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Lawrence Livermore Laboratory

SEISMIC ANALYSIS OF THE MIRROR FUSION TEST FACILITY BUILDING

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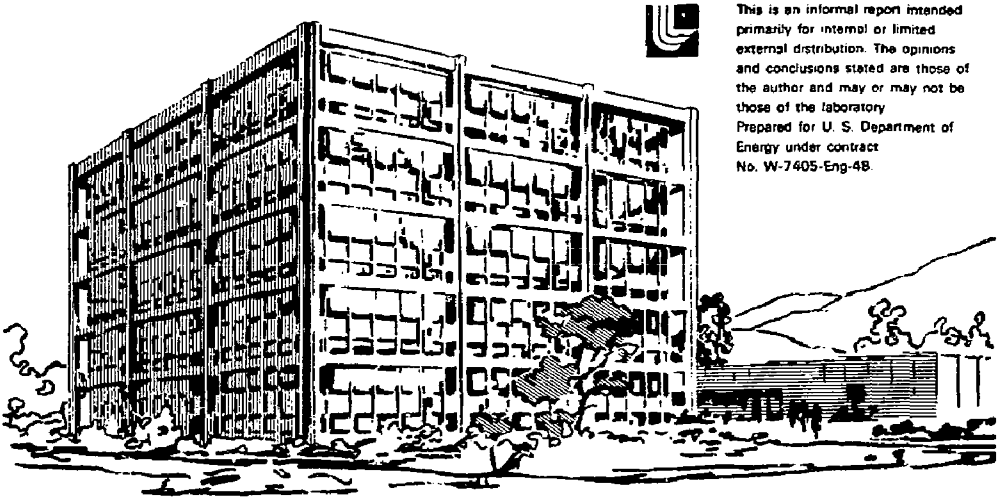
September 1, 1978



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ABSTRACT

This report describes a seismic analysis of the present Mirror Fusion Test Facility (MFTF) building at the Lawrence Livermore Laboratory. The analysis was conducted to evaluate how the structure would withstand the postulated design-basis earthquake (DBE). We discuss the methods of analysis used and results obtained. Also presented are a detailed description of the building, brief discussions of site geology, seismicity, and soil conditions, the approach used to postulate the DBE, and two methods for incorporating the effects of ductility. Floor spectra for the 2nd, 3rd, and 4th floors developed for preliminary equipment design are also included. The results of the analysis, based on best-estimate equipment loadings, indicate additional bracing and upgrading of connection details are required for the structure to survive the postulated design-basis earthquake. Specific recommendations are made.

INTRODUCTION

We conducted a seismic analysis of the MFTF building at the request of the MFTF Program Director. The building will house the experimental test equipment to be used as part of the MFE research program for the next 15 years. The decision to conduct the analysis was primarily based on economic considerations. The experimental programs to be conducted within the structure pose no significant health or safety hazards to the public or environment and as such, a safety-related analysis of this structure was not required. However, the economic impact of a failure of the structure during an earthquake justified both the analysis and any recommended strengthening of the structure.

Following our analysis, a major addition to the existing building was proposed. This addition would necessitate a reanalysis of the structure that could lead to significant changes in the conclusions and recommendations presented herein. The purpose of this report, then, is to document our analysis and subsequent recommendation to upgrade the present MFTF structure.

Because the structural analysis was based on economic rather than safety-related considerations, and because of the relatively short life span of the MFE Research Program, the DBE chosen was less severe than the site-specific DBE used to analyze and design critical facilities at the Livermore site.[1,2] However, the frequency content of the DBE developed for the Laboratory was retained for this analysis.

We also conducted a dead load analysis of the building and combined the results with those of the seismic analysis. The analysis of the structure was simplified somewhat because of the symmetrical nature of the building framing system. As a result, it was only necessary to model and analyze one half of the structure. A shield-block structure housed within the building was also analyzed for the postulated DBE. The analysis of this structure is not included in this report.

BUILDING DESCRIPTION

The MFTF building consists of a diagonally braced, steel frame structure with concrete floor diaphrams and a trussed roof system. It is rectangular in shape, and is approximately 159 x 220 x 100 ft high. Corrugated metal decking comprises the high-bay roof and exterior walls. A tar and gravel composite roof covers both the low bay areas over the 4-in. concrete roof slab and the corrugated metal decking on the high bay roof. All frame connections are riveted and it is believed that A7 structural steel was used throughout. All columns bear on cast-in-place belled piles varying from 10 to 28 ft in length and 18 to 24 in. in diameter. A 60-ton crane services the high-bay area. The structure was constructed in the early 1950's with a single bay added at the east end approximately a year after completion. Figure 1 shows the building during its original construction and Fig. 2 shows recent photos from three different angles. More structures have been added to the building

FIG. 1. MFTF building during construction in the early 1950's.

over the years but they are not structurally tied to the main building framework. (See Appendix A for plan and elevation drawings of Building 431.) The main lateral force-resisting elements of the building are diagonal bracing angle sections located along the exterior walls of the structure. We considered these bracing elements to be critical items and analyzed them to evaluate their ability to maintain structural integrity after experiencing ground vibrations from the postulated DBE.

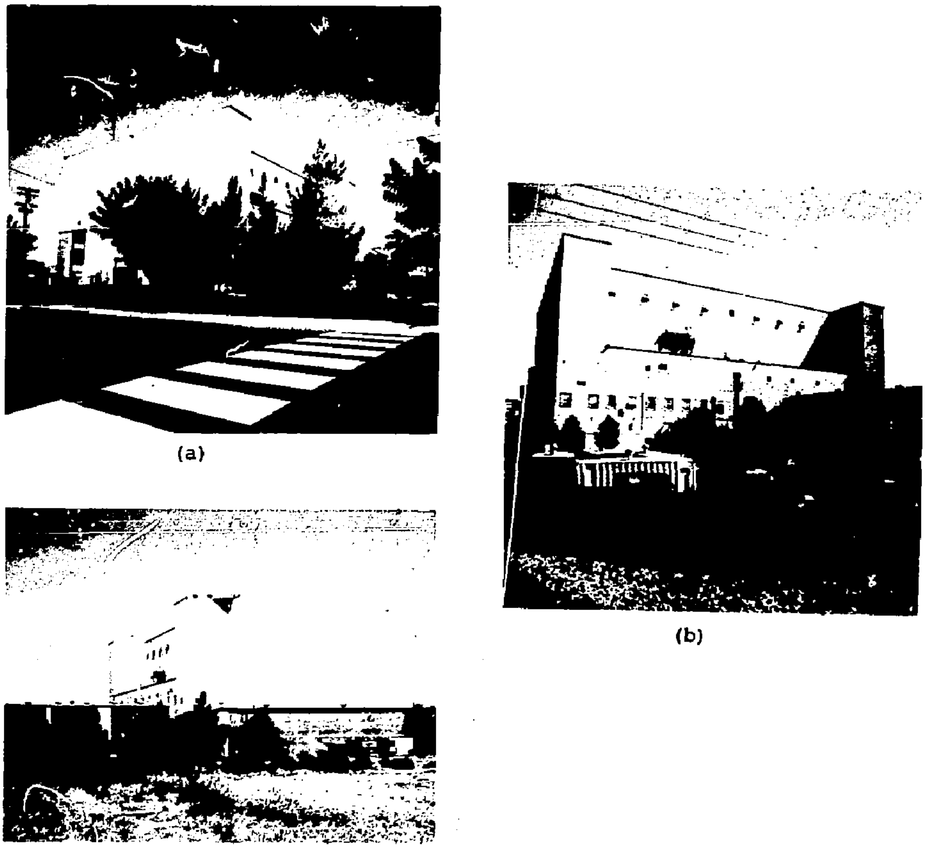

FIG. 2. MFTF building in July 1978; (a) view looking northeast, (b) view looking north, (c) view looking northwest.

SITE GEOLOGY, FAULTING, AND SEISMICITY

The Lawrence Livermore Laboratory is located in the southeast portion of the Livermore Valley. References 1 and 2 give a thorough review of the geology, faulting, and seismicity of the area based on a detailed literature search. John A. Blume & Associates, Engineers, conducted a separate review[3] to establish seismic design criteria for a plutonium facility at the Laboratory site.

GEOLOGY

The location and geological setting of the building site within the eastern portion of the Valley are shown in Fig. 3. The site rests on a gently sloping alluvial plain. Within a mile to the south are low hills called the Livermore Uplands; to the east and northeast are hills generally referred to as the Altamont Uplands.

Figure 4 shows two geological sections that are at right angles to each other and that intersect very near the building site. The locations of these sections are shown in Fig. 3. The site is blanketed by unconsolidated alluvial deposits directly overlying the Livermore formation. These formations are of recent to Plio-Pleistocene origin and consist of an active deposit of gravels, sands, and clays. The thickness of the deposit varies along the sections, and is estimated to be about 400 ft at the site. Older Miocene deposits underlie the Livermore formation. More detailed characteristics of the rock and soil deposits in the area are shown in Fig. 3.

