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MODIFICATION OF THE COLONY TOWER
FOR
PROJECT RIO BLANCO

by

John A. Blume
Sigmund A. Freeman
Kenneth K. Honda
Lloyd A. Lee

John A. Blume & Associates
Research Division
San Francisco, California

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MODIFICATION OF THE COLONY TOWER FOR PROJECT RIO BLANCO

CONTENTS

TABLES

FIGURES

FIGURES (Continued)

ABSTRACT

The Colony Tower, a 180-foot tall steel-frame experimental oil shale processing retort structure with heavy process equipment on various levels, is situated approximately 29 kilometers southeast of the RIO BLANCO emplacement well. The RULISON detonation of September 1969 was about the same distance southeast of the tower.

During Project RULISON, recordings of ground and structure response motions were obtained. Recorded motions showed greater responses in the higher frequency ranges than anticipated. Based on this experience and the predicted RIO BLANCO ground motion, it was determined that additional study to evaluate the expected tower performance during Project RIO BLANCO was necessary.

Utilizing response spectrum predictions for RIO BLANCO, subsequent analyses showed that several bracing members would be subjected to forces exceeding their yield strength and some would reach a level at which failure could occur. Further analyses were made with assumed bracing members installed and a final scheme for modified vertical bracing was established.

Modification, which consisted of fabricating and placing approximately 21 tons of structural steel and 750 pounds of welding electrode, was accomplished during the period from April 30 to May 14, 1973. To obtain measurements and indications of ground and structure response motions, 25 channels of L-7 velocity meters were installed, 19 of which were placed on the structure. No evidence of any damage was seen during the immediate post-detonation inspection of the tower.

The modification of the Colony Tower and the procedures used to determine these modifications exemplify the utility of current ground motion and structural response prediction technology for forecasting dynamic behavior of important structures subjected to ground motion from underground nuclear explosions.

INTRODUCTION

The Colony Tower is a 180-foot tall steel-frame experimental oil shale processing retort structure with heavy process equipment on various levels. Lateral forces are carried by a horizontal floor brace system to vertical diagonal bracing in the exterior frames. The emplacement well location for the RIO BLANCO detonation is approximately 29 kilometers northwest of the tower. The Project RULISON emplacement well is about the same distance southeast of the tower.

Prior to the RULISON detonation, a brief structural-dynamic study was made of the tower structure of The Colony Development Site (TOSCO), based on rough field measurements obtained from a field trip on March 21, 1969 (Ref. JAB-99-78, pp. 96 and 98). The results of the evaluation indicated that the predicted detonation-caused motion from RULISON (response spectra from Environmental Research Corporation -- ERC) would account for stresses less than 25% of those allowable. Therefore, no special precautions were recommended, other than evacuation of the facility because of the rockfall hazard. The above evaluation was noted to be approximate at best because of the lack of data. These data were not available because they were company-classified as proprietary. The evaluation was based on first mode response. The analysis indicated that the higher mode responses (e.g., 2nd and 3rd modes) would not be critical.

After the RULISON detonation, a review of the recorded motion was made. The results of this review indicated that the 1st mode responses were less than predicted but the 2nd and/or 3rd mode responses were substantially greater than anticipated. It was approximated that the higher mode responses may have resulted in stresses close to allowable design code stresses (Ref. JAB-99-78, pp. 129-130). A recommendation was made that it would be advisable to make a detailed analysis if future detonations were to be planned that would result in ground motion equal to or greater than RULISON.

As part of a structure bracing program for Project RIO BLANCO, CER-Geonuclear (CER) employed the services of John A. Blume & Associates, Engineers (BLUME), through an existing contractual relationship with the U.S. Atomic Energy Commission (AEC), to conduct structure response analyses and to design and implement a structural bracing program for the Colony Development Oil Shale Retort Tower (Figure 1), situated on Parachute Creek near Grand Valley, Colorado (Figure 2).

The analysis of the Colony Tower to predicted RIO BLANCO ground motion was based on structural data supplied by Colony Development and The Oil Shale Company (TOSCO) and on response spectrum predictions provided by ERC for the median predicted ground motion and corresponding standard geometric deviations. The structure was analyzed with the aid of a two-dimensional computer program (FRMSTC). A three-dimensional computer program (SAP) was used to check the results.

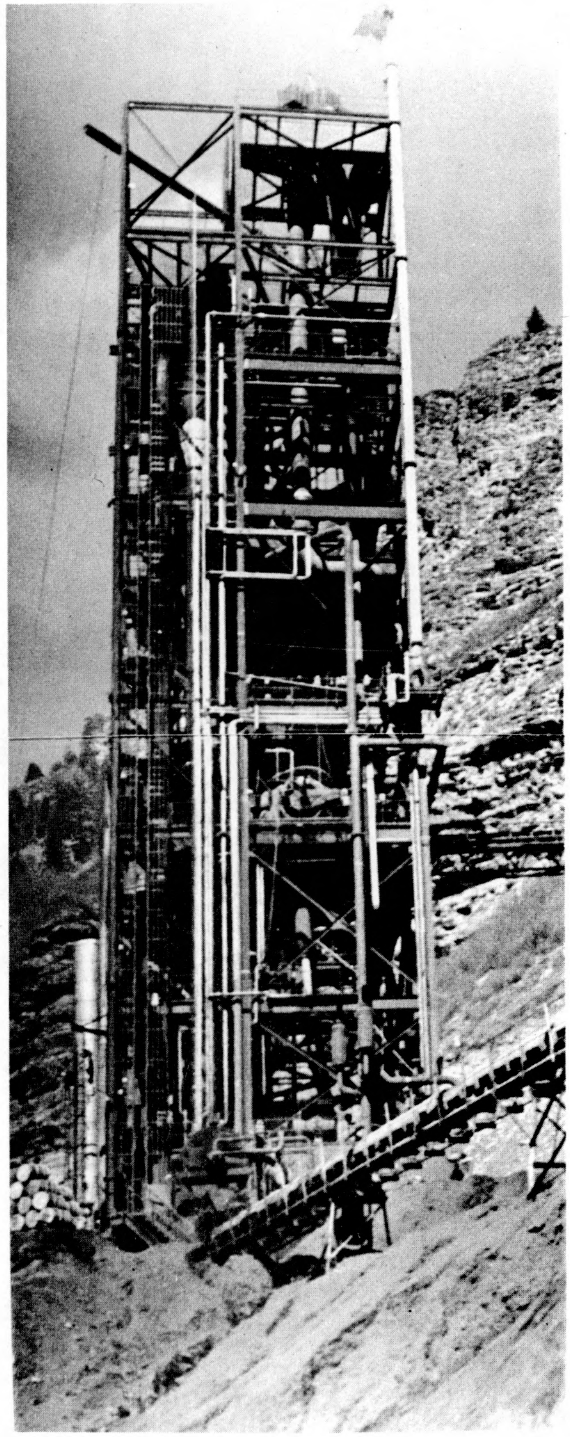


FIGURE 1 COLONY TOWER

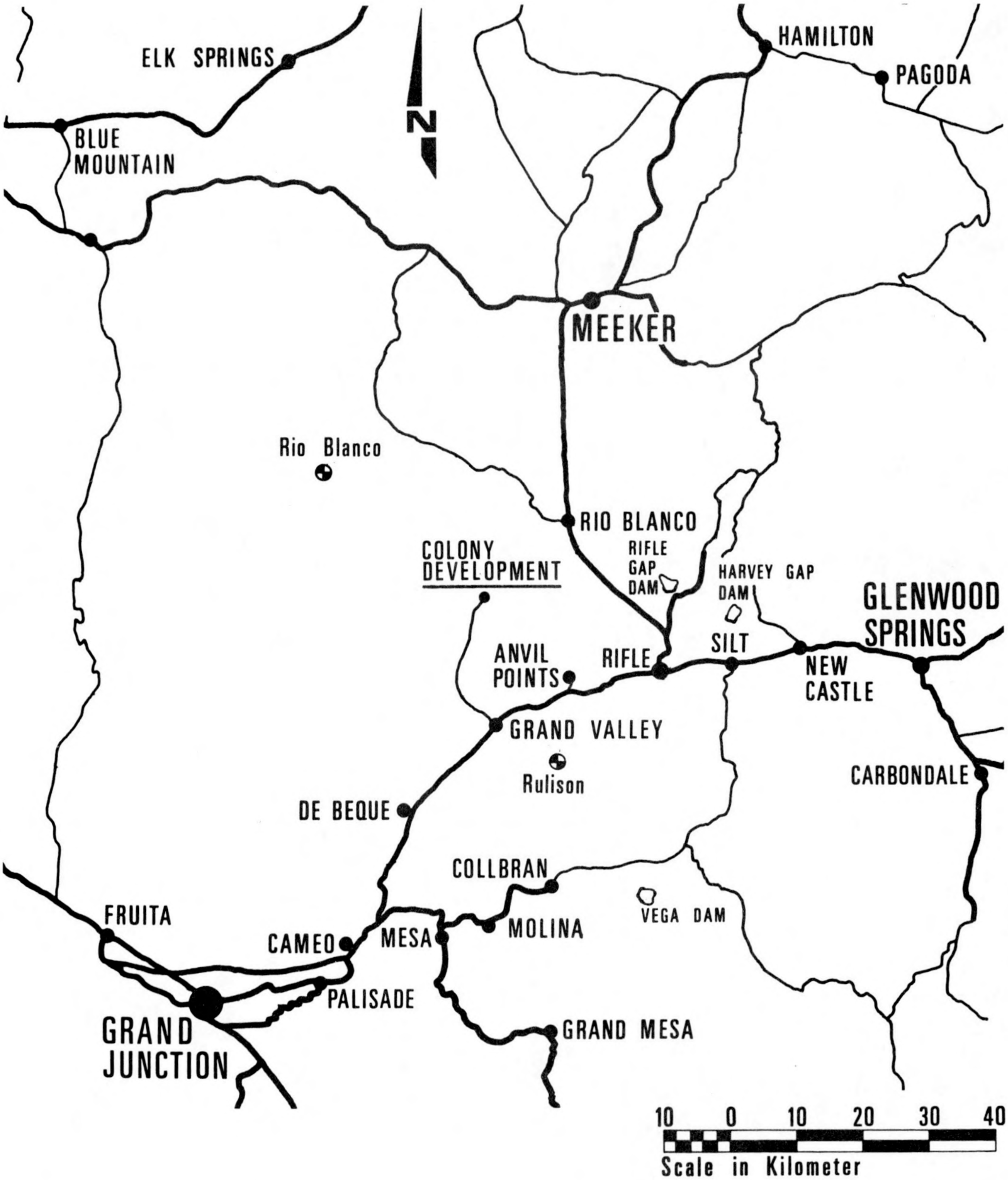


FIGURE 2 GENERAL AREA MAP

CRITERIA FOR ANALYSIS AND DESIGN MODIFICATIONS

Criteria for the analyses and the design modifications which would provide an adequate margin of safety were agreed upon by AEC and CER. The main provisions of the criteria are the following:

- Analysis and design were based on one geometric standard deviation ($+1\sigma$) times the median predicted response spectrum (Figure 3).
- The median predicted response spectrum was derived from the maximum values of two orthogonal directions of horizontal motion.
- Damping was assumed to be 2% of critical.
- Natural periods of vibration were considered to have a ±10% variation from the calculated value. The maximum spectral response value within this period band width was used as the demand value.
- The capacity of the structural members were based on the yield limit of the material. Where the yield limit was difficult to define (e.g., friction connections of high strength bolts), the capacity was taken at 1.5 times the allowable working stress.

