THREE DIMENSIONAL ELASTIC DYNAMIC ANALYSIS OF A 985 FOOT HIGH FREESTANDING COMMUNICATIONS TOWER

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SYNOPSIS

This paper reviews the design development of a 985' high free-standing communications tower recently constructed in San Francisco. Included is a description of the structural configuration of the tower and the criteria by which it was designed. The dynamic characteristics of a three-dimensional linear elastic mathematical computer model, devised to represent the physical structure, are presented. The dynamic response of this computer model to various levels of ground shaking, including both horizontal and vertical excitations, are summarized, evaluated and compared to the seismic force levels prescribed by the 1969 edition of the San Francisco Building Code. Also included in the comparison are the responses derived for the tower from wind tunnel studies and static wind design criteria.

1. INTRODUCTION

Historically, conceptual planning of the 985' high Mount Sutro Tower dates back to the mid-fifties. Over the intervening years, several tower configurations and site locations have been proposed. In 1968, the San Francisco Planning Commission approved the present location of the tower on Mt. Sutro. At an elevation of 825' above mean sea level, the tower site is located on the San Francisco Peninsula, some 2½ miles southwest of the San Francisco Civic Center. The contours of the site were such as to preclude the possibility of utilizing a guyed tower configuration. As a result, a free-standing tower design concept evolved. The geometric form of the triangular tower, plate 1.1, evolved as a result of aesthetic and functional considerations Initially, a stress-skin structural system was devised to support lateral loads as a 100% moment-resisting space frame. Subsequent design development replaced the stress-skin solution with a braced frame system in which pretensioned cables carry lateral loads to the foundation. The original design of the antenna structures, mounted on top of the main tower, plate 1.1, was governed by electrical constraints and wind considerations. Later development indicated that a concern for the stability of the then proposed antenna array during periods of strong ground shaking, was well founded. This concern prompted a revision of the original seismic design criteria and ultimately resulted in the adoption of a tri-level guying support system for each of the 3 antennas structures. Plate 1.1 reflects an artist's conception of the final configuration of the tower. Plate 1.2 depicts the tower nearing completion. Unless noted otherwise, all further references to the tower/antenna structures pertain to the final "as built" configurations.

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2. STRUCTURAL CONFIGURATION AND FOUNDATION

2.1 Tower Structure

Triangular in plan, the 770' high steel tower supports three 215' high antenna structures, designated as stacks "A", "B" and "C", mounted on top of the tower, plates 1.1 and 1.2. The tower comprises three principal columns or legs located at the apex of an equilateral triangle. Column spacing varies linearly from 150' o.c. at the tower base to 60' o.c. at the "waist" to 100' o.c. at the antenna base level. Horizontal trusses spanning between tower legs are located at elevations of 180', 375', 550', 650' and 770' above the tower base. 2"-3" dia. twin bridge strand cables, each pretensioned to a value of 25% of ultimate strength, provide necessary lateral and torsional bracing in each of the 15 panels (5 per face) formed by the intersection of truss and leg assemblies. Each leg assembly, Figure 2.1.1 comprises 3 WF sections, latticed together with single and double angle sections, to form a triangular leg having a 7'0" face dimension. All intermediate level trusses, Figure 2.1.2, comprise 3 WF sections again braced with angle sections to form a triangular latticed girder. The depth of the outer face of the trusses is typically 15'. The girder width is constant at 6'0". The upper level horizontal truss assembly, Figure 2.1.3, consists of a pair of vertical trusses, braced across at the upper and lower chord levels to form a 15'0" deep x 6'0" wide box girder. These box girders cantilever out 50' from each tower leg to provide support for the antenna guy system. Typically all steel sections are specified as A36, except A572, grades 50, 55, and 60, are utilized selectively in certain leg and truss sections. All connections specify the use of A490 high strength bolts. Nonstructural steel cladding encloses all leg and truss assemblies except for the upper two levels of truss girders. The three tower legs are each supported vertically by 14' x 14' reinforced concrete piers extending 14' down to the top of 53' x 53' x 10' thick conventional spread footings. Grade beams, 4' wide, 6' deep, interconnect the tower footings.

2.2 Antenna "A" Structure

A conventional superturnstile "bat-wing" antenna operating in the channel 2, 4/5 frequency range, stack "A" is a 215' high step-tapered circular tube column rigidly connected to the tower leg below. Outside diameters of the antenna sections vary from 28" at the base to 5" at the beacon level. Yield strengths typically were specified at 35/36 ksi except 50 ksi sections were required in the uppermost segments of the antenna. A tri-level fiberglass guy system provides deflection control and lateral support under the design conditions. Guying elevations are at 75.0, 127.0 and 183.0 above the antenna base level. At each guy level, four guy assemblies each comprising 4-5/8" diameter fiberglass rods pretensioned to approximately 25% of ultimate capacity, extend down to anchor points symmetrically located at the center of the upper truss girders and at the ends of the cantilever outriggers.

2.3 Antenna "B" Structure

The stack "B" configuration includes a channel 7, 10"/14" diameter travelling wave antenna, 76' high, located above a 34' high radomed channel 44, 20" polygon antenna which is in turn mounted on top of a 38' high polygon support. The lower portion of the stack configuration comprises a 63' high triangular trussed tower, having a 7' face dimension. A441, A242 and A36 grades of structural steel were specified for the travelling wave, polygon and trussed tower segments of the antenna respectively. A tri-level fiberglass guy arrangement similar to the stack "A" system provides deflection control and lateral support under the design conditions. Guying elevations are at 101.0, 135.0 and 179.0 above the antenna base. The lower level of guy assemblies comprise only 2-5/8" diameter fiberglass rods, while the middle and upper level guy assemblies consist of 4-5/8" diameter fiberglass rods.

2.4 Antenna "C" Structure

The antenna arrangement and structural configuration are very similar to those of stack "B". The uppermost antenna segment is a channel 9, 14" diameter travelling wave antenna, 70' in height. Immediately below, a channel 32 radomed 22" polygon, 37' high, is mounted on top of 28" polygon designed for UHF transmission in the channel 26 frequency range. Similar to stack "B", a 63' high triangular trussed tower, rigidly connected at its base to the top of the main tower, provides vertical support to the antenna "C" configuration. A441, A242, and A36 grades of steel are used respectively for the channel 9, channel 32, channel 26 and trussed tower segments of the antenna structure. Deflection control and lateral support, under design conditions, are effected by means of a tri-level guy system similar to that described for stack "B".

2.5 Foundation

The tower leg footings are founded on bedrock. The bedrock consists of sedimentary cherts, shales and sandstones, and altered igneous basalt of the Franciscan formation. The bedrock bearing capacity is well in excess of the 3,500 psf maximum bearing pressure anticipated during the design conditions. Ground water was encountered at a depth of 40' below bottom of footings during the soil investigation.

3. DESIGN LOADS

3.1 Live Loads

Catwalks, 50 psf
Unspecified equipment rack loads 100 psf

3.2 Special Loads

Stack "A"	54^T	
Stack "B"	34^T	
Stack "C"	37^T	
Standby antennas	4^T	each
FM antennas	0.5^T	each
Microwave antennas	0.2^T	each

3.3 Wind Loads

As specified by the Electronic Industry Association Code RS222A, dated November 1966 except as noted below;

Flat surfaces	50 psf
Round surfaces	33 psf

or those induced during the expected 100 year extreme winds and drag forces, Plates 3.3.1 and 3.3.2, by direction, whichever is greater.

3.4 Seismic Loads

As specified by the 1969 edition of San Francisco Building Code, Section 23 or those resulting from a linear elastic dynamic analysis of the tower/antenna configuration when subject to the recommended levels of ground shaking identified in Table 3.4.1, whichever is greater.

In order to increase the high frequency content in the type A and type B artificial ground motion records as published by Jennings, Housner and Tsai (1968)[1], the records used to represent vertical ground excitations were modified such that the time scale was reduced by 25%. In addition to the levels of ground shaking indicated in Table 3.4.1, the unfactored 8244 Orion Blvd. Feb. 9, 1971 ground motions recorded during the San Fernando earthquake, and the unfactored El Centro May 1940 NS component record, were used to "test" the tower/antenna structure.

4. DESIGN CRITERIA

4.1 Wind

Under the combined action of dead, live, special and equivalent static wind loads, as defined in section 3, the stresses in all elements of the tower, as determined by a linear elastic analysis, shall not exceed the allowable stresses permitted by the applicable San Francisco Building Code. Further, a factor of safety of 2 was prescribed for checking stability against overturning and sliding.

Electrical considerations imposed a further requirement that under an equivalent static wind of 10 psf (50 mph), the lateral deflections of a point on Stacks "A", "B", and "C", 170' above the antenna base level shall not exceed 4½" in any direction.

4.2 Seismic

The tower and antenna structures shall be reasonably expected to survive, without collapse, the combined action of dead, live, special loads and simultaneously applied ground excitations in three directions, as defined in Section 3. Further the tower and antenna structures shall conform with the seismic provisions of the applicable San Francisco Building Code.