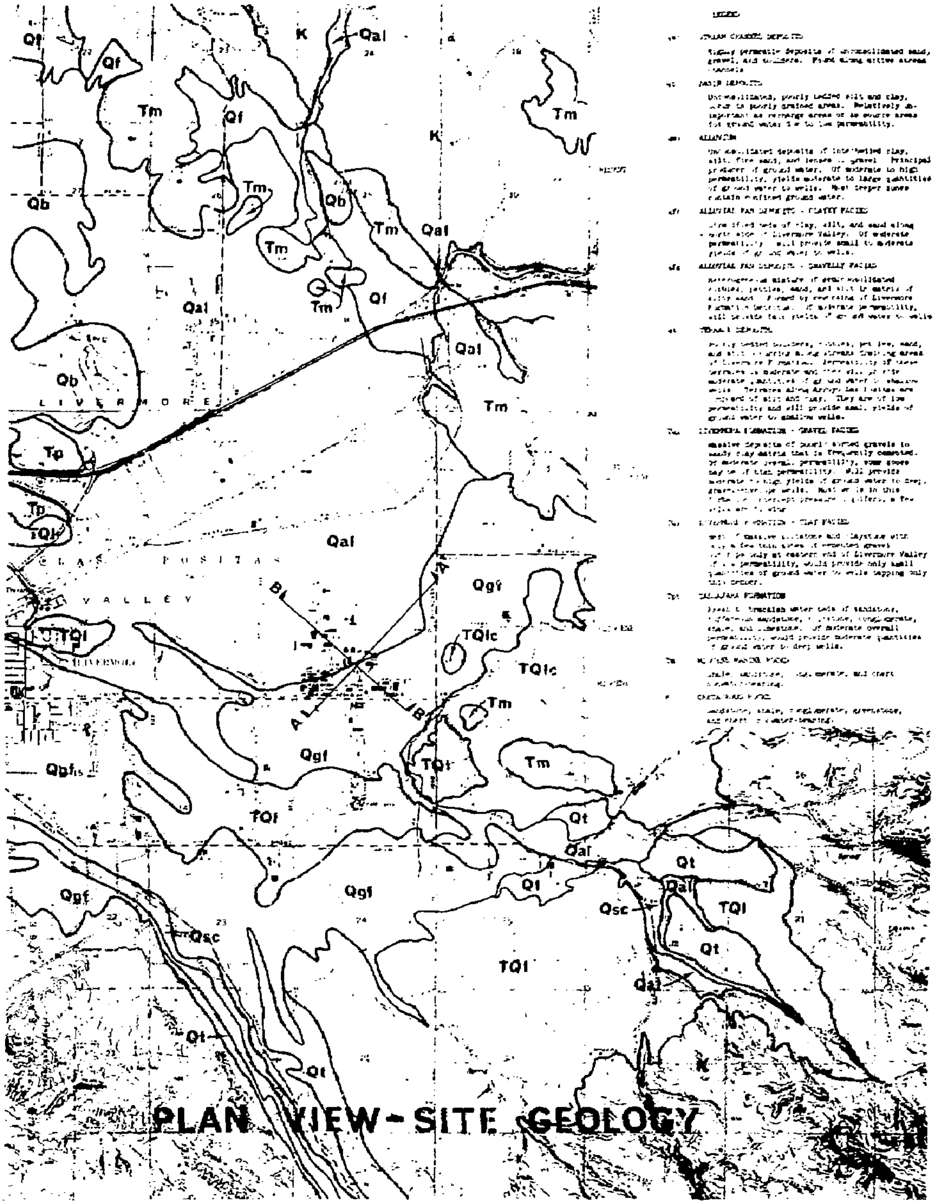

FIG. 3. Plan view of the site geology.

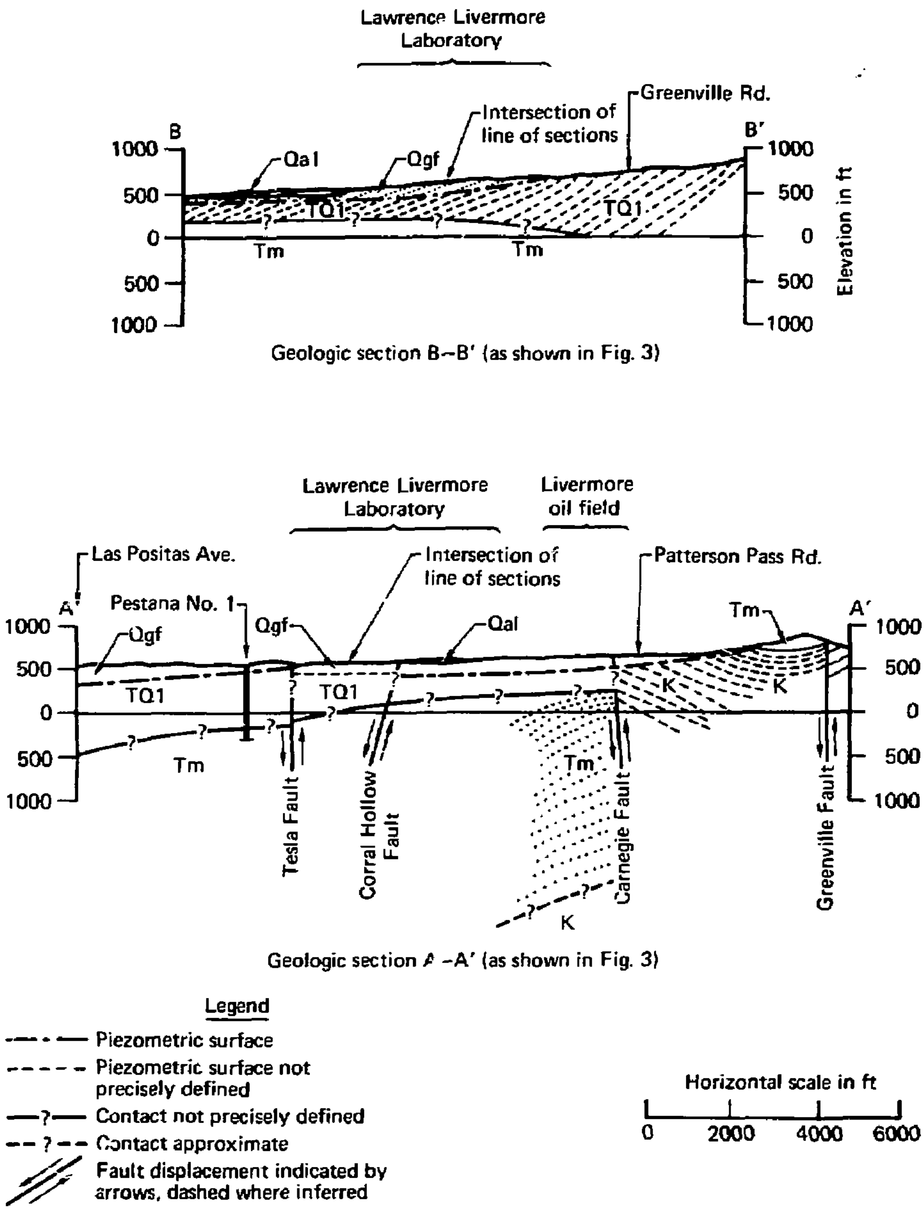

FIG. 4. Geological sections through the site (Ref. 2).

FAULTING AND SEISMICITY

The locations of the faults considered most pertinent to this study are shown in Fig. 5. Many of these faults transect the Livermore Valley in close proximity to the Laboratory, including the Tesla, Carnegie, Mocho, Greenville-Patterson Pass, and Corral Hollow faults. Also included are the San Andreas, Hayward, and Calaveras faults to the west. Not shown is the Las Positas fault currently under investigation by the USGS and John Blume and Associates. The presence of the Las Positas fault is not expected to change the DBE postulated for the Laboratory. These faults are all within the coastal portion of Central California, one of the most seismically active regions in the United States. Figure 5 also shows earthquake epicenters within a 60-mi radius of the Laboratory.

Historical earthquake records indicate that several destructive earthquakes have occurred along the San Andreas fault. The largest had a magnitude of 8.3* and occurred in 1906; the most recent was in 1957, with a magnitude of 5.3. Along the Hayward fault destructive earthquakes occurred in 1836 and 1868. Modified Mercalli intensities for these earthquakes are estimated at VIII and X. Along the Calaveras fault it appears that in 1861 there was a major earthquake with intensity estimated between VIII and IX.

Closer to the Laboratory there have not been any earthquakes larger than magnitude 4.5. In April 1943 an earthquake of magnitude 4.2 occurred in the vicinity of the Tesla fault near the intersection of East Avenue and South Vasco Road. In the same month two seismic shocks with magnitudes of 4.0 and 4.1 were recorded along the Mocho fault in the Livermore Valley.

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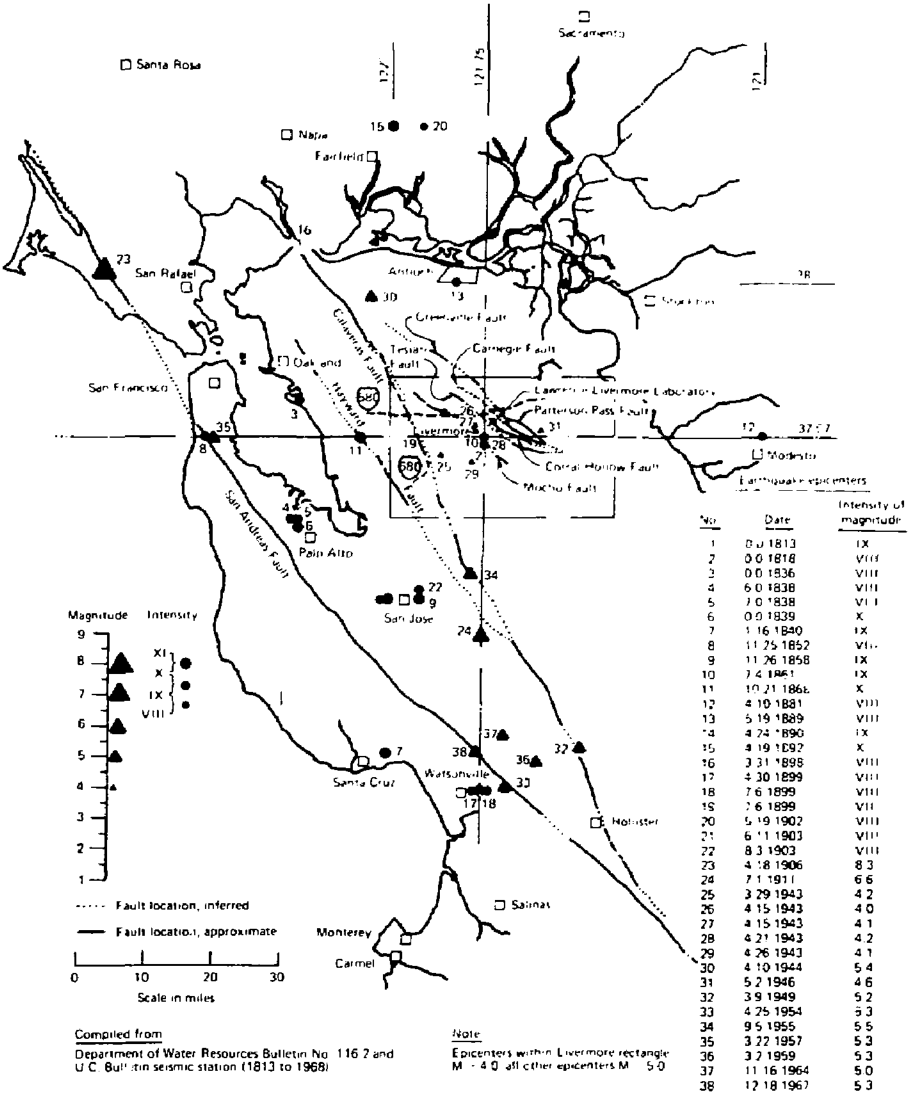
*Unless otherwise specified, all magnitudes in this report are based on the Richter scale.



No.	Date	Intensity or magnitude
1	0 0 1813	IX
2	0 0 1818	VIII
3	0 0 1836	VIII
4	6 0 1838	VIII
5	7 0 1838	VIII
6	0 0 1839	X
7	1 16 1840	IX
8	11 25 1852	VIII
9	11 26 1858	IX
10	7 4 1861	IX
11	10 21 1868	X
12	4 10 1881	VIII
13	5 19 1889	VIII
14	4 24 1890	IX
15	4 19 1892	X
16	3 31 1898	VIII
17	4 30 1899	VIII
18	7 6 1899	VIII
19	7 6 1899	VII
20	5 19 1902	VIII
21	6 11 1903	VIII
22	8 3 1903	VIII
23	4 18 1906	8 3
24	7 1 1911	6 6
25	3 29 1943	4 2
26	4 15 1943	4 0
27	4 15 1943	4 1
28	4 21 1943	4.2
29	4 26 1943	4 1
30	4 10 1944	5 4
31	5 2 1946	4 6
32	3 9 1949	5 2
33	4 25 1954	5 3
34	9 5 1955	5 5
35	3 22 1957	5 3
36	3 2 1959	5 3
37	11 16 1964	5.0
38	12 18 1967	5 3

FIG. 5. Seismicity and fault locations.

