ambient vibration tests on the completed building in 1971, and Nielsen and Teledyne performed man-excited tests in 1972.

Two mathematical models were developed for this building. For the "full composite model" the lateral stiffness matrix was derived assuming that the full area of concrete was acting on the columns and that a 15-foot wide section of floor slab was composite with the top chord of the trusses. For the "partial composite model", it was assumed that the concrete in the flexural tension zone of the columns provided no flexural stiffness and that a 7.5-foot wide section of the floor slab acted compositely with the truss top chords. Both models were assumed to be held against horizontal translation at the first floor level, and the columns were assumed to be fully fixed at the foundation pad level. The full composite model was additionally restrained from horizontal translation at the second floor level. (Values of moduli of elasticity were not presented.) Two-dimensional models were thus developed from the stiffness properties of the frame members and the calculated story weights. These refined models were obtained by adjusting the periods of the lower modes obtained from more basic models to agree with the predominant periods found from the earthquake accelerogram investigation.

There was good agreement between the computed periods of the structural models and the values measured by the ambient test after the earthquake. Also the periods computed by the partial composite model were within 10% of the observed earthquake values.

	PERIODS IN TRANSVERSE DIRECTION			PERIODS IN LONGITUDINAL DIRECTION		
	T_1	T_2	T_3	T_1	T_2	T_3
I. Computed Periods from:						
Full Composite Model	1.16	0.34	0.17	1.09	0.36	0.21
Partial Composite Model	1.46	0.47	0.26	1.21	0.42	0.26
II. Measured Periods from:						
Earthquake Records	1.44	0.44	0.24	1.09	0.36	0.21
During Construction - Man Excited (Nielsen, 1963)	0.88	0.29	--	0.91	0.29	--
Forced Vibration Test - Const. (Nielsen, 1963)	1.03	0.31	0.16	0.99	0.33	0.20
Ambient Tests (Teledyne, 1971)	1.11	0.35	0.16	1.05	0.33	0.20
Man-Excited Tests (Nielsen & Teledyne, 1972)	1.15	0.31	--	1.00	0.30	--

CASE III. THE SHERATON UNIVERSAL HOTEL

The Sheraton Universal Hotel (3) at 3838 Lankershim Blvd., No. Hollywood, California, is a 21-story reinforced concrete frame building with one basement. The typical floor plan is 183'-6" by 57'-10". At the first floor level, there is an adjacent one-story structure separated from the tower at the ground level by expansion joints.

Vertical loads are carried by columns and floor beams in both directions with two-way floor slabs between floor beams. Columns on the north and south sides are not prismatic, but taper from 20" x 18" at the top and bottom of each story to 20" x 15" at the mid-story height.

Lateral forces are resisted by column-girder frames in each direction (the building was designed as a ductile moment-resisting frame). The exterior north and south facades are 4-inch thich precast concrete panels connected to the spandrel beams by strap anchors and having a 3/8-inch gap all around. Slotted bolt holes at the top connections of the precast panels allow lateral movement of the panels in the frames. The exterior east and west facades consist entirely of windows between column lines. Most interior partitions consist of gypsum wallboard on metal studs, while some plaster partitions are used on the ground floor. Twelve-inch reinforced concrete shear walls extend on all four sides of the building in the basement. The foundation consists of reinforced concrete spread footings on soil consisting primarily of sandstones with deposits of salt and clay.

Accelerations during the 1971 San Fernando earthquake were recorded by seismometers located on the 21st floor, 11th floor, and basement level.

Acceleration seismograms were developed from the digitized time histories of acceleration at the different building levels. Estimates of the actual fundamental periods of the structure during the earthquake were made from time-history plots of acceleration at the roof level.

The mathematical model consisted of a cantilevered stem fixed at the base with masses lumped at floor levels. The masses were taken as the sum of the actual tributary dead weights, including estimated weights of furnishings, mechanical and electrical equipment, exterior walls, windows, and partitions. Lateral stiffness characteristics were computed for each element of the lateral load resisting system (apparently using the full section properties), with adjustments made in the moments of inertia and cross-sectional areas of members with different moduli of elasticity in order to express all stiffnesses in terms of only one modulus. The assumed modulus of elasticity for the concrete elements was determined from $E_c = W^{1.5}\ 33\sqrt{f'_c}$. The tapered exterior columns were modeled as prismatic members with a cross-section equal to the average of the top and mid-height sections. In the transverse direction alternate frames do not have girders in the center span where there are large openings for ducts and piping. Therefore these frames were modeled to have negligible bending stiffness for the horizontal members in the center span. The base of the cantilever was assumed fixed at the base and soil-structure interaction was assumed negligible. Floors were taken as rigid horizontal diaphragms. The building was assumed to be symmetrical with no eccentricity between the center of mass and center of rigidity.

COMPONENT	T_1	T_2	T_3	MEASURED PERIOD IN 1ST 6 SEC. OF EQ	MEASURED PERIOD AFTER 6 SEC. OF EQ
Transverse	2.215	0.790	0.484	1.4	2.1
Longitudinal	2.146	0.748	0.461	2.1	2.1

It is believed that the reason for the computed transverse period being longer than the measured period in the first 6 seconds of response to the earthquake is that the model did not account for the stiffness of the interior partitions located on the transverse column lines, which abut the columns at each end and the beam soffit at the top (these partitions consist of a double layer of gypsum board on each side). However, it is felt that after 6 seconds, enough energy had been generated to overcome the bond between the structural and non-structural elements.

BUILDINGS WHERE ONLY AMBIENT PERIODS HAVE BEEN MEASURED

Jerningham Apartments (4) in New Zealand. The building consists of 14 stories (16 floors) with a typical plan approximately 67' x 74'. The building is supported by reinforced concrete moment-resisting frames symmetrical about the N-S centerline but asymmetrical about all E-W axes. The two lowest stories have much less lateral flexibility than the rest of the framework in both the N-S and E-W directions. The frames on the east and west faces of the building are offset (instead of being contained in a single plane). In the initial design, almost all the lateral resistance was considered to be provided by the external spandrel-beam frames.

"Man-excited" vibration of the building was initiated at the roof level and the induced motions were detected and recorded by "conventional electronic apparatus" to obtain a reliable indication of the resonant frequency.

Because of the symmetry about the N-S centerline, the mathematical models neglected torsional effects for motion in the N-S direction; but because of the asymmetry about the E-W axes, torsional forces were expected to be induced for E-W disturbances, especially in the bottom of the structure where the shear wall in the E-W direction is located to the south of the building's center of mass. Since the structure is practically integral with the ground rock below the second floor level, it was modeled as being supported at this level on a rigid footing. One model assumed that the offset east and west face frames acted independently, while the second model assumed the two sections acted together. Preliminary analyses indicated that these frames should be treated compositely. Using a single bay approximation, the elastic stiffness properties of the elements were combined to form the building lateral stiffness matrix. Using the classical Stodola method, an iterative solution procedure provided the normal frequencies.