The $+1\sigma$ level of predicted response spectra is considered to provide an 84% probability of not being exceeded. The median prediction was based on the maximum values of two orthogonal directions of horizontal motion because of the uncertainties of directional effects. The 2% of critical damping criterion was considered to be a reasonable compromise for design purposes. Field measurements indicated 1.25% to 2% of critical damping at small amplitudes and experience indicated that damping (or an effective damping value to relate to the response) would probably increase at high amplitudes of motion to as high as 5% of critical damping. The ±10% variation in period was used to account for complexities of the structure and uncertainties in masses and mass distribution.

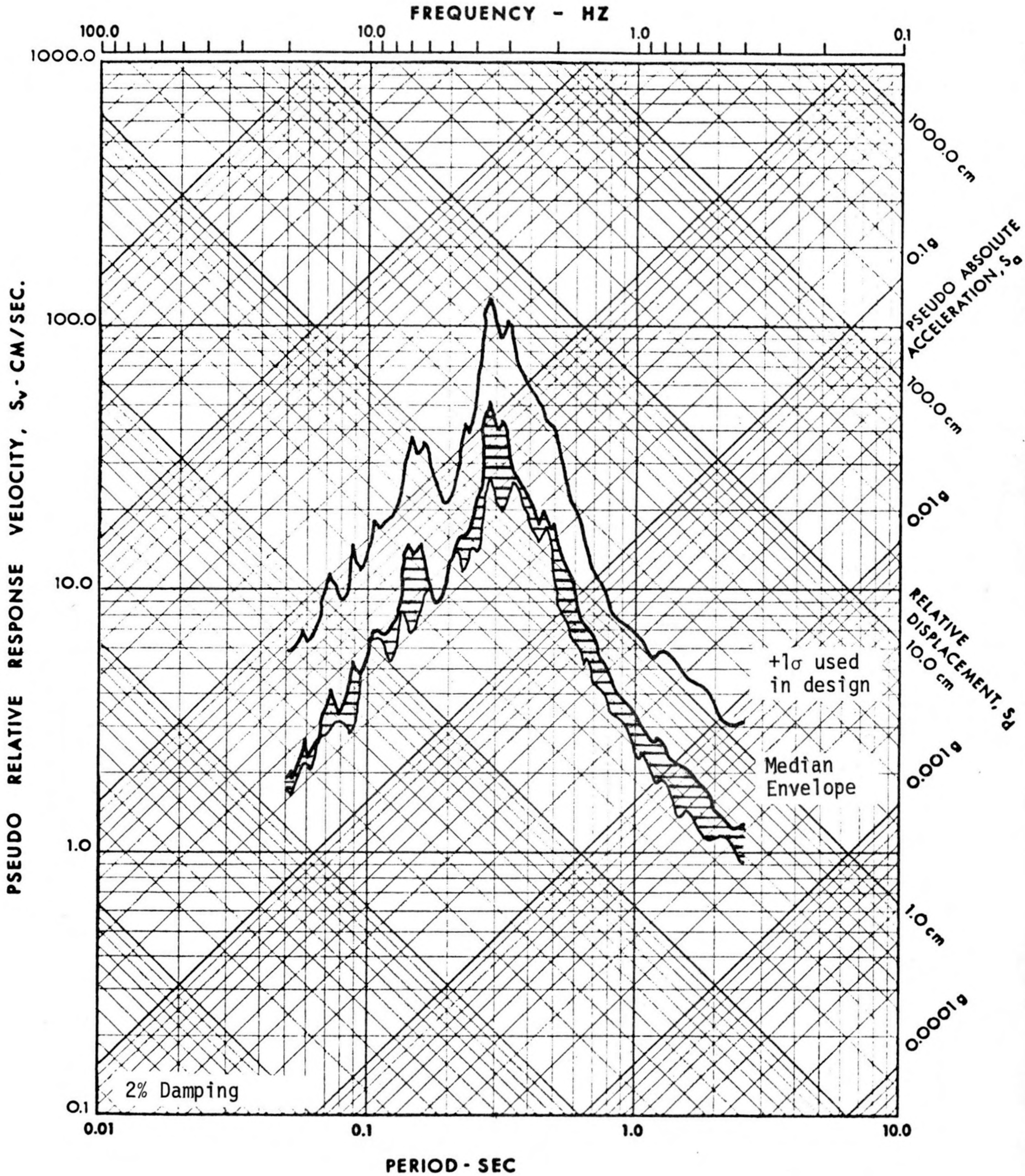


FIGURE 3 MEDIAN AND DESIGN (+1σ) SPECTRA

RESPONSE EVALUATION

Selection of design criteria for the modification of the tower structure was made with a view toward assuring that no damage would occur to the structure. Consequently, these criteria established a reasonable but clearly conservative approach to the prediction of response and stress in tower elements.

Therefore, in order to make a meaningful comparison of actual response (Figures 8 and 9) with predicted responses (Figure 3), it is necessary to compensate for the conservative weighting of the pre-detonation design criteria and assumptions. The resulting values are called "unweighted" and were obtained as follows:

- Instead of using the spectral values at the upper statistical level of $+1\sigma$, a value was selected which represented an average between the upper and lower bounds of the median response spectra envelope derived from the two orthogonal directions of horizontal motion. This average value is the one with the highest probability of actual occurrence.

- Instead of using the highest response value in the ±10% range of each of the calculated natural periods, a value was selected which represented an average value within the ±10% period range.

- Damping was assumed to be 2% of critical, as before.

Table 1 shows a comparison of predicted tower response, using three different sets of prediction assumptions, with actual recorded values. The prediction assumption sets are (1) the conservative pre-detonation predictions, weighted to assure the tower's safety, (2) the somewhat less conservative pre-detonation predictions using the median prediction of motion instead of the $\pm1\sigma$ level but retaining other safety aspects, and (3) the unweighted and statistically most probable response value selected as described above.

All of the above criteria tended to increase the probability (above the 84% value) that the design criteria would not be exceeded by the RIO BLANCO detonation. Some of this apparent conservatism was offset by necessary approximations in the structural analysis which may result in underestimation of stresses in some structural elements and overestimation of stresses in others.

The object of the modification program was to assure, within reasonable limits, that no damage would occur to the structure. Therefore, the modification design criteria should not be confused with a predicted response solution.

The results of the analysis of the then existing structure are illustrated in Figures 4 and 5 by showing the peak calculated story shears. As is shown in Figures 4 and 5, the median predicted motion would have caused overstress at the top level in the north-south direction only.

Motion at the $+1\sigma$ level would have caused overstress at the top three levels and the bottom three levels in the north-south direction and top three levels and bottom level in the east-west direction. To provide an adequate margin of safety, CER directed Blume through the AEC contractual relationship to proceed with a program to design the installation of adequate permanent bracing and to modify existing bracing to assure that no significant overstress would occur at the $+1\sigma$ level.

An initial bracing program was designed and the probable effects on the response of the tower determined by analytical means. This procedure was repeated until the calculated response and stresses satisfied the design criteria. Upon completion of the two-dimensional analysis (using computer program FRMSTC), a three-dimensional analysis (using computer program SAP) was performed on the final modified structure as a check.

The results of the analysis of the modified structure are illustrated in Figures 6 and 7 by showing the peak calculated story shears. A comparison of the pre-modified structure results and the modified structure results (Figures 4 and 5 versus Figures 6 and 7) shows that some