DESIGN BASIS EARTHQUAKE

We had to select a DBE before we could ascertain the structural integrity of the MFTF building. A DBE for the analysis and design of critical facilities at the Laboratory had previously been developed.[1,2] However, because of the noncritical nature of the MFTF experiments and the relatively short life of the program, this DBE was considered too severe. Based on a paper by D. L. Bernreuter[4] dealing with appropriate peak g values for noncritical facilities at the LLL site, we selected a peak ground acceleration of 0.25 g with the spectral frequency content remaining the same as the DBE spectrum developed for critical facilities at the Laboratory. This peak ground acceleration corresponds to an earthquake return interval of approximately 60 years.

RESPONSE SPECTRUM MODIFICATIONS

Once we had decided upon the basic shape and magnitude of the elastic response spectrum, we made additional modifications to account for the inherent damping and ductility of the structure. Because, for the purpose of this analysis, stress values at the yield level would be allowed, a value of 10% of critical damping was selected based on recommendations made by Newmark and Hall[5]. Table 1 is reproduced from Ref. 5 and indicates the recommended damping values for various structure types at various stress levels.

The major objective of the analysis of the MFTF structure was to ensure that the building would not collapse during ground motions at the level of the selected DBE. However, local failures and permanent deformations would be allowed. Based on this acceptance criteria, we decided to take advantage of the reserve inelastic capacity of the building by reducing the elastic response spectrum to account for the ductility inherent in the structure.

TABLE 1. Recommended damping values.

Type and condition of structure	Percentage of critical damping
Working stress, no more than about one-half yield point	
Vital piping	1 to 2
Welded steel, prestressed concrete, well-reinforced concrete (only slight cracking)	2 to 3
Reinforced concrete with considerable cracking	3 to 5
Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	
Vital piping	2 to 3
Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
Prestressed concrete with prestress left	7 to 10
Reinforced concrete	7 to 10
Bolted and/or riveted steel, wood structures, bolted joints	10 to 15
Wood structures with nailed joints	15 to 20

We investigated two methods of reducing the elastic spectrum to account for ductility. The first method was presented by Newmark and Hall[6] and is based on the inelastic response of a single degree of freedom system. The second method was presented by Montgomery and Hall[7] and is a modification of the first method for x-braced buildings. This modification results in a more conservative reduction because of the reduced hysteretic-energy-absorptive capacity of x-braced frames. As a conservative estimate, we used a ductility ratio of 2.0 in reducing the elastic spectrum. Figure 6 shows a composite plot of the 10%-damped elastic DBE spectrum, the same spectrum reduced for ductility using Newmark's method and finally using Hall's method. Because the main lateral force-resisting system in the MFTF building consists of x-braced frames, the more conservative Hall and Montgomery spectrum reduction was used for the analysis.

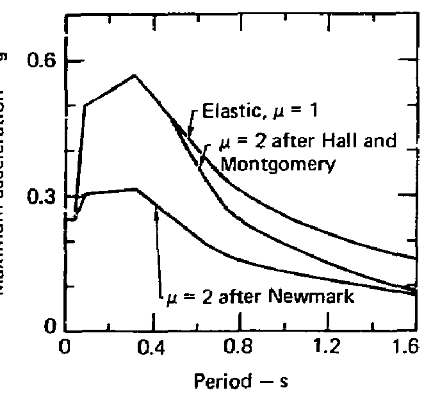


FIG. 6. 10%-damped DBE acceleration spectra.

EARTHQUAKE ANALYSIS

As previously mentioned, the symmetry of the building enabled us to model and analyze only one-half of the structure. The structure was analyzed by the response spectrum technique using the computer program SAP4.* A three-dimensional model of the structure that included beam, truss, and plate elements was used in the analysis. Figure 7 shows a computer plot of the structural model generated for the analysis. Truss elements were used to model the diagonal bracing system, beam elements to model the main framing system, and plate elements to simulate the floor and low bay roof diaphrams. Cross-sectional areas assigned to the truss elements were one-half of the actual areas of these members. This was done to correctly simulate the tension-only stiffness capacity of the actual x-bracing system. The truss elements used in the computer model can accept both tension and compression, thus the one-half area reduction used in the analysis has the net effect of simulating the true stiffness and lateral force capacity of the actual x-braced framing systems.

As stated earlier, riveted connections were used throughout the main framing system. These connections were assumed to be 100% moment-resisting. The connection of supporting columns to the pile foundations were modeled first as fully fixed and then as pinned about the weak axis of the column. The differences in responses from the two assumed support configurations were negligible. Soil-structure interaction was not included in the analysis model for the following reasons:

- For its size, the structure is relatively light and as such soil-structure interaction effects would be minimal.
- The inclusion of soil-structure interaction would have shifted the fundamental frequency of the structure to a lower value resulting in a decrease in spectral acceleration.

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*SAP4 is the Lawrence Livermore Laboratory version of the structural analysis program SAP IV by Bathe, Wilson, and Peterson, University of California, Berkeley, California.

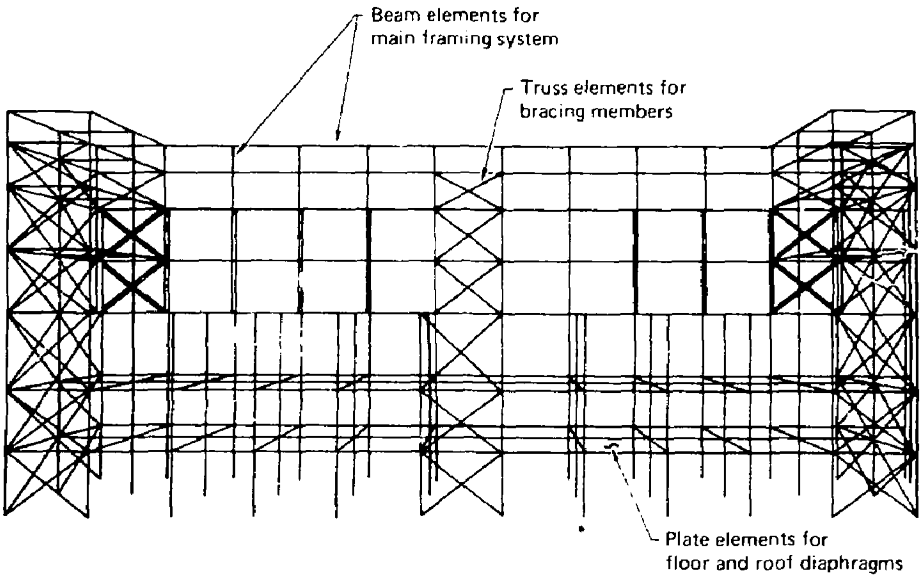

FIG. 7. Computer-generated plot of structural framing system.

Additional lumped masses were added to the model to account for the mass effect of the exterior metal sheathing and built-up roof system. No attempt was made to incorporate any additional stiffening of the structure caused by the presence of the exterior sheathing. Because of the way in which this sheathing is attached to the structure (sheet metal screws), we believe that its stiffness contribution is small when compared to the x-bracing systems, and that neglecting this effect introduces no significant error. Adjacent structures attached to the main MFTF building have been neglected in the analysis because these buildings are not structurally tied to the main framing system of the MFTF building.

The effect of equipment masses has been included in the analysis. Equipment masses and locations were based on proposed equipment layout drawings provided by MFTF project personnel. Equipment floor loadings are included in Appendix B. Because of the uncertainties associated with the final equipment layout, an additional loading configuration with a general floor loading plan was used and is also included in Appendix B.

The analysis was performed using three simultaneous components of response spectrum loading. The two orthogonal horizontal components were of 0.25-g maximum ground acceleration while the vertical component was two-thirds of this value. The vectoral sum of these three components is considerably greater than the 0.25-g maximum ground acceleration used as a basis for selecting the DBE. However, the response of the structure in its three orthogonal directions was essentially uncoupled so that the net effect of the three simultaneous input loadings was not significantly different from that of applying one direction of loading at a time.

The number of contributing modes required in the analysis was determined by making a comparison of analyses using 10 modes and four modes. The results indicated only very minor differences between the two analyses. Because of this, subsequent analyses were made using four modes, thus significantly reducing the computer time required.

ANALYSIS RESULTS

Tables 2 through 6 present the results obtained from the analysis. Table 2 summarizes the first four modes obtained in the analysis while Fig. 8 shows computer-generated plots of these mode shapes. Table 3 gives safety factors for the existing diagonal bracing systems in the north and south walls and Table 4 gives the same for the east and west walls. Table 5 presents the safety factors against buckling of the main support columns and Table 6 contains safety factors against uplift of the main support columns.

The results shown in the tables are based on the use of the generalized floor loadings. As can be seen, the current structure is not adequate to survive the postulated DBE.

TABLE 2. Period and direction of first 4 modes.

Mode	Period, s	Direction
1	0.82	East-west
2	0.73	North-south
3	0.50	East-west
4	0.47	North-south

TABLE 3. X-bracing in the north and south walls.

Description	Location, story	Load, thousands of pounds	Capacity,[a] thousands of pounds	Safety factor[b]
4 × 4 × 1/2	First	161	107	0.66
4 × 4 × 3/8	Second	120	78	0.65
4 × 4 × 5/16	Third	80	63	0.79

[a] Capacity of connections approximately equal to angle capacity.
[b] Based on yield capacity of angle section with one rivet hole subtracted from gross area.

TABLE 4. X-bracing in the east and west walls.

Description	Location, story	Load, thousands of pounds	Capacity,[a] thousands of pounds	Safety factor[b]
4 × 4 × 1/2	First	124	107	0.86
4 × 4 × 3/8	Second	91	78	0.86
4 × 4 × 5/16	Third	67	63	0.94

[a] Capacity of connections approximately equal to angle capacity.
[b] Based on yield capacity of angle section with one rivet hole subtracted from gross area.

TABLE 5. Column compressive loads and capacities.

Description	Location[a]	Load, thousands of pounds	Capacity,[b] thousands of pounds	Safety factor
	First story			
10 WF 45	15 B,F,G,&K	213	369	1.73
10 WF 45	15 A&L	274	354	1.29
10 WF 49	14 A&L	198	422	2.13
16 WF 88	12 A&L	215	747	3.47
	Second story			
10 WF 33	15 B,F,G.&K	126	289	2.29
10 WF 45	15 A&L	163	384	2.36
10 WF 49	14 A&L	135	444	3.29
10 WF 88	12 A&L	128	785	6.13
	Third story			
10 WF 33	15 B,F,G,&K	58	277	4.78
10 WF 33	15 A&L	78	264	3.38
10 WF 33	14 A&L	107	269	2.51
10 WF 88	12 A&L	61	751	12.[illegible]

[a]See Appendix A for location of columns. (South side analyzed, north side similar.)