COMPONENT	COMPUTED PERIOD	MEASURED PERIOD
N-S	0.54	0.50
E-W n	0.49	0.50

Similar studies were made on three university science buildings: Zoology Building (6 stories), Chemistry Building (8 stories), and Physics Building (8 stories). All three buildings are reinforced concrete, combined moment-resisting frame and shear walls. However, since these buildings are relatively stiff buildings supported on flexible soils, the mathematical model provided for foundation movement (both translational and rotational flexibility). Also, instead of man-excited vibration tests, small amplitude forced vibration tests were used to determine the actual periods of vibration.

	COMPUTED PERIOD		MEASURED PERIOD	
BUILDING	N-S	E-W	N-S	E-W
ZOOLOGY BUILDING	.24	.33	.24	.33
CHEMISTRY BUILDING	.31	.39	.31	.37
PHYSICS BUILDING	.31	.38	.32	.38

Office building for the Department of National Health and Welfare (5), at Tunney's Pasture in Ottawa. The building consists of 19 stories (17 floors, ground level, and basement) with a typical plan 140' x 88'. Structural steel columns run the full height of the building; and in the center of the building is a reinforced concrete core housing the elevator shafts and stairways. Floors are 11-inch reinforced concrete slabs. Outside walls consist of heavy precast window sections (each weighing about two tons) bearing on the floor slab at each floor level.

The wind-induced vibration of the building was measured with seven Willmore Mark II seismometers on the 17th, 14th, 11th, 8th, 5th, 2nd, and ground levels. These measurements were fed onto tape recordings, and the Fourier transforms were computed, from which the natural periods of vibration were determined.

Two mathematical models were made concurrently with the experimental program. One model assumed a shear type behavior for the structural frame with equal masses lumped at the floor levels. The second model assumed

COMPONENT	MEASURED PERIODS T_1	T_2	T_3	COMPUTED (FRAME ONLY) T_1	T_2	T_3	COMPUTED (FRAME + CORE) T_1	T_2	T_3
Transverse	1.28	0.30	0.20	1.39	0.65	0.40	0.48	0.162	0.097
Longitudinal	0.99	0.26	0.19	1.40	0.60	0.36	0.47	0.161	0.097
Rotational	0.89	0.28	0.18						

that the reinforced concrete core would contribute to the stiffness of the structure by assuming the core acted as a shear structure with lateral displacements equal to those of the surrounding frame (i.e., beams were considered infinitely stiff). In both models, the window sections on each floor forming the outside walls were considered as contributing to the mass but not the stiffness. The lateral stiffness matrix was generated and eigen values for lateral motion about the two principal axes of the building were obtained by digital computation.

Discussion of discrepancies.

If the window sections on each floor forming the outside walls had been considered as contributing to the stiffness as well as to the mass, the building would have been stiffer in the longitudinal direction than in the transverse direction and the periods computed for frame action only would be closer to the measured values. The reason for such low values for the case of frame and core combined is that the stiffness of the core was very large compared to that of the frame. However, if flexure of the beams were allowed (instead of infinitely stiff), the core would not have performed integrally with the frame.

San Diego Gas and Electric Company Building (6) at 101 Ash Street in San Diego, California. The tower of this building consists of 21 floor levels plus a roof above grade and two levels below grade with a typical plan of 180' x 70'. Floor construction is cellular steel decking topped with 2 inches of concrete (5-inch reinforced concrete slabs are used in areas subject to heavier loads). Below the first floor, all steel framing is fireprofed with reinforced concrete; above the first floor, the frame is fireproofed by metal lath and plaster facing on the columns and a sprayed layer of Zonolite plaster elsewhere. Interior walls are 4-inch and 6-inch metal lath and plaster. Exterior walls are glass and lightweight metal panels. All four sides of the tower are faced with reinforced concrete fins (6" x 18" x 27') attached to the structural frame at each floor level. The tower is structurally independent of the adjacent 2-story U-shaped building and has a foundation consisting of 14- to 22-ft spread footings.

Two eccentric mass vibration generators (developed at C.I.T.) were installed on the 20th floor. Steady-state sinusoidal vibrations were induced. Motions were measured by Statham accelerometers, amplified by Miller C-3 carrier amplifiers, and recorded on a CEC recording oscillograph. A stationary synchro was added for the purpose of measuring the phase between the excitation and the response. Resonant frequencies were determined by applying the half-power method to the response curves of the 90° out-of-phase components. In this manner, the first six modes for E-W, N-S, and torsional components of the response were determined and compared with those frequencies obtained from ambient vibration tests: the two test techniques yielded "substantially the same natural frequencies."

The analytical study of this building was performed in parallel with the testing program. Basic parametric studies were made, first using a program for two-dimensional analysis, and then one for three-dimensional analysis. The mathematical model accounted for the structural frame, floor beams and slabs, fireproofing, and the masses and stiffnesses of non-structural elements (by an approximate technique).

	MEASURED PERIODS			COMPUTED PERIODS		
COMPONENT	T_1	T_2	T_3	T_1	T_2	T_3
N-S	2.62	0.91	0.50	2.76	0.93	0.53
E-W	2.54	0.83	0.44	2.60	0.86	0.46
Torsional	2.35	0.81	0.46	2.16	0.73	0.41

The office building for Bethlehem Steel Co., Pacific Coast Division in San Francisco is a 15-story steel moment-resisting frame structure. Floors are lightweight concrete fill on cellular steel decking. There is exterior marble facing over fireproofed columns and concrete-encased spandrel sections. The rigid frame joints are high-strength bolted connections. There is horizontal steel bracing at the second, third, and fourth floor levels to distribute resulting shear forces at those levels. The foundation consists of driven H-piles in old bay deposits and varied clay and sand strata.

Twenty vibration recordings of wind-induced motions were made during construction using portable seismographs and vibration meters in numerous locations in the building. At times, as high as the fourth natural mode of vibration of the building could be isolated.

The mathematical models for this building assumed shear-beam behavior, concentrating masses at the floor levels. Joint rotations of the columns and girders were included in the analysis, while axial deformations of the columns were neglected. It was decided that non-structural components (fireproofing and partitions) should be included in the calculations of stiffnesses (since it was found that neglecting

them resulted in as much as 39% variation from the measured periods, while including them reduced the variation to as low as 2.2%).
The periods were calculated twice: when the steel frame as complete and the concrete slab placing had begun, and when the building was complete except for part of the ceiling, partitions, glass, and similar items. When the period was calculated the second time, the masses were calculated from the total design building weights, and stiffness factors were calculated assuming no composite action of the various materials (e.g., composite action of steel decking welded to the steel girders).