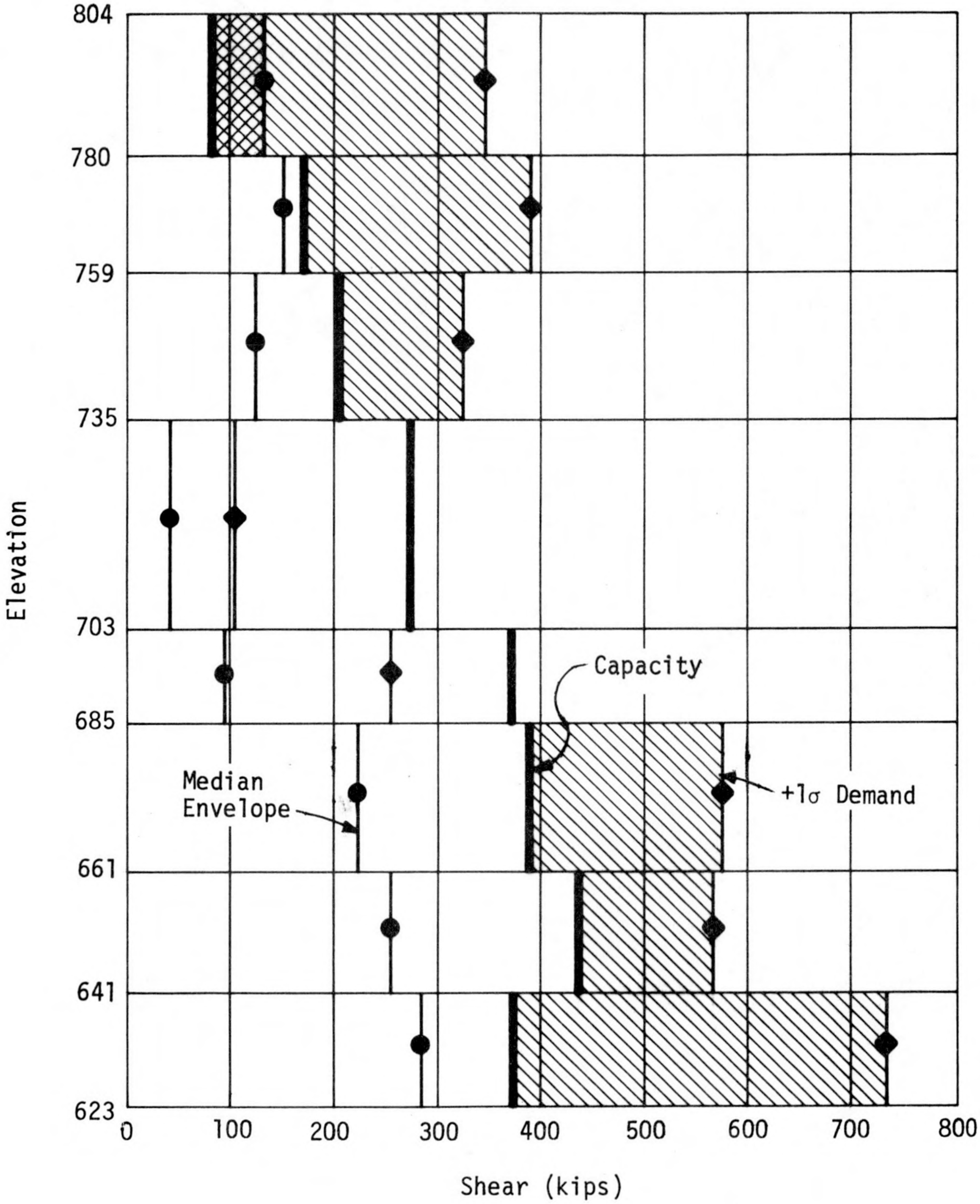


FIGURE 4 COMPARISON OF CAPACITY AND DEMAND STORY SHEAR LONGITUDINAL (N/S) MODEL L2-MO

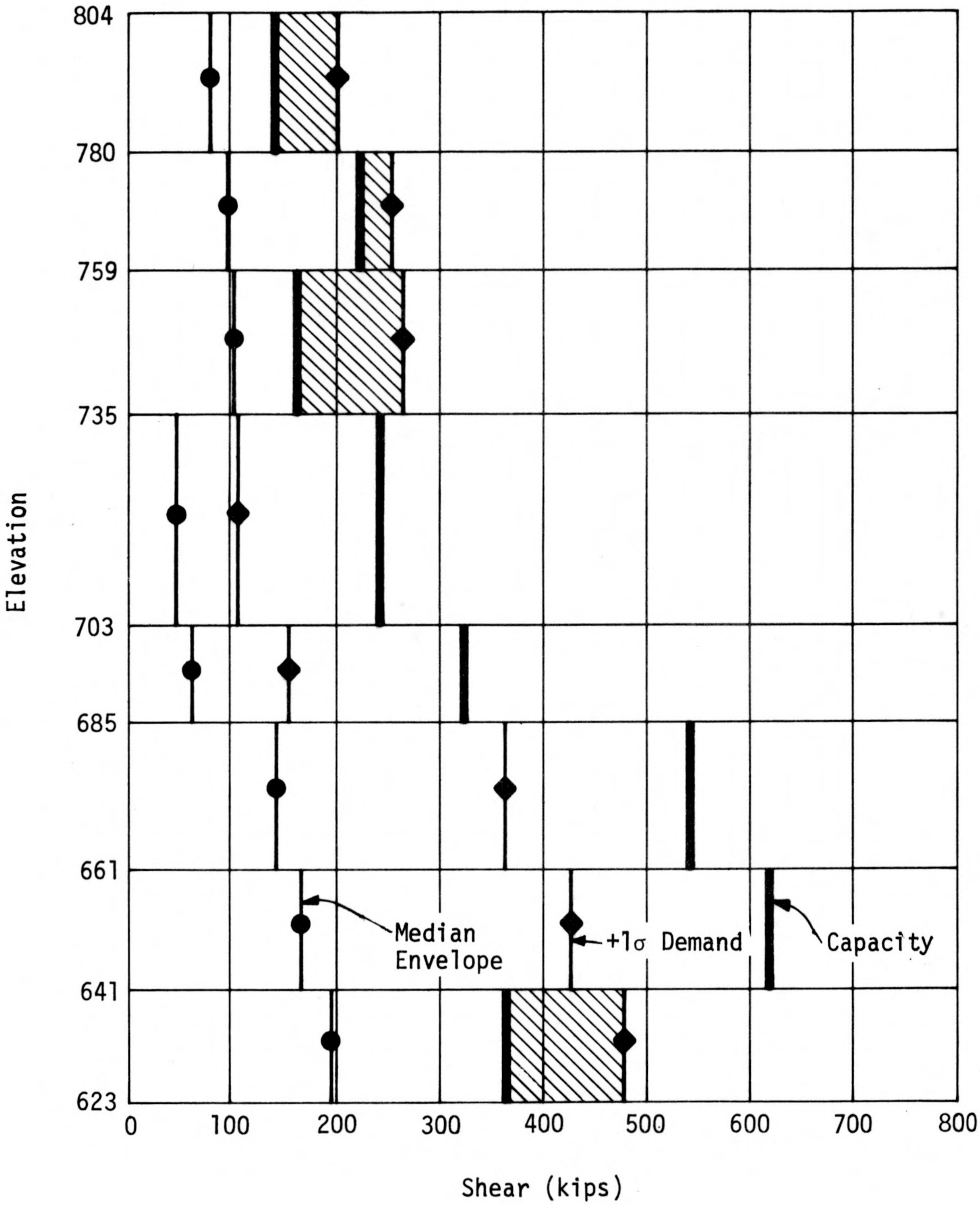


FIGURE 5 COMPARISON OF CAPACITY AND DEMAND STORY SHEAR TRANSVERSE (E/W) MODEL T2-MO

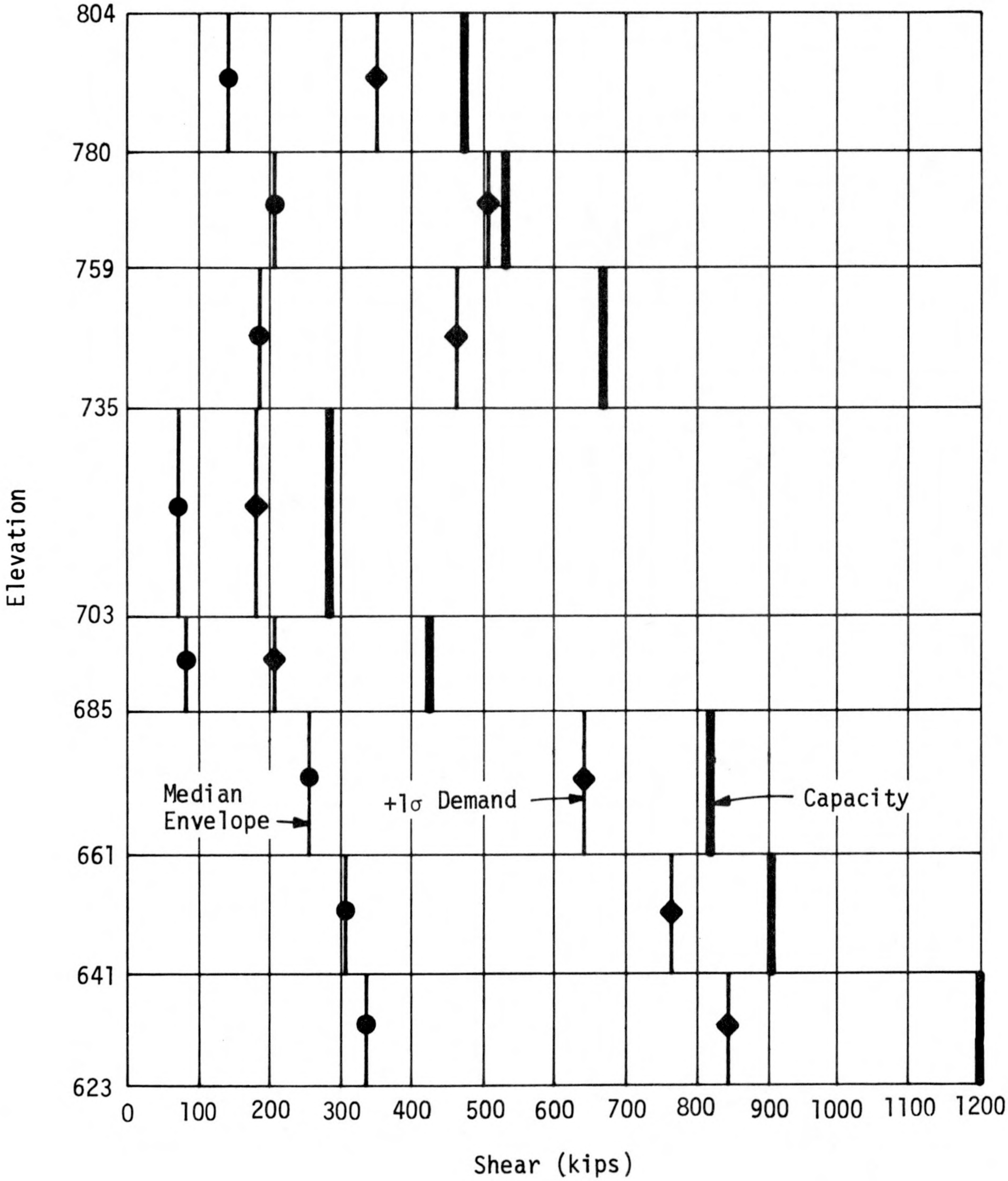


FIGURE 6 COMPARISON OF CAPACITY AND DEMAND STORY SHEAR LONGITUDINAL (N/S) MODEL L2-M6

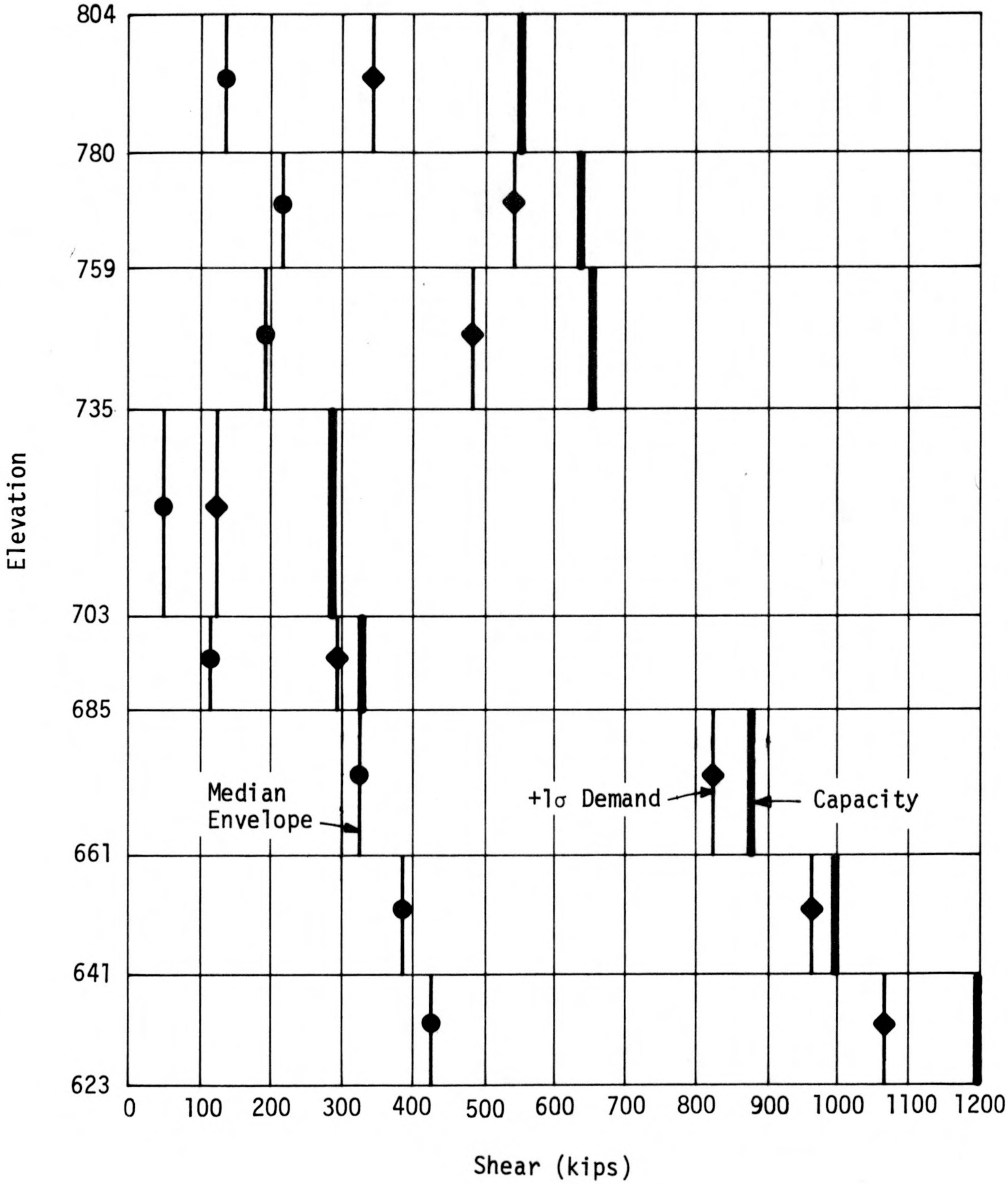


FIGURE 7 COMPARISON OF CAPACITY AND DEMAND STORY SHEAR TRANSVERSE (E/W) MODEL T2-M6

of the increase in structure capacity was offset by similar but smaller increases in demand. These increases in demand occurred because the shorter periods of vibration of the structure resulting from the stiffening effects of the bracing intersected higher response accelerations on the predicted response spectrum.

