Investigation of the June 20, 2001, Partial Collapse of the Mast Climbing Platform at Cambridge, MA

U.S. Department of Labor
Occupational Safety and Health Administration
Directorate of Construction

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REPORT

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Report prepared by
Mohammad Ayub, PE
Dinesh Shah, PE

Background:

The Directorate of Construction, National OSHA Office, was requested to provide assistance in the investigation and causal determination of the June 20, 2001, collapse of mast climbing platform during construction of a parking garage at Massachusetts Institute of Technology (MIT), Cambridge, MA. At your request, I examined the failed scaffold parts accompanied by the Area Office personnel on October 31 and November 28, 2001. After inspecting the failed scaffold framing, reviewing the pertinent documents, and conducting structural computations, we provide you the following report.

Incident:

The incident occurred at the construction site of a parking garage for MIT at Pacific and Landsdowne Sweet in Cambridge, MA on June 20, 2001, around 11:15 a.m. Three construction workers were hurt, one of them seriously, when they fell during the partial collapse of the platform. The workers on the scaffold fell 26 feet, and cinderblocks and bricks they had been laying on the exterior of the garage toppled over them. The incident had a potential of catastrophic proportions.

The entire mast-climbing platform did not fail. Only the side, which had four sections with the total length of 20 feet, collapsed (figure 6, & 7). The other side, which had three sections, remained intact and so did the tower and the fixed platform (figure 8 & 9).

Description of the project:

The project consisted of construction of a six-story parking garage, one of five buildings in a $750-million construction project near Landsdowne and Pacific streets in Cambridge, MA. The general contractor was William A. Berry & Son of Denver, CO. The subcontract for the masonry work was awarded to Milton based D.J. Construction Co. The scaffold for the masonry work was supplied and erected by Julian Crane & Equipment Co. of Watertown, MA.

The scaffold consisted of a mast climbing platform, 20 foot long fixed platform plus extensions on either side supported in the center by a single tower. At one end of the fixed platform, three sections, each 5-foot long were installed, while at other end of the fixed platform, four sections, each 5-foot long were installed. Operations Manual for Model No. 6000 published by the manufacturer, Dunlop Equipment Inc., shows a maximum of three sections on either side of the tower with a total length of 50 feet. The total length of the platform erected at the site, however, was 55-foot. Each section was approximately 54.5” x 39.75” x 25 3/8” high. The sections consisted of top and bottom chord of structural tubing (TS) 1 ½” x 1 ½”, while verticals and diagonals of TS 1 $^{1}/_{8}$” x 1 $^{1}/_{8}$”. These sections were interconnected by SAE ¾” diameter bolt at top tension chords, and with contact connection at bottom compression chords. Prior to the incident, three workers were laying bricks and cinderblocks on the exterior of the building from the 20-foot long platform, comprising of last four sections. It was determined, after the incident, that the masonry contractor had placed cinderblocks and bricks weighing 4890 pounds over the last four sections.

Observation of the failed section:

The four sections involved in the incident were salvaged by the North Boston Area Office and placed in a trailer for future examination by interested parties. Observation of the sections revealed that only one section, hereafter called Section No.1, which was placed closest to the fixed platform, was damaged (figure 1, & 2). The remaining three sections were practically intact, free from any visual distress (figure 5). A number of observations were made during the site visit:

1. The structural framing layout of all four sections was different. No two sections were identical. This indicated that sections were mismatched and came form different models of mast climbing platforms. This invalidated the load chart posted at the site (figure 3, & 4). A competent person, knowledgeable in structural design, must analyze and determine the safe load carrying capacity of the platform before it could be placed in service.
2. The two bottom horizontal members parallel to the longitudinal axis of the platform were sharply bent and deformed. Due to the deformation of the two bottom members, other members were also deformed and distressed.
3. The thickness of the structural tubes was reduced from their usual .125" to about 0.11" due to corrosive damage. It was observed that paint was applied over rusted steel.
4. As the original drawings were not available, field measurements of the member sizes and dimensions were taken.

