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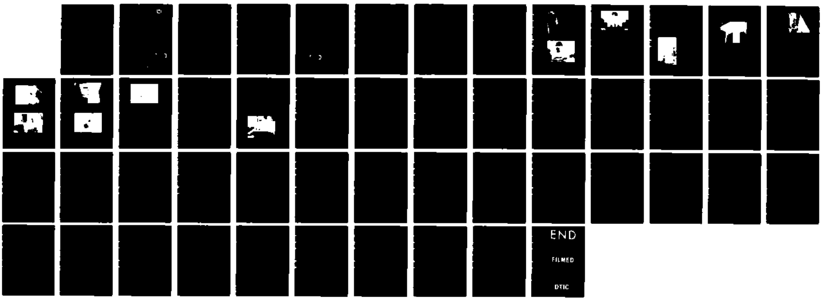

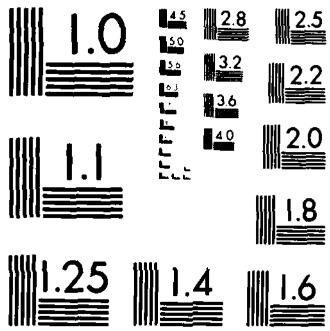

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September 1984

US Army Corps of Engineers

Cold Regions Research & Engineering Laboratory

Secondary stress within the structural frame of DYE-3: 1978-1983

H. Ueda, W. Tobiasson, D. Fisk, D. Keller and C. Korhonen

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DEW line ice cap station DYE-3 was moved sideways 210 ft and placed on a new foundation in 1977, then raised 27 ft in 1978. Secondary forces within the structural steel framework were measured in 1978, 1981, 1982 and 1983. The overall level of secondary stresses had increased but through 1983 the columns were still within their stress limitations. Some localized overstress is expected in 1984. The concept of using above-surface trusses to resist wind loads and brace the eight columns has proven to be satisfactory. It has eliminated the subsur-

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face enclosures used in the past to protect subsurface trusses, enclosures that proved to be the structural weak link of the original facility; their elimination has resulted in a stronger facility that is easier to maintain. The measurements and findings of this program were used in the development of the design to extend the life of DYE-3 to be implemented in 1984. That work should reduce the level of secondary stresses in the frame.

PREFACE

This report was prepared by Herbert Ueda, Mechanical Engineer, Engineering and Measurement Services Branch, Technical Services Division; Wayne Tobiasson, Research Civil Engineer, Civil Engineering Research Branch, Experimental Engineering Division; David Fisk, Mechanical Engineering Technician, Engineering and Measurement Services Branch, Technical Services Division; Donald Keller, Civil Engineering Technician, Geotechnical Research Branch, and Charles Korhonen, Research Civil Engineer, Civil Engineering Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding was provided by the DEW System Office, 4700th Air Defense Squadron, U.S. Air Force Tactical Air Command through MIPR S83-174, 24 June 1983.

In addition to the authors, the following CRREL personnel participated in the investigation: B. Coutermarsh, D. Prince, G. Thurston, K. Kaufmann, and M. Saban. B. Coutermarsh and D. Garfield of CRREL technically reviewed this report.

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CONVERSION FACTORS: U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

These conversion factors include all the significant digits given in the conversion tables in the ASTM Metric Practice Guide (E 380), which has been approved for use by the Department of Defense. Converted values should be rounded to have the same precision as the original (see E 380).

Multiply	By	To obtain
inches	25.4	millimetres
feet	0.3048	metres
tons (short)	907.1847	kilograms
pounds (force) per square inch	0.006894757	megapascals
kips	448.222	newtons

SECONDARY STRESSES WITHIN THE STRUCTURAL FRAME OF DYE-3: 1978-1983

by

H. Ueda, W. Tobiasson, D. Fisk, D. Keller and C. Korhonen

INTRODUCTION

The primary loads to which a structural system is subjected consist of dead loads such as its own weight, live loads from equipment and personnel, and imposed loads such as those caused by wind or snow. In addition, secondary loads can be introduced into a structural system, creating secondary stresses within its members. This can happen during construction as a result of imperfect fits, for example, or after, when supports settle differentially, move laterally or tilt. When a large structure such as DEW Line Ice Cap Station DYE-3 is founded on snow, the development of secondary stresses is inevitable and the magnitudes of these stresses can become critical.

In 1977, because of large secondary stresses within its structural frame and excessive distortions of portions of its substructure, DYE-3 was moved sideways 210 ft onto new footings (Tobiasson 1978). The following summer the building was raised 27 ft. Another lift in 1984, along with additions to the truss system, will complete the program started in 1977 to extend the useful life of the station to 1990.

During the fall of 1978, CRREL personnel made initial measurements of the forces within the structural frame. Subsequent measurements were made in 1981, 1982 and 1983 as part of the structural performance monitoring program that CRREL has been conducting for the U.S. Air Force since 1973 (Tobiasson and Ueda 1972, Tobiasson et al. 1974).

TECHNIQUE

Figure 1 shows the location of DYE-3 in Greenland. Exterior views of DYE-3 are shown in Figure 2; elevation and plan views of the DYE-3 structural system are shown in Figure 3. A detail of the trusses and collars at column N1 is shown in Figure 4. The 2600-ton building is supported by eight columns and can be leveled and raised by hydraulic lifts located at each column on the second floor where the building hangs from 6-in.-diameter threaded rods. The two 80-ton truss networks are independent of the building. They rest on

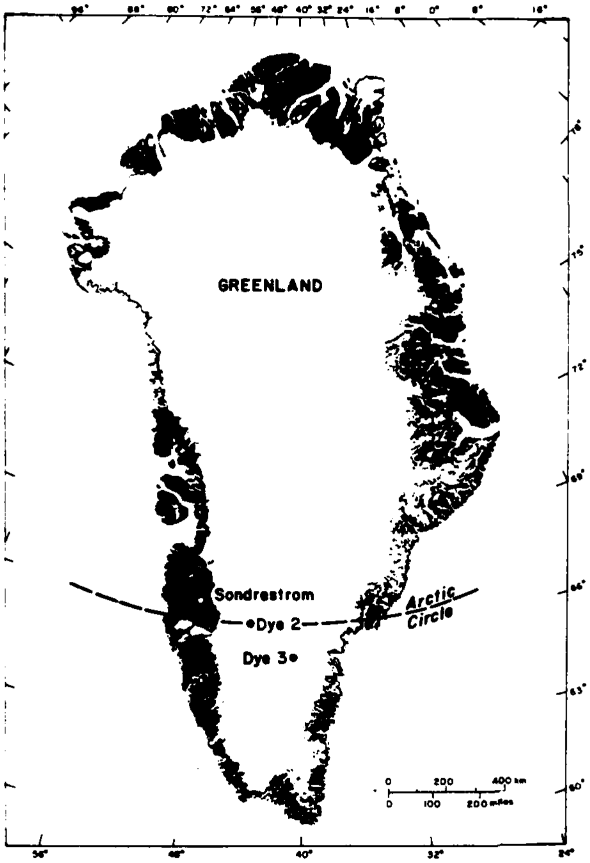
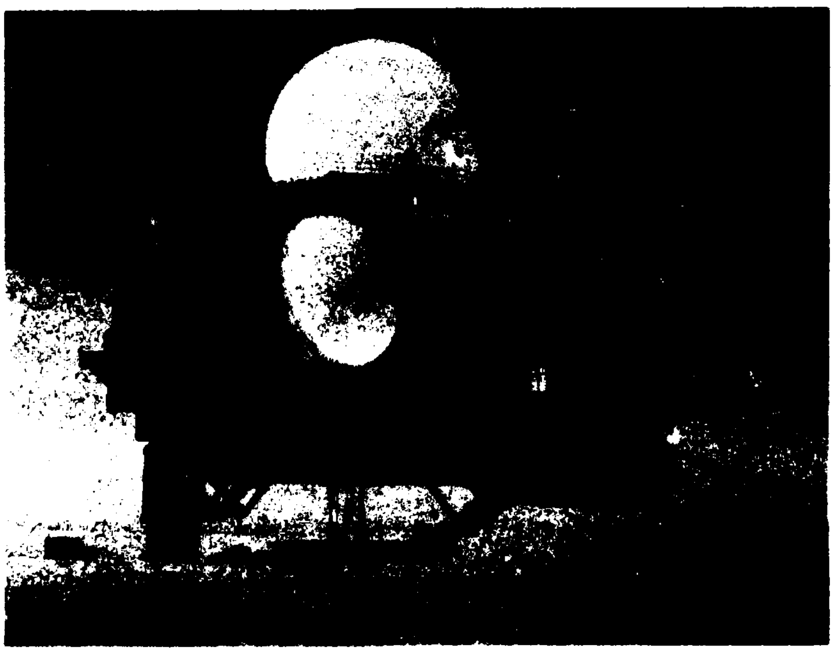
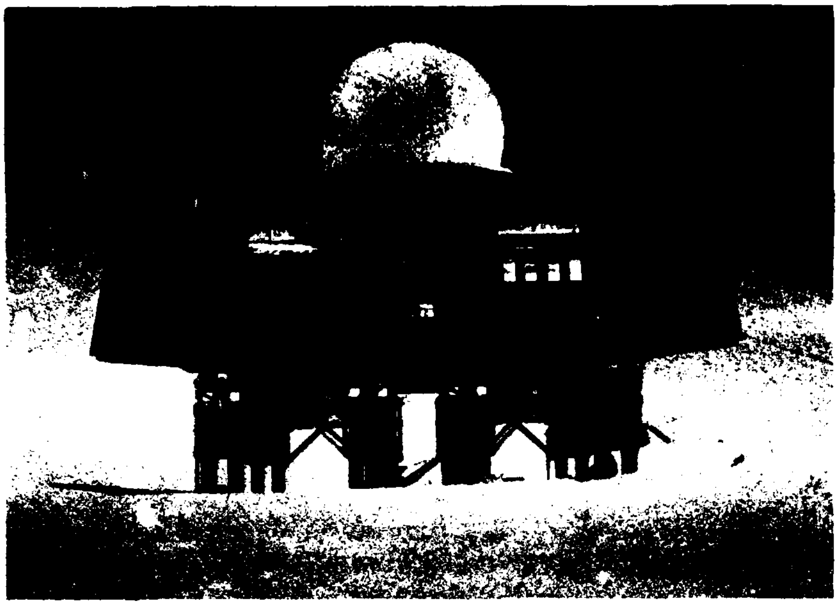


Figure 1. Location of DYE-3.



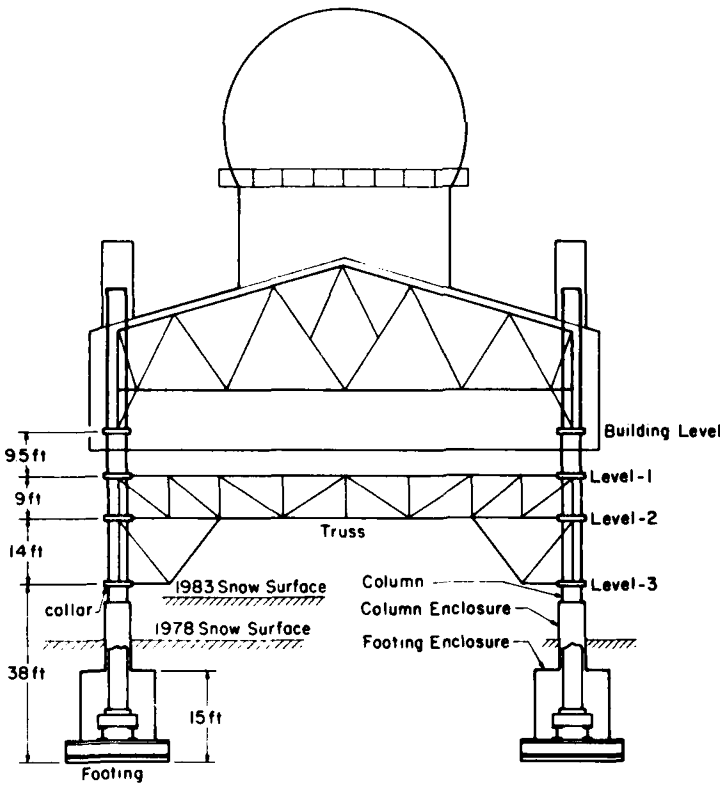
a. From the southeast.

Figure 2. Exterior views of DYE-3 (1981).