	MEASURED PERIODS			COMPUTED PERIODS		
COMPONENT	T_1	T_2	T_3	T_1	T_2	T_3
Transverse	1.49	0.47	0.27	1.59	0.56	0.33
Longitudinal	1.32	0.42	0.23	1.36	0.50	0.31

Discussion of results.

It was felt that the computed periods were in "general concurrence" with the measured values; that non-structural items should be included in the period calculations; and that assuming shear-beam behavior was valid.

The Sir Alexander Campbell Building (8), Canadian Post Office Department at Ottawa has ten floors above ground level (with a penthouse and one basement). A typical floor plan is 266' x 74'. The frame and floor slabs are of reinforced concrete. Exterior walls are non-load-bearing 4-inch or 8-inch brick walls. Interior partitions are of light-weight metal stud construction or 3-inch block (not designed to provide lateral resistance). Columns are rectangular and have approximately the same section from the foundation to the fifth floor and are halved from the sixth floor to the roof. The foundation consists of 22-inch diameter piles driven 40 feet to solid limestone rock.

Wind-induced vibrations were recorded by six Willmore II seismometers located on floors 9, 8, 7, 6, 4, and 2. A Honeywell-Brown analyzer was used to perform the Fourier analyses of the records and an analog computer was used to determine the phase relation between the vibrations of the different floors.

For the mathematical model, it was assumed that masses were lumped at the floor levels and that all the floor masses were equal. Girders were assumed to be infinitely stiff (shear-beam behavior) and the building was on a rigid base.

COMPONENT	MEASURED PERIOD	COMPUTED PERIOD
Transverse	0.69	0.75
Longitudinal	0.59	0.90

The _Canadian Imperial Bank of Commerce Building_ (8) in Montreal has 44 stories above ground plus three basement floors. The typical floor plan is 140' x 100'. The building has a structural steel frame with bolted connections. Exterior columns are fireproofed in concrete. The floors consist of corrugated metal deck units supported by purlins and topped by reinforced concrete. The curtain walls are precast concrete faced with slate, and the interior has floating partitions.

The foundation consists of footings on bedrock 48 feet below street level.

Wind-induced vibrations were recorded by six Willmore II seismometers located on different floors of the building. A Honeywell-Brown analyzer was used to perform the Fourier analyses of the records and an analog computer was used to determine the phase relation between the vibrations of the different floors.

For the mathematical model, it was assumed that masses were lumped at the floor levels and that all the floor masses were equal. Girders were assumed infinitely stiff (shear-beam behavior) and the building was on a rigid base.

COMPONENT	MEASURED PERIOD	COMPUTED PERIOD
Transverse	4.65	3.27
Longitudinal	4.65	3.86

Discussion of discrepancies.

It is believed that the large discrepancy in computed and measured building periods indicates that shear-beam behavior is not a realistic assumption for this building.

The CIL House in Montreal (8) has 34 floors above ground level plus 4 basement floors with a typical floor plan of 168' x 112'. It has a structural steel frame with welded connections. Floor construction in the basement, ground level, mechanical floors, and roof are concrete slabs formed in place; elsewhere, floors consist of 3-inch steel deck-with-concrete fill. In the areas where there are concrete slabs, beams and columns are fireproofed with concrete; elsewhere, asbestos is used for fireproofing. The curtain wall consists of lightweight aluminum. Interior partitions are of lightweight aggregate slag. The building is founded on bedrock.

Wind-induced vibrations were recorded by six Willmore II seismometers at different floor levels of the building. A Honeywell-Brown analyzer was used to perform the Fourier analyses of the records and an analog computer was used to determine the phase relation between the vibrations of the different floors.

For the mathematical model, it was assumed that masses were lumped at the floor levels and that all floor masses were equal. Girders were assumed infinitely stiff (shear-beam behavior) and the building was on a rigid base.

COMPONENT	MEASURED PERIOD	COMPUTED PERIOD
Transverse	4.46	2.58
Longitudinal	3.93	3.01

It is believed that the large discrepancy indicates that shear-building behavior is not a realistic assumption.

BUILDINGS WHERE PERIODS HAVE BEEN MEASURED DURING RESPONSE TO EARTHQUAKE EXCITATION

The Muir Medical Center (9) at 7080 Hollywood Blvd., Hollywood, California, consists of an 11-story tower surrounded by single-story commercial facilities (separated structurally by a 2-inch seismic joint). There is also a single-story basement garage. The typical floor plan for the tower is 89' x 144'. The structural system consists of 9-inch flat slabs on columns and deep spandrel beams designed to work together as a moment-resisting frame. However, the perimeter basement walls are designed to serve as shear walls to resist seismic forces. The foundation consists of drilled-and-belled caissons and drilled, cast-in-place piles founded on a firm soil layer.

During the February 9, 1971 San Fernando earthquake, accelerations were recorded by AR-240 paper tape recorders (by Teledyne) located in the basement, on the sixth floor, and the roof. The natural periods of vibration of the building were estimated from the acceleration time histories from the sixth floor and roof. Also, past earthquake ambient vibration periods were measured.

Two 2-dimensional, lumped mass, distributed stiffness models were made (one for each principal building axis). The masses included the structural floor system, columns, partitions, exterior walls, windows, mechanical equipment, and electrical equipment. Altogether, 14 lumped masses were used in the model. The base was assumed rigidly attached to the ground, which was assumed infinitely stiff. Possible effects of soil-structure interaction were assumed to be included in the recorded basement accelerations. The influence of building rocking was assumed to be negligible. Stiffness properties were computed using full concrete sections without regard for the transformed area of steel reinforcement. Interior slab stiffnesses were computed to the face of the columns. All joints between beams and columns were considered to be rigid and moment-resisting. The computer program used to calculate the building periods was a modified version of FRMSTC (originally developed at the University of California, Berkeley). Concrete floor slabs were assumed to act as rigid horizontal diaphragms and the building was considered symmetric (i.e., the center of mass and center of rigidity coincide).

	INITIAL COMPUTED PERIODS				EARTHQUAKE PERIODS				POST-EARTHQUAKE AMBIENT PERIODS
COMPONENT	T_1	T_2	T_3	T_4	T_1	T_2	T_3	T_4	
Transverse	1.60	0.536	0.311	0.218	1.60	--	0.30	--	1.14
Longitudinal	1.49	0.493	0.283	0.197	1.43	0.425	--	0.189	1.02

The KB Valley Center (10) (Independence Bank Building) at 15910 Ventura Blvd., Los Angeles, California, is a 16-story office tower (plus one basement) structurally separated from the adjacent 4-story parking structure by a 2-inch seismic gap. The typical floor plan is 87' x 165'. The structure is a steel moment-resisting frame. Lightweight concrete slabs are used in composite construction with beams. The four frames which form the perimeter of the tower are designed to resist lateral loads, while all the remaining frames are designed for only vertical loads. All girder-to-column connections are fully welded. The building is founded on firm dense soil layers by means of driven Raymond step-tapered piles interconnected with reinforced concrete tie beams where necessary to provide lateral resistance.