[b]Based on AISC formulas with safety factors removed.

TABLE 6. Column uplift loads and capacities.

Description	Location	Load, thousands of pounds	Capacity, thousands of pounds	Safety factor[a]
10 WF 45	15 B,F,G,&K	101	83	0.82
10 WF 45	15 A&L	196	83	0.42
10 WF 49	14 A&L	136	83	0.61
16 WF 88	12 A&L	136	130	0.96

[a]Based on column anchor bolts capable of developing a working stress of 0.6 Fy with a 33% stress increase factor for earthquake loading.

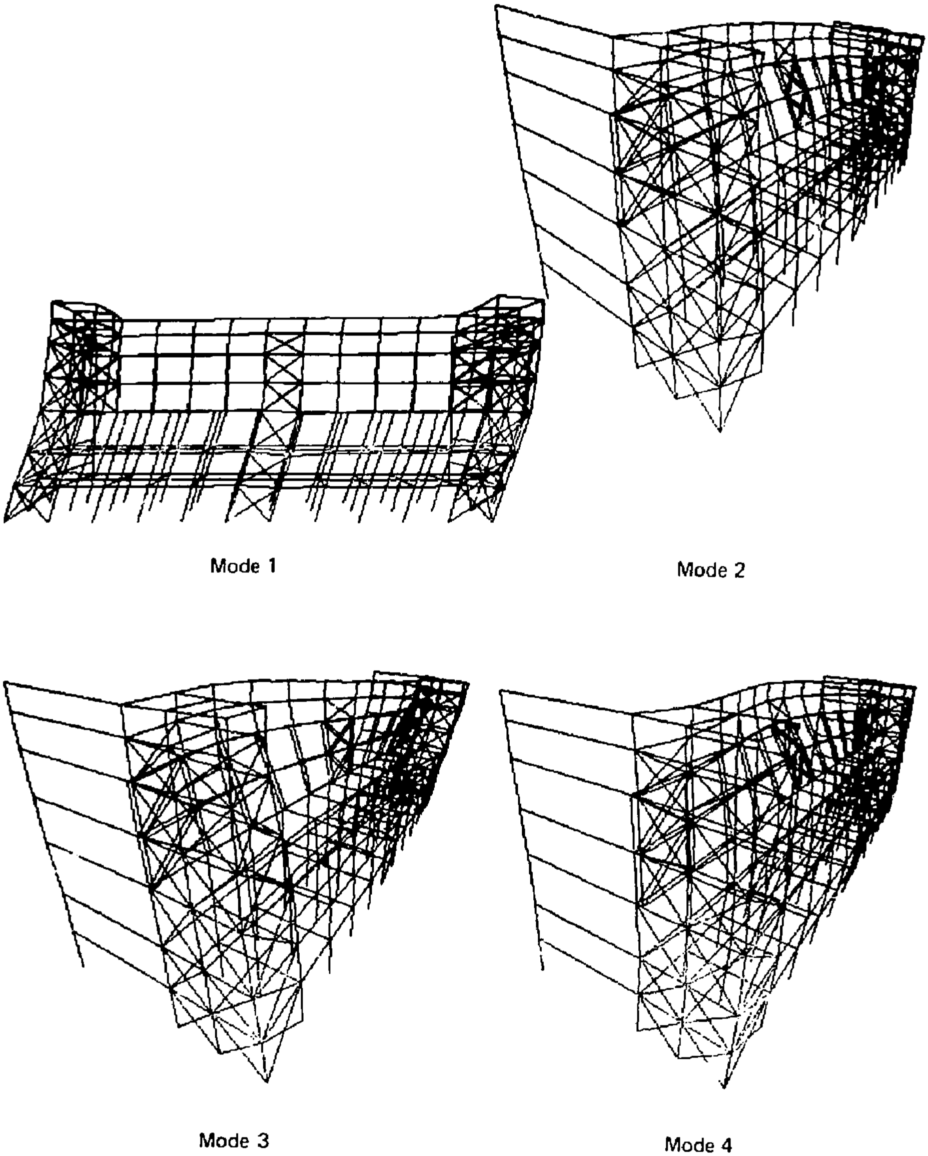

FIG. 8. First four mode shapes.

CONCLUSIONS AND RECOMMENDATIONS

The results of the analysis, based on the best estimate of equipment loadings, indicates that the diagonal bracing system and uplift anchorages must be upgraded for the structure to survive the postulated DBE. We proposed and analyzed several modifications and quickly discovered that simply increasing the number or size of the existing diagonal braces was not the most effective solution. In fact, it had the counter-productive effect of stiffening the structure and shifting the fundamental frequency toward the peak of the response spectrum. To strengthen the diagonal bracing system but not add additional stiffness to the structure, we decided to recommend high-strength cables to replace existing diagonals on the first two stories. Additional bays of bracing were also recommended to spread the load out and reduce the uplift problem. Even so, additional anchorage for the four corner columns would still be required.

Appendix C contains a description and location of the proposed modifications to the MFTF building (in its current configuration) to increase its seismic capacity to a level consistent with the proposed DBE. Also included are sketches of proposed connection details for the high-strength cable modifications.

Preliminary floor spectra for equipment design have been included in Appendix D. These spectra were generated using the Biggs[8,9] and Kapur[10] approximate methods for the building as modified structurally and with the latest equipment loadings.

ACKNOWLEDGMENT

The author wishes to acknowledge the assistance of Robert Murray for his work in providing the preliminary floor spectra for equipment design, and for his support throughout this project.

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APPENDIX A

SELECTED "AS-BUILT" DRAWINGS

The following selected drawings indicate the building layout, column locations, and main structural components of the MFTF structure.*

* A complete set of construction drawings can be obtained from Plant Engineering, Lawrence Livermore Laboratory, Livermore, California 94550.

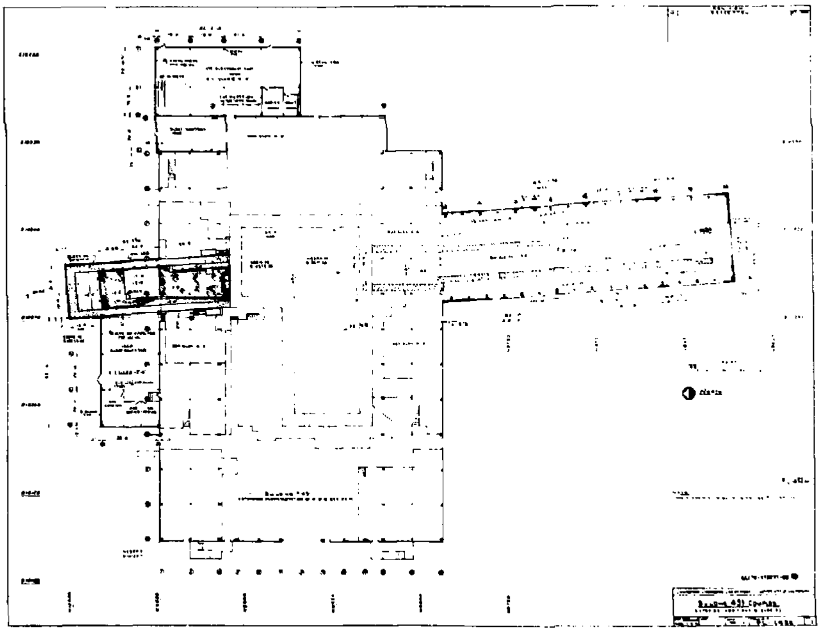


FIG. A-1. Building 431.