5. MODELLING

5.1 Mathematical Computer Models

Several analytical models were devised to represent the physical structure. Preliminary sizing of members was done with the aid of a very simple two dimensional computer model which proved very effective in establishing the overall dynamic characteristics of both tower and antenna structures and in exposing the very considerable interaction between them. In consequence, more detailed three-dimensional finite element computer models were devised for use in subsequent analyses.

Two complimentary modelling approaches were followed. The first required that all structural members in the tower/antenna structure, Plate 5.1.1 be modelled as discrete elements. In total, this model comprised; 5571 elements; 1993 joints; and 6339 degrees-of-freedom. Three element types were used namely; the beam-column; the truss; and the boundary element types. In the second approach, the tower antenna structure was idealized to the extent that the tower leg assemblies and the tower truss and box girders were represented as single beam elements. Nodal points were established at the mid-height and midspans of these leg and girder assemblies. In this case, the model comprised; 201 elements; 126 joints; and 675 degrees-of-freedom. Only beam-column and boundary element types were utilized in the model. To determine individual member forces in the various assemblies, isolated "free body" models comprising all structural elements were appropriately developed and analyzed as sub-structures.

Because of the vast amount of computational effort required, the model developed using the former approach was used only for performing static analyses including dead, live and wind load conditions. The idealized or simplified model devised in the second approach, was used extensively in determining the dynamic behaviour of the tower/antenna structure during periods of strong ground shaking. The "free body" models of the second approach, were confined to critical assembly and haunch configurations.

5.2 Physical Model

An 8' high, physical model was built for use in a low speed wind tunnel analysis. The primary purpose of the analysis was to substantiate the adequacy of using the design wind loads prescribed in Section 3. As such, the rigid aluminium model was static in nature and no attempt was made to achieve dynamic similitude between the model and the real structure. Thus, wind dynamics effects were not investigated in the analysis.

6. DYNAMIC CHARACTERISTICS

6.1 Background

Preliminary analyses quickly revealed that the antenna systems had to be considered as an integral part of the structure as a whole. Resonance effects between the tower and the antenna structures, as originally designed and supported, were very significant, for all levels of ground excitation. Amplification of lateral motion reaching an order of magnitude of 10 to 20, assuming linear elastic behaviour, were predicted at the top of the antenna structures. Obviously, amplifications of this order could not be tolerated. This resonance phenomenon, Plates 6.1.1 through 6.1.3, was attributed to the virtual coincidence of the lateral fundamental periods of vibration of the antenna structures with those of the support tower. In order to verify this, the antennas were excited as independent structures located at ground level. In such cases it was found that during period of strong ground shaking, the antennas, as originally designed, would not be unduly distressed. Extensive efforts were made to effect a "shift" in the tower and antenna frequencies. Because of severe electrical constraints, this shift in frequencies was best accomplished by guying all of the antenna structures with a tri-level fiberglass guy system.

6.2 Natural Frequencies and Mode Shapes

The modal analysis of the lumped mass, idealized tower/antenna computer model, included fifty-five modes of vibration. Of these 25 were associated with the support tower and 10 for each of the three antenna structures. The antenna modes of vibration, figure 6.2.1, were all translational, 5 along the weak axis, 2-2. of the antenna guying configuration and 5 along the strong axis, 1-1. Of the 25 modes of vibration for the support tower, 5 were torsional modes, 10 were translational and 10 were vertical modes of vibration. Five translational modes were considered about both the N-S and E-W axes of the tower. Figure 6.2.2 illustrates the principal planes of vibration of the fundamental and second translational modes of the tower. The 10 vertical modes included 5 "breather" modes, 5 modes associated with the horizontal truss girders and the cantilever mode of vibration of the outrigger at the top of the support tower.

The undamped natural periods of vibration for the first and second modes of the support tower and each of the three antenna are identified and ranked in descending order of period in Table 6.2.1. The modal shapes associated with modes 1 through 12, as listed in Table 6.2.1, are illustrated in figures 6.2.3 through 6.2.6.

Mode 27 corresponds to the fundamental cantilever mode of vertical vibration of the outrigger assemblies located at the top of the support tower. Modes 37 and 48 are the first and second "breather" modes respectively of the support tower. Mode 51 represents the fundamental mode of vertical vibration of the lower truss girders.

7. DYNAMIC RESPONSE

7.1 Computer Simulations

Elastic time-dependent deterministic responses of the three-dimensional idealized model of the tower/antenna structure were determined for various levels and types of ground shaking. In all cases the base of the tower was excited, simultaneously, vertically and in two perpendicular horizontal directions. Constant, 2% of critical, viscous damping was assumed in all modes of vibration. Masses were concentrated at the nodal points. The dynamic response was computed using the normal mode method. The time interval of integration was set at 0.0125 seconds. Response parameters monitored during the simulation, include nodal accelerations and displacements in addition to element moments, shears and axial forces.

7.2 Response Summary

Table 7.2.1 summarizes some of the significant maximum dynamic responses of the tower/antenna structure for three levels of ground shaking, Section 3, recommended for use in the design of the tower. These are compared to corresponding values obtained from a code level seismic static analysis, those resulting from the application of the design wind load, and those derived from the wind tunnel study.

In order to assess the importance of vertical ground motions on the tower structure, the maximum vertical displacements at certain nodal points, attributable to the vertical component of ground shaking, were isolated from the resultant displacements. These values are presented in Table 7.2.2 for a type "A" ground motion, modified as noted in Section 3.

Because of virtual symmetry of the tower/antenna structure and the boundary constraints applied during the computer simulation, namely that the vertical excitations applied to each tower leg were identical and in phase, the horizontal displacements accompanying the vertical displacements listed in Table 7.2.2 were negligible.

Selected time history responses are included as figures 7.2.1 through 7.2.6. They are representative of many such curves drawn to depict the dynamic behaviour of critical elements within the structure.

7.3 Response evaluation

As a result of the very significant interactions between the support tower and antenna structures during seismic disturbances, the structural design of all 3 antenna structures and the upper tier of the support tower was governed ultimately by seismic considerations. As might be deduced from the response summary, the structural design of the lower four tiers of the support tower is governed by the equivalent static design wind load. This arbitrary static 50/33 psf wind load is two to four times the loads determined in the low speed wind tunnel study, using extreme 100 year wind velocity predictions for the tower site. The wind tunnel studies thus provided some assurance of the adequacy of the 50/33 psf wind load. However, because of possible significant dynamic effects in the wind load patterns, no reduction was made in the original design wind criteria except that the one-third increase in allowable stresses, as provided for in the 1969 edition of the San Francisco Building Code, was permitted.

The design of the tower footings was controlled primarily by uplift considerations resulting from the design wind load. This implies that factors of safety of at least 5 and 8 are provided for against overturning during a Type "A" ground motion representing a San Andreas 8-8 1/2 M event on a fault segment adjacent to the tower site and during an extreme estimated 100 year windstorm, respectively.

The effectiveness of the tri-level guying system is seen with reference to Plates 6.1.1 through 6.1.3. The 20% - 25% shift in fundamental period, attributed primarily to the guying system, is sufficient to avoid resonance conditions between the antenna structures and the support tower. Sufficient pretension was specified in the guy assemblies to overcome the maximum dynamic "compression" loads and thus validate the assumptions made in the linear elastic analysis.

Of particular importance in the support tower design are the secondary or sidesway moments induced in the leg and truss assemblies. The magnitude of such moments, whether resulting from wind or seismic load conditions, controlled the design of the leg and truss sections. The tower X-braced cables effectively carry virtually all the lateral loads down to the foundation. Sufficient pretension was introduced in these cables during erection, to exceed the maximum wind or seismic loads imposed, thus validating the assumptions inherent in a linear elastic analysis.

Within the support tower, amplifications of lateral accelerations and displacements are not significant. The contributions of lateral displacement at the tops of the antenna structures resulting from rotation of the support tower are of little consequence.

8. CONCLUSION

The Type "A" series ground motion, representing an 8-8 1/2 M San Andreas event on a fault segment adjacent to the tower site, is clearly the most critical event of those considered in the design of the Mt. Sutro Tower. The tower, as designed and constructed, should be expected to survive, without collapse, and remain essentially elastic during a Type "A" series ground motion or lesser seismic disturbance.

9. REFERENCE

1. Jennings, P.C., "Simulated Earthquake Motions", Proceedings of the Fourth World Conference on Earthquake Energy, Santiago, Chile, Vol. 1, January 1969, A-1,pp. 145.