Accelerations in the basement, on the ninth floor and on the roof were recorded by SMA-1 70 mm film recorders (manufactured by Teledyne). The average of periods taken from the accelerograph records at the ninth floor and roof were considered the actual periods of vibration of the building during the quake.

A 2-dimensional, lumped mass, distributed stiffness mathematical model was developed for each of the two principal building axes. Masses consisted of the structural floor system, columns, partitions, exterior walls, windows, mechanical equipment, and electrical equipment tributary to each floor level. The base of each model was assumed rigidly attached to the ground which was assumed to be infinitely stiff. Any possible effects of soil-structure interaction were assumed to be included in the recorded basement accelerations. Effects of the building rocking on the soil were assumed negligible. In calculating the stiffness properties, the moment of inertia of the girders was based on the full steel section without consideration for the transformed area of the concrete slab which acts compositely. Similarly, moments of inertia and axial area of those columns partially encased in concrete were based on full steel section, neglecting any composite action with the concrete. Girder stiffnesses were calculated to the centerline of the columns, and column stiffnesses were established between clear story heights. All joints between girders and columns were modeled as fully rigid and moment-resisting. The concrete floor slabs were assumed to act as rigid horizontal diaphragms with no eccentricity of stiffness center to mass center. A modified version of FRMSTC (originally developed at the University of California, Berkeley) was used to compute the undamped natural periods of vibration of the building.

COMPONENT	COMPUTED PERIODS T_1	T_2	T_3	T_4	EARTHQUAKE PERIODS T_1	T_2	T_3	T_4	POST-EARTHQUAKE AMBIENT PERIOD
Transverse	3.37	1.29	0.801	0.567	3.00	1.11	0.695	--	2.27
Longitudinal	3.43	1.33	0.816	0.576	3.20	1.20	0.75	0.50	2.37

The Holiday Inn (11) at 1640 Marengo St., Los Angeles, California, is a 7-story reinforced concrete structure with no basements. The typical floor plan is approximately 63' x 160'. Floors are reinforced concrete flat slabs with spandrel beams around each floor perimeter. Lateral forces are resisted by the columns and slabs in the interior of the building and by the columns and spandrel beams around the perimeter of the building on the exterior. The exterior end walls consist of plaster on metal studs. At the west end of the north face, the structure has four bays of brick masonry walls between the ground level and the second floor. However, these brick walls are separated from the exterior columns by 1-inch expansion joints and from the underside of the second floor spandrels by half-inch expansion joints, and are not expected to have a significant effect on the building response. Interior partitions consist of gypsum wallboards on metal studs attached to both the floor and ceiling slabs. The building is supported by concrete piles in mostly silt and silty sand.

Accelerations during the 1971 San Fernando earthquake were recorded on the first floor, fourth floor, and roof. Time history plots of recorded transverse and longitudinal absolute accelerations were obtained from the digitized time histories. Acceleration plots for the fourth floor and roof were reviewed for periodicity and estimates were made of the fundamental period during the earthquake.

For the mathematical model, masses were lumped at floor levels. Masses were calculated as the sum of the actual tributary dead weights (including estimated weights of furnishings, mechanical and electrical equipment, exterior walls, windows, and partitions). The model consisted of a cantilevered stem fixed at the base. Effects of soil-structure interaction were assumed negligible. Member stiffness properties of each slab, girder, and column were determined by accounting for the effects of reinforcement and taking clear spans as effective member lengths. All joints were considered to be rigid and moment-resisting. Floor slabs were assumed to act as rigid horizontal diaphragms. In order to account for varying moduli of elasticity, member sections were adjusted proportionately to express them all in terms of a single modulus of elasticity. The building was assumed symmetrical with no eccentricity between the center of mass and center of rigidity. Thus a linear elastic 2-dimensional analysis was made using the FRMSTC-4 computer program to determine the periods of vibration.

COMPONENT	COMPUTED PERIOD	MEASURED PERIOD IN 1ST 5 SECS OF EQ	MEASURED PERIOD FOR ENTIRE EQ
Transverse	0.88	0.63	1.15
Longitudinal	0.79	0.60	1.10

The computed period was longer than the period measured in the first 5 seconds of earthquake ground motion. It is believed this difference was due to the fact that the mathematical model didn't account for the stiffness contributed by the partitions. The computed period was shorter than the period measured for the duration of the earthquake. It is believed this difference is due to the fact that yielding of the girders resulted in non-linear behavior with the columns remaining largely elastic.

The Holiday Inn (12) at 8244 Orion Avenue, Van Nuys, California, is the sister building to the Holiday Inn at Marengo. It is a 7-story reinforced concrete structure with no basement. The typical floor plan is approximately 63' x 160'. Floors are reinforced concrete flat slabs with spandrel beams around the perimeter of the floors. Lateral forces are resisted by the frame action of columns and slabs on the interior and by the columns and spandrels on the exterior frames. The exterior end walls consist of cement plaster on metal studs. At the west end of the south (longitudinal) face, the building has cement plaster extending from the ground to the roof; cement plaster extends only to the second floor level along the remaining south face area. Interior partitions are gypsum wallboard on metal studs. The foundation consists of concrete piles on silt and silty-sand with lesser deposits of sand and clay.

The accelerations caused by the 1971 San Fernando Earthquake were recorded by seismometers located at the roof, fourth, and ground floor levels. Time history plots of recorded transverse and longitudinal accelerations were obtained from digitized time histories. Using the plots of accelerations at the roof level, estimates were made of the actual fundamental period of the structure during its response to the earthquake.

The mathematical model assumed cantilevered masses lumped at each floor level, where the masses were obtained by summing the tributary dead weights (including mechanical equipment and partitions). Slabs were assumed to act as rigid horizontal diaphragms. Member section properties were adjusted to account for varying moduli of elasticity. The building was assumed to be symmetrical with no eccentricity between the center of mass and center of rigidity. Non-structural element stiffnesses could not be incorporated into the model directly, so approximations and engineering judgment were used when inputting the idealized structural elements. Soil-structure effects were assumed negligible. A computer analysis was made using the FRMSTC-4 program to determine the natural periods.

	COMPUTED PERIODS			MEASURED PERIOD IN	MEASURED PERIOD FOR
COMPONENT	T_1	T_2	T_3	1ST 7 SECONDS OF EQ	ENTIRE EQ
Transverse	0.88	0.288	0.164	0.7	1.6
Longitudinal	0.791	0.266	0.156	0.7	1.5

The computed period was for the bare structural frame. If the interior and exterior partitions and walls were included in the model, the computed period would be reduced (toward the measured 0.7 second period). The fact that the measured periods for the entire duration of the earthquake are longer than the computed periods can be explained by attributing the loss of stiffness to the yielding of some members.