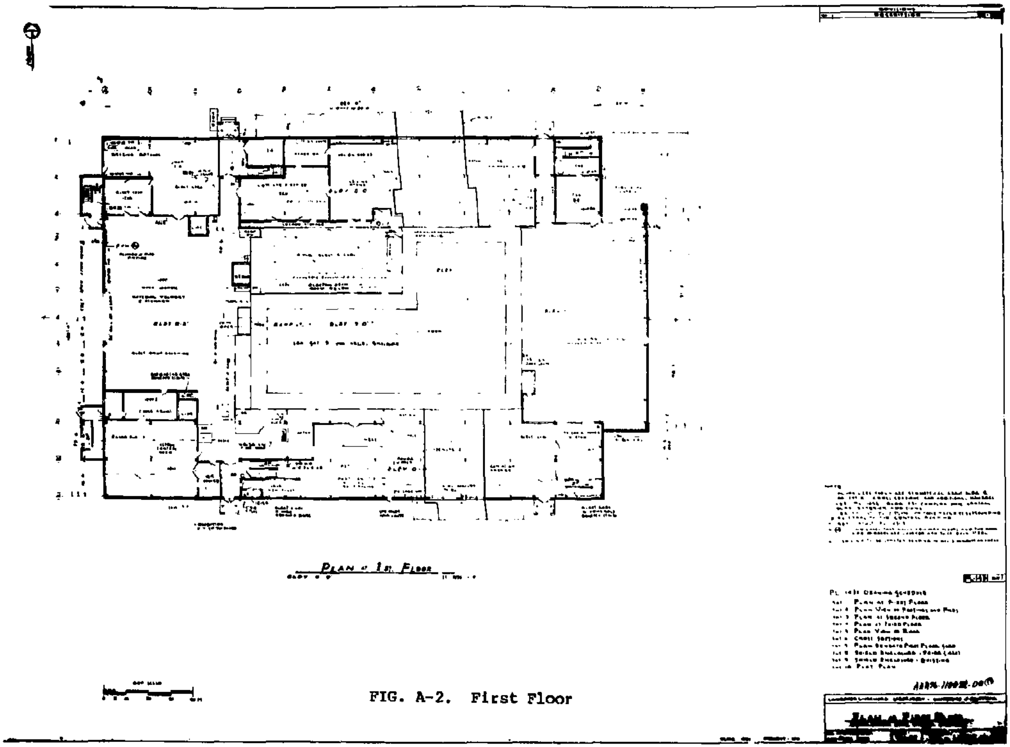


FIG. A-2. First Floor

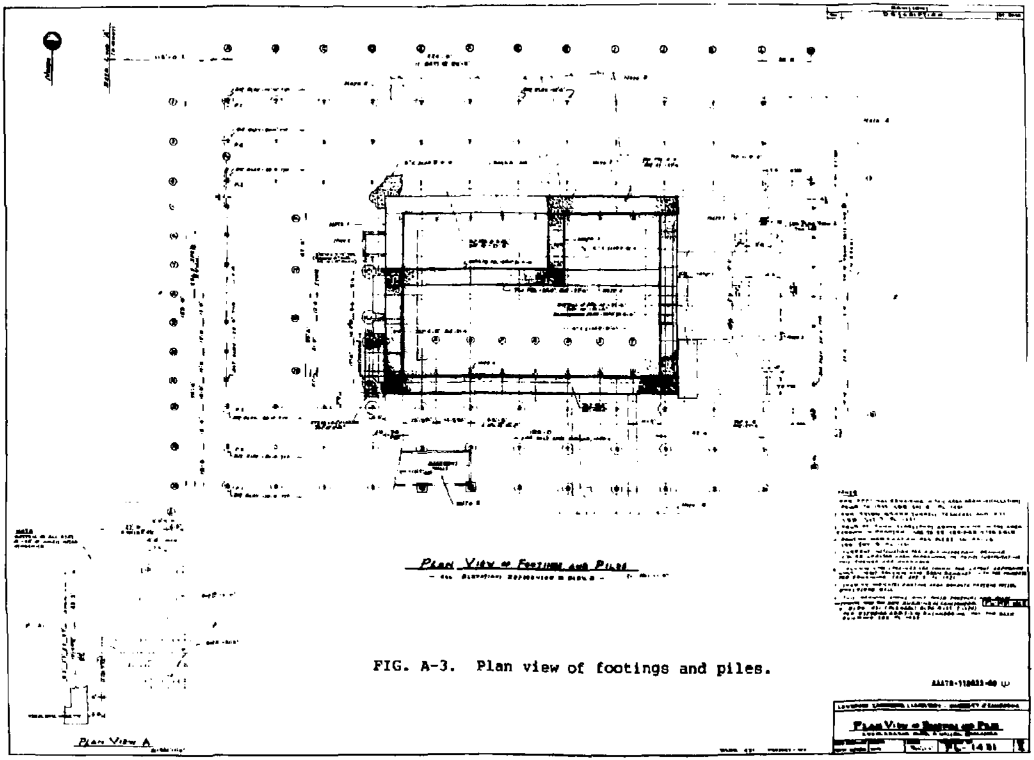


FIG. A-3. Plan view of footings and piles.

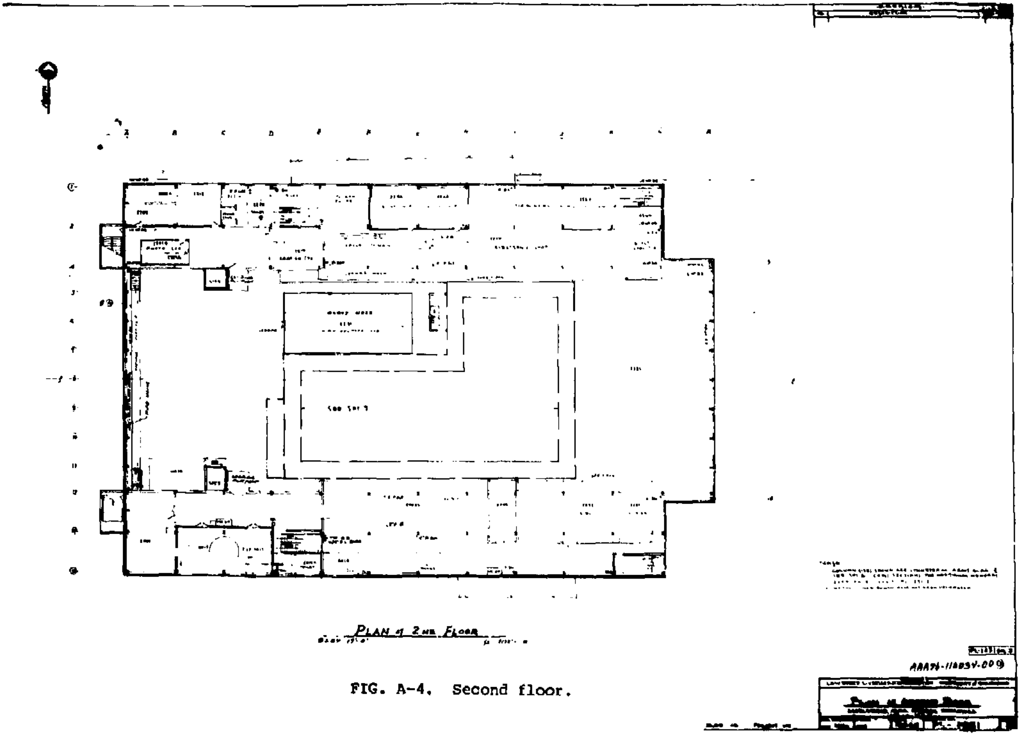


FIG. A-4. Second floor.

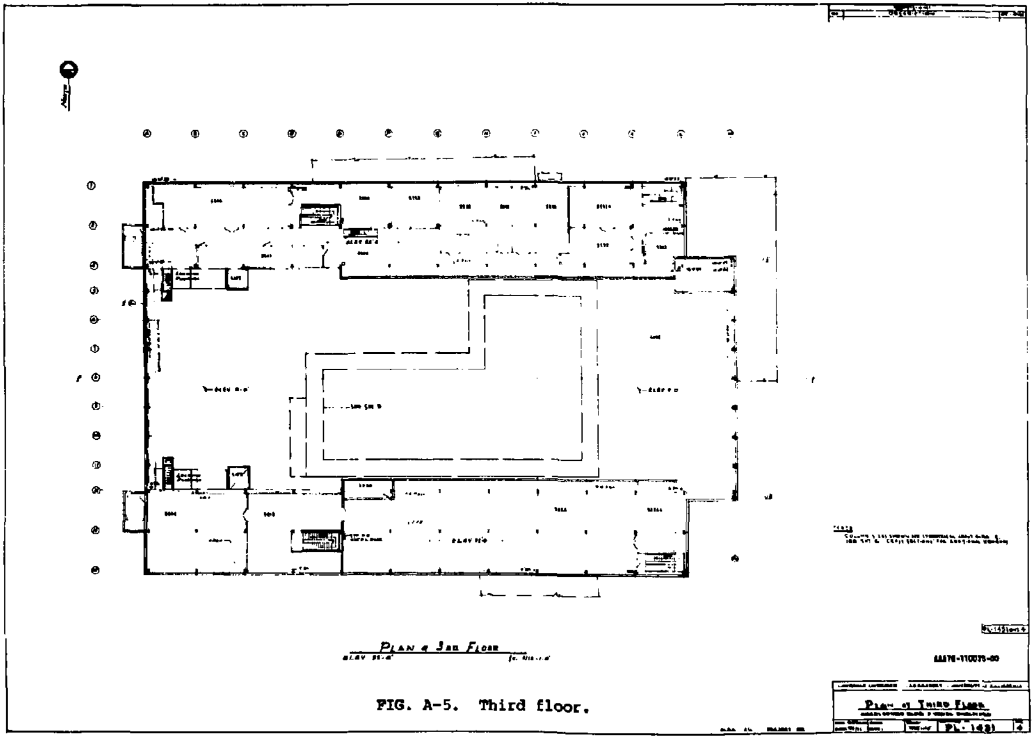


FIG. A-5. Third floor.

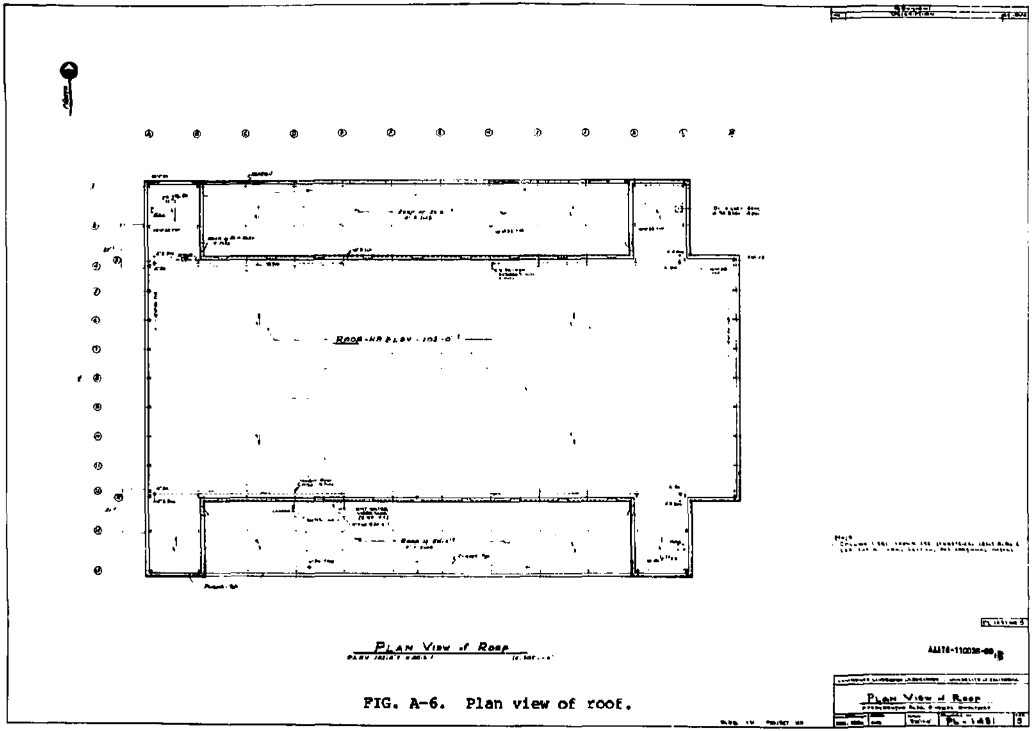
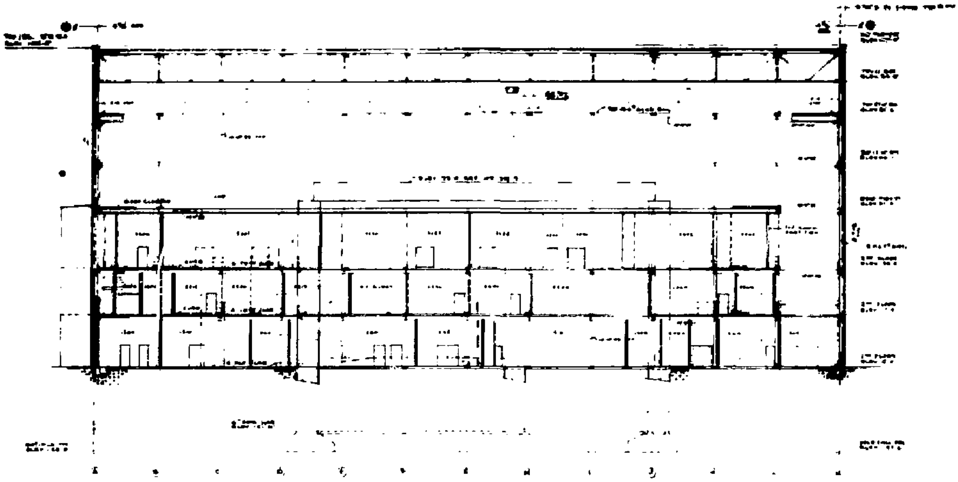


FIG. A-6. Plan view of roof.