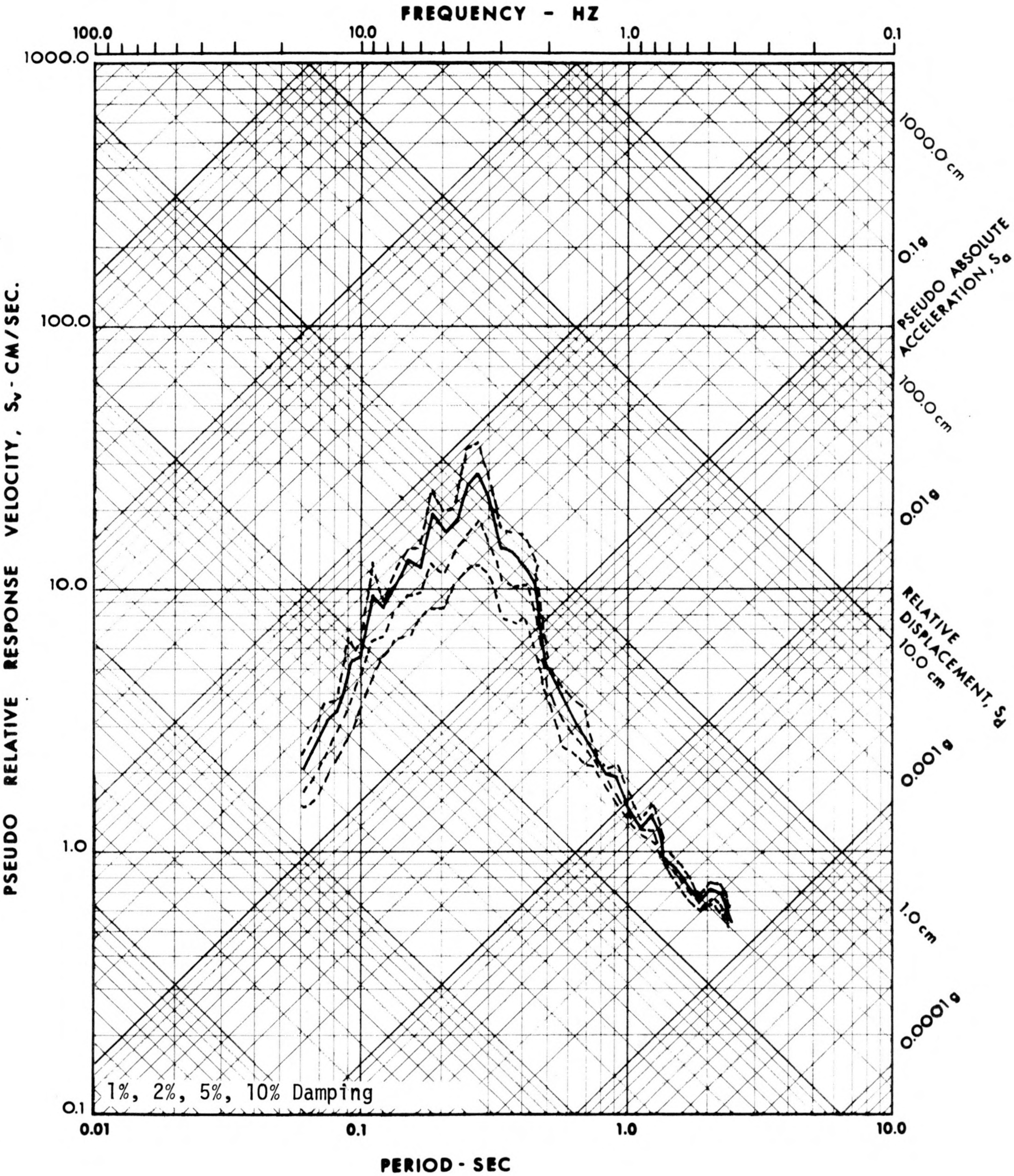


FIGURE 8 COLONY TOWER · RIO BLANCO
LONGITUDINAL

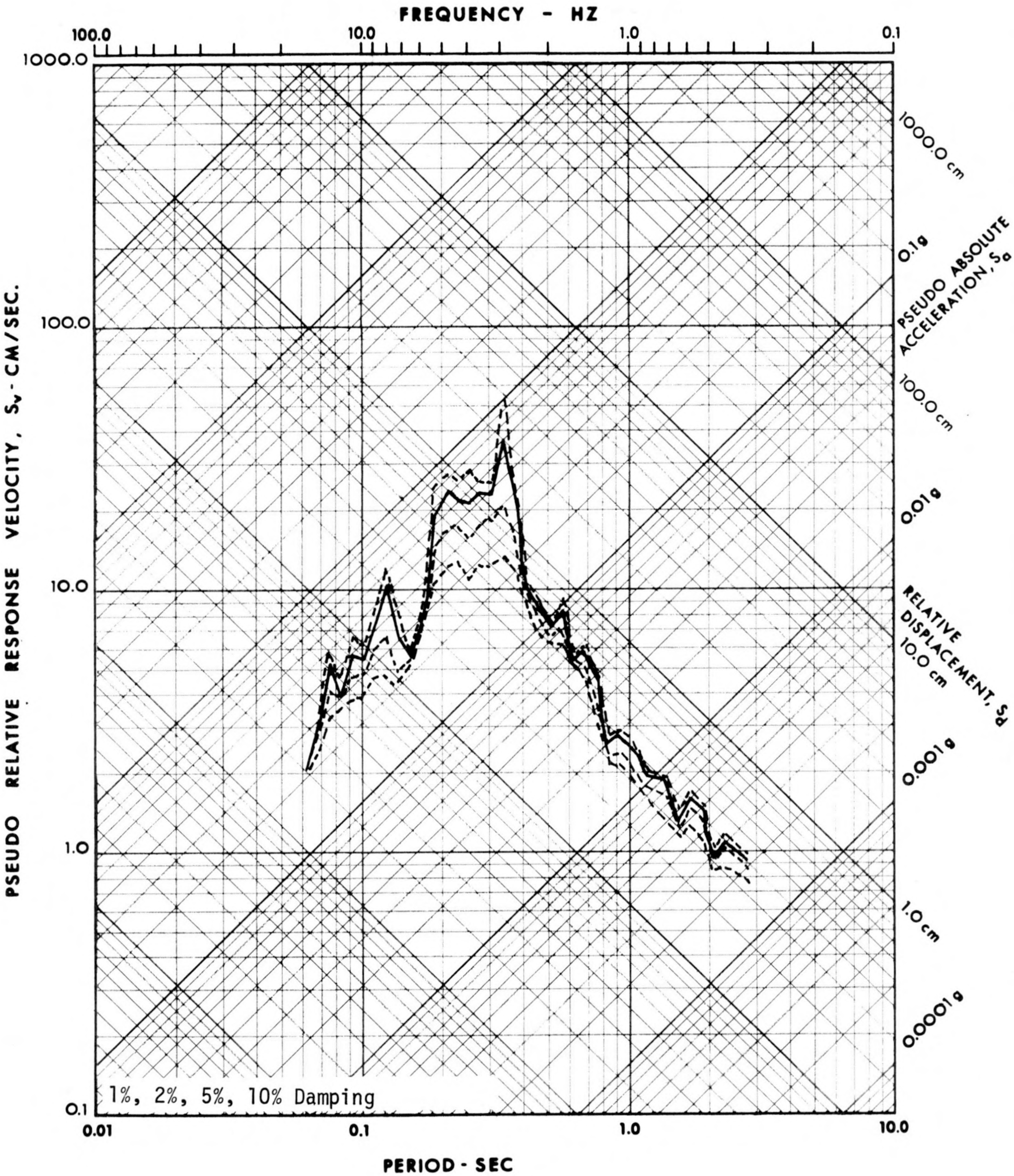


FIGURE 9 COLONY TOWER · RIO BLANCO
TRANSVERSE

TABLE 1 COMPARISON OF TOP LEVEL RESPONSE*

		LONGITUDINAL (N/S)				TRANSVERSE (E/W)			
		1ST MODE	2ND MODE	3RD MODE	RMS* SUM	1ST MODE	2ND MODE	3RD MODE	RMS* SUM
	PERIODS (SEC)								
1	MODEL M0	1.017	0.344	0.215		1.20	0.371	0.207	
2	MODEL M6	0.916	0.261	0.171		1.15	0.289	0.169	
3	MEASURED	1.02	0.34	0.21		1.22	0.36	0.20	
	SPECTRAL ACCELERATION FOR 2% DAMPING (G)								
	DESIGN CRITERIA								
4	AT +1 SIGMA	0.06	2.99	1.49		0.04	3.01	1.50	
5	AT MEDIAN	0.03	1.20	0.60		0.02	1.20	0.60	
	ENVELOPE								
6	UNWEIGHTED	0.024	0.55	0.40		0.013	0.72	0.40	
7	FROM RECORDINGS								
a	A. MEASURED T	0.009	0.29	0.51		0.009	0.42	0.75	
b	B. CALCULATED T	0.014	0.64	0.49		0.011	0.50	0.39	
	SPECTRAL VELOCITY (CM/SEC)								
8	UNWEIGHTED	3.3	22	10		2.3	32	10	
9	FROM RECORDINGS								
a	A. MEASURED T	1.5	14.9	16.0		1.9	24.5	24.0	
b	B. CALCULATED T	1.9	26.0	13.5		1.9	22.0	9.6	
	TOP LEVEL VELOCITY (CM/SEC)								
10	UNWEIGHTED	4.6	14	3	15*	3.3	20	2.4	21*
11	FROM RECORDINGS								
a	A. MEASURED T	2.1	9.5	4.8	11*	2.7	15.4	5.8	17*
b	B. CALCULATED T	2.6	16.6	4.1	17*	2.7	13.9	2.3	14*
12	MEASURED				14.6				6.04 & 10.3

* Notes on the following page clarify the measurements listed in each row of Table 1.

Notes Relating to Table 1

Row No.

1. Periods listed in the row following "Model M0" are those calculated for the first three modes of response in each direction of the tower before it was modified. (These periods agree well with some obtained from field measurements of the tower before modification).

2. Periods listed in the row following "Model M6" are those calculated for the modified tower. The periods shortened because of added stiffness.

3. Periods obtained from the recordings made during RIO BLANCO.

4. Spectral acceleration (in g's) at each period using the $+1\sigma$ spectral response prediction, and therefore weighted for tower safety purposes.

5. Spectral acceleration (in g's) at each period using median prediction response spectrum values and therefore less weighted for tower safety.

6. Unweighted spectral accelerations, which were obtained as described by taking the average of the upper and lower envelope of the predicted spectrum at each period band. These represent the predicted spectral values with the highest probability of actual occurrence.

7. Spectral acceleration value at each period from the spectra constructed from the recorded ground motion.

   a. Using measured periods from row 3