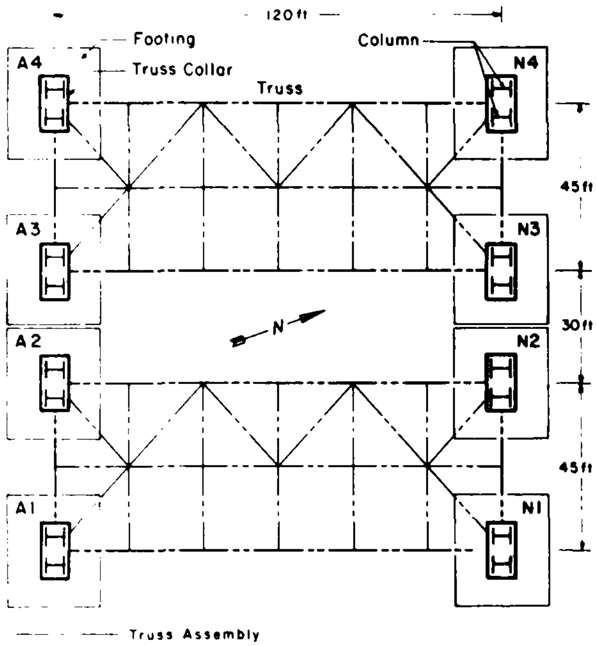
b. From the southwest.

Figure 2 (cont'd).



a. Elevation cross section.

Figure 3. DYE-3 structural system.



b. Plan view showing footing, column and truss layout.

Figure 3 (cont'd). DYE-3 structural system.

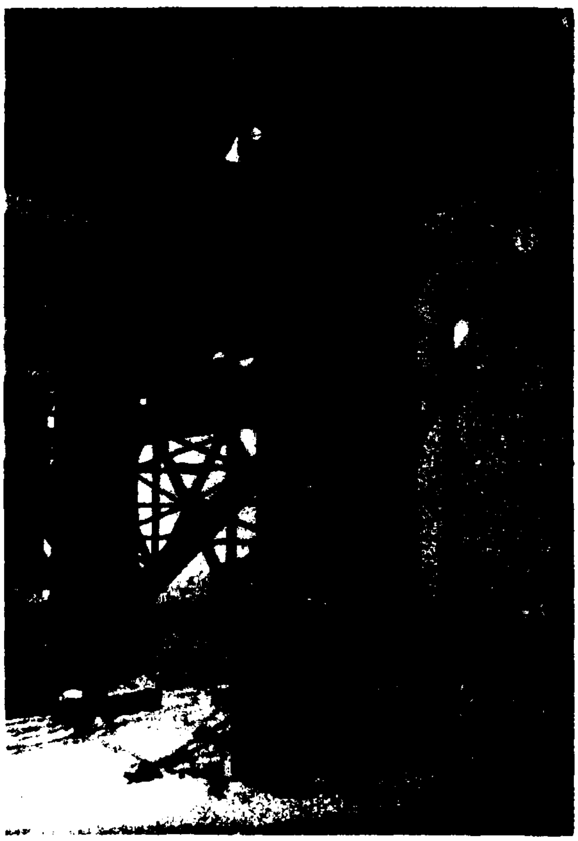


Figure 4. Three collars at each column transfer lateral loads between the trusses and the columns (1 - level 1 collar, 2 - level 2 collar, 3 - level 3 collar).

steel beams and channels attached to the columns (Fig. 5). To permit periodic lifting and leveling of the trusses, they are not connected to the columns in a conventional manner; instead, a rectangular collar surrounds the column at each connection point and sway bolts (Fig. 5) that act as set screws transmit lateral loads between the collar and the column.

There are 12 large sway bolts at each collar (Fig. 6) and there are three collars at each column. They are designated levels 1, 2 and 3 as shown

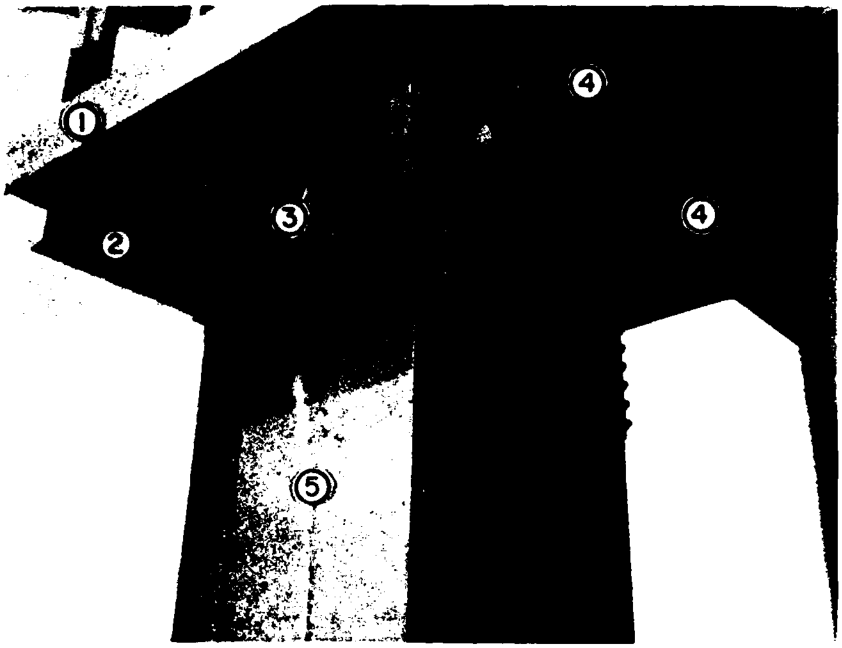


Figure 5. Truss support points and collar assembly (1 - collar, 2 - channel that supports truss, 3 - sway bolt [1 of 12 at each collar], 4 - beams that support truss, 5 - column half).

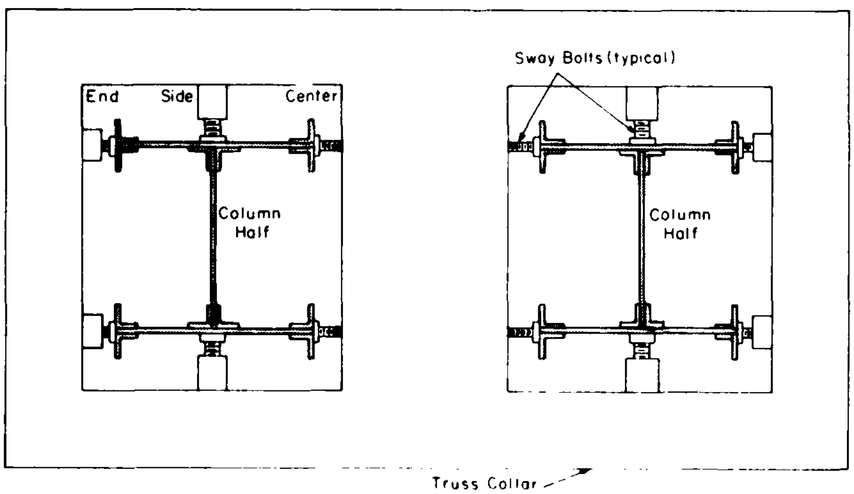


Figure 6. Plan view of typical truss collar and column showing the 12 sway bolts.

Figure 7. Loosening an unloaded sway bolt.

in Figure 3a. The sway bolts located on the first floor of the building are also arranged as shown in Figure 6.

Details of how the sway bolts are checked are described by Tobiasson et al. (1974). Initially, the tester places a wrench on each bolt. Those bolts that are easily turned (i.e., those sustaining little or no load) are backed off (Fig. 7). Working with one loaded sway bolt at a time, the tester then places a hydraulic ram alongside the loaded sway bolt (Fig. 8). Before the tester backs off a loaded sway bolt, a dial extensometer is placed on the opposite side of the column to monitor the relative movement between the column and the collar (Fig. 9). Pressure is increased in the ram until the loaded sway bolt is unloaded. The now unloaded bolt is backed off (Fig. 10) and hydraulic pressure on the ram is then slowly released until the system is returned to its original position as indicated by the dial extensometer. The hydraulic pressure recorded at this time is considered to represent the sway bolt load. Finally, the load is transferred from the ram back to the bolt and the system is returned to its original position, within 0.001 in. In this fashion the lateral interacting loads between the building and columns and the trusses and columns is determined.

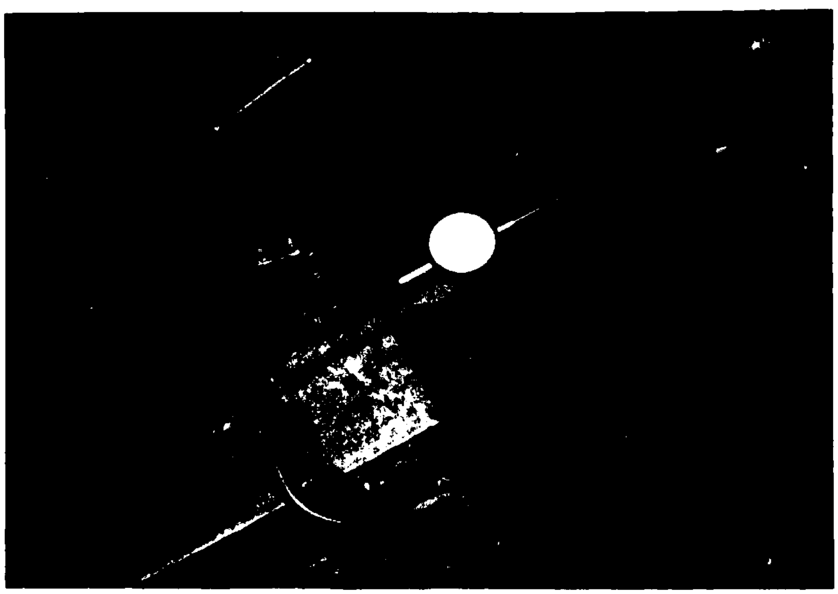
Sway bolt measurements taken in 1978, 1981, 1982 and 1983 as well as column bending moments determined from these measurements are presented in

a. Adjacent to sway bolt in the building.

b. Adjacent to truss collar sway bolt.

Figure 8. Ram placement.

a. Between a truss collar and a column.



b. Between the building and a column.

Figure 9. Dial extensometer placement.

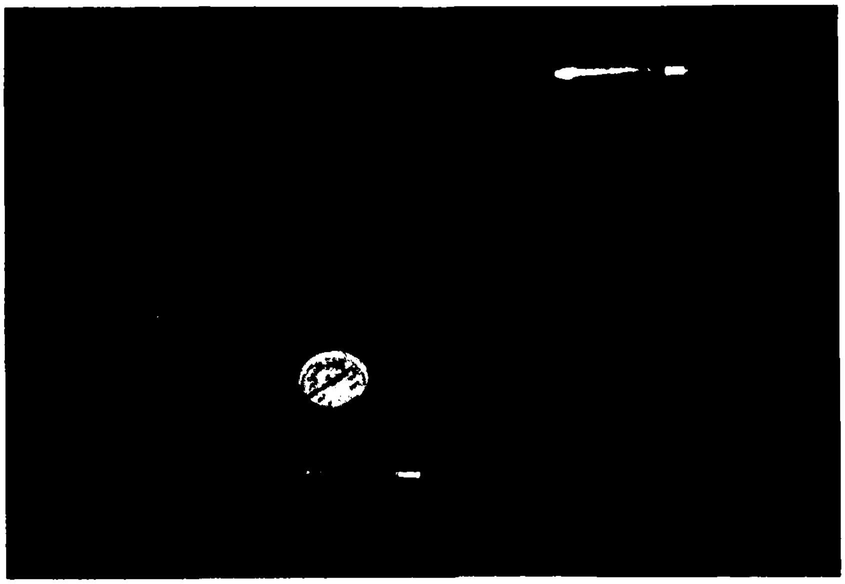


Figure 10. Backing off the "loaded" sway bolt once its load has been transferred to the ram.

Appendix A. Using these data, and equilibrium considerations, we have calculated column bending moments and stresses in two perpendicular directions. To determine axial compressive stresses, we used column load measurements made by the contractor (Danish Arctic Contractors) during the building lift in 1978.

The allowable stress for a column under combined axial compression and bending is defined in the American Institute of Steel Construction (1980) Manual of Steel Construction as follows:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \leq 1.0 \qquad (1)$$

where

f_a = axial stress (lb/in.^{2})

f_{bx} = bending stress in the x-direction (lb/in.^{2})

f_{by} = bending stress in the y-direction (lb/in.^{2})

F_a = axial stress permitted if no bending stresses exist (lb/in.^{2})

F_b = bending stress permitted if no axial stresses exist (lb/in.^{2}).

This dimensionless equation applies only when the axial stress is 15% or less of the maximum allowable axial stress with no bending present (i.e., when $f_a/F_a < 0.15$).

The column is considered overstressed whenever the combined stress factor exceeds 1.0. Before DYE-3 was moved in 1977, combined stress factors as high as 2.3 were present in some portions of the columns. Stress factor calculations are also presented in Appendix A.