The Bank of California (13), at 15250 Ventura Blvd., San Fernando, California is a 12-story reinforced concrete structure (no basement) with a typical floor plan 60' x 161'. A typical floor is a 4 1/2-inch slab on pan joints which span to girders. Lateral forces in each direction are resisted by reinforced concrete frames consisting of columns and girders: in the transverse direction, exterior frames run from the ground to the roof, while the interior frames run from the ground to the third floor; in the longitudinal direction, exterior frames run from the ground to the roof, while the interior frame is designed to carry only vertical loads. At the second floor level of the west longitudinal frame, the spandrel is set back about two feet from its typical location at other floors. Two 11'6" x 8" thick concrete shear walls on this frame extend from the ground to the third floor. Also a 10-inch thick shear wall runs from the ground level to the second floor approximately 30 feet beyond the east face of the tower. Interior partitions are gypsum board on metal studs. Exterior walls consist of 6'6" glass windows resting on 2'6" high metal stud walls. The foundation consists of concrete piles on moderately firm to firm soils (silt and silty sand).

Accelerations during the 1971 San Fernando earthquake were recorded by seismometers located on the roof, seventh, and ground floor levels. The natural periods of vibration of the building were estimated from the time history plots of acceleration (obtained from digitized time histories).

Two mathematical models were made assuming cantilevered masses lumped at floor levels, where the masses were obtained by summing the tributary dead weights (including mechanical equipment and partitions). Slabs were assumed to act as rigid horizontal diaphragms. Member section properties were adjusted to account for varying moduli of elasticity. The building was assumed to be symmetrical with no eccentricity between

between the center of mass and center of rigidity. Soil-structure effects were assumed negligible. One model (Model A) was made considering all frames as capable of carrying lateral loads (by virtue of their concrete dimensions). A second model (Model B) considered only those frames specifically reinforced for lateral loads as capable of carrying lateral loads. The models were analyzed using the FRMSTC-4 computer program to determine the natural periods of vibration.

	COMPUTED PERIOD		MEASURED PERIOD FROM	MEASURED PERIOD FROM
COMPONENT	MODEL A	MODEL B	5-15 SEC. OF EQ	15-28 SEC. OF EQ
Transverse	1.335	1.557	2.0	2.5
Longitudinal	0.851	0.926	2.5	1.8

It is believed that the measured periods are longer than the computed periods due to the yielding of structural and non-structural elements.

The <u>Bunker Hill Tower</u> (14) at 611 West Sixth St., Los Angeles, California, is a 32-story steel frame structure (no basement) with a typical floor plan 90' x 125'. The floor system consists of 5-inch lightweight concrete slabs on steel beams and girders. Lateral forces are resisted by the perimeter girders and columns. The interior girders and columns are designed for vertical loads only. The exterior walls are glass window walls; interior partitions are gypsum board on metal studs. Structurally-separated concrete block walls are used at the lower floors below the plaza level. Beams and girders are fireproofed with gypsum board. The building is supported on concrete spread footings and grade beams on caisson foundations on primarily firm shale and sandstone.

Accelerations during the 1971 San Fernando earthquake were recorded by seismometers located on the roof, 16th, and ground floor levels. The natural periods of vibration of the building were estimated from the time history plots of acceleration at the roof level.

The mathematical model consisted of cantilevered masses lumped at each floor level, with the masses also accounting for the weights of mechanical equipment and partitions. The lateral stiffness characteristics were computed for each element of the lateral load resisting system. Twice the moment of inertia of the steel girders was used to approximate the composite action with the concrete slab. Floors were taken as rigid horizontal diaphragms. The building was assumed to be symmetrical with no eccentricity between the center of mass and the center of rigidity. The base of the cantilever was assumed fixed, and the effects of soil-structure interaction were assumed negligible.

	COMPUTED PERIODS			MEASURED PERIOD AFTER 20 SEC.
COMPONENT	T_1	T_2	T_3	OF EQ SHAKING
Transverse	3.979	1.440	0.833	4.0
Longitudinal	3.508	1.274	0.749	3.4

TABLE 1: Buildings with Periods Determined from Ambient Vibration Tests

BUILDING NAME	STORIES	BASE-MENTS	FRAME	SOURCE OF VIBRATION	DATE OF STUDY	COMPUTED PERIODS	MEASURED PERIODS	PERCENT DIFFERENCE
Jerningham Apartments	14	2	CMRF	MAN-EXC	1965	0.54	0.50	8.0
						0.49	0.50	2.0
Zoology Building	6	-	CMRF/CSW	FORCED	1965	0.24	0.24	0
						0.33	0.33	0
Chemistry Building	8	-	CMRF/CSW	FORCED	1965	0.31	0.31	0
						0.39	0.37	5.4
Physics Building	8	-	CMRF/CSW	FORCED	1965	0.31	0.32	3.1
						0.38	0.38	0
Dept. of Nat'l Health & Welfare	18	1	SMRF	WIND	1964	1.39 & 0.48	1.28	8.6 & 62.5
						1.40 & 0.47	0.99	41.4 & 52.5
San Diego Gas & Electric Co. Bldg.	21	2	SMRF	FORCED	1971	2.76	2.62	5.4
						2.60	2.54	2.4
						2.16	2.35	8.1
Bethlehem Steel Co. Bldg.	15	1	SMRF	WIND	1960	1.59	1.49	6.7
						1.36	1.32	3.0

Table 1 Continued

BUILDING NAME	STORIES	BASE- MENTS	FRAME	SOURCE OF VIBRATION	DATE OF STUDY	COMPUTED PERIODS	MEASURED PERIODS	PERCENT DIFFERENCE
Alexander Campbell Bldg.	10	1	CMRF	WIND	1966	0.75	0.69	8.7
						0.90	0.59	52.5
Canadian Imperial Bank of	44	3	SMRF	WIND	1966	3.27	4.65	29.7
Commerce Building						3.86	4.65	17.0
The CIL House	34	4	SMRF	WIND	1966	2.58	4.46	42.1
						3.01	3.93	23.4
Muir Medical Center (Post-EQ)	11	1	CMRF	WIND	1971	1.60	1.14	40.3
						1.49	1.02	46.1
KB Valley Center (Post-EQ)	16	1	SMRF	WIND	1971	3.37	2.27	48.5
						3.43	2.37	44.7
Certified Life Bldg. (Pre-EQ)	14	-	CSW	WIND	1972	0.68	0.81	16.0
						0.78	0.88	11.4
Bldg. 180 at JPL	9	1	SMRF	MAN-EXC	1972	(SEE CASE STUDY)		0
				FORCED				to
				WIND				3.15