   b. Using calculated period from row 2

8. Unweighted predictions of spectral velocity.

9. Recorded spectral velocities.

   a. Using periods from row 3
   b. Using periods from row 2

10. Unweighted predictions of roof level motion corresponding to spectral values shown in rows 6 and 8. Columns 4 and 8 present combined modal response values using a root-mean-square (RMS) procedure.

11. Roof motion based on spectra from recordings. These correspond to spectral values shown in rows 7 and 9 and would be the tower motion values predicted from a perfect pre-detonation prediction of the response spectrum at the tower location.

12. Recorded roof motion.

Comparison of predicted and recorded motion (rows 10, 11, and 12) is reasonable. Comparison of the RMS response with actual measurement at the roof level as shown in rows 10 and 11 shows that the tower model was quite good in the longitudinal direction but overpredicted response in the transverse direction.

Table 2 shows a comparison of responses at other levels of the structure. The M6 series of mathematical models were subjected to the recorded ground motions and the responses at various levels were obtained and compared with the recorded motions at these levels. In the longitudinal direction, the comparison is quite good for all levels. In the transverse direction it is not as good, although within reason.

The inaccuracies in the prediction for the transverse direction are probably caused by the extremely irregular placement of large equipment weights in the tower resulting in response with torsional component. Additional complexities occur because of flexible floor diaphragms and resulting internal relative movement of these large equipment masses. Additional study would be required for a definitive evaluation of the translational response velocities.

TABLE 2 COMPARISON OF RESPONSE AT VARIOUS LEVELS (CM/SEC)

LEVEL	LONGITUDINAL (N/S)		TRANSVERSE (E/W)			
	MEASURED AT CENTER	PREDICTED	N END	MEASURED CENTER	S END	PREDICTED
804	14.6	16.7	10.3		6.04	14.1
781	7.78	5.7	6.75		5.10	7.0
735	7.71	7.6	3.83	13.5	7.99	11.2
685	11.8	13.3	5.25		7.25	6.85

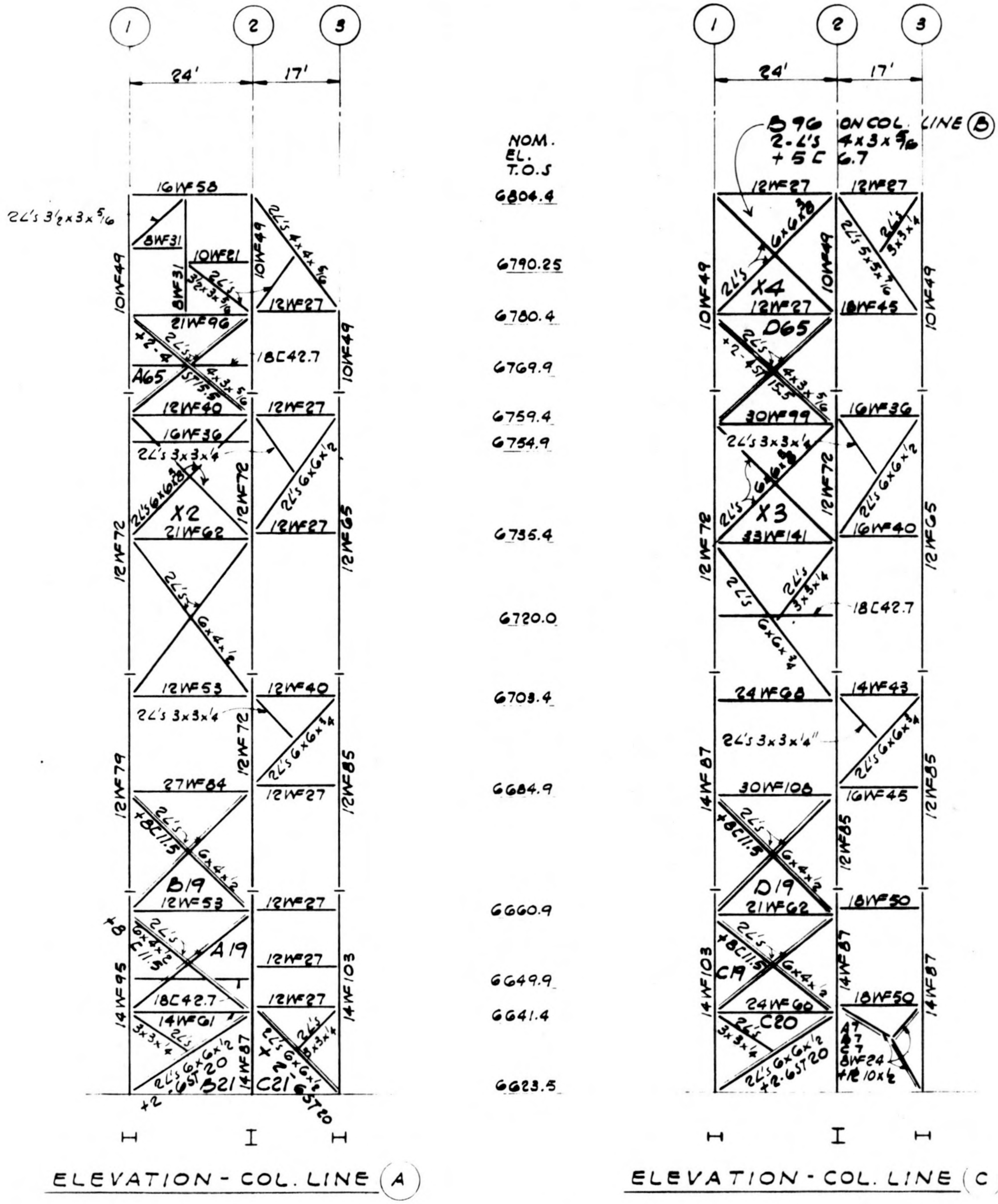
FIELD IMPLEMENTATION

Stearns-Roger, Inc., was selected by CER as the contractor because the firm had worked extensively on earlier tower modifications and security agreements were still in effect. On April 19, 1973, the Blume resident engineer met with representatives from Stearns-Roger at the Colony Development at Parachute Creek to inspect the tower and the facilities available to the contractor.