MEASUREMENT DIFFICULTIES

In 1978, 13 sway bolts could not be backed off because of interfering bolt or rivet heads that made it impossible to turn them, and their loads had to be determined using a load-displacement technique (Tobiasson and Ueda 1974). The technique consists of measuring the load at various displacements and using the plot of this relationship to estimate the zero displacement load. A typical determination is shown in Figure 11. In that example, the sway bolt load is considered to be 8.4 kips. In 1981, all of the interferences were successfully removed, thereby simplifying the measurement process considerably and increasing measurement accuracy.

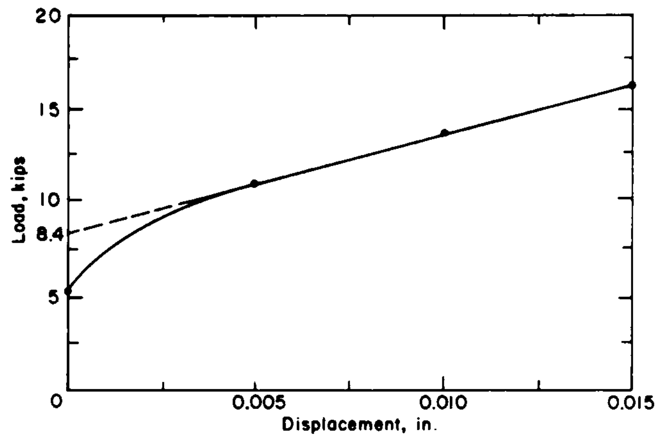


Figure 11. Typical load-displacement curve for a sway bolt that could not be backed off.

The truss network consists of two major framed assemblies, each encompassing four columns as shown in Figure 3. Three collars transfer the lateral truss load to each column (Fig. 4). The two major assemblies weigh approximately 160 kips each. As shown in Figure 5, they are supported on channels and beams attached to each column below the level-3 collar. Friction at these supports makes it difficult to isolate and measure sway bolt loads for the level-3 collars. We estimate that the friction force effects on a collar could be as high as 25 kips.

In 1981, an attempt was made to reduce friction there during sway bolt readings by supporting the four corners of each level-3 collar on friction-reducing roller devices (Fig. 12 and 13) designed by Ueda and Tobiasson. Unfortunately, the center portion of each level-3 collar, which rests on two beams (Fig. 5), did not lift when the rollers were inserted at the four corners because of high flexibility in the collar assembly; therefore, considerable frictional resistance was still present between the column halves.

Two sets of level-3 sway bolt readings are presented in Appendix A, 1981 Measurements, one with and one without the rollers installed. The rollers did not change level-3 sway bolt loads much. An unsuccessful attempt was made in 1982 to insert Teflon sheets between all bearing surfaces of the level-3 collars, but again the center portion of the collars could not be lifted. Consequently, loads measured on the level-3 collars are only a rough indication of the actual loads there. Level-1 and level-2 collar loads were measured more accurately. When the building and trusses are raised in 1984, low-friction bearing surfaces will be provided for the truss system. Teflon and stainless steel sheets will be inserted between the contact surfaces at these points, using a design developed cooperatively

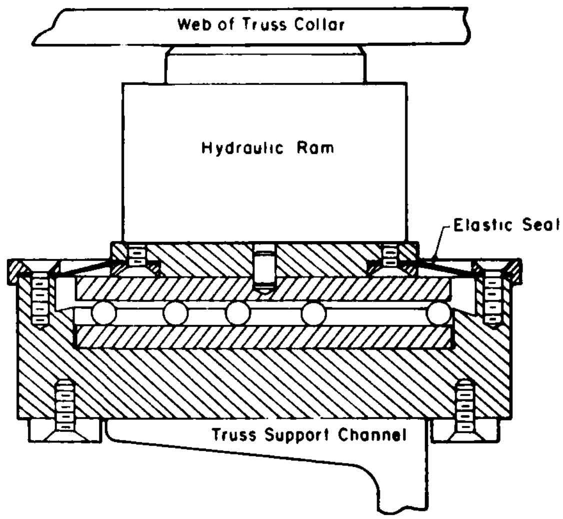


Figure 12. Cross section of friction-reducing roller devices.

Figure 13. Friction-reducing roller device in place.

by CRREL and Danish-American contractors. This should reduce friction by about 90%.

DISCUSSION OF RESULTS

Free-body diagrams of each column and the connecting trusses are presented in Appendix B. The free-body diagrams of the trusses assume simple, two-dimensional plane configurations. In reality the connecting trusses form two three-dimensional truss assemblies (Fig. 3b), each of which encompasses four columns.

A summation of the forces on the truss free-body diagrams indicates that they are not in equilibrium, i.e., the loads imposed on the trusses do not sum to zero; there are unbalanced loads. When the three-dimensional nature of the trusses is considered, the imbalance is reduced, but it is not eliminated. Some of the factors contributing to this measured imbalance are 1) the assumption in the truss free-body diagrams of a simple, two-dimensional plane configuration, 2) the portion of the load not measureable because of the friction at the level-3 collar discussed earlier, 3) the degree of accuracy of our measuring technique, and 4) the building wind loads during a force measurement. We considered factors 3 and 4 to be minor; factor 1 may explain some of the unbalanced load, but we believe factor 2 (friction) to be the largest contributor to the imbalance. Examination of Appendix B indicates that significant imbalances exist that make it difficult to accurately define the secondary stresses in the structure. Once frictional resistance is essentially eliminated at the level-3 collars, a far better understanding of the secondary stresses in the DYE-3 structural frame should be possible. As discussed earlier, efforts will be made during the next scheduled lifting operation in 1984 to reduce friction at all truss support points.

Imbalances also exist on the building (Table 1), but they cannot be blamed on truss friction. We believe lateral resistance between the columns and the building on the second floor, where the building hangs from the columns on 6-in.-diameter rods, explains most of the building imbalance. A tilted building may also contribute to the imbalance.

Friction problems notwithstanding, there was a significant increase and change in direction of some of the collar loads from 1978 to 1981 (compare Fig. B1 and B2). We anticipated changes because the 1978 measurements were made immediately after the building had been raised and many of the lateral loads then should have been relatively low. However, the magnitude of some

Table 1. Imbalance in building load (kips).

	1978	1981	1982	1983
Across column rows*	35.0	34.9	42.7	34.2
Along column rows**	13.4	18.9	13.9	28.5

*All are in the A to N direction (see Fig. 2b).
**All are in the column 4 to column 1 direction (see Fig. 2b).

of the changes surprised us. One building level load at column N2 changed from 3.0 kips in one direction in 1978 to 49.7 kips in the opposite direction in 1981. By 1982, the same load increased further to 65.0 kips. In 1983, the load was 89.9 kips.

In 1978, no collar load exceeded 50 kips. In 1981, 12 loads exceeded 50 kips with two of these over 100 kips. In 1982, 14 loads exceeded 50 kips with four of these over 100 kips. In 1983, 18 loads exceeded 50 kips with four exceeding 100 kips. Of the loads exceeding 100 kips, three of the four were at the same position in 1982 and 1983.

Before taking readings in 1983, we reduced the load on several highly loaded sway bolts with the objective of reducing stress concentrations in the structural frame. Had we not done this, we expect even more sway bolts would have had loads exceeding 50 kips in 1983. The combined stress factors calculated for the lower end of the columns from the four surveys are summarized in Table 2. These factors have been determined from eq 1. The bending moments are calculated from the measured forces and it is assumed that one-half of the moment on each full column is carried by each column

Table 2. Combined stress factors at the base of each column.

Column	1978	1981*	1981	1982	1983
A1	0.25	0.45	0.43	0.66	0.79
A2	0.34	0.55	0.54	0.78	0.59
A3	0.68	0.37	0.43	0.33	1.00
A4	0.36	0.16	0.16	0.19	0.32
N1	0.53	0.55	0.57	0.42	0.64
N2	0.35	0.20	0.20	0.32	0.33
N3	0.66	0.54	0.53	0.23	0.49
N4	0.72	0.21	0.23	0.54	0.41
Average	0.49	0.38	0.39	0.43	0.57

* Friction-reducing roller used.

half. It is evident from the measurements in Appendix A that the column halves are not sharing collar loads in most cases, but since the column halves are tied together at several places we expect that there is some overall load.

In 1978 the average stress factor was 0.49 with a high of 0.72 and a low of 0.25. In 1981 both sets of readings, one without the friction reducing roller devices in place and one with them in place, resulted in considerably lower average factors of 0.38 and 0.39. In 1982, the average factor increased to 0.43 with a high of 0.78, which was well below the allowable value of 1.00. In 1983 the average factor was 0.57 with a high of 1.00 at column A3. The jump at column A3 from 0.33 on 1982 to 1.00 in 1983 is probably the result of the load adjustments prior to the 1983 measurements. While those adjustments reduced seven very high sway bolt loads, those loads were transferred to other locations, and in the case of column A3, large bending moments were generated at its base. We are convinced that the load adjustments were needed but it is obvious that in such a highly indeterminate structure such as this, load adjustments pose potential risks as well as benefits.

The column axial stress f_a in eq 1 is composed of the stresses created by a portion of the building and truss weight and the weight of the column itself. It should be noted that the stress factor, f_a/F_a, is 0.15 for columns A1 and A3, which is the recommended limit for use of eq 1 (Table A6). Any changes in building load or addition of column extensions could cause f_a/F_a to exceed 0.15, thereby limiting the usefulness of the equation.

High pinching and spreading loads (i.e., loads on a column at a collar in opposing directions) existed at the building level in 1978 (see, for example, sway bolt loads at column A2, building level, in Figure A1a). This condition usually occurs between column halves. Pinching and spreading loads were also present in 1981 and 1982. In 1983 all of the pinching loads were eliminated prior to load measurements.

Secondary stresses are accumulating in the DYE-3 structural frame because of differential settlement and tilt of the footings, and horizontal distortion of the footing system by lateral flow of the ice cap. Horizontal distortion was measured some years ago at DYE-3 (Flax et al. 1971). Differential settlements and footing tilts measured since the sideways move in 1977 are presented in Figures 14 and 15 respectively.

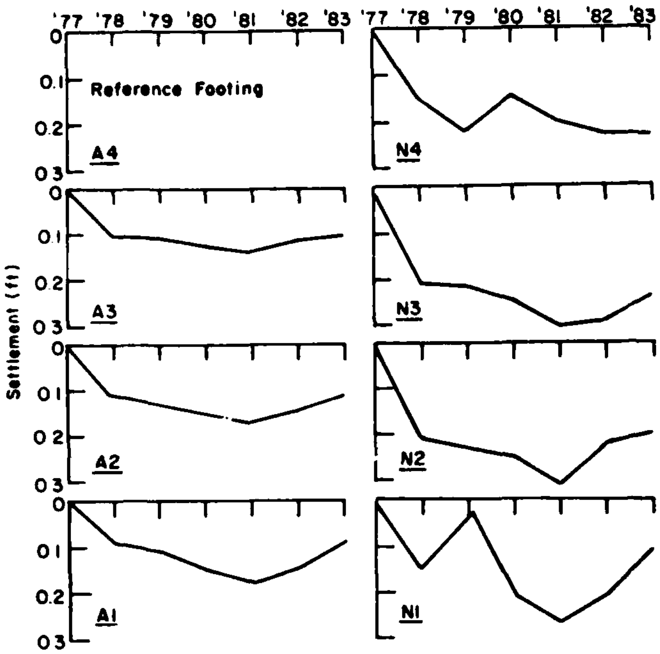


Figure 14. Footing settlements relative to footing A4, 1977-83.