Discussion:

D.J.Construction Co. (DJ), the masonry contractor, contracted with Julian Crane & Equipment Corporation (Julian) to furnish and erect mast-climbing platform at the site. Julian erected five units at the jobsite. One of the erected units was Model 6000, involved in the incident, which was believed to have been manufactured in United Kingdom. Julian acquired the unit from Dunlop Mastclimbers LLC in 1995 and since then is the owner of the scaffold. Julian posted a load chart at the site showing the magnitude of loads that can be safely placed over the main platforms and on extensions (figure 3, & 4). It is believed that the load chart belonged to the 6000 Model. For a maximum platform length of 50 feet, the total distributed load was given as 6100 pounds. The 50 feet length of the platform was comprised of 20 feet long fixed platform and three sections, each five feet long, attached on either side of the fixed platform. A total load of 6100 pounds over a length of 50 feet translated to a uniformly distributed load of 122 pounds per foot. So, for a section of five feet length, the maximum recommended load to be placed was 610 pounds on each section, up to a maximum of three sections on either side of the tower.

Julian, however, erected four sections on one side and three sections on the other, making the platform 55 feet long instead of 50 feet. The decision to make the platform 55 feet long was, reportedly, taken jointly by Julian and DJ. Julian did not had a load chart for four extensions on one side and three on the other. The load chart posted at the site was meant for three sections on either side of the tower. Julian did not determine the safe load carrying capacity of the platform with four sections, as configured at the site, by a competent person, and was therefore unable to ascertain whether the configured platform met the 4:1 factor of safety requirement, as per OSHA standards. After the incident, Julian faxed a document to North Boston Area Office, showing permissible loads on platform with four sections on either side. The one page document was signed and sealed by David Herr, PE of DH Engineering Associates, Boston, MA on April 24, 1998. The document, however, does not indicate whether it pertained to 7000 or 6000 model. In any case, the professional engineer indicated a maximum load of 400 pounds on the fourth extension and 325 pounds each on three sections. The load chart was neither provided to DJ nor posted at the site. Instead, as stated above, the load chart with thee extensions were posted at the site that became irrelevant in view of the actual configuration of the platform. In fact, Julian left DJ in a state of sheer ambiguity as far as the safe loading of the platform is concerned.

The bricks and block were recovered from the incident scene and were weighed. The Area Office has determined that the total load placed on the four extensions that failed were 5340 pounds including the weight of three workers, which amounted to a load of 1335 pounds per section. Please note that even for three-section configuration, the maximum combined load to be placed on the three sections was 610 X 3 = 1830 pounds. It is a common knowledge that when more sections are added to the tower to lengthen the platform, carrying capacity is reduced, as was evident from the load chart posted at the site. In this event, DJ had a full knowledge of the amount of safe load to be placed on the three sections configuration where the total length of the platform was 50 feet, as shown on "Maximum Platform Loadings" at the site. DJ, instead of reducing the load, increased it by 275% and placed the heavier load over a longer platform. Incidentally, DJ was afforded a number of opportunities to rectify the situation before the incident when the workers complained that the platform was deflecting and could not be moved up due to excessive loading. These opportunities were missed.

Structural Analysis:

As stated earlier, four sections were added to the fixed platform on one side and three on the other. Failure occurred on one side only, the side with the four sections. The most stressed and damaged section, Section # 1, was the one closest to the fixed platform (figure 1, & 2). The other three remained intact after the failure (figure 5). The Section # 1 was therefore analyzed to determine the stresses in each member and to determine whether or not the members were overstressed to the point of failure. Reactions due to the actual loads placed over the other three sections were applied on Section # 1. A total load of 5340 pounds including the weight of three workers were assumed, based upon the weight of the bricks and blocks recovered after the incident. The load was assumed to be uniformly distributed over the length of the platform (20 feet) and a width equal to 16 inches. This was based upon eyewitness statements that the bricks and blocks were placed towards the side of the building. Yield strength of steel was assumed to be 46 ksi. Member sizes and dimensions were based upon field measurements. Commercially available, STAAD III program was used to perform the three dimensional structural analysis.