During the tower inspection, it became evident that removal and replacement of existing bracing members with heavier members should be avoided if at all possible. Bracing members were too long and there were too many obstructions from piping and process equipment to make member removal and replacement feasible. Instead it was decided to stiffen the existing double angle bracing members by the addition of steel channels, tees, and wide-flange shapes as appropriate for each particular vertical brace panel. All fastening of stiffening members to brace members would be accomplished by welding, with details supplied to the Resident Engineer. (Refer to Figures 10, 11, and 12.)

All bolted and welded connections were reviewed in the field for adequacy and, if found inadequate, welding details were supplied by the Resident Engineer. The AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" allows welds to be proportioned to function in conjunction with high strength bolts used in friction-type connections in resisting the transfer of stress across faying surfaces; however, every effort was made to provide complete reliance on welds if existing bolts were inadequate for postulated loads.

In general, basic sizes and lengths of stiffener members were developed in the office. Final sizes were dependent on the immediate availability of structural steel shapes from suppliers in Grand Junction or Denver, Colorado. As it developed, the required steel was available from Grand Junction. All steel was shipped unfabricated but cut to length and marked for the particular bracing panel. The first load arrived on



NOTE: BRACE PANEL NUMBERS (D65, X3 ETC.) ARE GIVEN FOR NEW OR STIFFENED BRACE PANELS

FIGURE 10 EXTERIOR ELEVATIONS, LINES A AND C

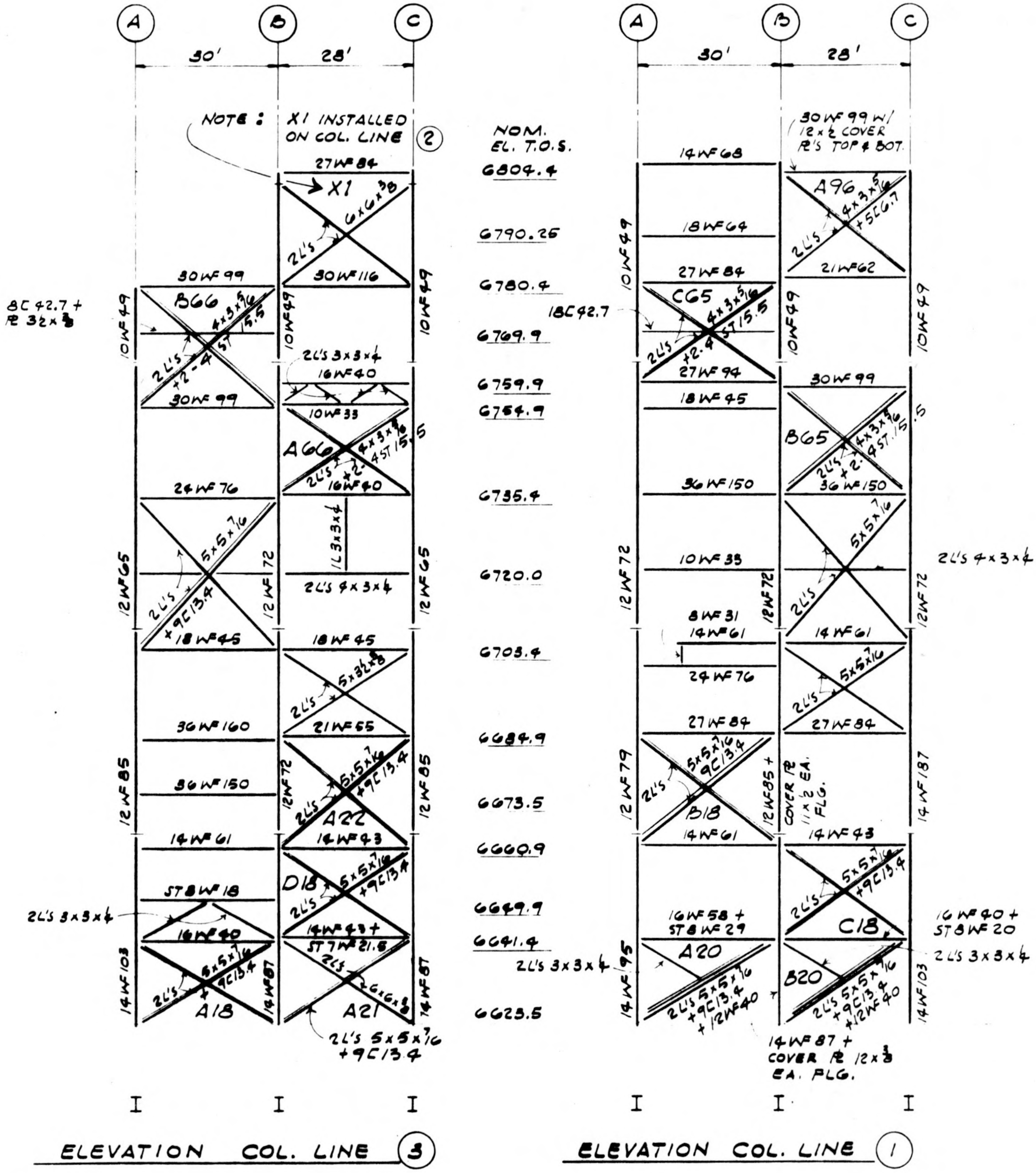
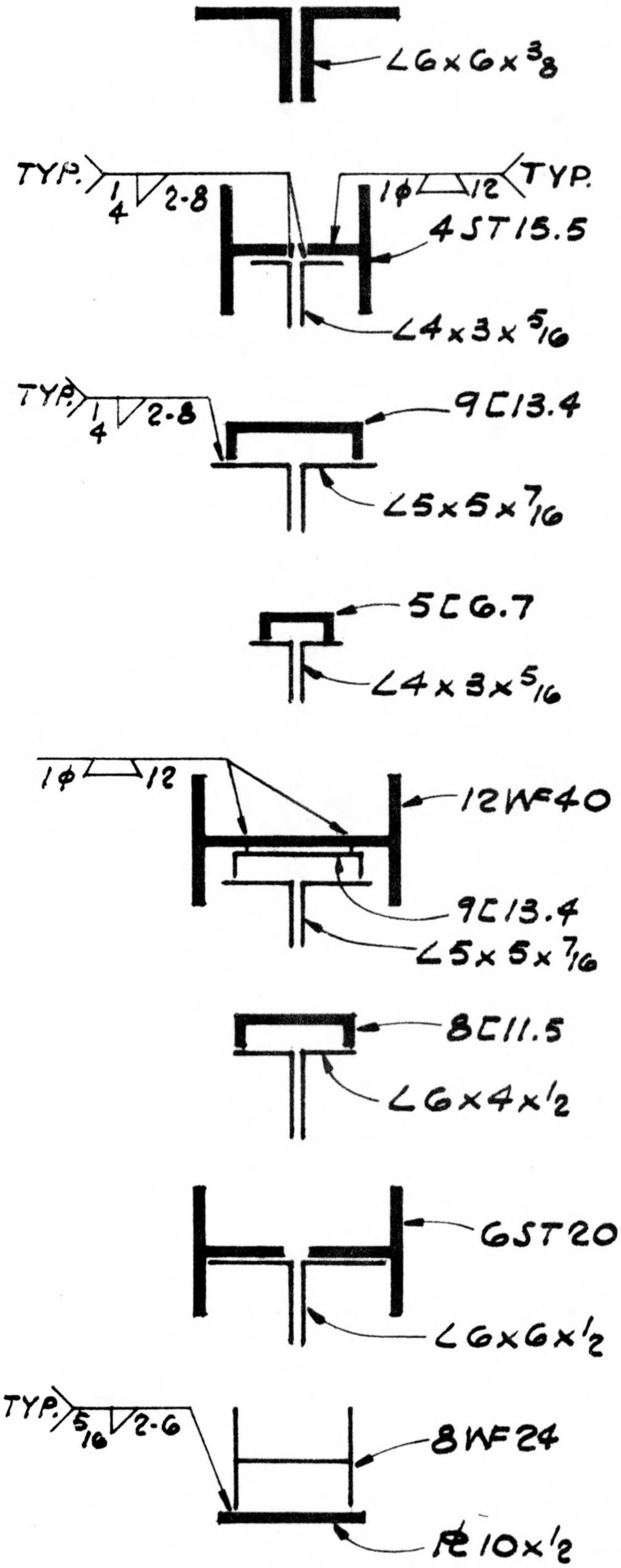


FIGURE 11 EXTERIOR ELEVATIONS, LINES 1 AND 3

FOR BRACE PANEL NUMBER



X1, X2, X3, X4 & A21*

A65, B65, C65, D65,
A66 & B66

A18, B18, C18, D18,
A21* & A22

A96 & B96

A20 & B20

A19, B19, C19 & D19

C20, B21 & C21

A7, B7 & C7

*REFER TO ELEVATION FOR LOCATION

FIGURE 12 STIFFENED BRACE CROSS SECTIONS

May 2, after initiating the steel order on Monday, April 30, and the final load arrived Friday, May 4. All fabrication and replacement of stiffening members was accomplished in the field from sketches developed by the Resident Engineer.