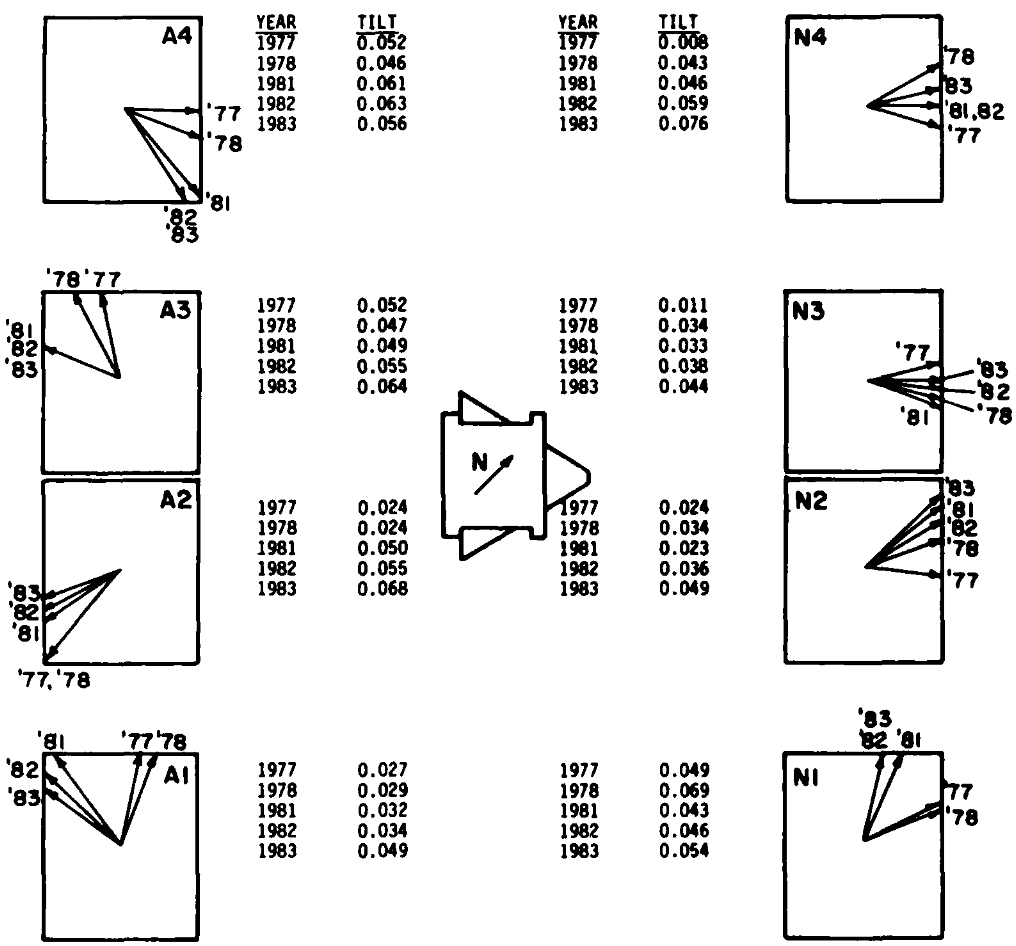


Figure 15. Footing tilts, 1978-83 (in./ft). Arrows indicate downward tilt directions.

The settlement of DYE-3 relative to the bedrock over a mile below is not known, but it probably amounts to a few feet per year. Absolute settlement of this sort is of no particular concern but differential settlement among the eight footings certainly is. In a highly indeterminate structure such as this, differential settlements can induce significant secondary stresses in the structure frame. The settlements presented in Figure 14 are referenced to footing A4 since it has settled less than the other seven footings.

The largest differential settlements occurred just after the building was moved. During the past 5 years, differential settlement among the footings has reached 0.3 ft. When assessing the impact of differential settlement, it is important to consider the distance between footings. For example, the 0.12 ft differential between footings A3 and A4, which are only 45 ft apart, may induce greater secondary stresses on the A2-A3 truss and the building frame in that area than the 0.23 ft differential between footings A4 and N4, which are 120 ft apart. When the building and trusses are raised in 1984, they will be releveled which, in principle at least, removes all secondary stresses in the frame caused by differential settlement and tilt of the footings.

Footing tilt measurements are summarized in Figure 15. For some footings, 1983 tilt is about the same as the initial "built-in" tilt. Footings A3, A4 and N1 fall into this category. The tilt of the other footings has increased over the past 5 years.

Do bending stresses in the columns cause the footings to tilt or do the tilting footings cause bending stresses in the columns? If bending stresses caused the footings to tilt, their tilt would increase over the years in the direction of the bending. Since this is not generally the case, it appears that the tilting footings are inducing bending stresses in the columns and other stresses in the trusses and the building frame. The snow surface under the building is lower than the surface surrounding the building. The extra overburden pressure surrounding the building tends to cause the footings to tilt away from the center of the building. In addition, strength and density differences of the snow on which the footings are founded causes them to tilt. That snow is far from homogeneous since it has been pushed around and built up by operations in that area since 1959.

All CRREL sway bolt measurements and stress calculations were furnished to and studied by Metcalf and Eddy engineers when the DYE-3 1984 life extension design was developed. Computer-assisted structural analyses done by Metcalf and Eddy in 1983 on the DYE-3 structure system show that footing tilts account for a larger portion of the secondary stresses in the structural frame than do differential settlements.

When DYE-3 is lifted in 1984, each column base will be shimmed in an effort to uniformly distribute its load at the four points where the column attaches to the footing. This should eliminate bending moments at the base of that column. However, as other columns are subjected to this procedure and their bending moments are redistributed, it is expected that some bending stresses will develop in each column. It is hoped that the net effect will be to reduce the overall level of secondary stress in the structural frame. Subsequent sway bolt measurements will determine if this has been accomplished. If, with time, secondary stresses of concern develop in the columns because of footing tilt, the columns can again be shimmed, one by one, where they attach to the footings to relieve bending stresses.

SUMMARY AND RECOMMENDATIONS

Sway bolt load measurements were made at DYE-3 in 1978, after completion of the life extension program, and again in 1981, 1982 and 1983. Significant increases in loads have been discovered at several locations, but accurate determination of all interaction loads has not been possible because of frictional resistance present where the trusses bear on the columns. Attempts have been made to reduce the friction there but they have met with only limited success. Plans have been made to support the trusses on Teflon and stainless steel sheets when the building is lifted in 1984. This should reduce friction by about 90%. The number of sway bolt loads beneath the building that exceed 50 kips has increased from none in 1978 to 14 in 1982 to 18 in 1983, with four loads exceeding 100 kips in 1982 and 1983. In 1983, high pinching or opposing loads were eliminated at all levels.

The combined stress factor, which is used to determine allowable column stresses under combined bending and axial compression, dropped from an average of 0.48 in 1978 to 0.38 in 1981. A column is considered overstressed if the computed factor equals or exceeds 1.0. In 1982, the average increased to 0.43 with a high value of 0.78 at the base of the column. In 1983, the average increased to 0.57 with a high value of 1.00 at the base of column A3. We expect that localized overstresses will be present in 1984 -- a major reason why the life extension work is needed in 1984.

The measurements presented in this report were used in developing the approach to the 1984 life extension design for DYE-3. When the DYE-3 building and trusses are lifted and leveled in 1984, secondary stresses because of differential settlement will be eliminated. By shimming the base of each column at that time, it should be possible to reduce secondary stresses caused by footing tilt.

It has been over 6 years since DYE-3 was moved sideways onto a new foundation and over 5 years since the building was raised 27 ft. Although secondary stresses have accumulated in the structural frame, allowable stresses have not been exceeded through 1983. Some localized overstress is expected in 1984 and it is therefore appropriate that the building and trusses be lifted and leveled then. The concept of using above-surface trusses and eliminating the problematic below-surface truss enclosures of previous designs has proven to be highly satisfactory. The life extension work planned for 1984 should significantly reduce the level of secondary stresses in the DYE-3 structural frame, preparing it for several more years of useful life on the ever-distorting Greenland Ice Cap.

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Tobiasson, W. (1978) Snow studies associated with the sideways move of DYE-3. Proceedings of the 36th Eastern Snow Conference, 7-8 June, Alexandria Bay, New York.

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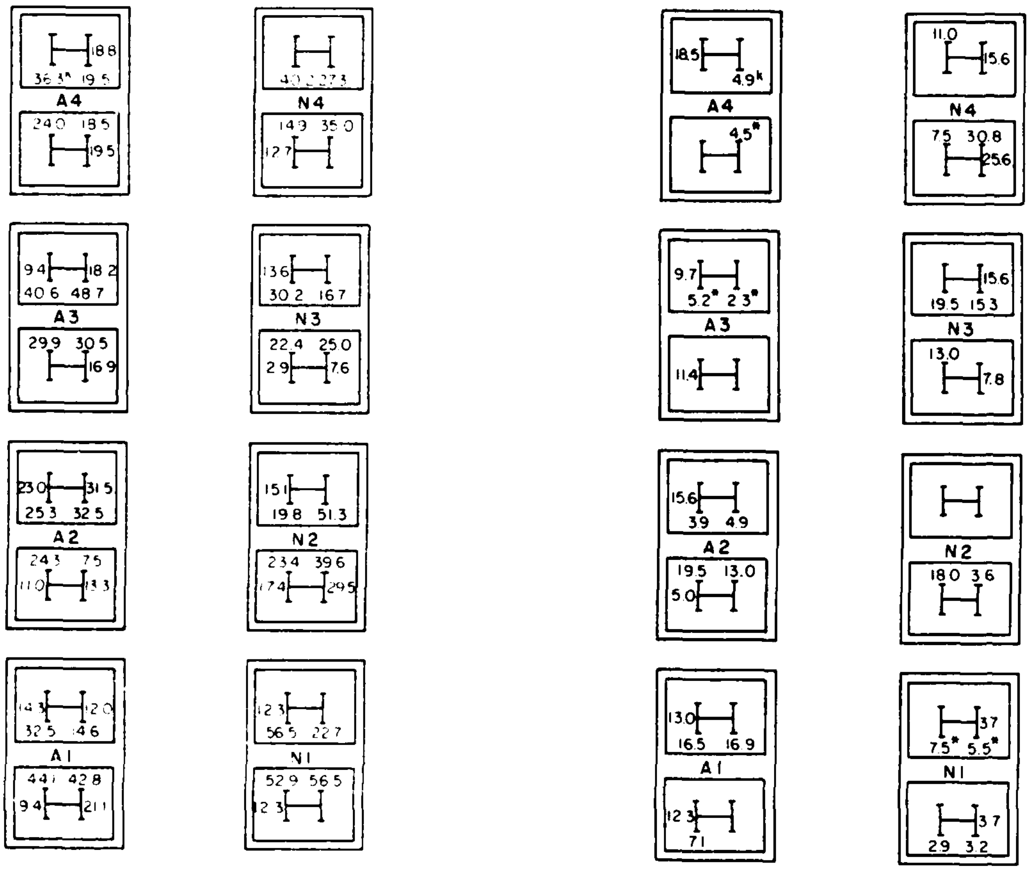
APPENDIX A: SWAY BOLT MEASUREMENTS AND COLUMN BENDING MOMENTS.

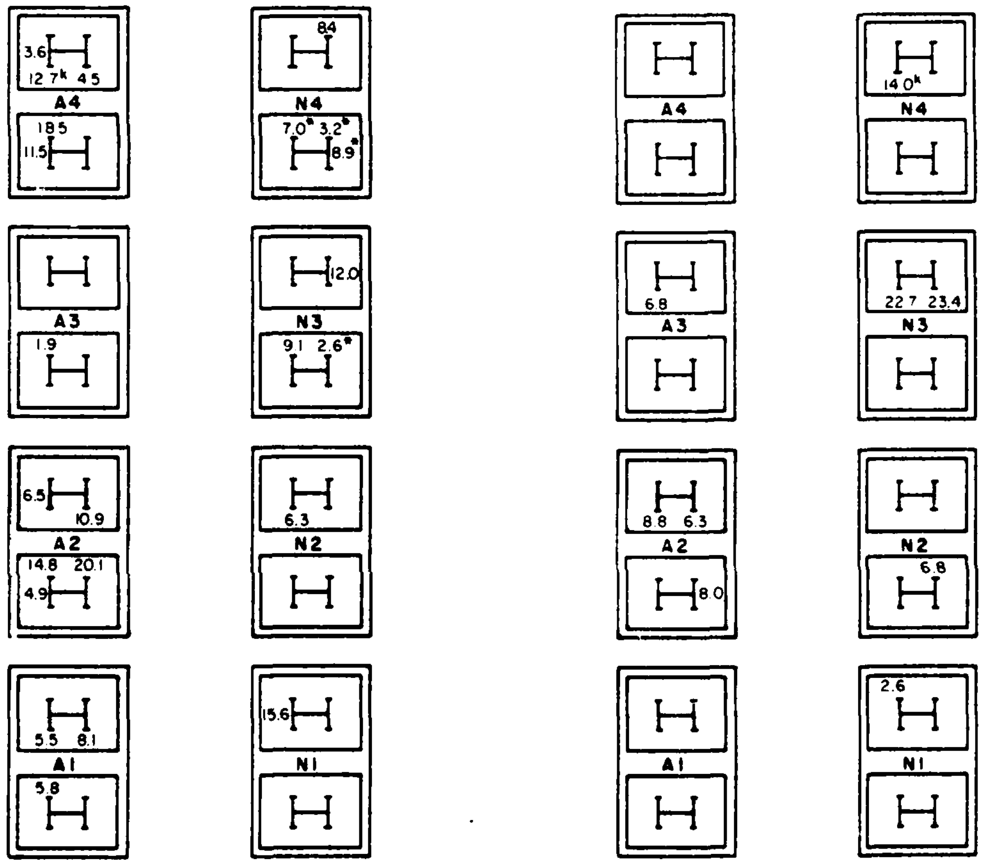
1978 Measurements



a. Building level.

b. Level 1.

Figure A1. Individual sway bolt load measurements (in kips [1000 lb], * = load displacement measurement).



c. Level 2.