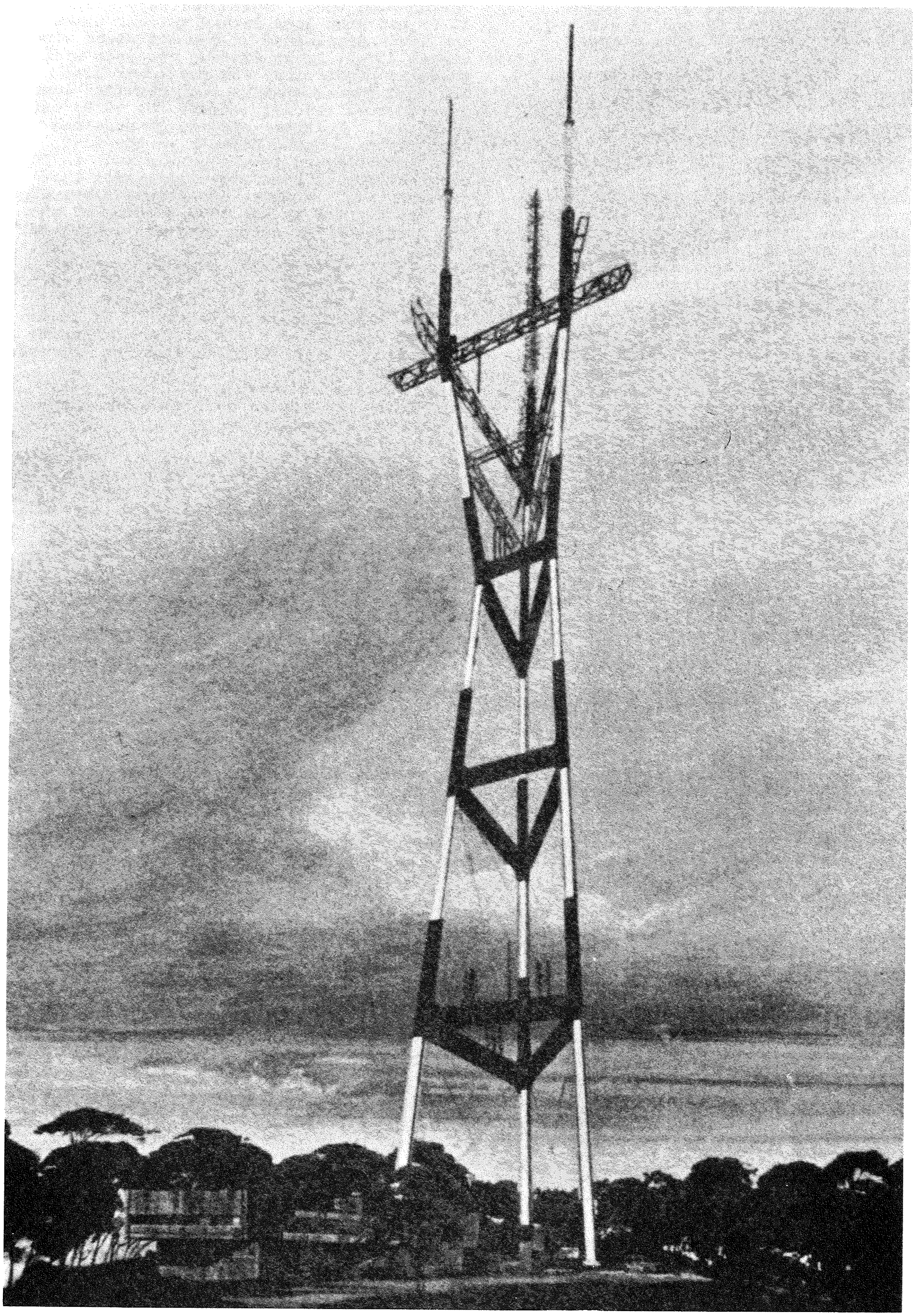


PLATE 1.1 ARTIST'S CONCEPTION OF THE PROPOSED MT. SUTRO TOWER

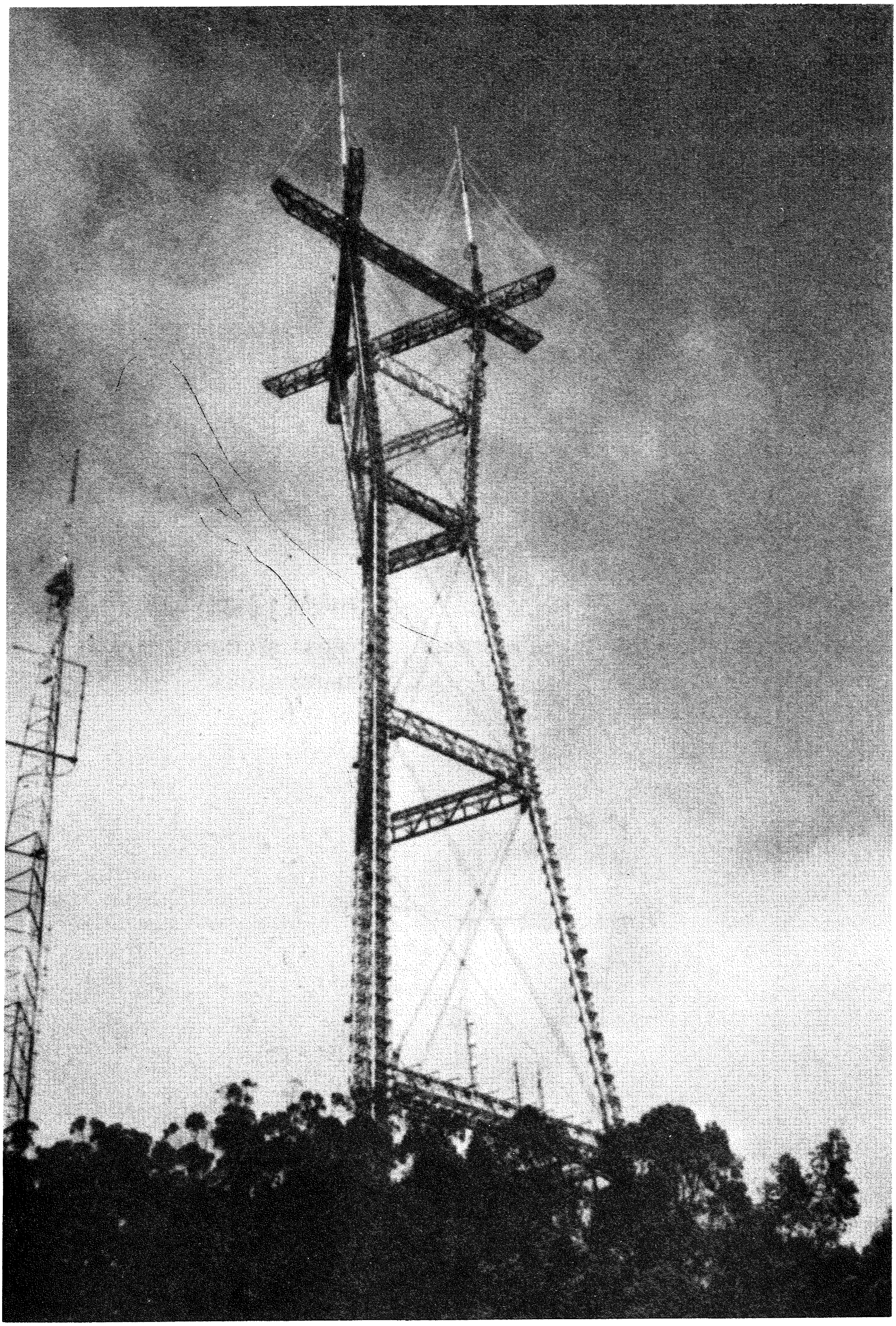


PLATE 1.2

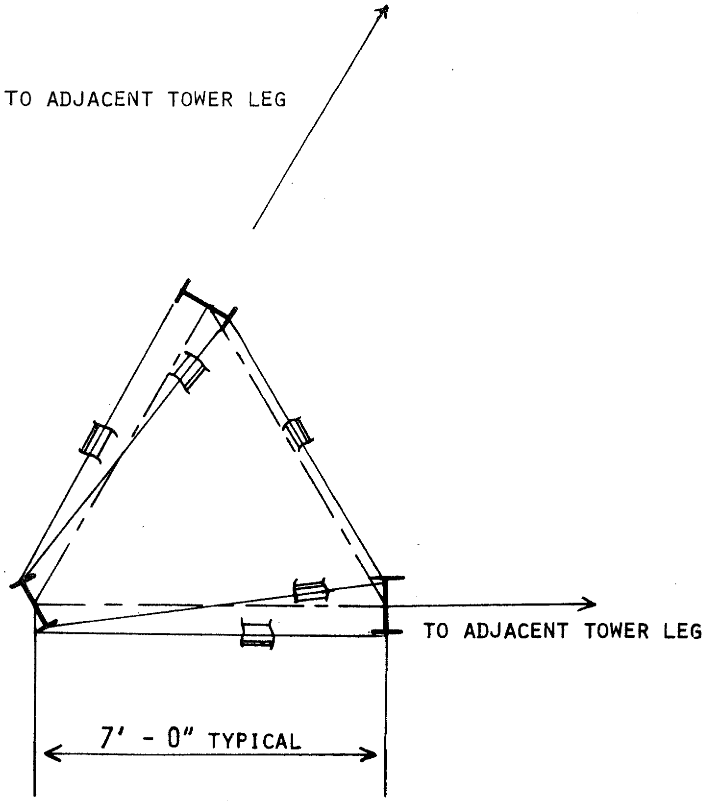


FIGURE 2.1.1

CROSS-SECTION OF TYPICAL TOWER LEG ASSEMBLY

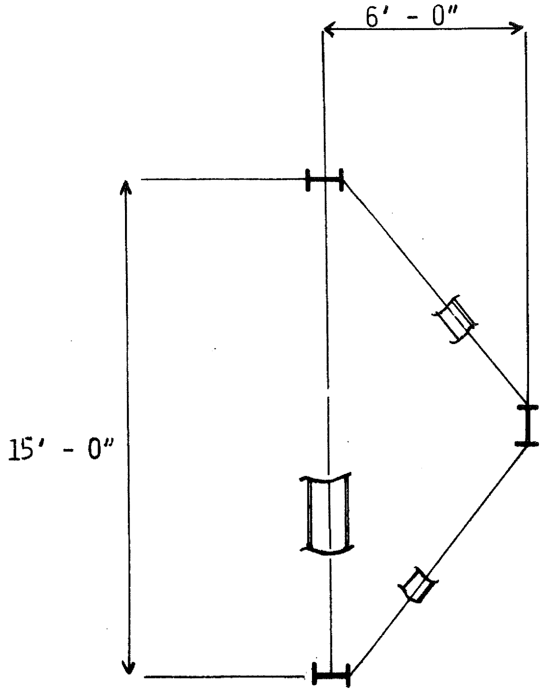


FIGURE 2.1.2

CROSS-SECTION OF TYPICAL TRUSS GIRDER

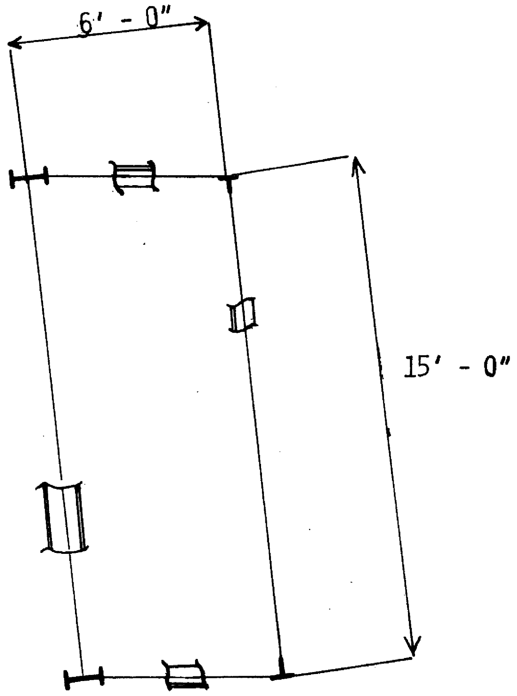
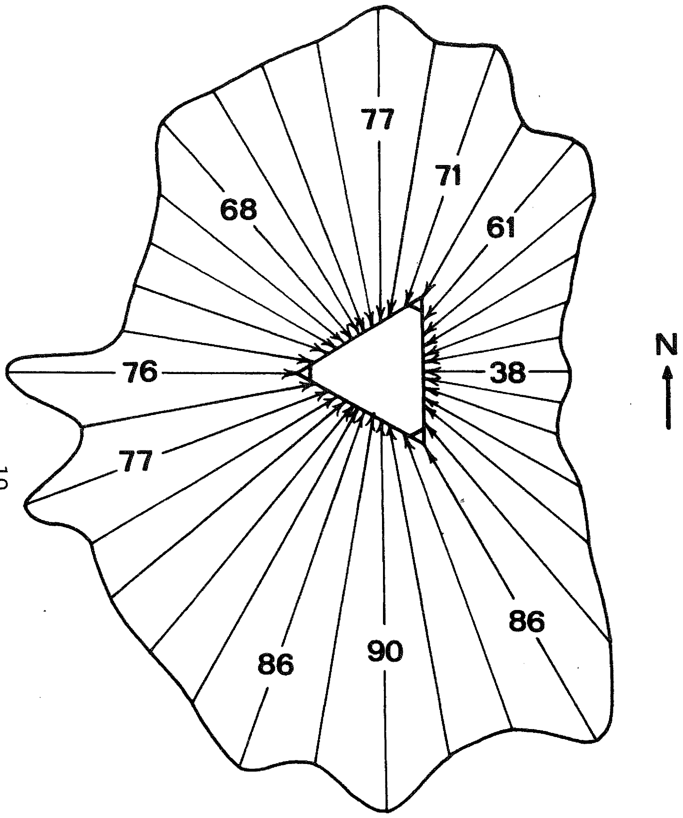


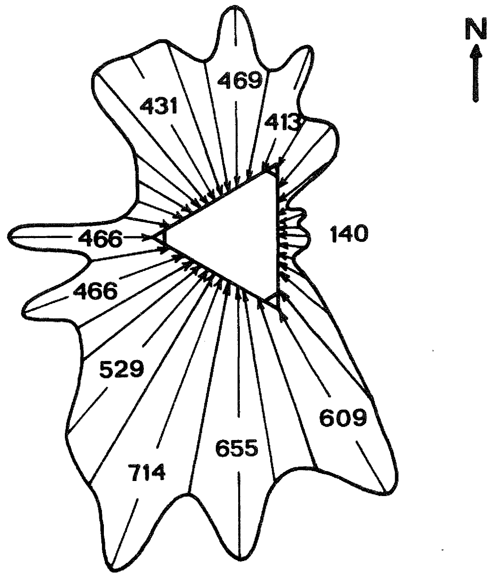
FIGURE 2.1.3

CROSS-SECTION OF TRUSSED BOX GIRDER AT ANTENNA BASE LEVEL



EST. 100 YR. EXTREME WINDS (mph)
OAKLAND + 500m.

PLATE 3.3.1



TOTAL TOWER DRAG FORCES (kips)
FOR EXTREME 100 YR. WINDS

PLATE 3.3.2

LOCAL EVENT	EARTHQUAKE RECORD	DURATION SECONDS	SCALE FACTOR	
			HORIZONTAL COMPONENT	VERTICAL COMPONENT
8 - 8 1/2 M	Caltech type A1, A3	60	1.0	0.8
8 - 8 1/2 M	Caltech type A2, A4	60	1.0	0.8
7 - 7 1/2 M	Caltech type B1	45	1.0	0.8
7 - 7 1/2 M	Caltech type B2	45	0.9	0.7
6 1/2 - 7 M	San Fernando E/Q Feb. 9, 1971, recorded at 445 So. Figueroa St., L. A.	30	1.0	1.0
5 1/2 - 6 M	Daly City E/Q, 1957 as recorded Golden Gate Park	30	1.0	1.0