(a)



(b)

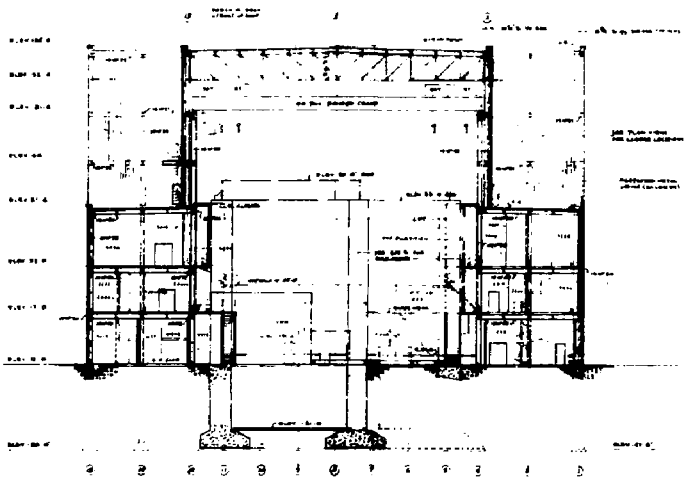


FIG. A-7. (a) Section looking north, (b) section looking west.

APPENDIX B

EQUIPMENT FLOOR LOADING PLANS

The following drawings show the equipment floor loading plans used in the analysis of the MFTF building. The first three drawings represent the location and equivalent uniform floor loadings of equipment as provided by MFTF project personnel. The last three drawings represent a conservative general equipment loading plan also used in the analysis. This generalized equipment loading plan was used because of the uncertainties associated with the equipment layout supplied by MFTF personnel.

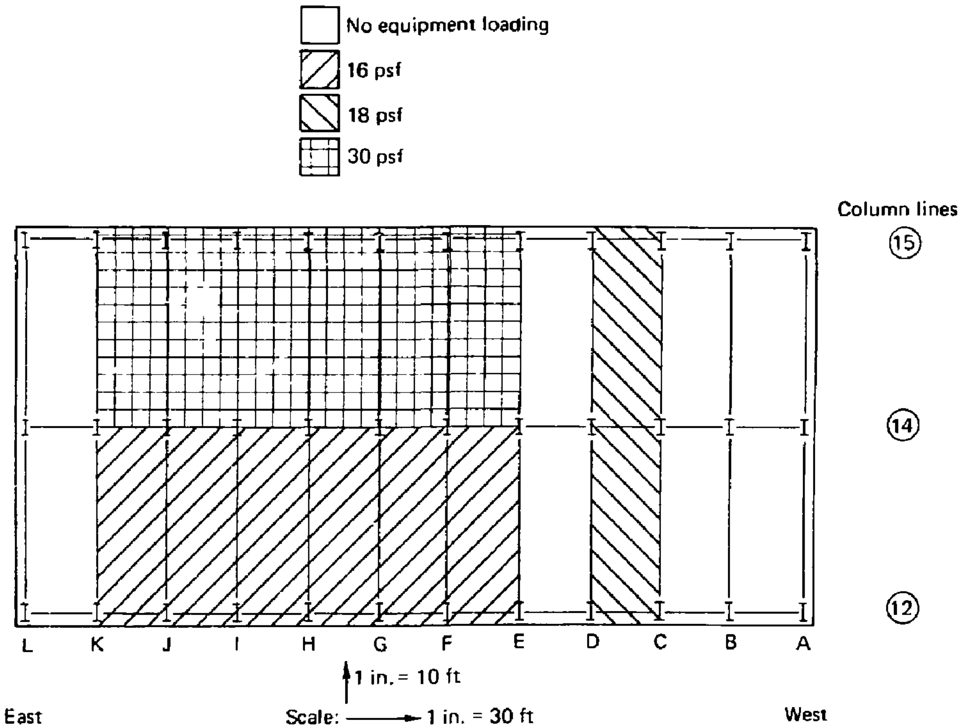


FIG. B-1. Second floor equipment loads (supplied by MFTF personnel).

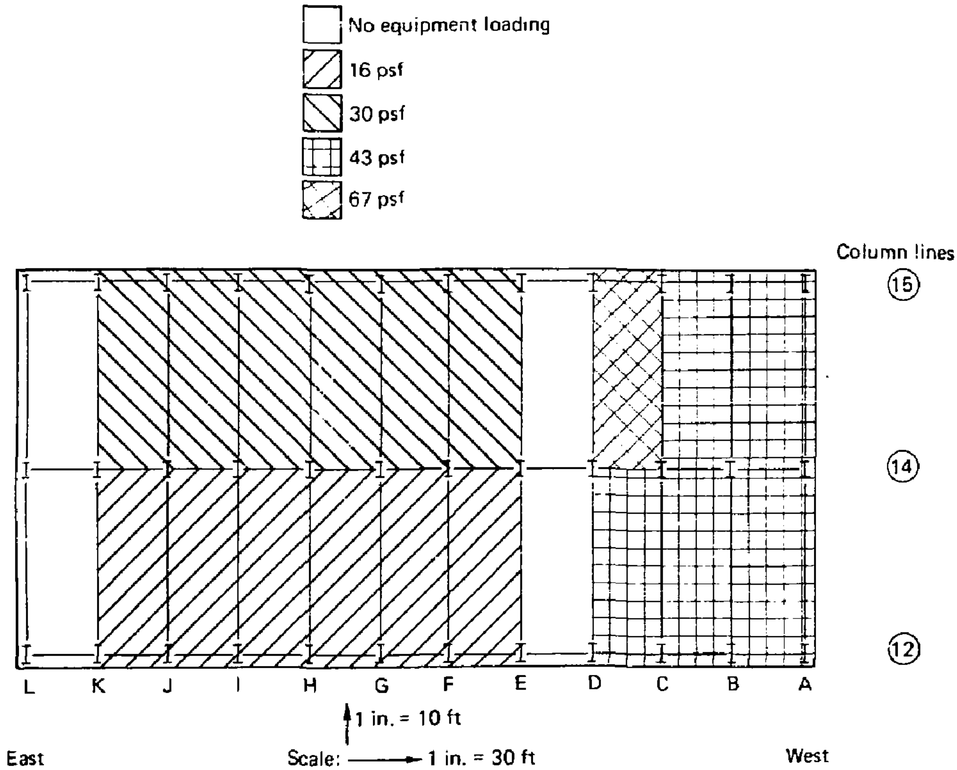


FIG. B-2. Third floor equipment loads (supplied by MFTF personnel).

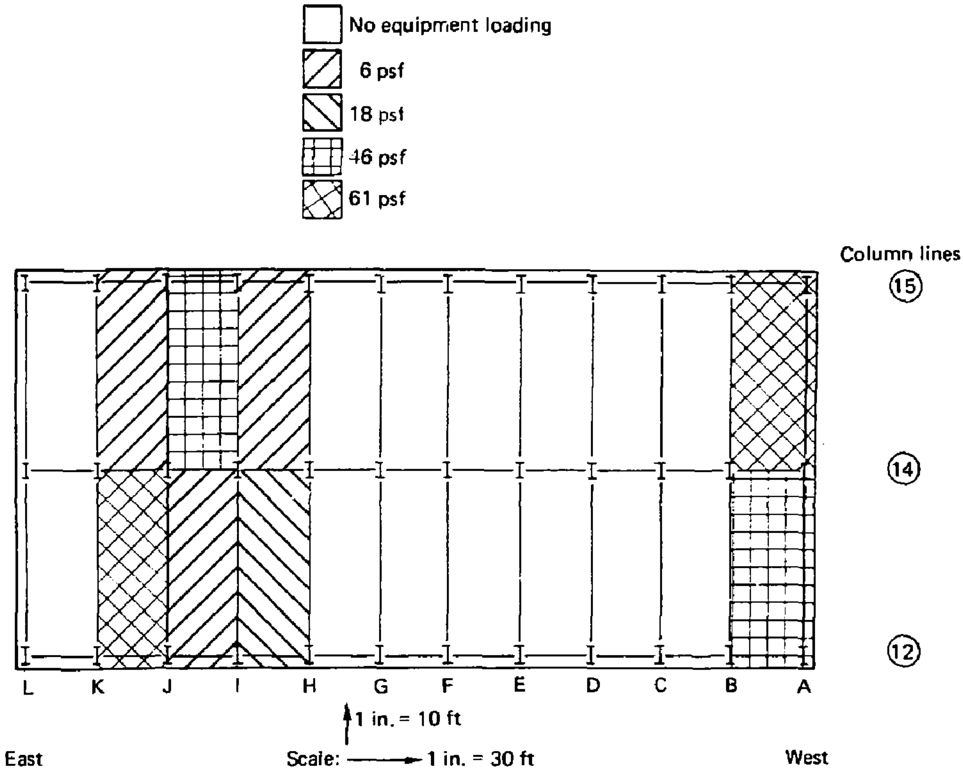


FIG. B-3. Low bay roof equipment loads (supplied by MFTF personnel).

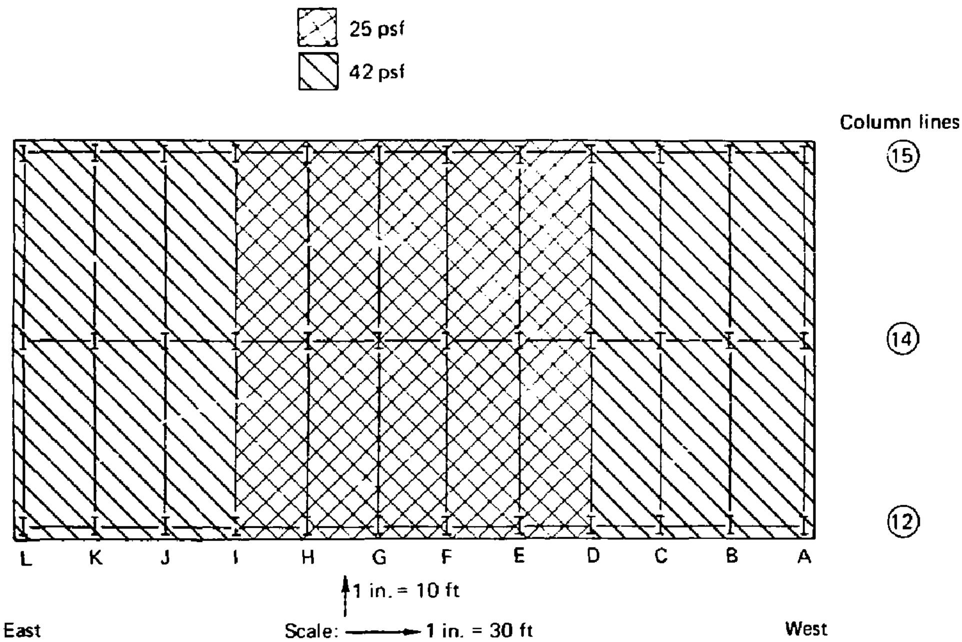


FIG. B-4. Generalized second floor equipment loads.