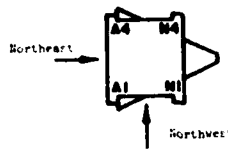
d. Level 3.

Figure A1 (cont'd). Individual sway bolt load measurements (in kips [1000 lb], * = load displacement measurement).

Table A1. Column bending moments.

DYE 3

Date Aug 1978



LEVEL COLUMN

Viewing direction: Northwest	A1	A2	A3	A4	N1	N2	N3	N4
1 (level immediately beneath building)	89	103	244	364	-234	-29	-85	-26
2	-54	14	286	542	-389	-56	46	321
3	-276	-282	350	608	-848	-98	417	984
Base of column	-706	-639	474	735	-1733	-179	1132	2264
Viewing direction: Northeast								
1 (level immediately beneath building)	378	-152	-275	-126	287	-77	5	-167
2	372	-83	-602	-250	387	45	-187	118
3	253	361	-1085	-423	542	145	-321	822
Base of column	23	809	-2200	-758	912	523	-1825	1802

Clockwise moments are positive
Moments are in ft-kips

Table A2. Combined stress factor (analysis of column halves).

Dye 3

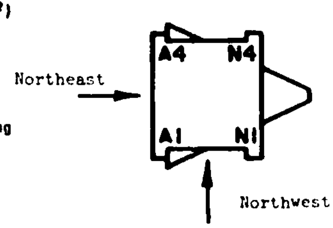
Date Aug 1978

Collar level Bottom

Column	$\frac{P}{A} = f_a$	$\frac{f_a}{F_a}$	M_{xx}	$\frac{M_{xx}}{S_{xx}} = f_{bx}$	$\frac{f_{bx}}{F_b}$	M_{yy}	$\frac{M_{yy}}{S_{yy}} = f_{by}$	$\frac{f_{by}}{F_b}$	$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b}$
A1	2.94	0.15	-353	1.76	0.09	12	0.10	0.01	0.25
A2	1.79	0.09	-319	1.59	0.08	405	3.46	0.17	0.34
A3	2.94	0.15	237	1.18	0.06	-1100	9.41	0.47	0.68
A4	2.13	0.11	367	1.83	0.09	-379	3.24	0.16	0.36
N1	2.13	0.11	-867	4.32	0.22	456	3.90	0.11	0.53
N2	2.20	0.11	-89	0.44	0.02	261	2.23	0.22	0.35
N3	2.60	0.13	566	2.82	0.14	-913	7.81	0.39	0.66
N4	2.40	0.12	1132	5.64	0.28	901	7.71	0.39	0.72
									0.49 Ave.

P = Axial load (kips)
A = Column cross-sectional area (167 in.^{2})
f_a = Axial stress (ksi)
f_{bx} = Bending stress across column rows (ksi)
f_{by} = Bending stress along column rows (ksi)
F_a = Allowable axial stress with no bending stress (20 ksi)

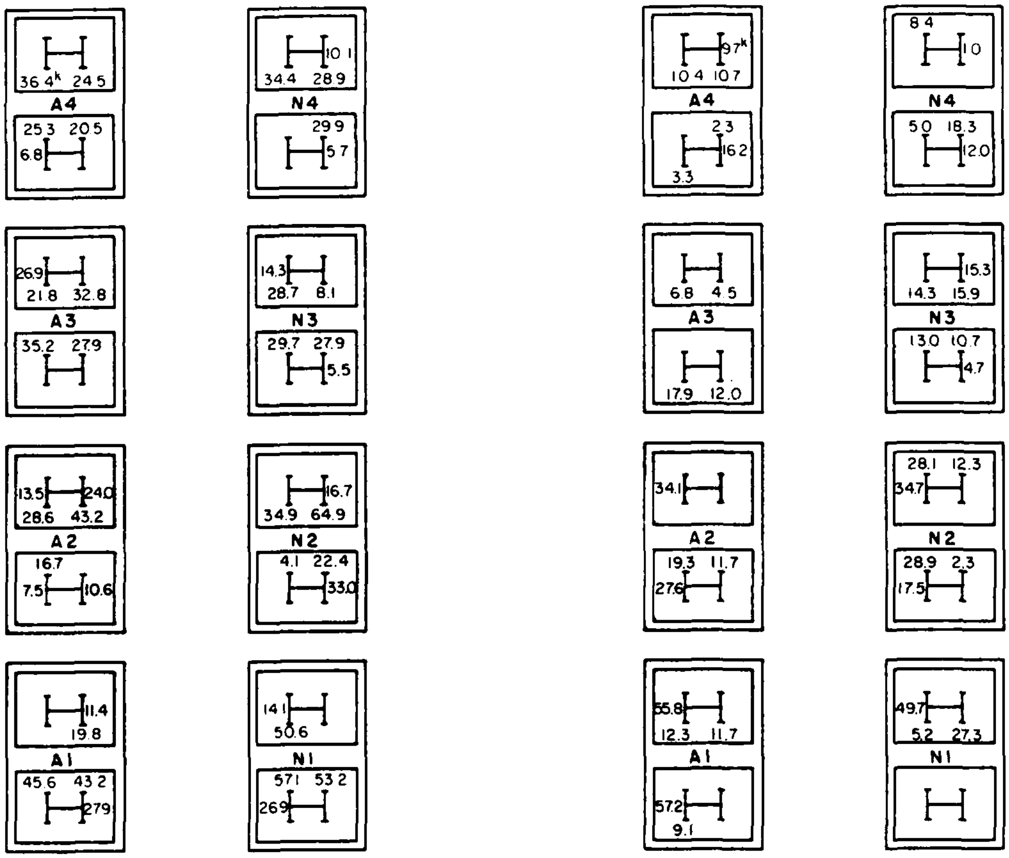
VIEWING DIRECTION



F_b = Allowable bending stress with no axial stress (20 ksi)
M_{xx} = Bending moment across column rows (ft kips)
M_{yy} = Bending moment along column rows (ft-kips)
S_{xx} = Section modulus across column rows (2410 in.^{3})
S_{yy} = Section modulus along column rows (1403 in.^{3})

Clockwise moments are positive.

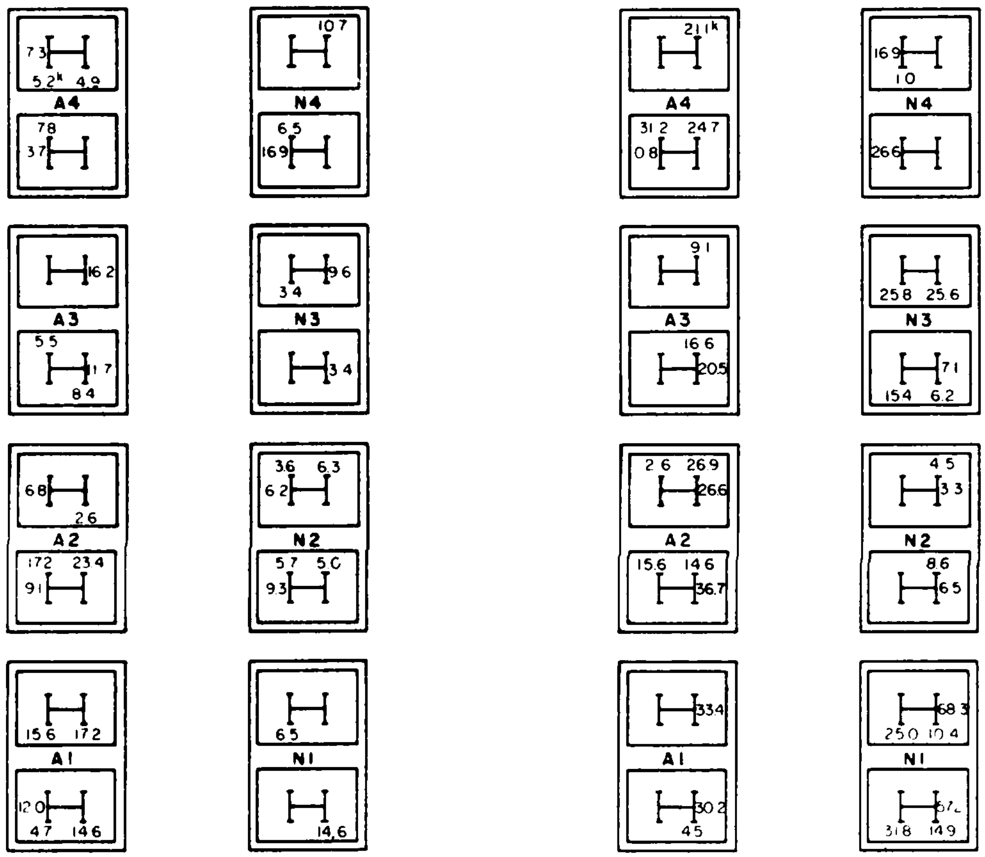
1981 Measurements



a. Building level.

b. Level 1.

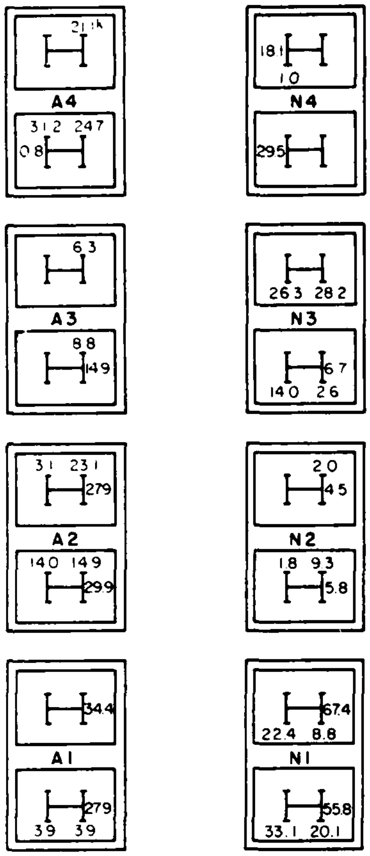
Figure A2. Individual sway bolt load measurements (in kips [1000 lb]).



c. **Level 2.**

d. **Level 3, with friction-reducing devices.**

Figure A2 (cont'd). Individual sway bolt load measurements (in kips [1000 lb]).



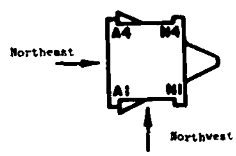
e. Level 3, without friction-reducing devices.

Figure A2 (cont'd).

Table A3. Column bending moments.

DYE 3

Date Jun 1981



LEVEL COLUMN

Viewing direction: Northwest	A1	A2	A3	A4	N1	N2	N3	N4
1 (level immediately beneath building)	-373	-129	256	65	390	-472	84	-150
2	290	304	498	-107	1206	-450	-17	-409
3	1490	1200	484	-221	2476	-198	-356	-576
Base of column	2086	1219	-97	-418	1536	24	-1201	277
(without roller devices)	2122	1367	54	-418	1598	10	-1190	388
Viewing direction:								
1 (level immediately beneath building)	-656	524	-81	143	-567	696	-198	317
2	-979	740	214	478	-812	712	-326	332
3	-752	546	712	1031	-897	447	-479	116
Base of column	-193	-1442	979	19	1155	-417	1198	-276
(without roller devices)	-104	-1317	1265	19	1217	-417	1147	-276

Clockwise moments are positive
Moments are in ft-kips

Table A4. Combined stress factor (analysis of column halves).

a. With roller devices.

Dye 3

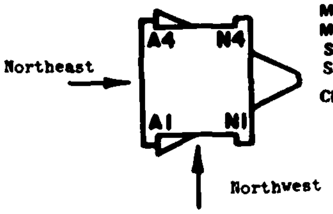
Date June 1981

Collar level Bottom (with roller devices)

Column	$\frac{P}{A} = f_a$	$\frac{f_a}{F_a}$	M_{xx}	$\frac{M_{xx}}{S_{xx}} = f_{bx}$	$\frac{f_{bx}}{F_b}$	M_{yy}	$\frac{M_{yy}}{S_{yy}} = f_{by}$	$\frac{f_{by}}{F_b}$	$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b}$
A1	2.94	0.15	1043	5.19	0.26	-97	0.83	0.04	0.45
A2	1.79	0.09	609	3.03	0.15	-721	6.17	0.31	0.55
A3	2.94	0.15	-48	0.24	0.01	490	4.19	0.21	0.37
A4	2.13	0.11	-209	1.04	0.05	9	0.08	0.00	0.16
N1	2.13	0.11	768	3.82	0.19	577	4.94	0.25	0.55
N2	2.20	0.11	12	0.06	0.00	-209	1.79	0.09	0.20
N3	2.60	0.13	-601	2.99	0.15	599	5.12	0.26	0.54
N4	2.40	0.12	139	0.69	0.03	-138	1.18	0.06	0.21
									0.38 Ave.