TABLE 3.4.1

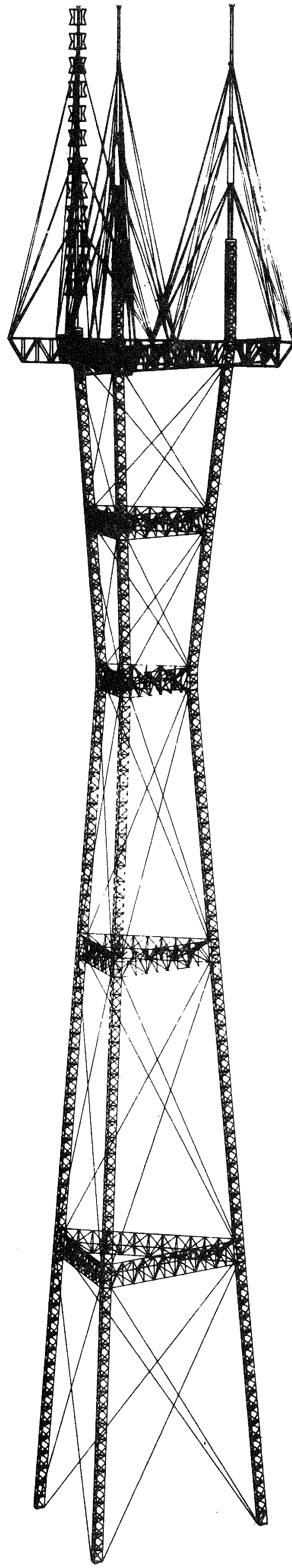


PLATE 5.1.1 "FULL" COMPUTER MODEL

MT. SUTRO TOWER

ANTENNA BASE MOTION

ACCELERATION SPECTRUM FOR

EL CENTRO EQ.. (N-S) MAY 1940

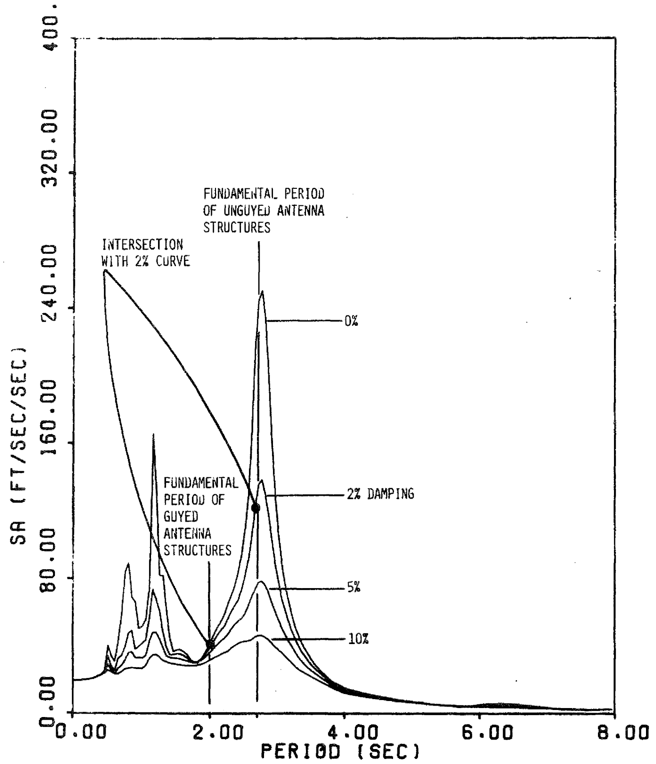


PLATE 6.1.1

MT. SUTRO TOWER

ANTENNA BASE MOTION

VELOCITY SPECTRUM FOR

EL CENTRO EQ.. (N-S) MAY 1940

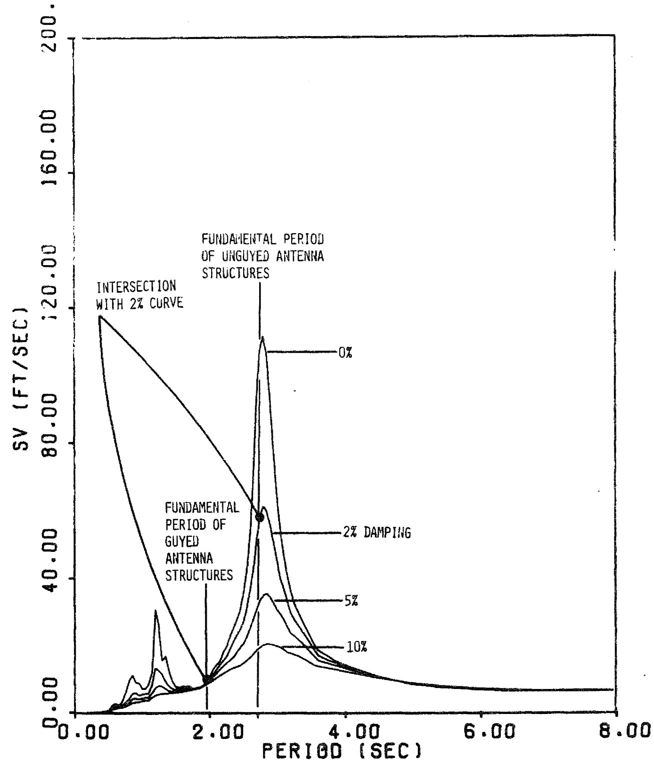


PLATE 6.1.2

MT. SUTRO TOWER

ANTENNA BASE MOTION

DISPLACEMENT SPECTRUM FOR

EL CENTRO EQ.. (N-S) MAY 1940

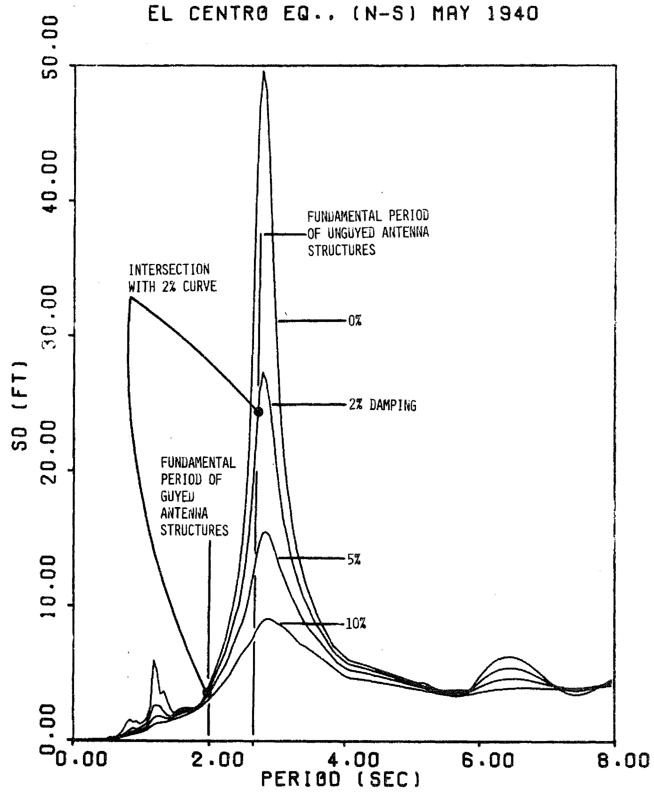
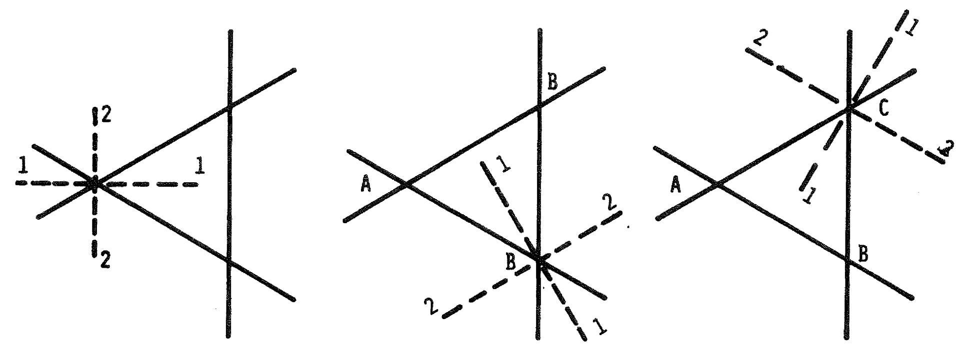
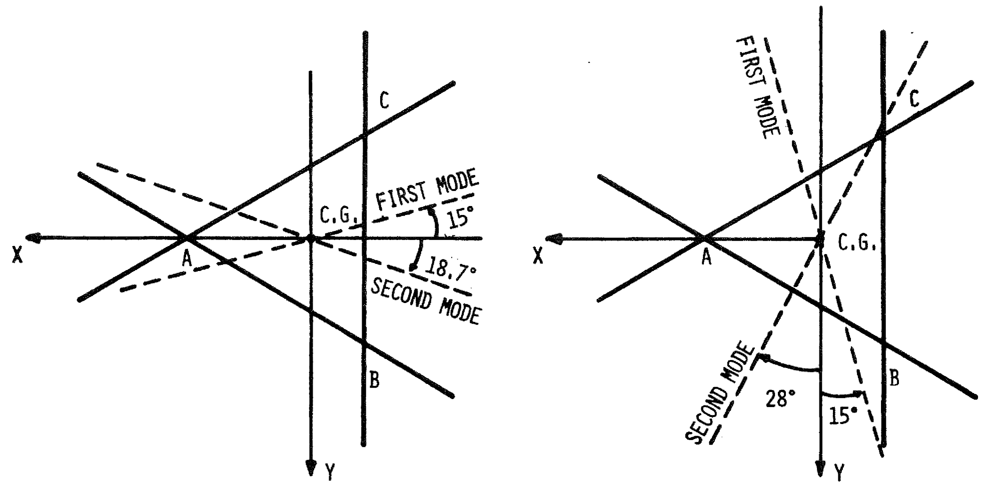


PLATE 6.1.3



a. ANTENNA -A- b. ANTENNA -B- c. ANTENNA -C-

FIGURE 6.2.1 PRINCIPAL PLANES OF VIBRATION OF THE ANTENNAS



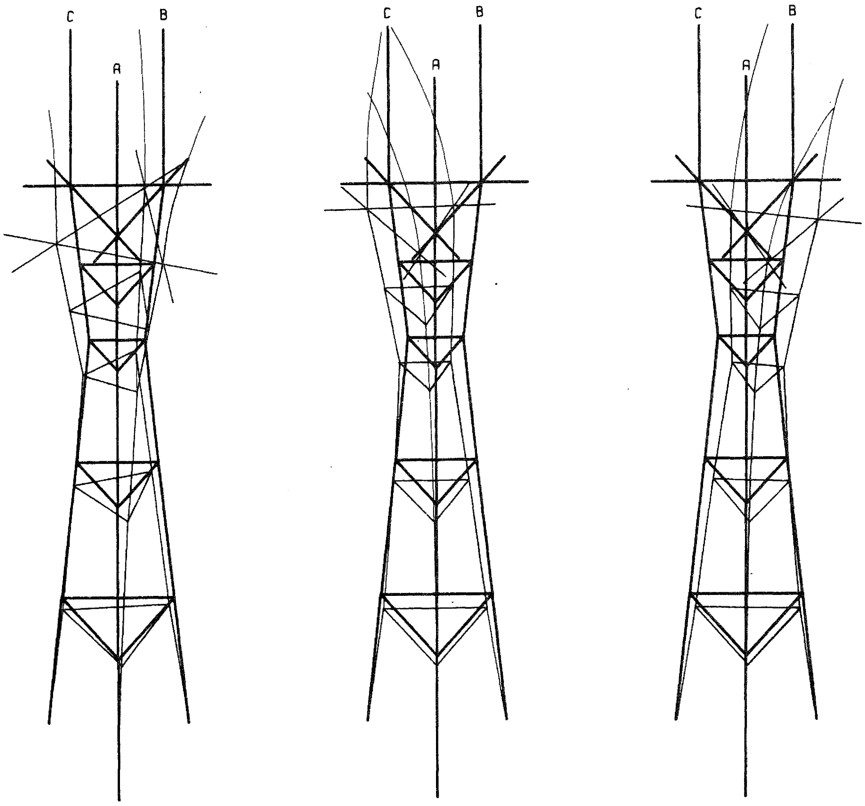
a. E-W DIRECTION b N-S DIRECTION

FIGURE 6.2.2 TRANSLATIONAL MODES OF VIBRATION OF THE MAIN TOWER

MODE	PERIOD	TOWER										ANTENNA 'A'				ANTENNA 'B'				ANTENNA 'C'			
		E-W DIRECT.		N-S DIRECT.		TORSION		VERTICAL				DIRECT. 1-1		DIRECT. 2-2		DIRECT. 1-1		DIRECT. 2-2		DIRECT. 1-1		DIRECT. 2-2	
		1	2	1	2	1	2	C	T	1	2	1	2	1	2	1	2	1	2	1	2	1	2
1	4.23					X																	
2	3.22			X																			
3	3.18	X																					
4	1.99													X									
5	1.92																					X	
6	1.76																	X					
7	1.50																			X			
8	1.48											X											
9	1.34															X							
10	1.30						X																
11	1.20				X																		
12	1.16		X																				
13	0.98														X								
14	0.93																						X
15	0.89																		X				
16	0.86																				X		
17	0.85												X										
18	0.82																X						
27	0.46							X															
37	0.29									X													
48	0.12										X												
51	0.05								X														