Our analysis indicated that a bottom horizontal member parallel to the longitudinal axis of the platform was stressed beyond its ultimate capacity to the point of failure, due to the loads placed over four sections. The horizontal member was a compression member whose failure load was computed to be 16,700 pounds, based upon the full thickness of 1/8". However, if the thickness is assumed to be 0.1", as measured in the field, the failure load is reduced to 13,900 pounds. The member was subjected to a compressive load of 19,300 pounds due to the actual loads placed over four sections, and hence the failure. Compressive members are highly sensitive to loads beyond their buckling strength and often fail in a catastrophic way without giving any visible warnings. As one of the bottom horizontal member failed, the loads were redistributed to other members, and overall failure occurred resulting in the collapse of the scaffold. Other members were subjected to stresses within their capacity, though many of which did not meet the 4:1 requirements. The ultimate capacities of the members were computed in accordance with the LRFD provisions of the American Institute of Steel Construction, the nationally recognized and accepted standard of the industry.

Findings:

Based upon the above, the following findings are presented:

1. The cause of the collapse of the scaffold was overloading of the platform. The four sections were loaded well in excess of their safe capacities.
2. Overloading of the platform resulted in buckling of one of the bottom compression members of the section closest to the fixed platform.
3. Corrosion reduced the wall thickness of the tubular framing members of the sections, as observed during the investigation, thus compromising the load carrying capacity of the sections.
4. The OSHA requirements of 4:1 factor of safety were not met. Hence 1926.451 (a)(l) was violated.
5. The masonry contractor did not follow the manufacturer's instructions for safe loading of the platform. The masonry contractor placed loads on four sections that far exceeded the recommended load for three sections. The greater the number of sections, the lesser load must be placed.
6. The scaffold erector erected the platform with four sections without providing a load chart to the masonry contractor for that configuration. The erector provided a load chart for a platform comprising of three sections only.
7. The masonry contractor failed to pay due attention when informed of platform deflections and inability of the platform to ascend, clearly indicative of heavier loads.
8. The erector placed dissimilar sections on the platform, thus jeopardizing the structural integrity of the scaffold and the validity of the load chart. The erector did not obtain the services of a professional engineer to compute the load carrying capacity of the erected platform with dissimilar sections. 1926.451 (a)(6) was violated.

Figure 1:

Damaged First Section of Mast Platform

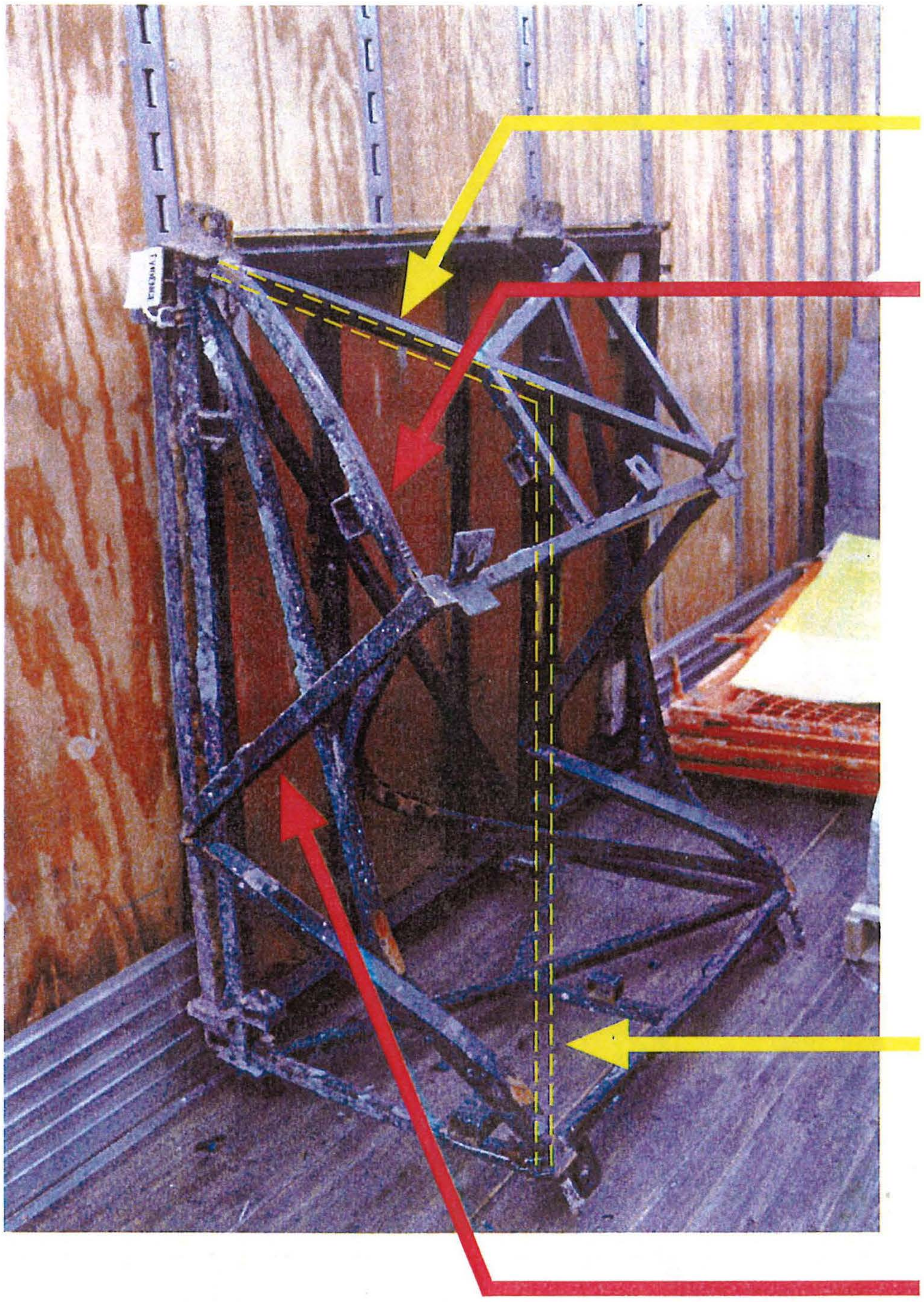