P = Axial load (kips)
A = Column cross-sectional area (167 in.^{2})
f_a = Axial stress (ksi)
f_{bx} = Bending stress across column rows (ksi)
f_{by} = Bending stress along column rows (ksi)
F_a = Allowable axial stress with no bending stress (20 ksi)

VIEWING DIRECTION



F_b = Allowable bending stress with no axial stress (20 ksi)
M_{xx} = Bending moment across column rows (ft-kips)
M_{yy} = Bending moment along column rows (ft-kips)
S_{xx} = Section modulus across column rows (2410 in.^{3})
S_{yy} = Section modulus along column rows (1403 in.^{3})
Clockwise moments are positive.

Table A4 (cont'd).

b. Without roller devices.

Dye 3

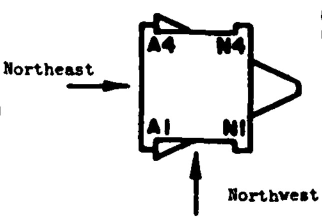
Date June 1981

Collar level Bottom (without roller devices)

Column	$\frac{P}{A} = f_a$	$\frac{f_a}{F_a}$	M_{xx}	$\frac{M_{xx}}{S_{xx}} = f_{bx}$	$\frac{f_{bx}}{F_b}$	M_{yy}	$\frac{M_{yy}}{S_{yy}} = f_{by}$	$\frac{f_{by}}{F_b}$	$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b}$
A1	2.94	0.15	1061	5.28	0.26	-52	0.44	0.02	0.43
A2	1.79	0.09	684	3.41	0.17	-659	5.64	0.28	0.54
A3	2.94	0.15	27	0.13	0.01	633	5.41	0.27	0.43
A4	2.13	0.11	-209	1.04	0.05	9	0.08	0.00	0.16
N1	2.13	0.11	799	3.98	0.20	608	5.20	0.26	0.57
N2	2.20	0.11	5	0.02	0.00	-209	1.79	0.09	0.20
N3	2.60	0.13	-595	2.96	0.15	573	4.90	0.25	0.53
N4	2.40	0.12	194	0.97	0.05	-138	1.18	0.06	0.23
									0.39 Ave.

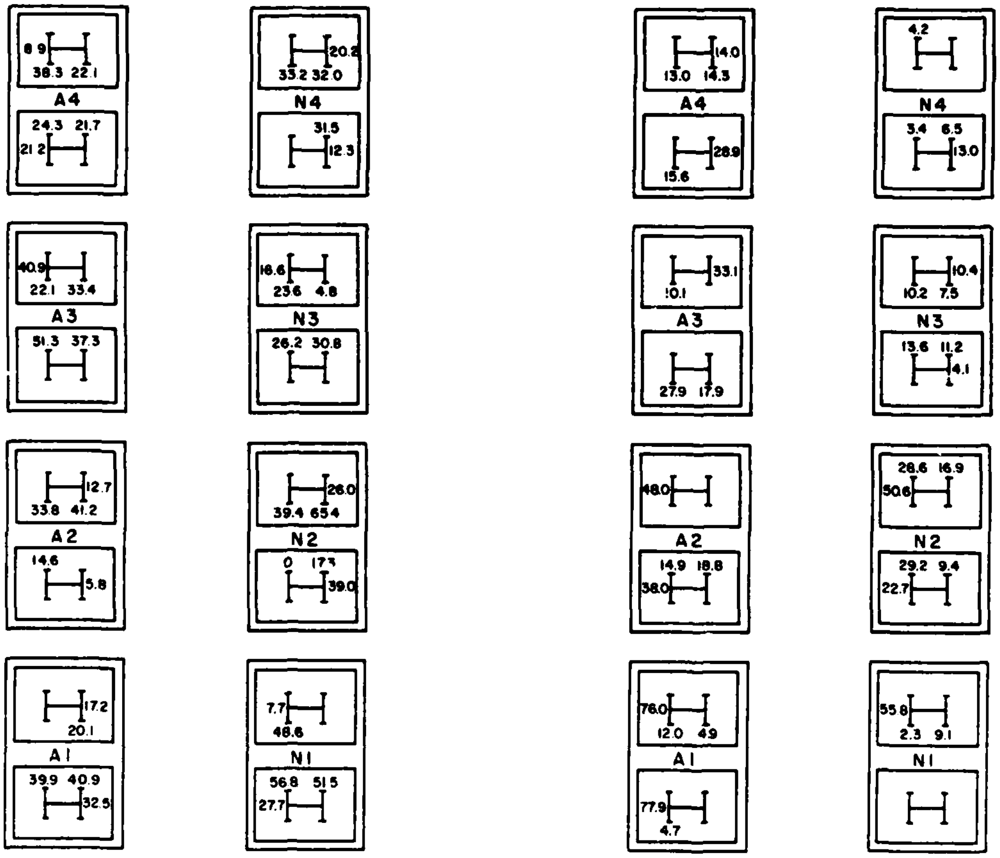
P = Axial load (kips)
A = Column cross sectional area (167 in.^{2})
f_a = Axial stress (ksi)
f_{bx} = Bending stress across column rows (ksi)
f_{by} = Bending stress along column rows (ksi)
F_a = Allowable axial stress with no bending stress (20 ksi)

VIEWING DIRECTION



F_b = Allowable bending stress with no axial stress (20 ksi)
M_{xx} = Bending moment across column rows (ft-kips)
M_{yy} = Bending moment along column rows (ft-kips)
S_{xx} = Section modulus across column rows (2410 in.^{3})
S_{yy} = Section modulus along column rows (1403 in.^{3})
Clockwise moments are positive.

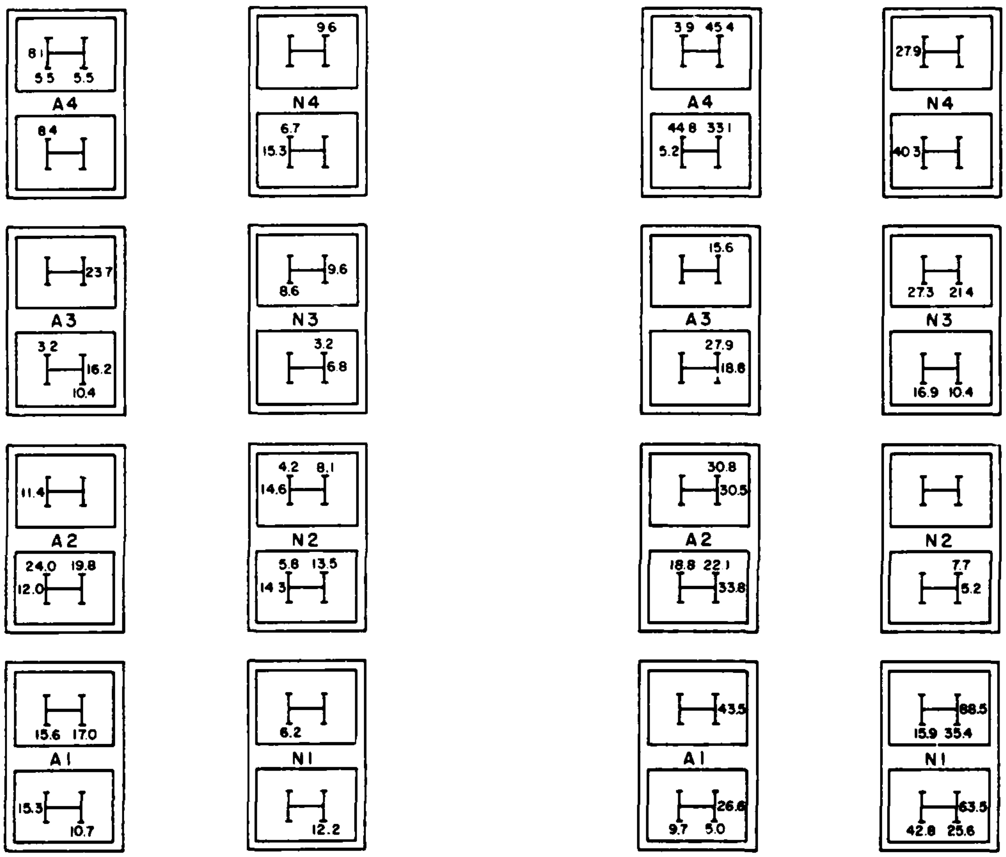
1982 Measurements



a. Building level.

b. Level 1.

Figure A3. Individual sway bolt load measurements (in kips [1000 lb]).



c. Level 2.

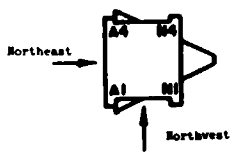
d. Level 3.

Figure A3 (cont'd).

Table A5. Column bending moments.

DYE 3

Date 8/4/82



LEVEL COLUMN

Viewing direction: Northwest	A1	A2	A3	A4	N1	N2	N3	N4
1 (level immediately beneath building)	-472.2	-175.8	388.6	286.0	336.3	-617.5	157.7	-308.8
2	465.6	431.8	458.8	170.8	1157.1	-542.8	176.6	-718.2
3	2138.6	1704.4	9.4	105.0	2433.9	-22.0	-23.6	-1141.0
Base of column	3472.4	2422.6	-1365.0	118.4	792.3	842.0	-409.7	-115.0
Viewing direction: Northeast								
1 (level immediately beneath building)	576.6	-573.8	314.4	-136.8	567.2	-831.2	271.7	-320.2
2	928.6	-814.1	109.3	-652.5	1001.8	-861.8	593.0	-496.6
3	869.8	-574.7	-310.8	-1491.1	1420.4	-467.0	1017.2	-542.8
Base of column	359.4	1822.9	53.8	326.0	-1004.2	502.2	-216.7	-631.8

Clockwise moments are positive
Moments are in ft-kips

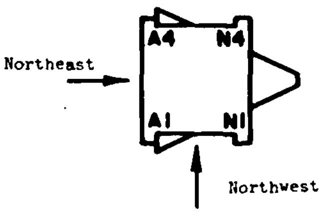
Table A6. Combined stress factor (analysis of column halves).

Dye 3
Date 5 Dec 82
Collar level Bottom

Column	$\frac{P}{A} = f_a$	$\frac{f_a}{F_a}$	M_{xx}	$6\,\frac{M_{xx}}{S_{xx}} = f_{bx}$	$\frac{f_{bx}}{F_b}$	M_{yy}	$6\,\frac{M_{yy}}{S_{yy}} = f_{by}$	$\frac{f_{by}}{F_b}$	$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b}$
A1	2.94	0.15	1736	8.64	0.43	180	1.54	0.08	0.66
A2	1.79	0.09	1211	6.03	0.30	911	7.80	0.39	0.78
A3	2.94	0.15	-683	3.40	0.17	27	0.23	0.01	0.33
A4	2.13	0.11	59	0.29	0.15	163	1.39	0.07	0.19
N1	2.13	0.11	396	1.97	0.10	-502	4.29	0.21	0.42
N2	2.20	0.11	421	2.10	0.10	251	2.15	0.11	0.32
N3	2.60	0.13	-205	1.02	0.05	-108	0.93	0.05	0.23
N4	2.40	0.12	- 58	0.29	0.01	-316	2.70	0.14	0.54
									0.43 Ave

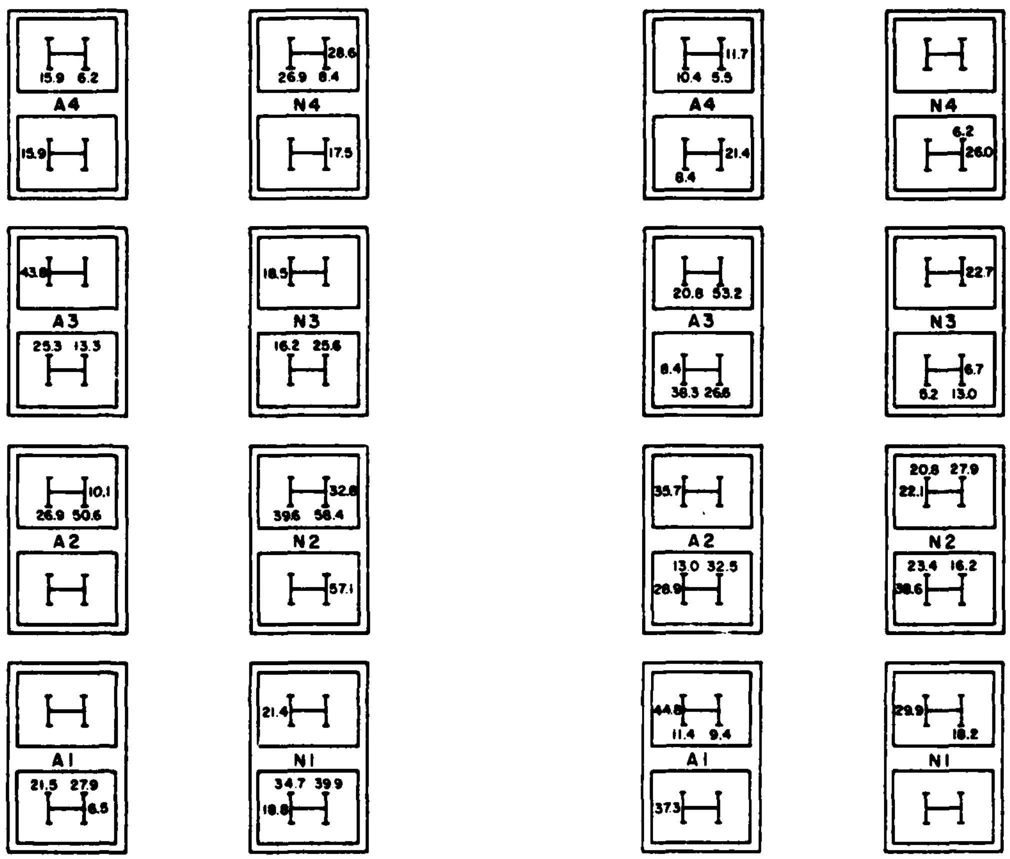
P = Axial load (kips)
A = Column cross-sectional area (168 in.^{2})
f_a = Axial stress (ksi)
f_{bx} = Bending stress across column rows (ksi)
f_{by} = Bending stress along column rows (ksi)
F_a = Allowable axial stress with no bending stress (20 ksi)

VIEWING DIRECTION



F_b = Allowable bending stress with no axial stress (20 ksi)
M_{xx} = Bending moment across column rows (ft-kips)
M_{yy} = Bending moment along column rows (ft-kips)
S_{xx} = Section modulus across column rows (2410 in.^{3})
S_{yy} = Section modulus along column rows (1403 in.^{3})
Clockwise moments are positive.