TABLE 6.2.1 DISTRIBUTION OF NATURAL PERIODS

MOUNT SUTRO TOWER

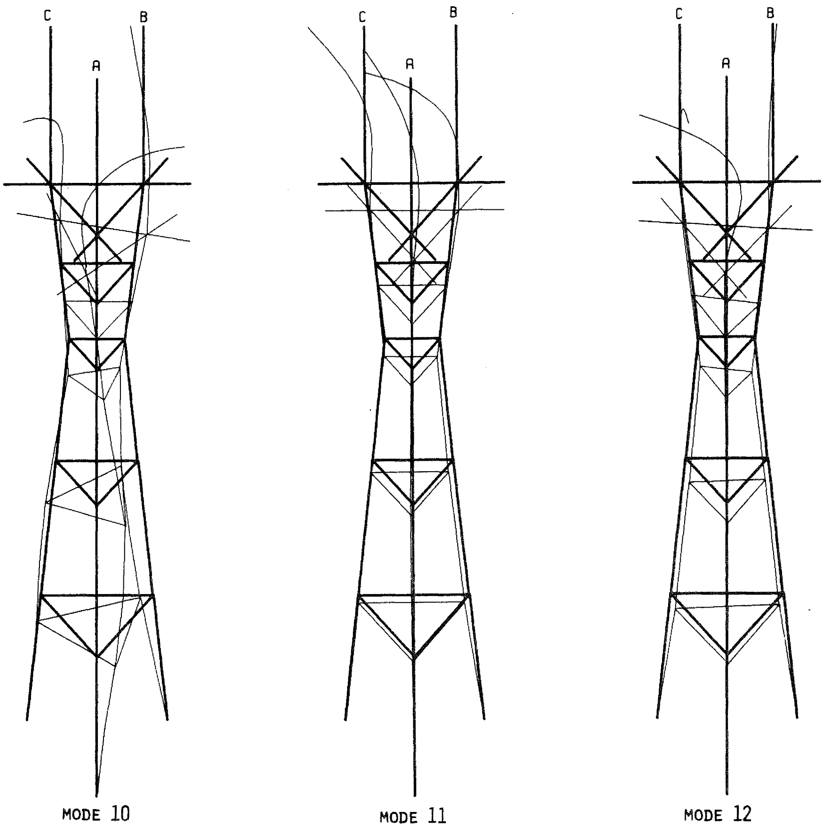


MODE 1 T_T = 4.23 SECS.

MODE 2 T_{NS} = 3.22 SECS.

MODE 3 T_{EW} = 3.18 SECS.

FIGURE 6.2.3 SUPPORT TOWER - FUNDAMENTAL MODES



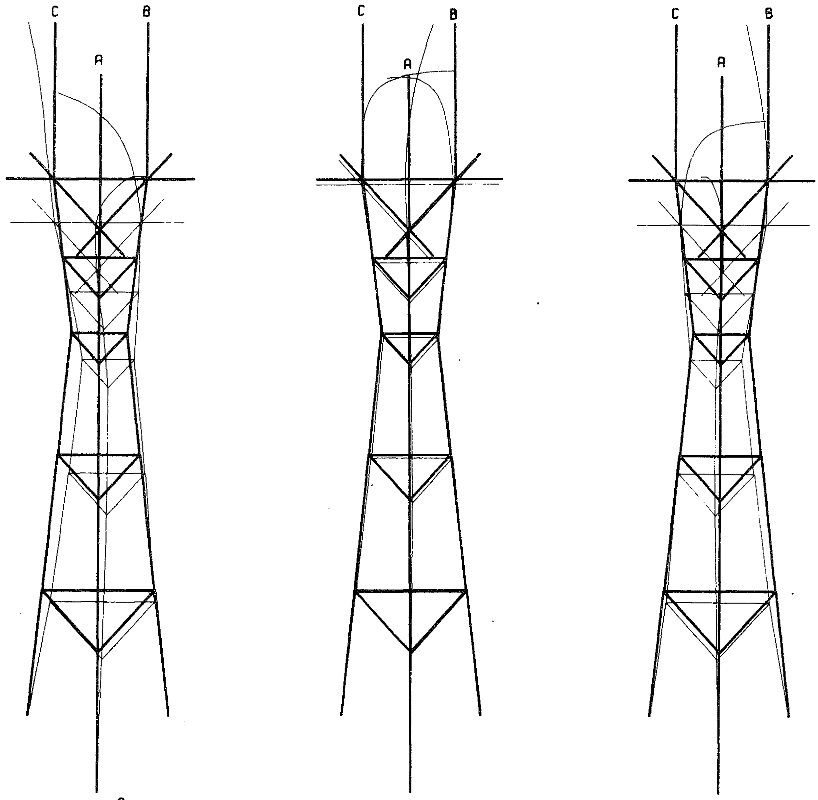
MODE 10 TORSION T = 1.30 SECS.

MODE 11 N-S T = 1.20 SECS.

MODE 12 E-W T = 1.16 SECS.

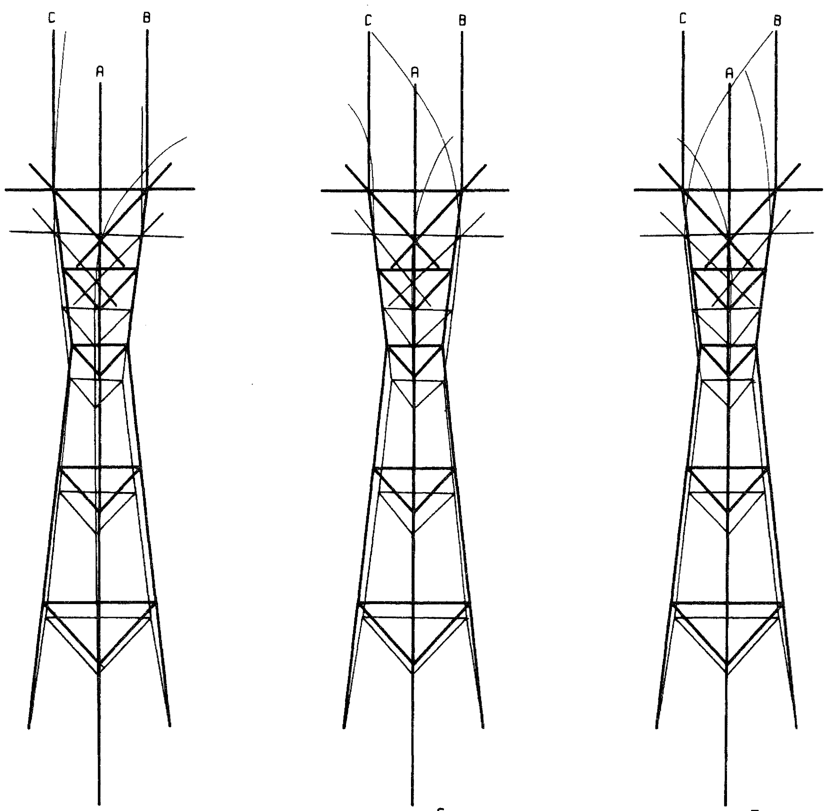
FIGURE 6.2.4 SUPPORT TOWER - SECOND MODES

MOUNT SUTRO TOWER



MODE 8	MODE 9	MODE 7
ANTENNA -A-	ANTENNA -B-	ANTENNA -C-
T = 1.48 SECS.	T = 1.34 SECS.	T = 1.50 SECS.

FIGURE 6.2.5 ANTENNA STRUCTURES - FUNDAMENTAL MODES, DIRECTION 1-1



MODE 4	MODE 6	MODE 5
ANTENNA -A-	ANTENNA -B-	ANTENNA -C-
T = 1.99 SECS.	T = 1.76 SECS.	T = 1.92 SECS.