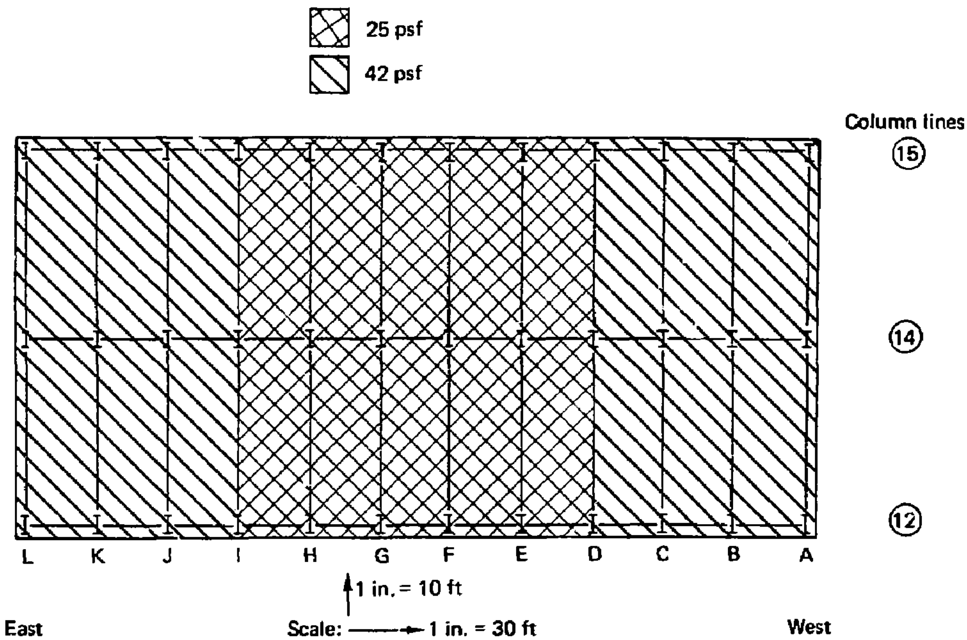


FIG. B-5. Generalized third floor equipment loads.

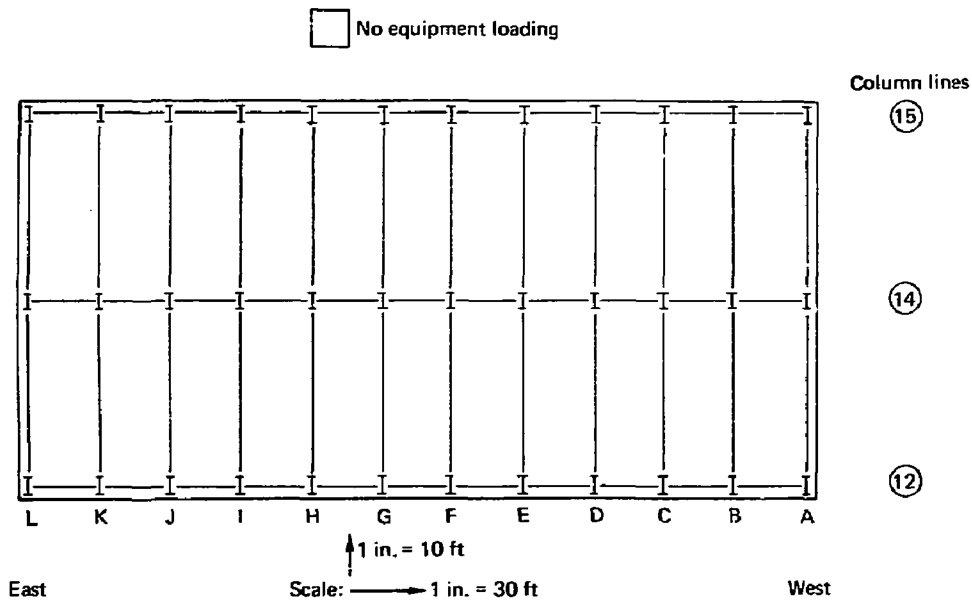


FIG. B-6. Generalized low bay roof equipment loads.

APPENDIX C

RECOMMENDED MODIFICATIONS

The recommended modifications to the MFTF building are discussed and their locations shown. Also shown are sketches of the proposed connection details for the high-strength cable modifications.

SOUTH SIDE OF STRUCTURE (See Fig. C-1)

First Story

Replace existing diagonal bracing on exterior wall (column line 15) with 1-1/4-in. o.d. high-strength cable. Add two additional bays of bracing with 1-1/4-in. o.d. high-strength cables.

Second Story

Same modifications as first story.

Third Story

Upgrade connection details of bracing on exterior wall by replacing existing rivets with high-strength bolts.

Tower Sections Above Third Story

No modifications required.

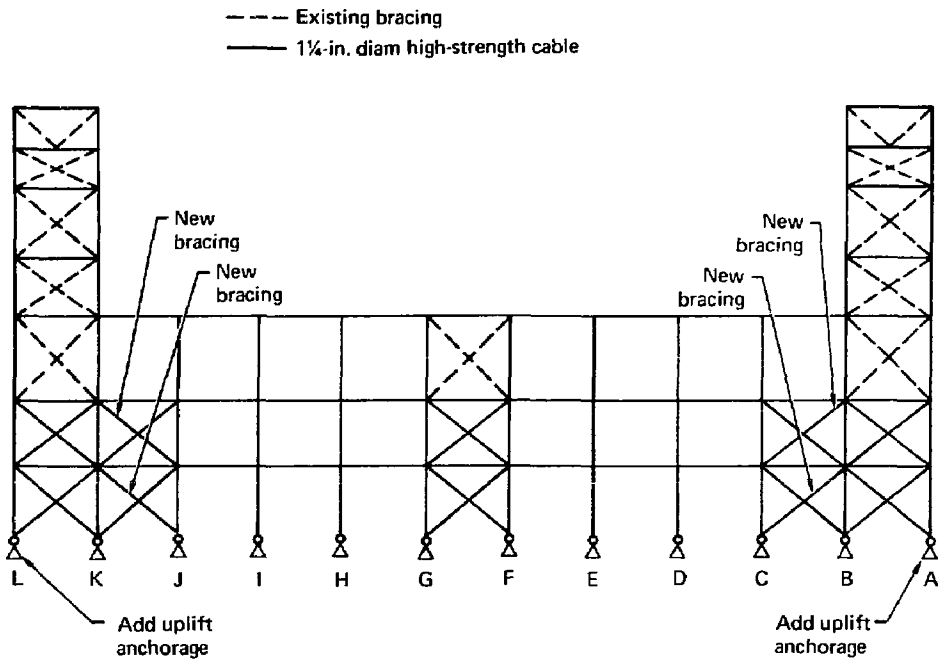


FIG. C-1. Column line 15 modifications.

NORTH SIDE STRUCTURE (See Fig. C-2)

First Story

No modifications needed if shield block structure is adequately tied (or can be made adequately tied) to the second floor slab. Otherwise, modifications similar to the South side would be required. We are not planning to evaluate this connection. We suggest Plant Engineering or the A&E Firm review the available details.

Second Story

Replace existing bracing (column line 1) with 1 1/4-in. o.d. high-strength cables.

Third Story

Upgrade connection details of bracing on exterior wall by replacing existing rivets with high-strength bolts.

Tower Sections Above Third Story

No modifications required.

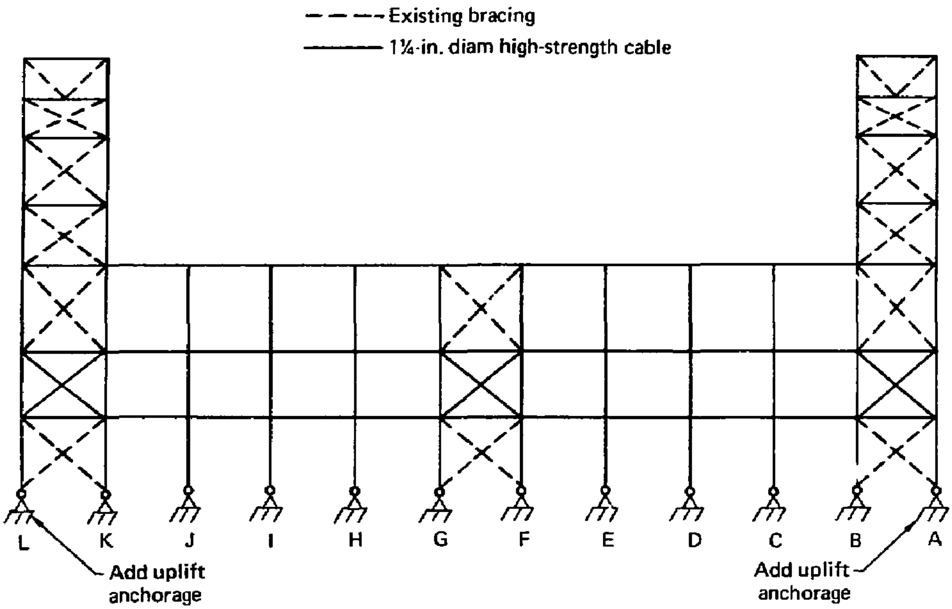


FIG. C-2. Column line 1 modifications.

EAST SIDE OF STRUCTURE (See Fig. C-3)

First Story

Replace existing diagonal bracing on exterior wall with 1 1/4-in. o.d. high strength cables.

Second Story

Same modifications as on first story.

Third Story

Upgrade connection details of bracing on exterior wall by replacing existing rivets with high-strength bolts.

Tower Sections Above Third Story

No modifications required.