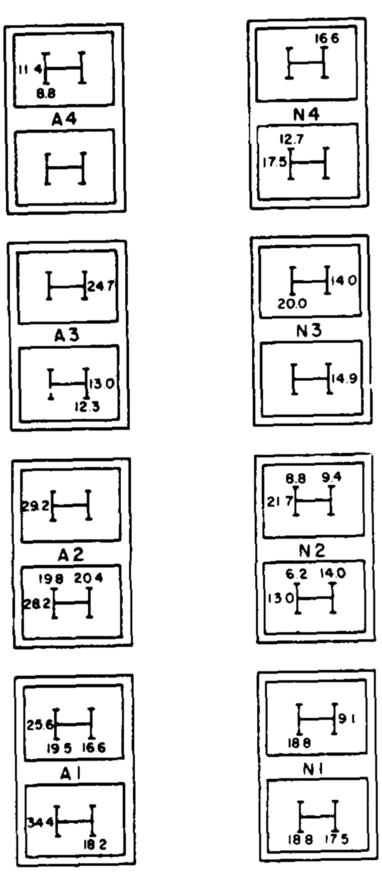
1983 Measurements



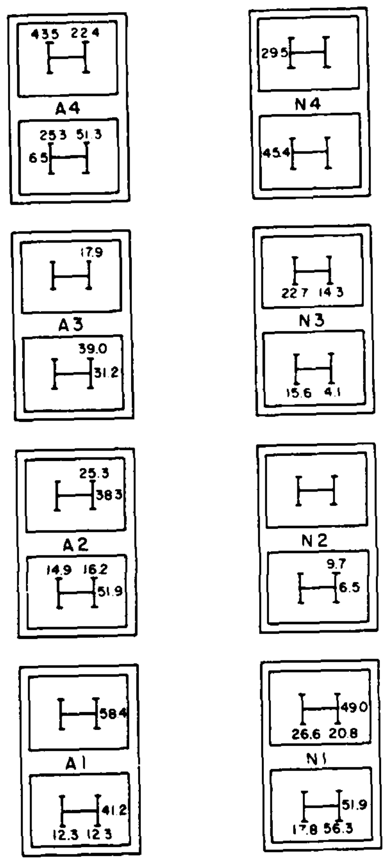
a. Building level.

b. Level 1.

Figure A4. Individual sway bolt load measurements (in kips [1000 lb]).



c. Level 2.



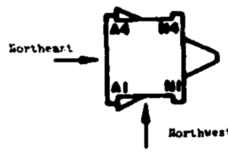
d. Level 3.

Figure A4 (cont'd).

Table A7. Column bending moments.

DYE 3

Date 1 Aug. 83



LEVEL COLUMN

Viewing direction: Northwest	A1	A2	A3	A4	N1	N2	N3	N4
1 (level immediately beneath building)	-61.8	-96.0	416.1	151.1	381.9	-854.1	175.8	-438.0
2	618.7	394.6	885.9	-3.8	1012.8	-1116.9	77.7	-1086.9
3	2517.1	1961.2	1088.9	-85.0	1866.8	-1039.9	-479.6	-1851.3
Base of column	3489.1	2547.1	638.0	-66.1	789.5	-1066.9	-1554.2	-1303.2
Viewing direction: Northeast								
1 (level immediately beneath building)	469.3	-736.3	366.7	-210.0	708.7	-931.0	397.1	-335.4
2	726.7	-1024.3	-536.0	-627.6	1216.3	-1018.3	609.5	-597.3
3	366.9	-909.5	-2112.4	-1400.4	1234.5	-616.5	659.9	-594.5
Base of column	-991.2	834.8	-3616.3	956.8	-2010.9	420.3	-773.8	-589.1

Clockwise moments are positive
Moments are in ft-kips

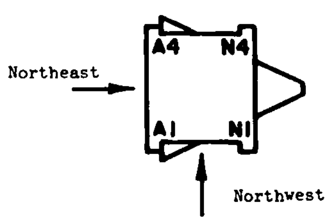
Table A8. Combined stress factor (analysis of column halves).

Dye 3
Date 1 August 1983
Collar level Bottom

Column	$\frac{P}{A} = f_a$	$\frac{f_a}{F_a}$	M_{xx}	$\frac{M_{xx}}{2}\frac{12}{S_{xx}} = f_{bx}$	$\frac{f_{bx}}{F_b}$	M_{yy}	$\frac{M_{yy}}{2}\frac{12}{S_{yy}} = f_{by}$	$\frac{f_{by}}{F_b}$	$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b}$
A1	2.94	0.15	3489	8.69	0.43	-991	4.24	0.21	0.79
A2	1.79	0.09	2547	6.34	0.32	835	3.57	0.18	0.59
A3	2.94	0.15	638	1.59	0.08	-3616	15.47	0.77	1.00
A4	2.13	0.11	-66	0.16	0.01	957	4.09	0.20	0.32
N1	2.13	0.11	789	1.97	0.10	-2010	8.60	0.43	0.64
N2	2.22	0.11	-1067	2.66	0.13	420	1.80	0.09	0.33
N3	2.60	0.13	-1554	3.87	0.19	-774	3.31	0.17	0.49
N4	2.40	0.12	-1303	3.24	0.16	-589	2.52	0.13	0.41
									0.53 Ave.

P = Axial load (kips)
A = Column cross-sectional area (168 in.^{2})
f_a = Axial stress (ksi)
f_{bx} = Bending stress across column rows (ksi)
f_{by} = Bending stress along column rows (ksi)
F_a = Allowable axial stress with no bending stress (20 ksi)

VIEWING DIRECTION



F_b = Allowable bending stress with no axial stress (20 ksi)
M_{xx} = Full column bending moment across column rows (ft-kips)
M_{yy} = Full column bending moment along column rows (ft-kips)
S_{xx} = Section modulus across column rows (2410 in.^{3})
S_{yy} = Section modulus along column rows (1403 in.^{3})
Clockwise moments are positive.

APPENDIX B: FREE-BODY DIAGRAMS FOR COLUMNS AND CONNECTING TRUSSES.

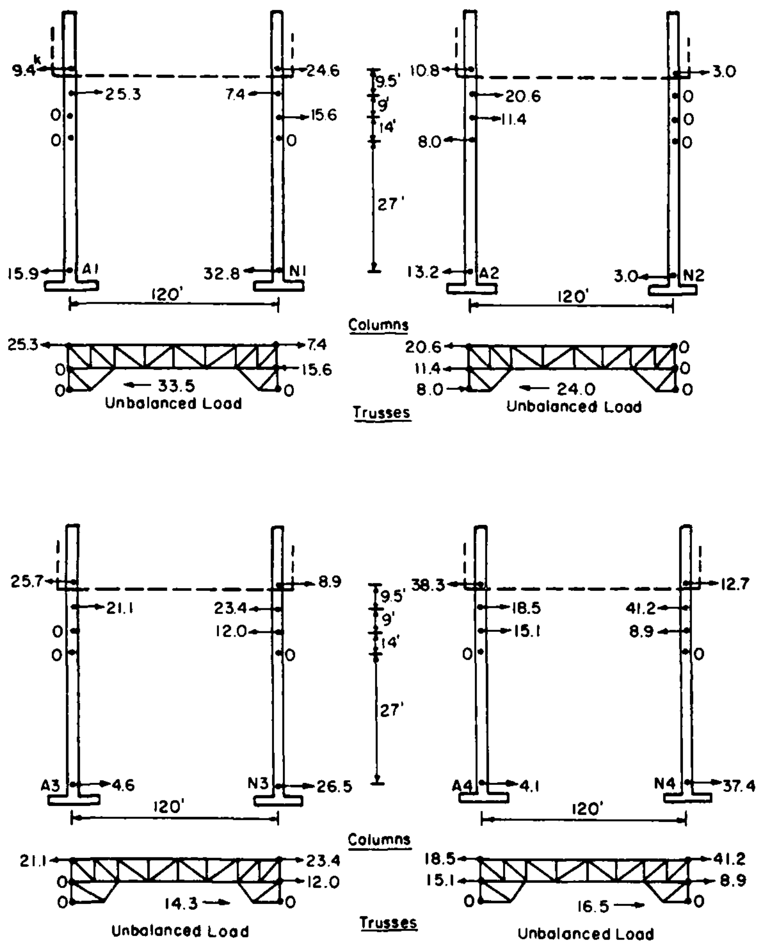


Figure B1. 1978.

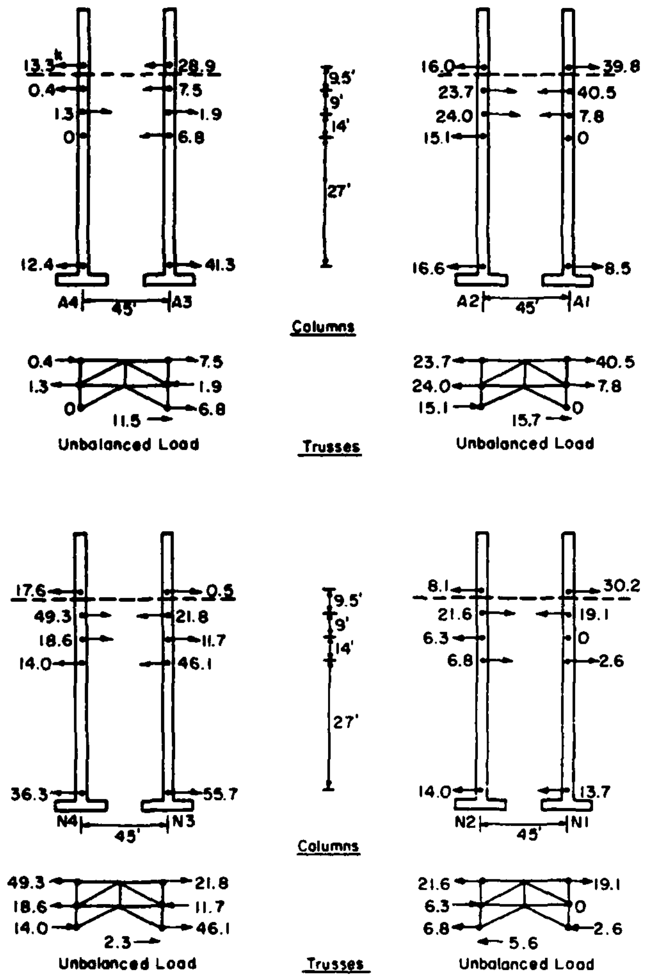


Figure B1 (cont'd). 1978.