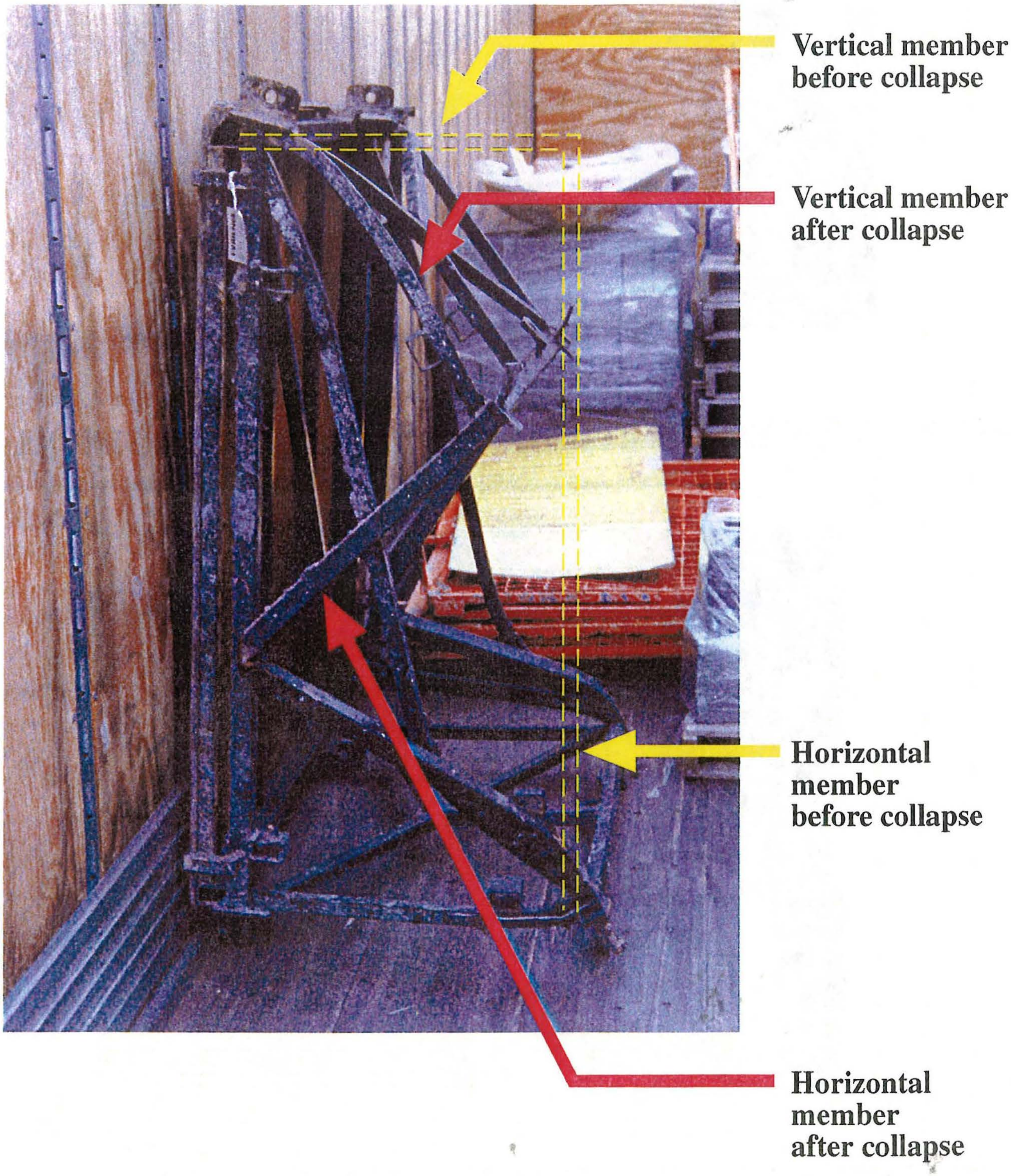