FIGURE 6.2.6 ANTENNA STRUCTURES - FUNDAMENTAL MODES, DIRECTION 2-2

	MAXIMUM RESPONSES	TYPE "A" SERIES		SAN FERNANDO E/Q 2/9/73 recorded at 8244 Orion Blvd. Los Angeles		SAN FERNANDO E/Q 2/9/71 recorded at 445 So. Figeuroa St. Los Angeles		Equivalent Static Design Wind Load (50/33 psf)		Wind Tunnel Study		Code Seismic
		N-S (A1)	E-W (A2)	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	E-W & N-S
ANTENNA BASE LEVEL	Acceleration (G)	0.59	0.38	0.28	0.21	0.10	0.14	-	-	-	-	-
	Deflection (FT)	4.10	1.91	2.05	0.90	1.02	0.56	3.67	4.60	1.92	1.35	-
TOWER BASE LEVEL	Acceleration (G)	0.39	0.37	0.26	0.11	0.14	0.13	-	-	-	-	-
	Shear (KIP)	1273 (31%)	942 (23%)	559 (13.5%)	458 (11%)	225 (5.5%)	248 (6%)	1568	1845	744	535	278 (6.7%)
	O.T.M. x 10^6 (KIP-FT)	0.57	0.32	0.29	0.13	0.10	0.14	0.66	0.80	0.31	0.22	0.24 (J=0.41)

Total weight of tower/antenna structure = 4130^K

(%) shears are expressed as a percentage of total weight of tower/antenna structure

TABLE 7.2.1 TOWER RESPONSE SUMMARY

LEVEL	Vertical Displacements (Relative to ground)		
	Truss Midspan	Leg ℄	Outrigger Tip
Beacon	-	1.48"	-
Antenna Base	1.55	1.48"	1.41
Upper Truss	-	1.19"	-
Waist	-	0.98"	-
Intermediate Truss	-	0.59"	-
Lower Truss	-	0.27"	-
Tower Base	-	0	-

TABLE 7.2.2

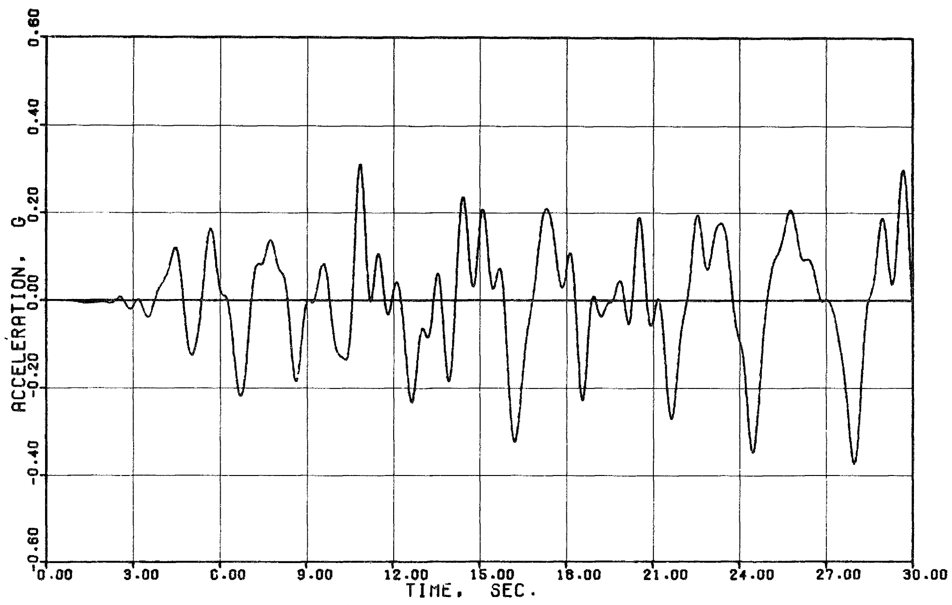


FIGURE 7.2.1 MOUNT SUTRO - DYNAMIC ANALYSIS
ACCELERATION TIME HISTORY AT BASE OF ANTENNA *A*,E-W-DIRECTION
DUE TO EARTHQUAKES A1/A2

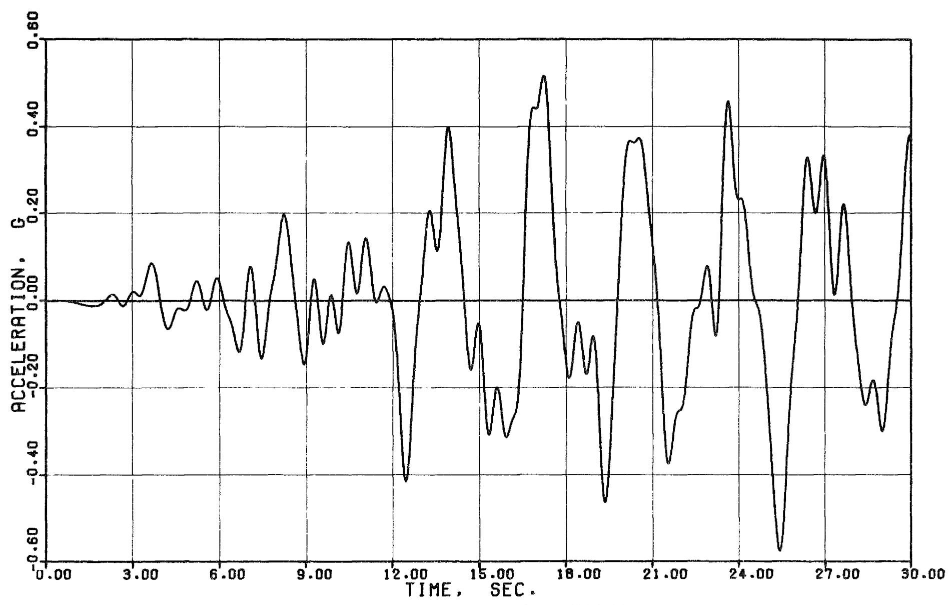
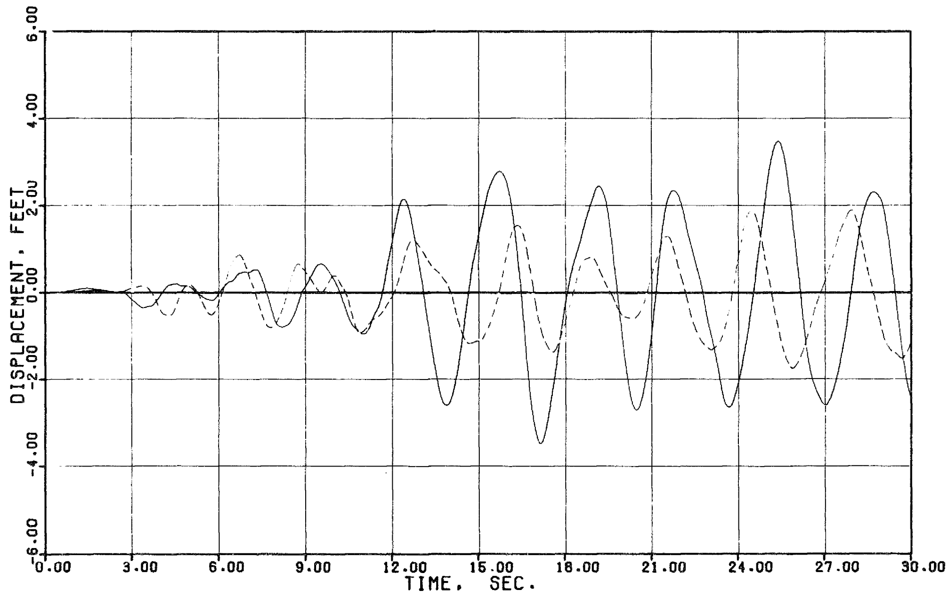


FIGURE 7.2.2 MOUNT SUTRO - DYNAMIC ANALYSIS
ACCELERATION TIME HISTORY AT BASE OF ANTENNA *A*,N-S-DIRECTION
DUE TO EARTHQUAKES A1/A2



——— CURVE - DISPLACEMENT IN SOFT DIRECTION
- - - - CURVE - DISPLACEMENT IN STRONG DIRECTION

FIGURE 7.2.3 MOUNT SUTRO - ANTENNA *A*
TIME HISTORY OF DISPLACEMENTS
AT ANTENNA BASE (NODE 43), DUE TO EARTHQUAKES A1/A2

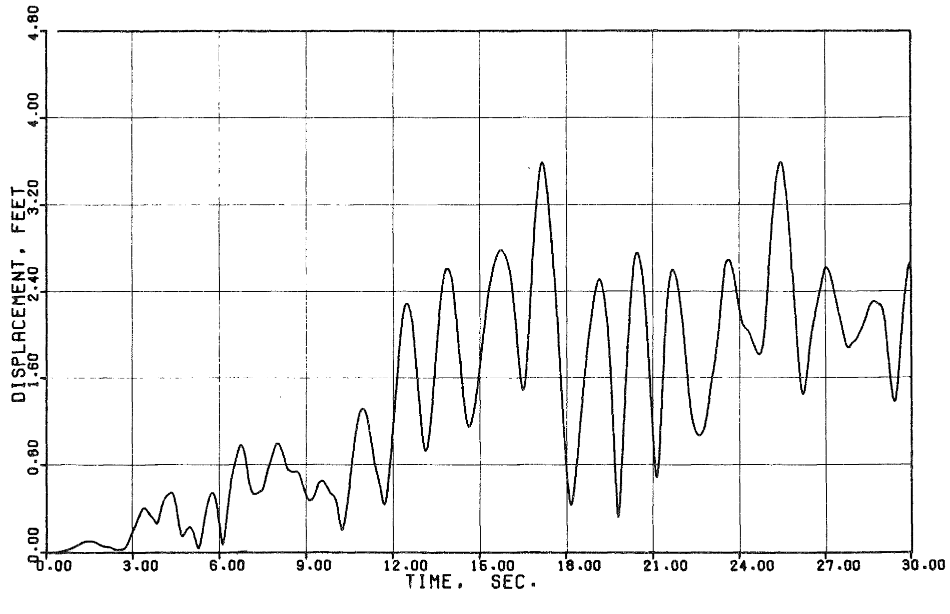


FIGURE 7.2.4 MOUNT SUTRO - ANTENNA *A*
TIME HISTORY OF RESULTANT DISPLACEMENTS
AT ANTENNA BASE (NODE 43), DUE TO EARTHQUAKES A1/A2