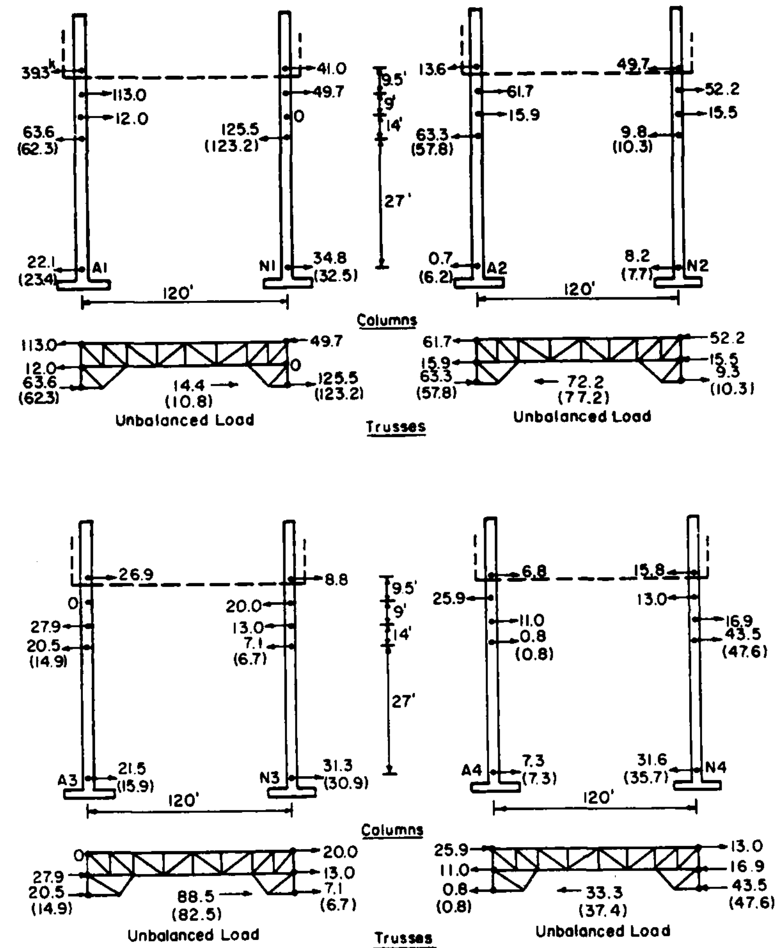


Figure B2. 1981 (values in parentheses were measured without friction-reducing devices).

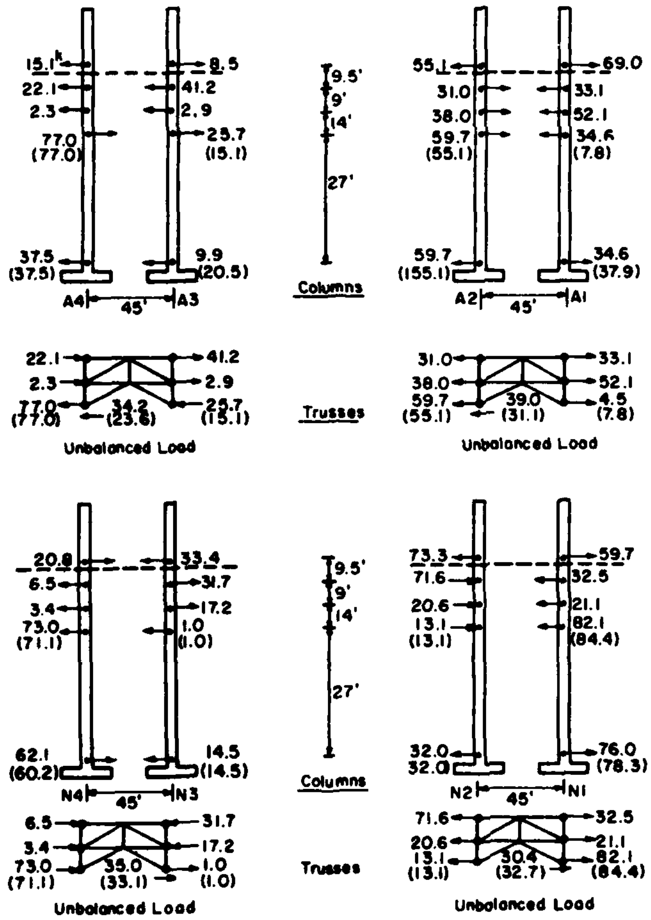


Figure B2 (cont'd). 1981 (values in parentheses were measured without friction-reducing devices).

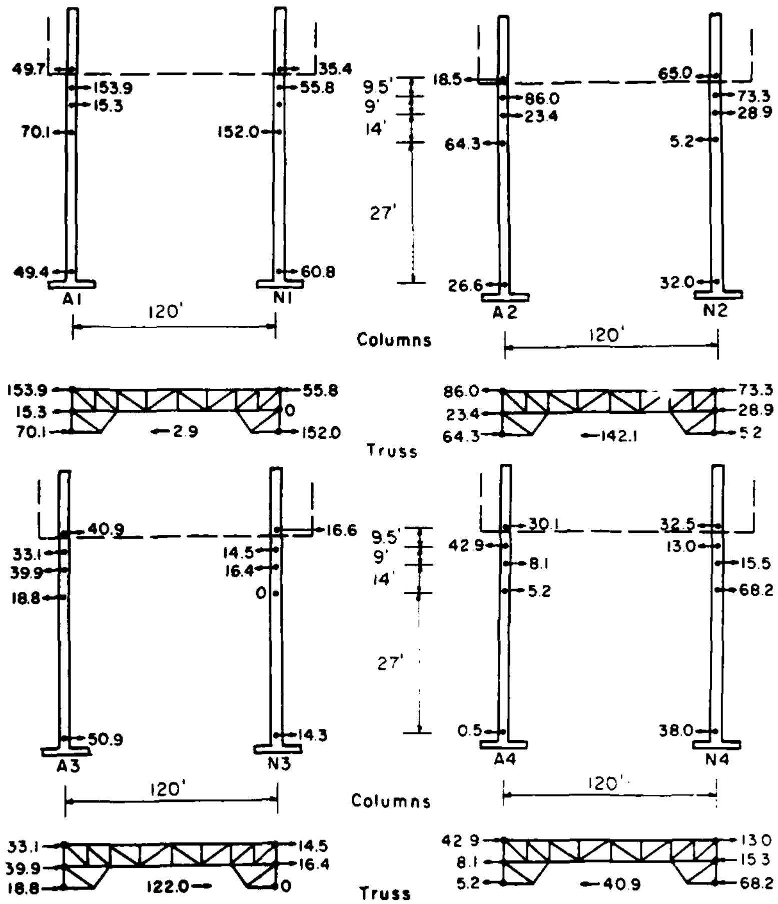


Figure B3. 1982.

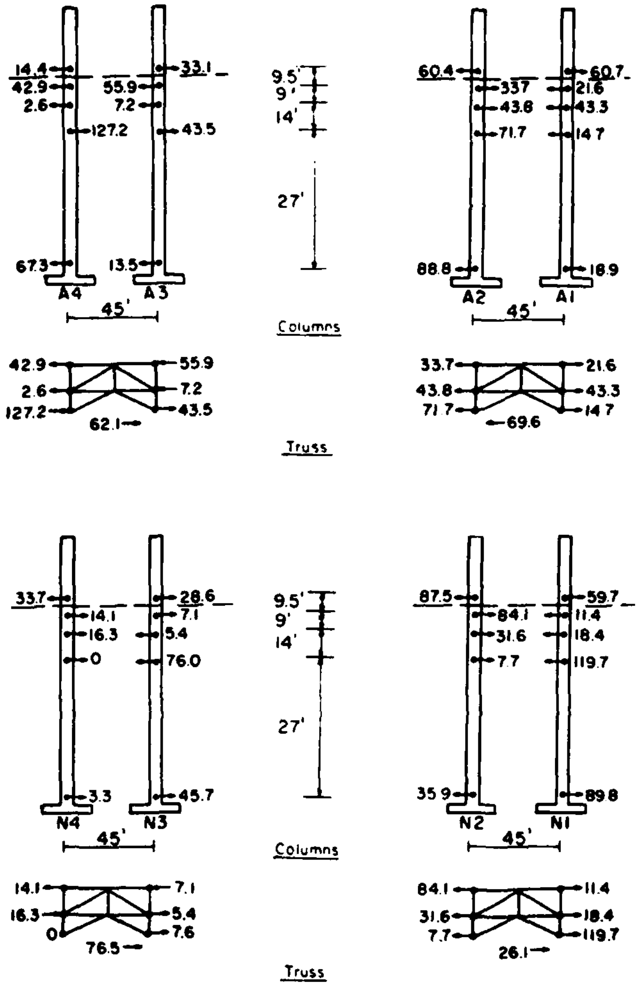


Figure B3 (cont'd). 1982.

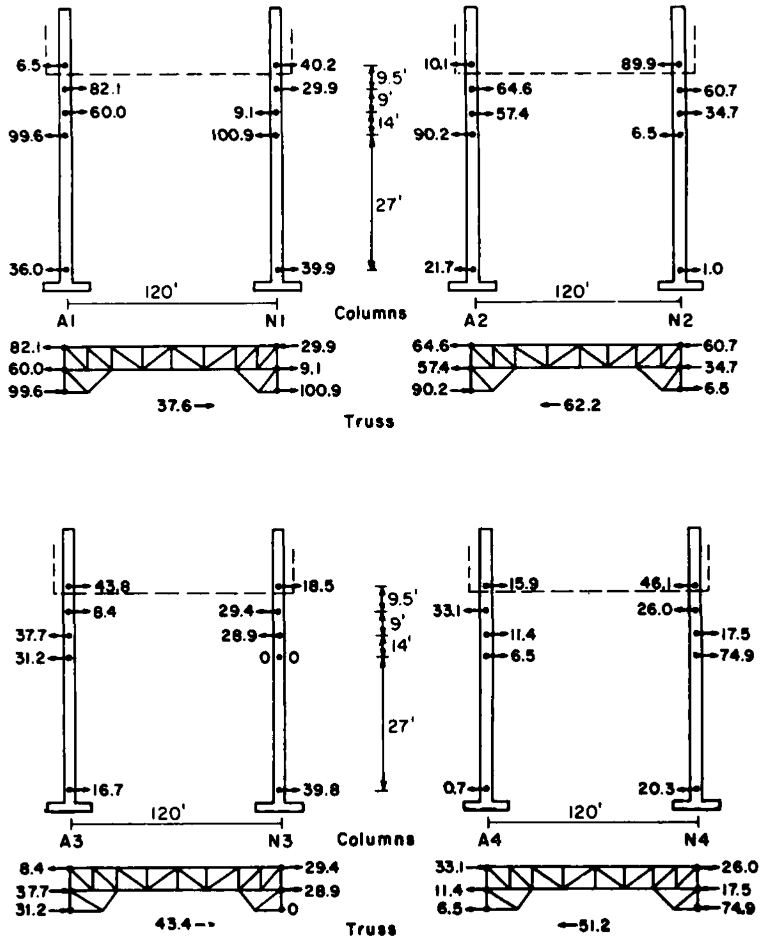


Figure B4. 1983.

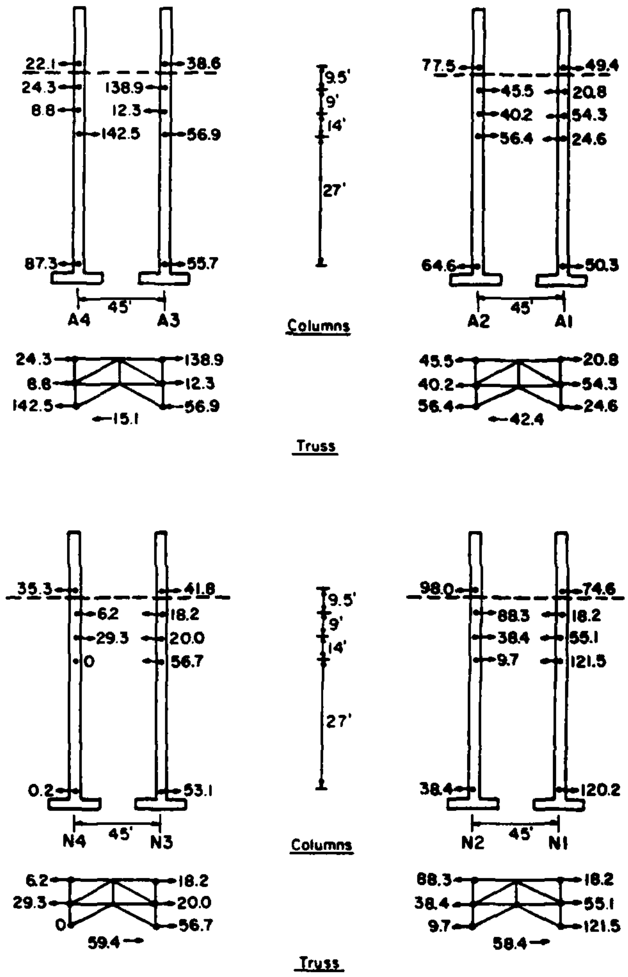


Figure B4 (cont'd). 1983.

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