TABLE 2: Buildings with Periods Determined from Earthquake Records

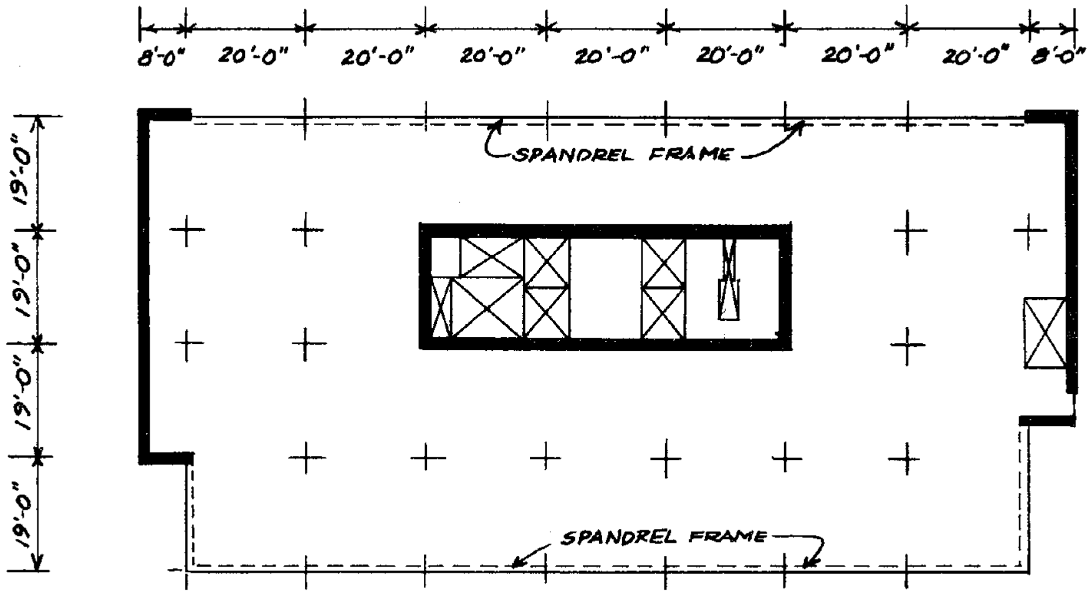
BUILDING NAME	STORIES	BASE-MENTS	FRAME	DATE OF STUDY	COMPUTED PERIODS	MEASURED PERIODS	PERCENT DIFFERENCE
Certified Life Building*	14	-	CSW	1972	0.68	1.08	37.0
Bldg. 180 at JPL (Partial Composite Model)*	9	1	SMRF	1972	1.46	1.44	1.4
					1.21	1.09	11.0
Sheraton Universal Hotel	21	1	CMRF	1971	2.215	1.4 - 2.1	58.2 - 5.5
					2.146	2.1	2.2
Muir Medical Center*	11	1	CMRF	1971	1.60	1.60	0
					1.49	1.43	4.2
The KB Valley Center*	16	1	SMRF	1971	3.37	3.00	12.3
					3.43	3.20	7.2
Holiday Inn (Marengo)	7	-	CMRF	1971	0.88	0.63 - 1.15	39.8 - 23.5
					0.79	0.60 - 1.10	31.7 - 28.2
Holiday Inn (Orion)	7	-	CMRF	1971	0.88	0.7 - 1.6	25.7 - 45.0
					0.79	0.7 - 1.5	12.9 - 47.3
The Bank of California (Model B)	12	-	CMRF	1971	1.557	2.0 - 2.5	22.2 - 39.7
					0.926	1.8 - 2.5	48.5 - 63.0

Table 2 Continued

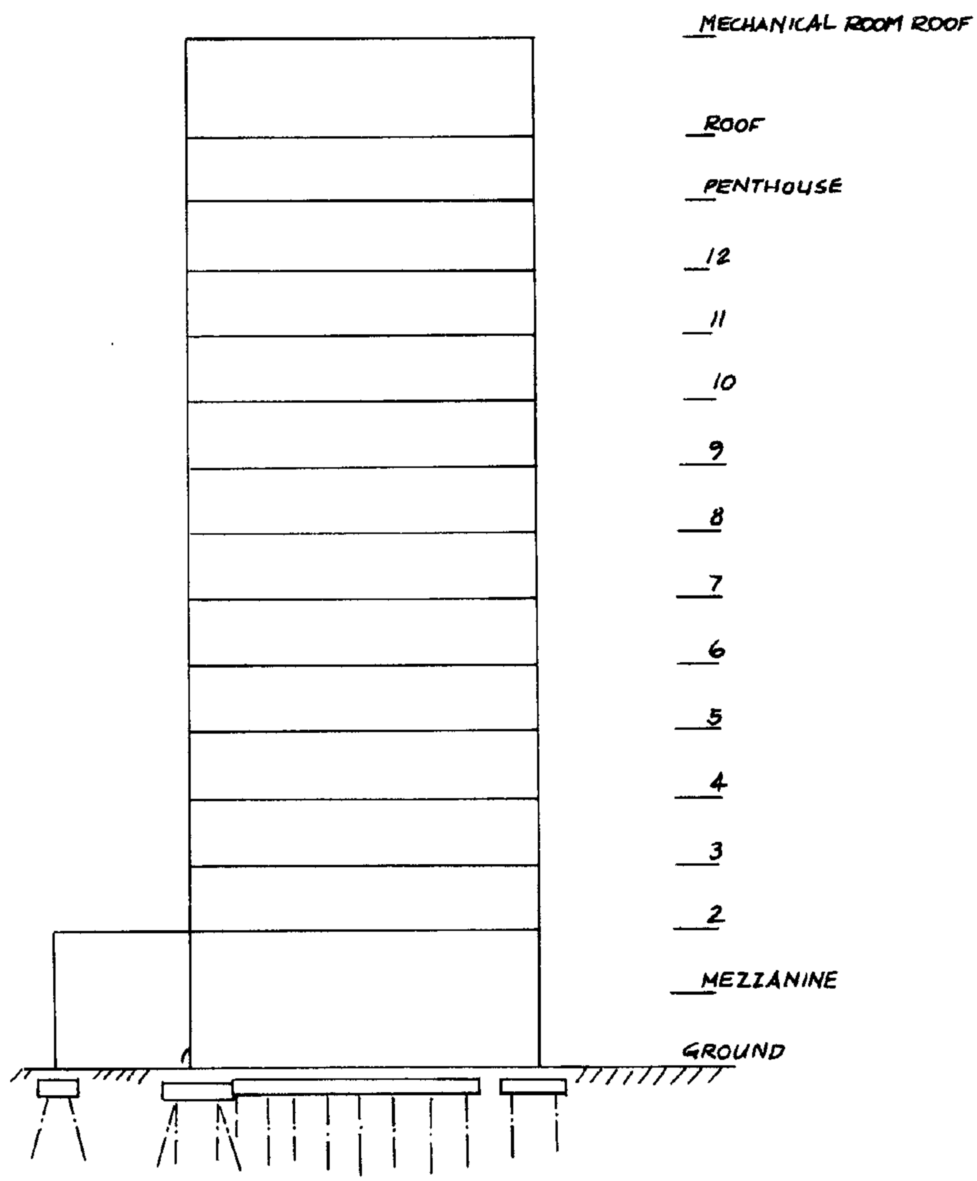
BUILDING NAME	STORIES	BASE-MENTS	FRAME	DATE OF STUDY	COMPUTED PERIODS	MEASURED PERIODS	PERCENT DIFFERENCE
Bunker Hill Tower	32	-	SMRF	1971	3.979	4.0	0.5
					3.508	3.4	3.2

* See Table I also.