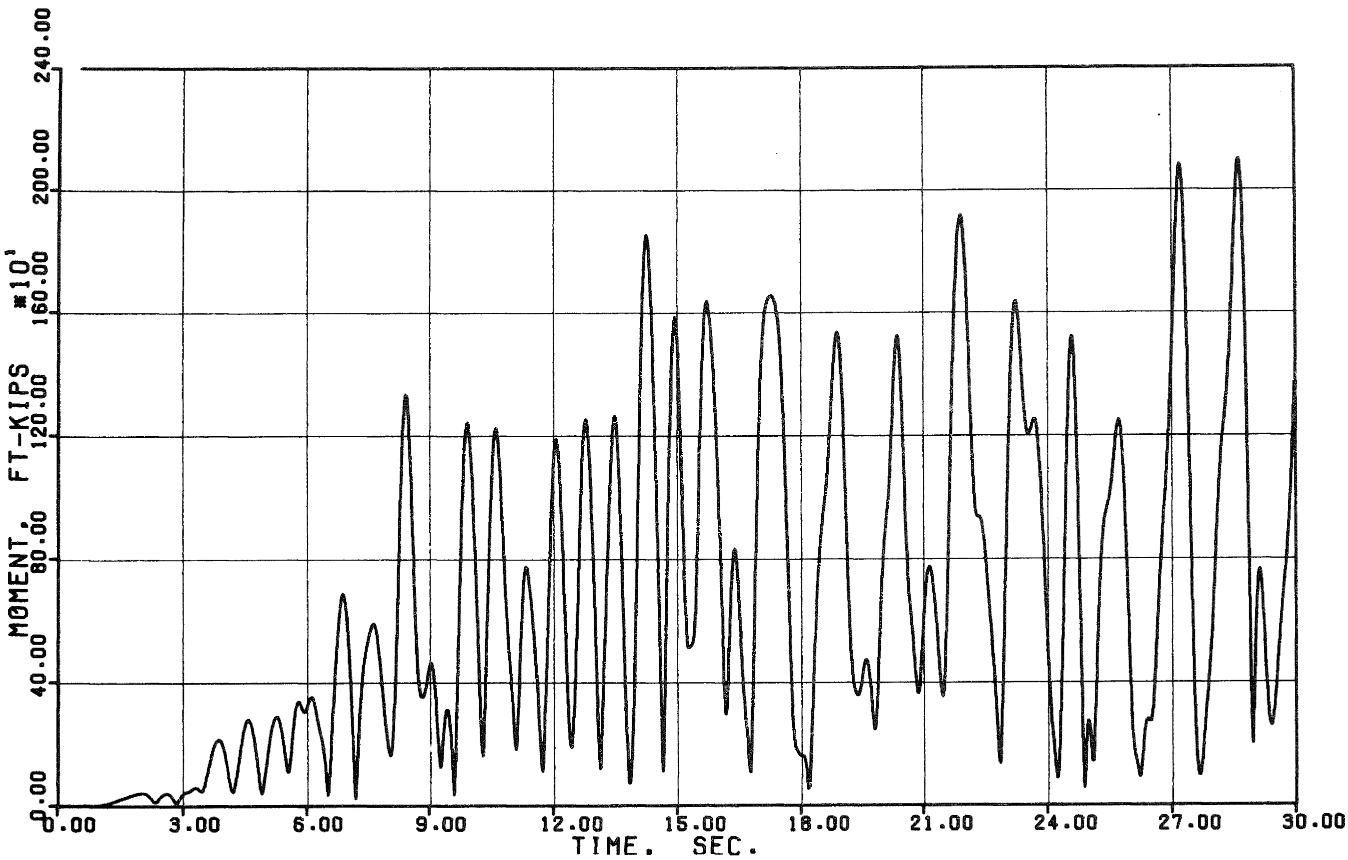


FIGURE 7.2.5 MOUNT SUTRO - ANTENNA *A* (STEP III-4)
TIME HISTORY OF RESULTANT BENCING MOMENT
AT ANTENNA BASE, DUE TO EARTHQUAKES A1/A2

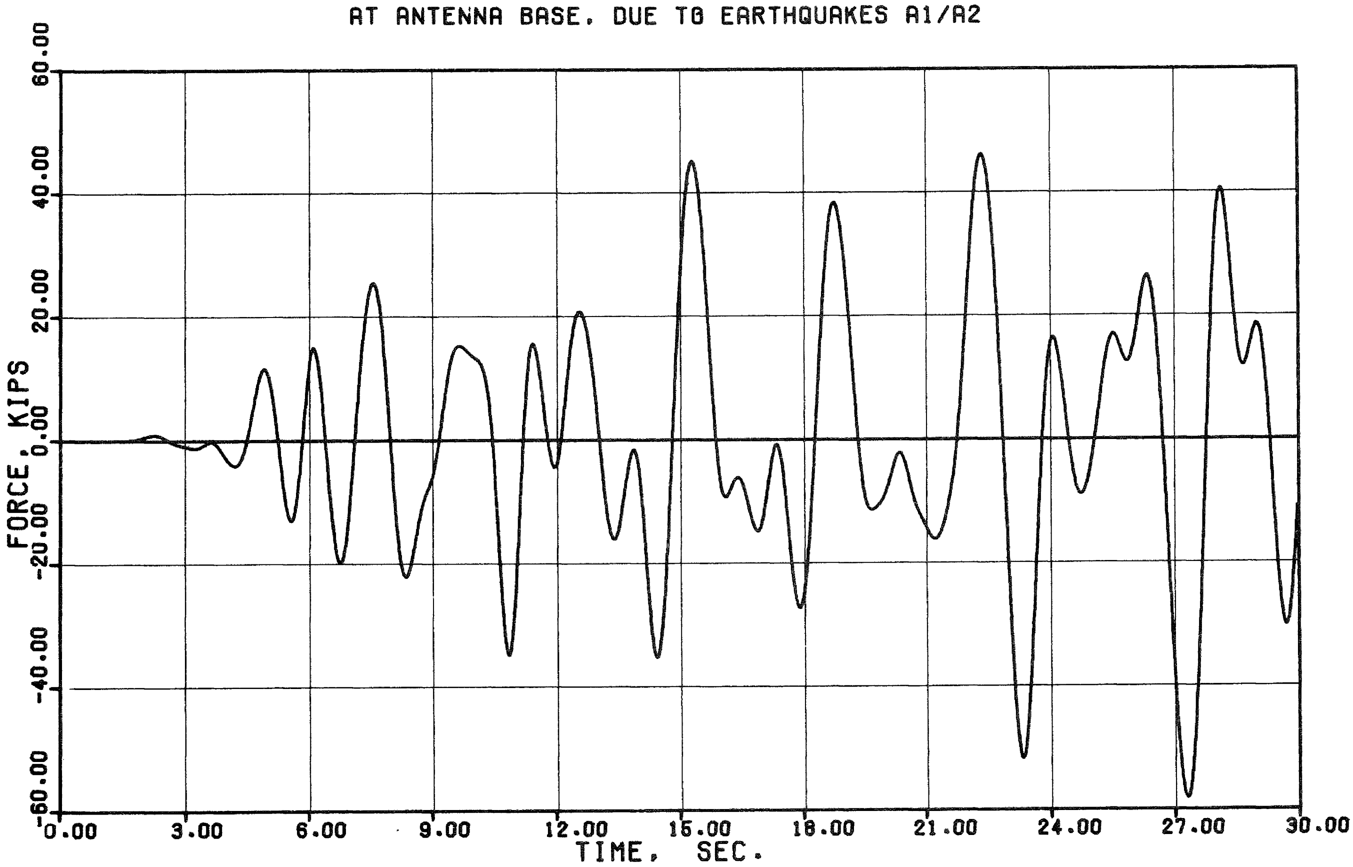


FIGURE 7.2.6 MOUNT SUTRO - ANTENNA *A* (STEP III-5)
TIME HISTORY UF RESULTANT CABLE FORCES
OF THE CABLE CONNECTING NODES 66 AND 118