Fabrication of stiffener members involved cutting members to length, burning slots in channels to clear existing gusset plates, coping flanges, and burning 1-inch ϕ holes at 12-inch centers for plug welds in the stem of structural tee shapes. Material was then carried by the crane to the material hoist and lifted to the proper level of the tower. From this point, small tugger hoists and snatch blocks were used to slide and lift the stiffener member into position for tack welding to the brace members. Chain hoists were often required to further assist in the placement of members.

In developing field fabrication drawings, every effort was made to utilize welds in the down-hand and horizontal positions. This meant that stitch welds and plug welds would provide the principal means of attaching stiffener members to existing bracing. Welding stresses used to proportion welds attaching stiffeners to braces were normal working stresses allowed by applicable code.

Due to the high slope of the diagonal brace panels, the welders operated from hanging scaffolds. Four of these scaffolds were of the one-man electrically operated "Sky Climber" type and two were electrically operated platform scaffolds 12 feet and 24 feet long, respectively. In addition, four more one-man scaffolds utilizing chain hoists were built on the job. The constant interference of piping and process equipment with scaffold movement and steel erection, and several days of inclement weather (snow, rain, cold) made steel erection and welding production less than ideal. Regardless, the work progressed according to schedule and with a 100% safety record. The work involved fabricating and placing approximately 21 tons of structural steel shapes (134 pieces) and 750 pounds of welding electrode to strengthen the existing vertical bracing system for the tower.

The horizontal bracing system at each floor level was reviewed in the field by the Resident Engineer. In general, the existing horizontal bracing had been placed to carry lateral loads back to the outside columns. However, the alignment of the horizontal bracing generally suffered due to conflict with vertical pipes and process vessels which penetrate each level. Due to the number of large floor penetrations, it is questionable that a better horizontal bracing system pattern could be provided. Horizontal members reviewed at the site were felt to be marginal but adequate, with the possibility of some minor slippage in the friction-type bolted connections.

The work was conducted on a 10-hour day, 7-day working week schedule, and was completed on May 14. The Project RIO BLANCO detonation occurred at the scheduled time on May 17.

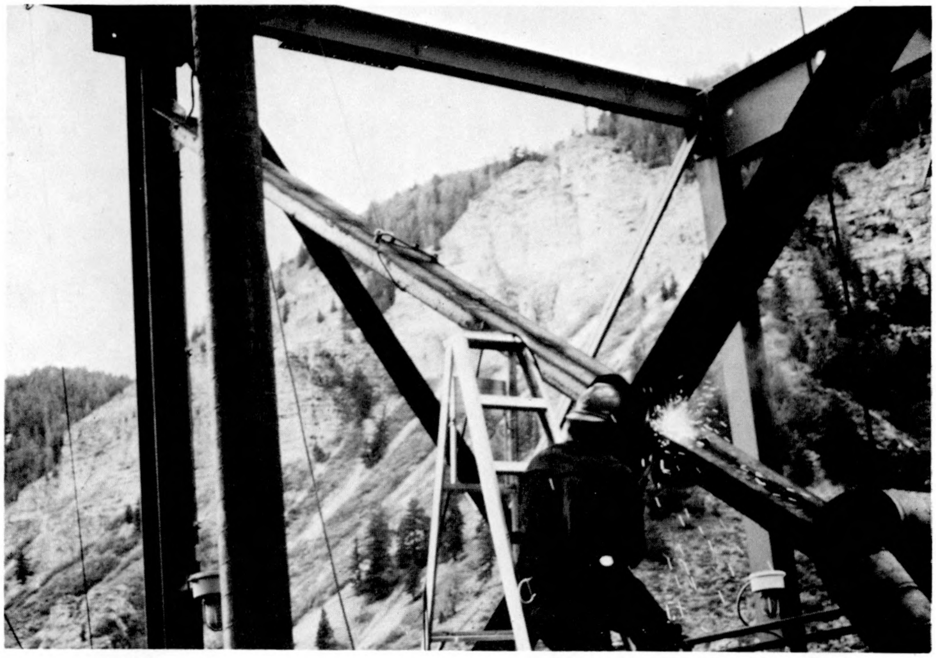
Following the detonation of RIO BLANCO, the tower structure was closely inspected by Blume's Resident Engineer. No damage and no indication of adverse tower response could be found.

To provide immediate post-detonation visual verification of tower response, the Resident Engineer had gone through the tower prior to the detonation and placed sealing wax tell-tales. About 30 tell-tales were so placed to indicate movement, if any, by cracking of the brittle wax. Heavy equipment mounts, bolted structural connections, and free hanging pipe guides were so marked throughout the tower.

The location of each of the wax tell-tales was pointed out to the post-detonation inspection party. The condition of the wax markers was closely examined and photographed by several of the inspecting engineers. While a few of the wax markers were cracked, no visual evidence of damage or adverse response could be found.

Figures 13-18 are photographs of tower bracing, bracing details, and sealing wax tell-tales.

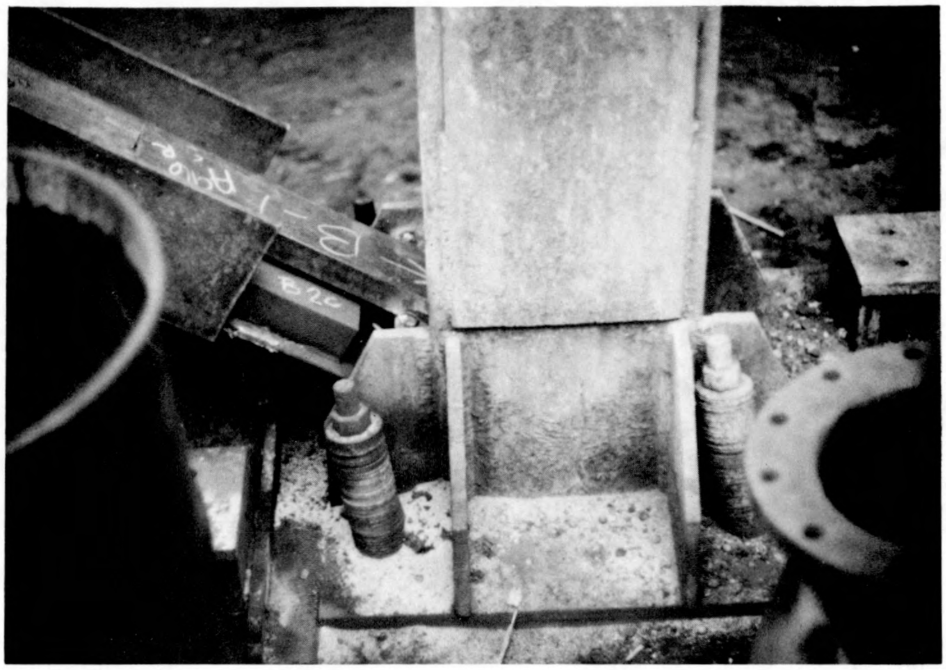
During the implementation period and since the detonation of RIO BLANCO, there has been some mention of possible removal of bracing modifications to restore the tower to its condition prior to RIO BLANCO. It is the opinion of the Resident Engineer that any efforts to restore the tower by removing modifications would be misguided. Besides incurring needless expense, such an effort could only result in damaged structural members and reduced tower strength. Plug welds, which were extensively required to effect the modifications, are difficult if not impossible to remove.



a. Installing brace X1

b. Brace D65. Note intersection detail

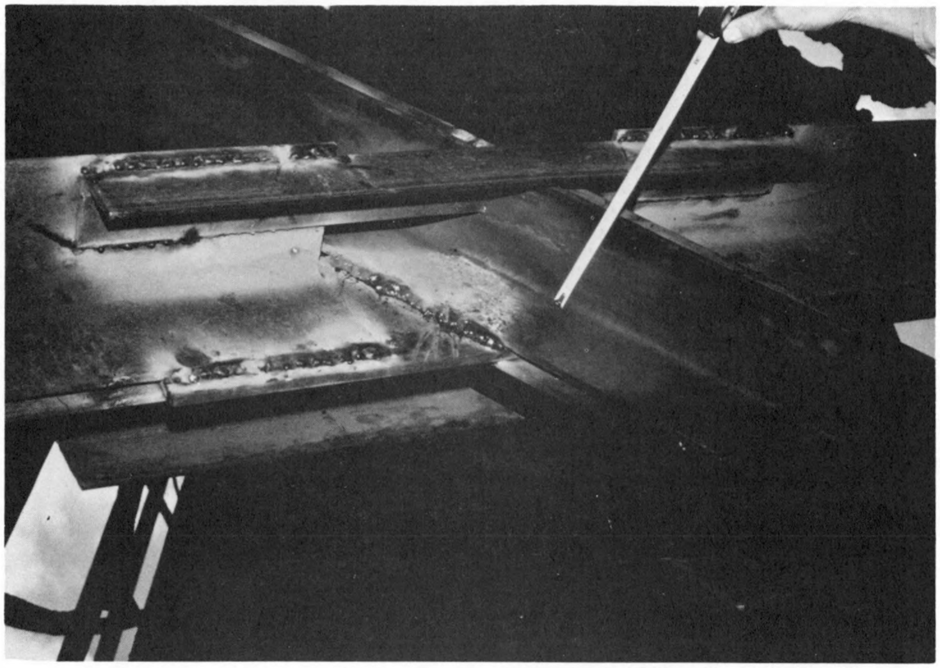
FIGURE 13 BRACE INSTALLATION AND INTERSECTION DETAILS