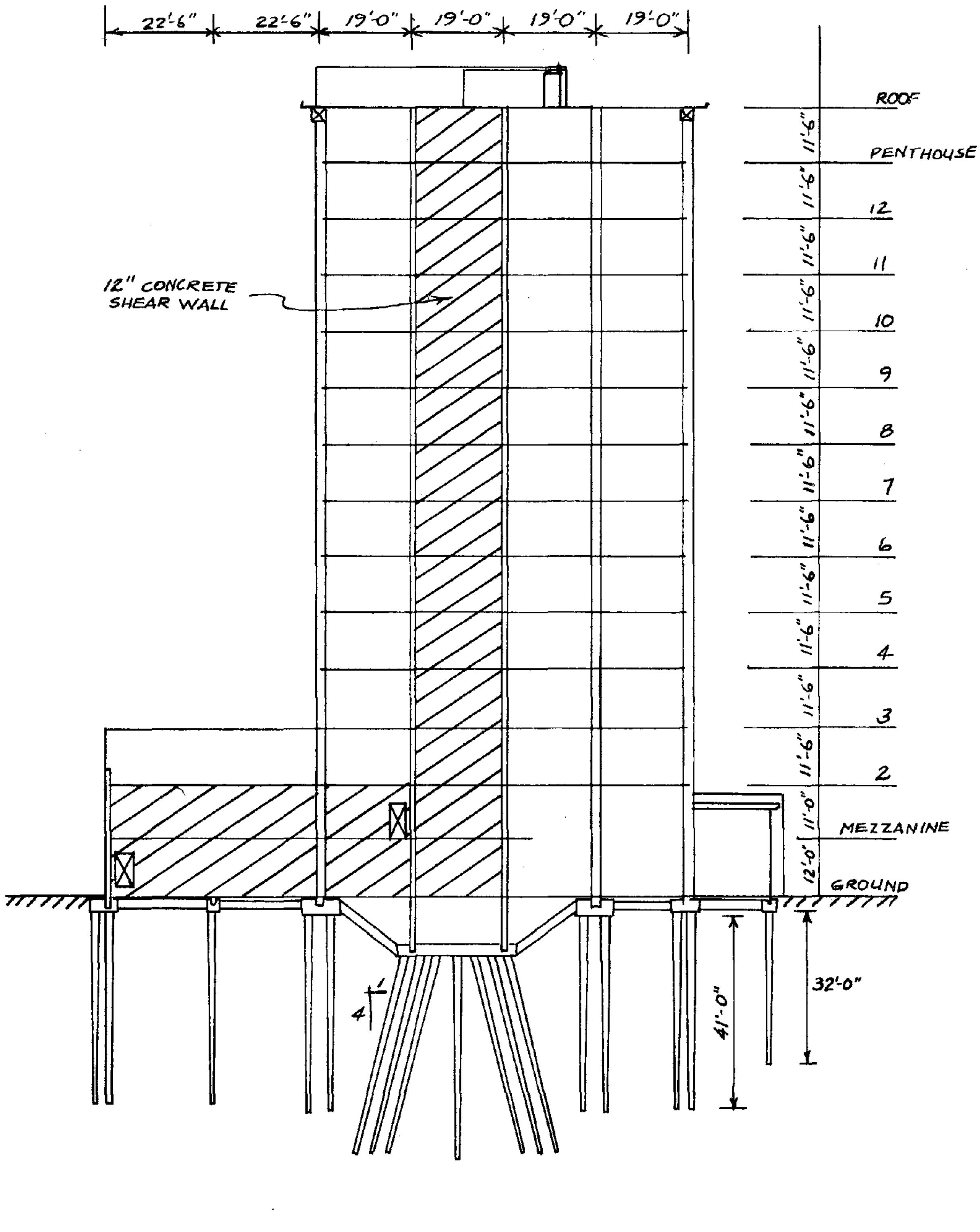
CERTIFIED LIFE BUILDING



TYPICAL FLOOR PLAN

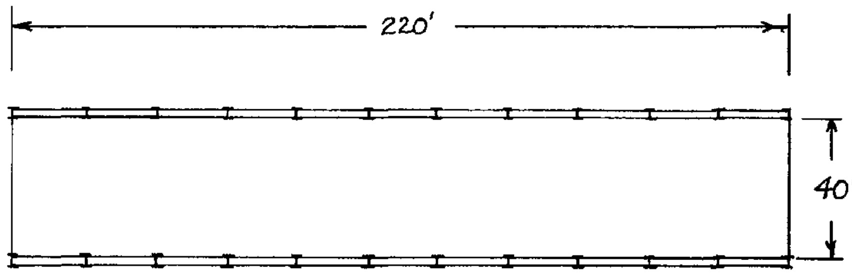


EAST-WEST ELEVATION



NORTH-SOUTH ELEVATION

BLDG. 180 AT JPL



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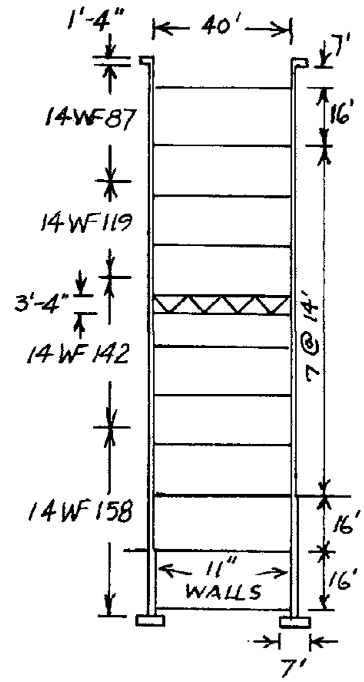