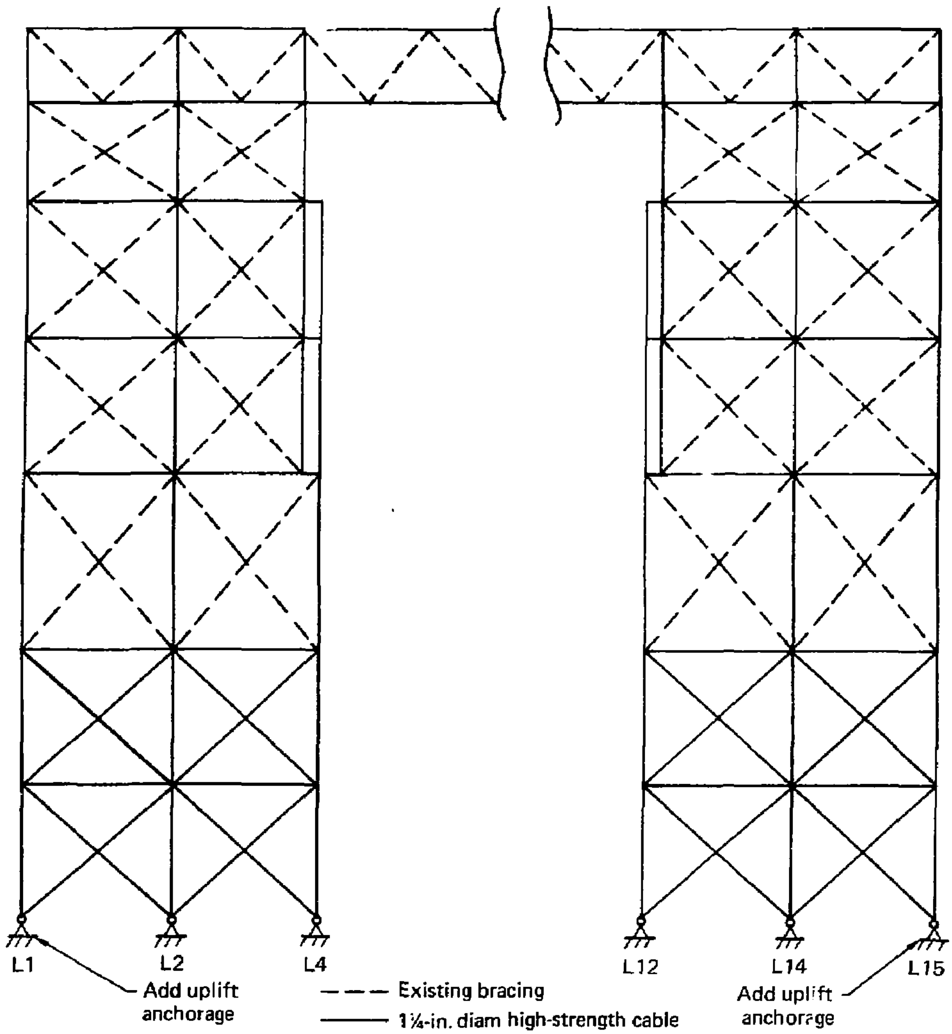


FIG. C-3. East wall modifications.

WEST SIDE OF STRUCTURE (See Fig. C-4)

First Story

Replace existing diagonal bracing on exterior wall with 1 1/4-in. o.d. high-strength cables.

Second Story

Replace existing diagonal bracing in bays A1 - A2 and A14 - A15 with 1 1/4-in. o.d. high-strength cables (Fig. C-5). Upgrade connection details of diagonal bracing in bays A2 - A4 and A12 - A14 by replacing existing rivets with high-strength bolts.

Third Story

Upgrade connection details of bracing on exterior wall by replacing existing rivets with high-strength bolts.

Tower Section Above Third Story

No modifications required.

UPLIFT ANCHORAGE

Provide positive tie-down system to anchor columns A1, A15, L1, and L15 against uplift forces occurring as a result of overturning moments produced during a seismic event.

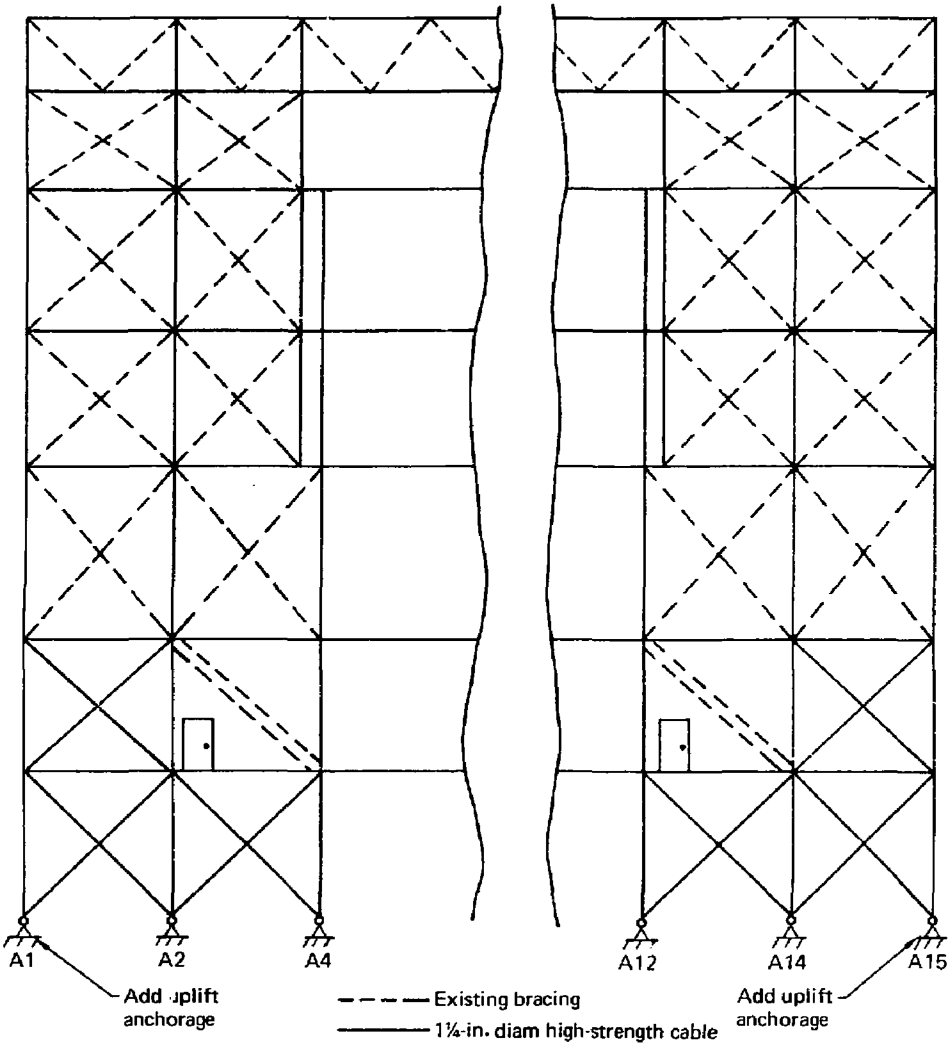


FIG. C-4. West wall modifications.

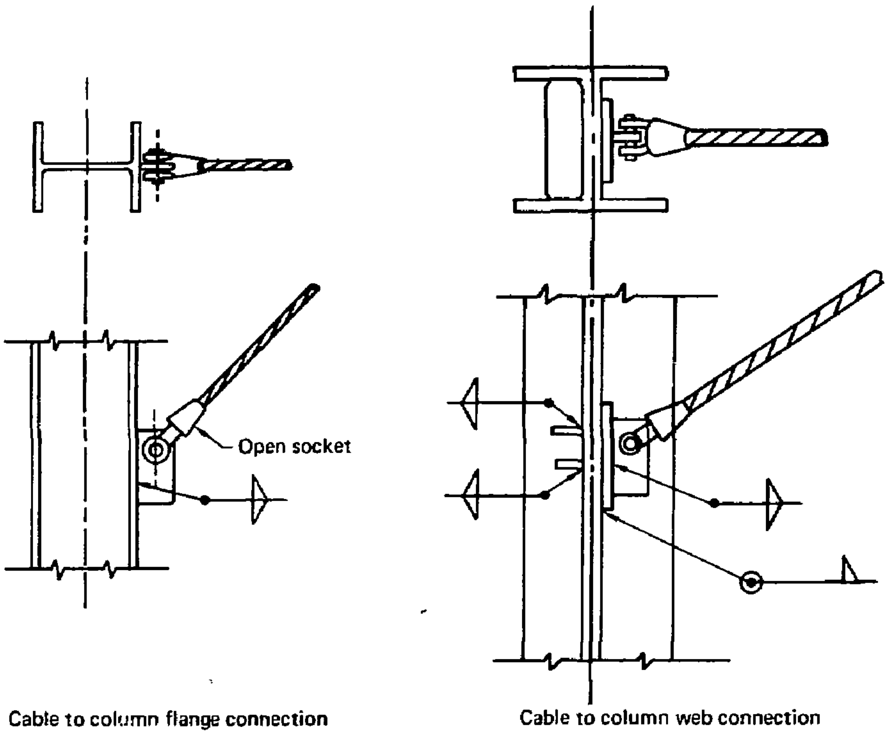


FIG. C-5. Proposed connection details for high-strength cables.

SPECIFICATIONS FOR NEW MATERIAL

One and 1/4-in. o.d. high-strength cables with a minimum breaking strength of 180 kips, and a minimum modulus of elasticity of 24,000,000 psi. High-strength bolts of ASTM A325 or equivalent are necessary.

GENERAL

A visual inspection of beam-column connections, column splices, diagonal bracing connections and anchorage details should be made prior to completion of building modifications to ensure these details will function as intended.

APPENDIX D

FLOOR SPECTRA

Floor spectra for equipment design were developed by both the Biggs and Kapur approximate methods for the MFTF building with the recommended upgrading modifications incorporated and with the latest equipment loadings. Horizontal floor spectra are presented in Figs. D-1 through D-3 for the second, third, and fourth floor levels respectively for 1%, 2%, and 5% equipment damping. Vertical spectra may be obtained by multiplying the horizontal spectral accelerations by two-thirds.

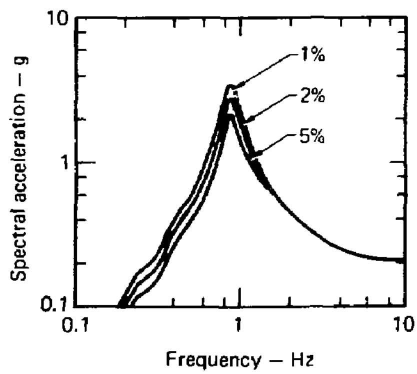


FIG. D-1. Floor spectra, second floor.

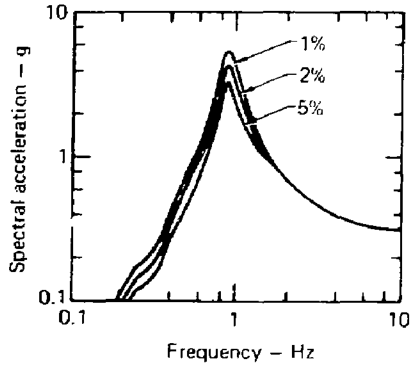


FIG. D-2. Floor spectra, third floor.

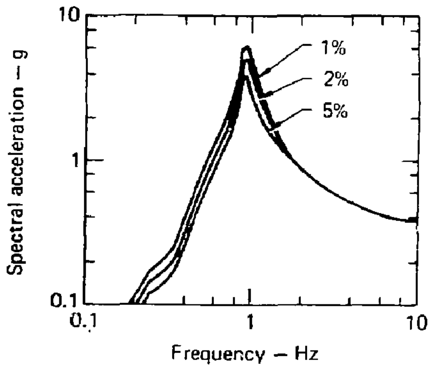


FIG. D-3. Floor spectra, fourth floor.