a. Brace B20 at Col. B-1

b. Bottom connection of Brace B20 at Col. B-1

FIGURE 14 BRACE CONNECTION DETAILS



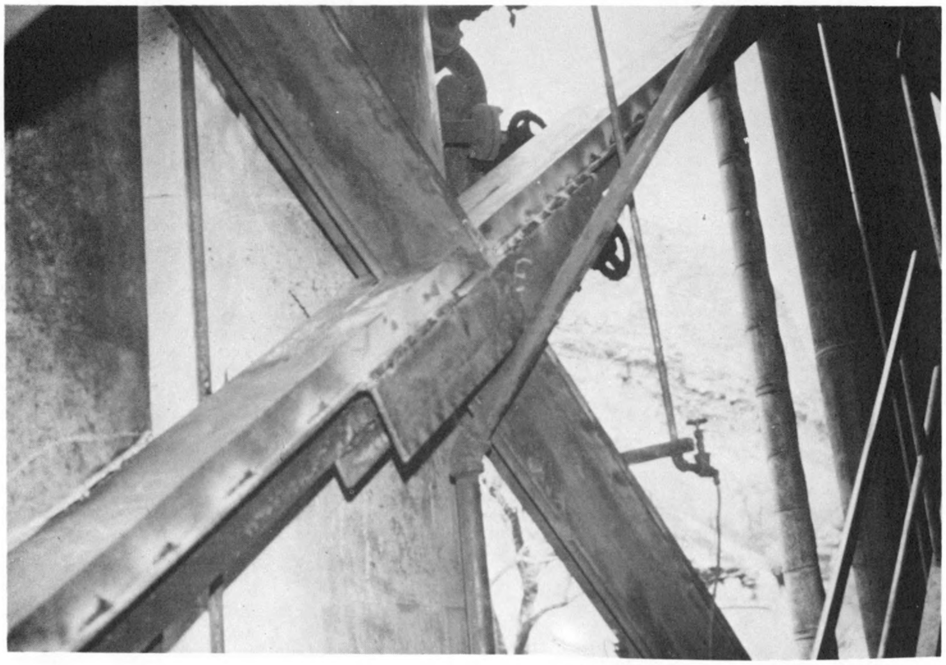


FIGURE 15 VIEWS OF INTERSECTION DETAILS OF STIFFENED BRACES

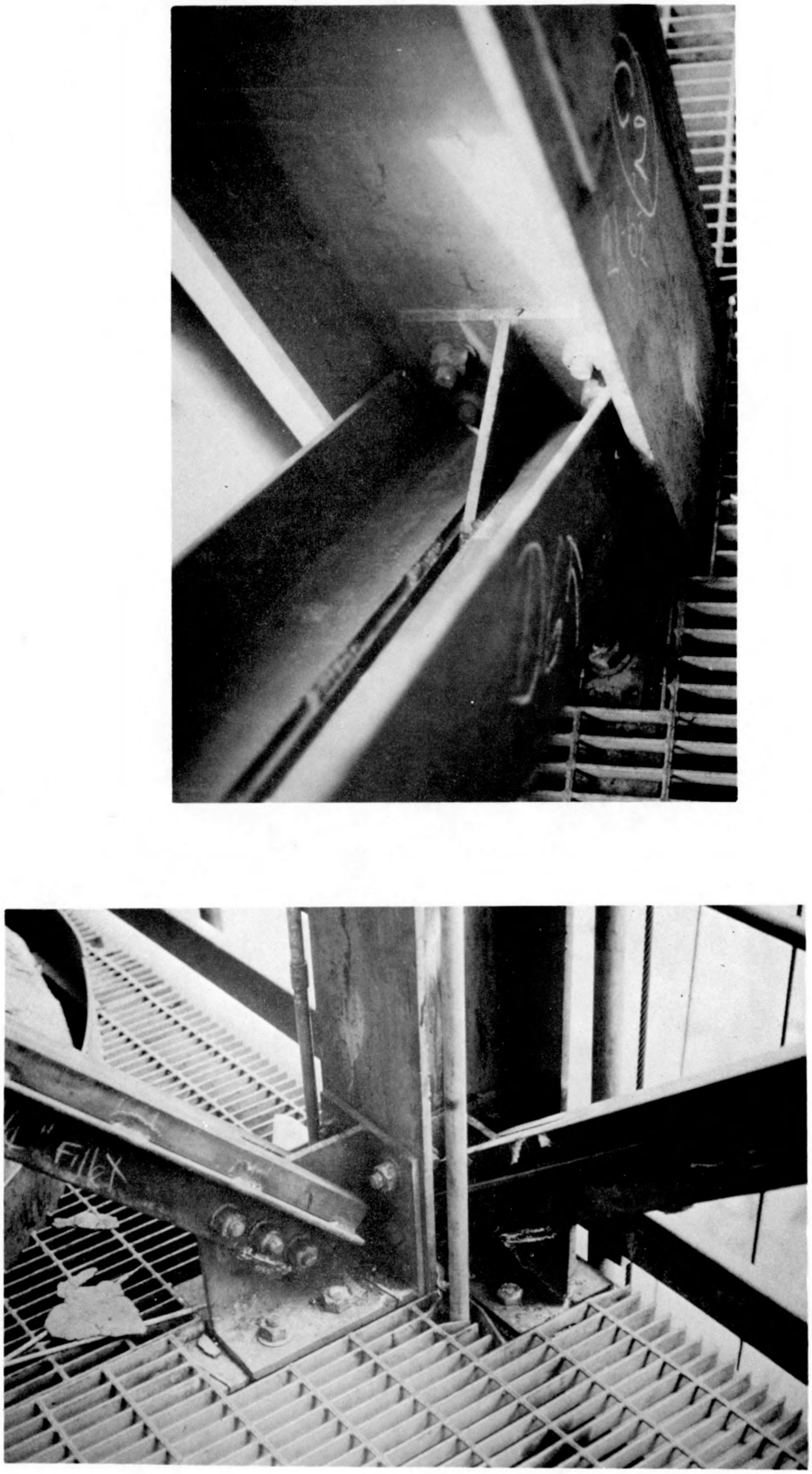
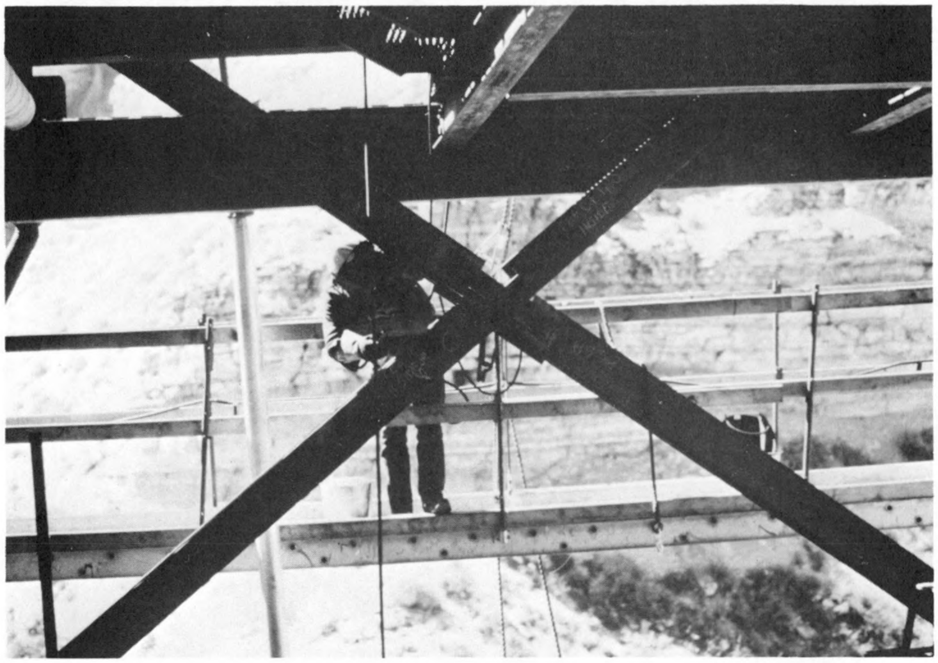
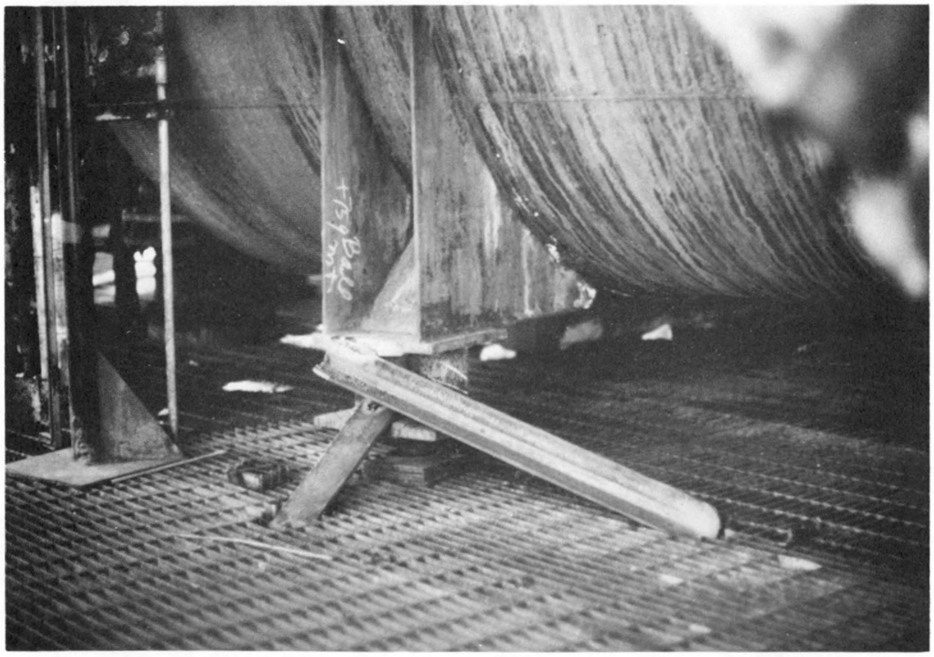


FIGURE 16 BOTTOM VIEW OF STIFFENED BRACES D65 (TOP) AND A96. B96 (BOTTOM)



A. Installing brace B66



b. Equipment tie down

FIGURE 17 BRACE INSTALLATION AND EQUIPMENT TIE DOWN DETAILS

FIGURE 18 SEALING WAX "TELL-TALES"

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