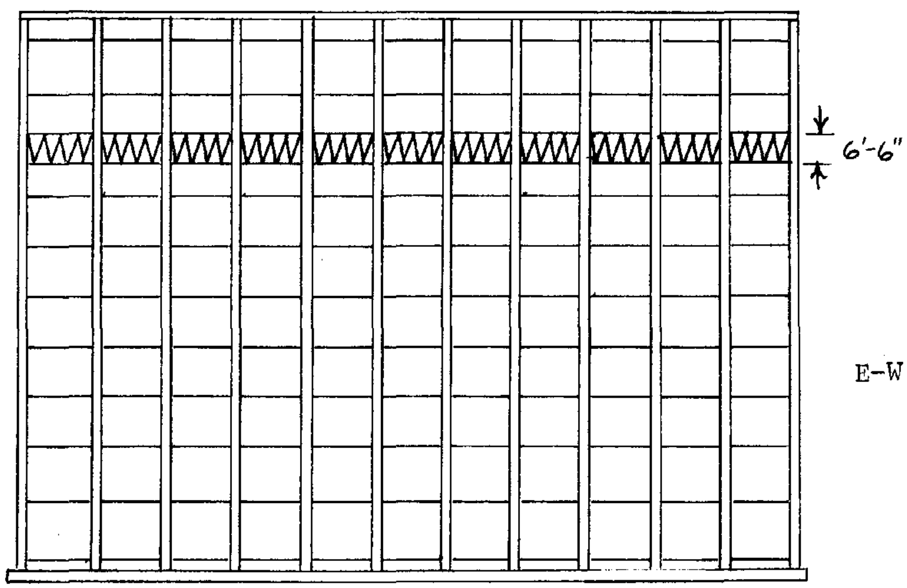
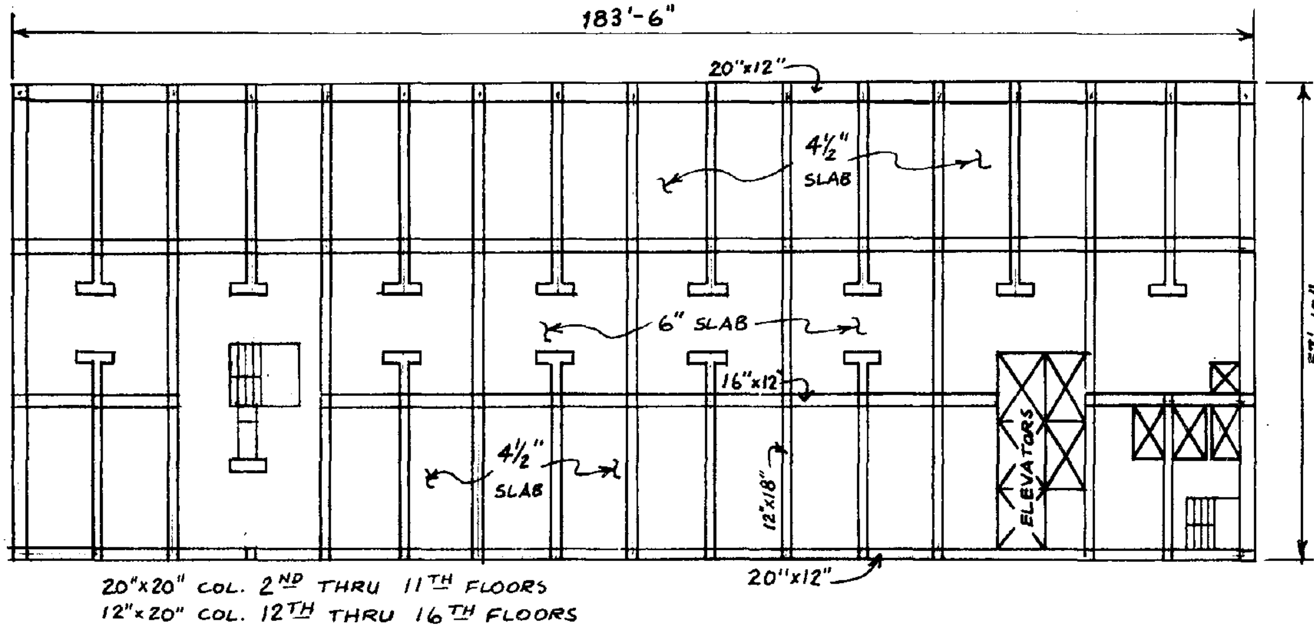


E-W ELEVATION



N-S ELEVATION



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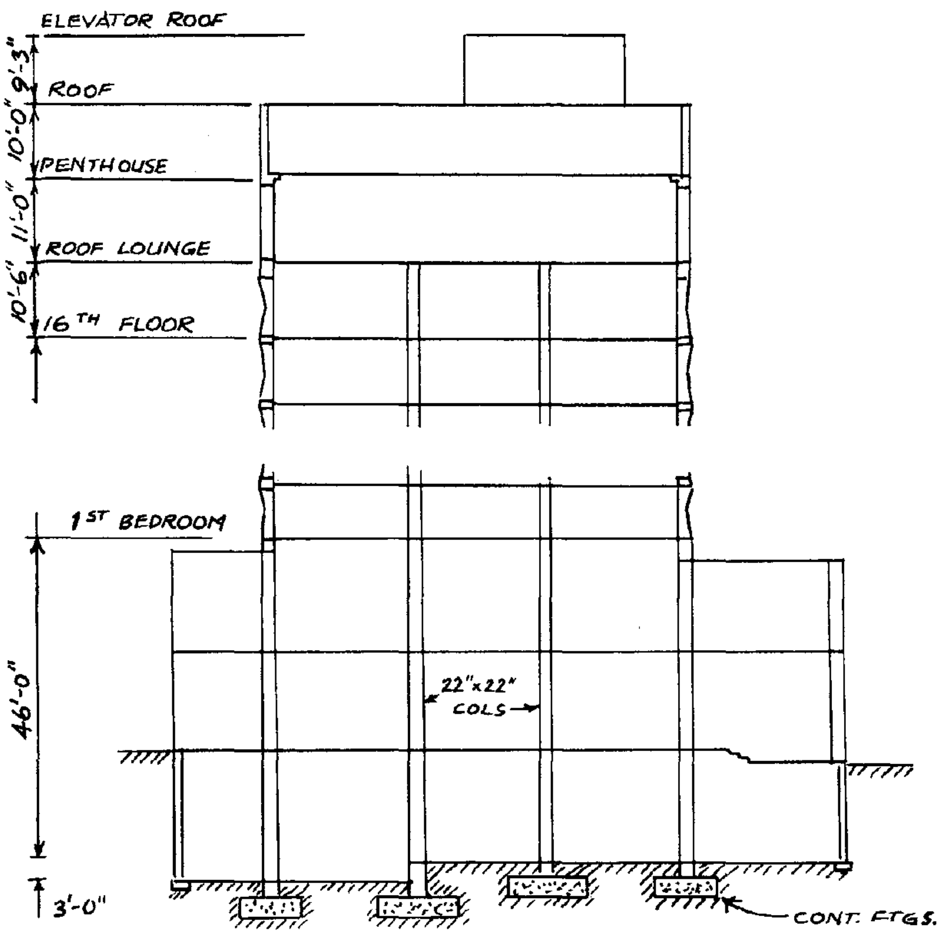

TYPICAL TRANSVERSE SECTION

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