Figure 2:

Damaged First Section of Mast Platform



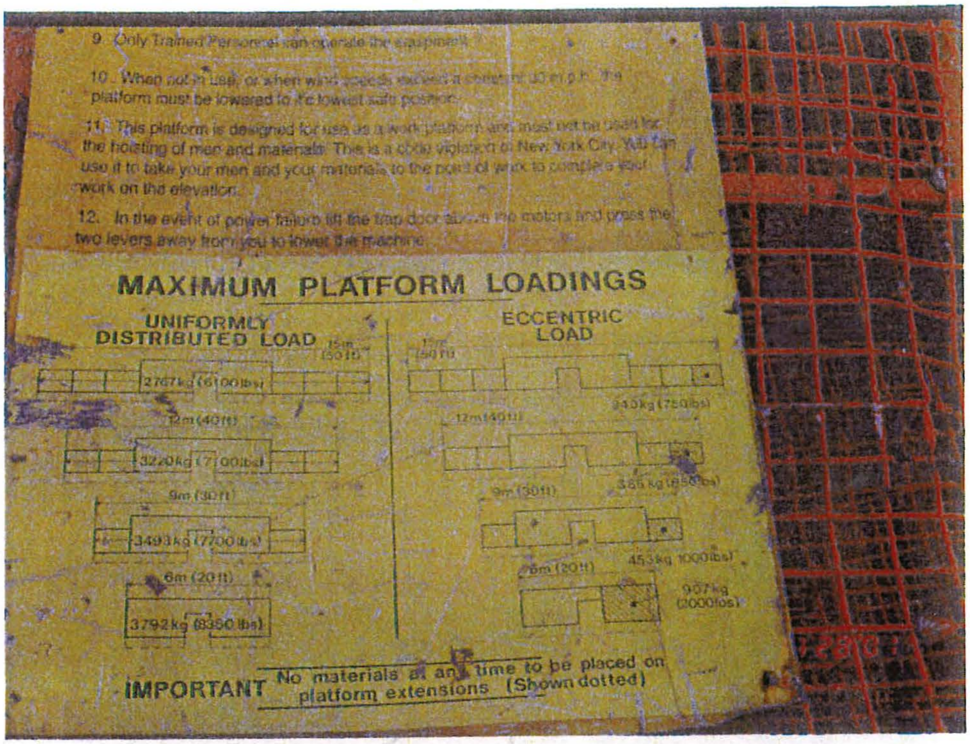


FIGURE 3 (LOAD CHART FOR SAFE LOAD CAPACITY OF THE SCAFFOLD)

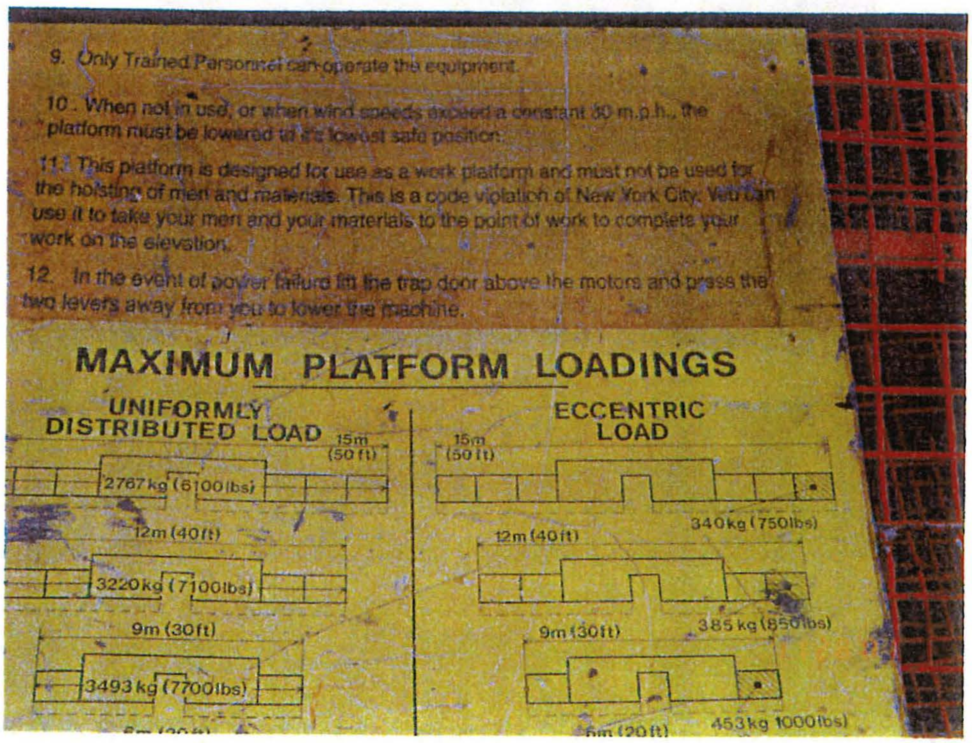


FIGURE 4 (LOAD CHART FOR SAFE LOAD CAPACITY OF THE SCAFFOLD)

FIGURE 5 (UNDAMAGED LAST THREE SECTIONS OF MAST PLATFORM)

FIGURE 6 (PARTIALLY COLLAPSED PLATFORM AT ONE END OF TOWER)

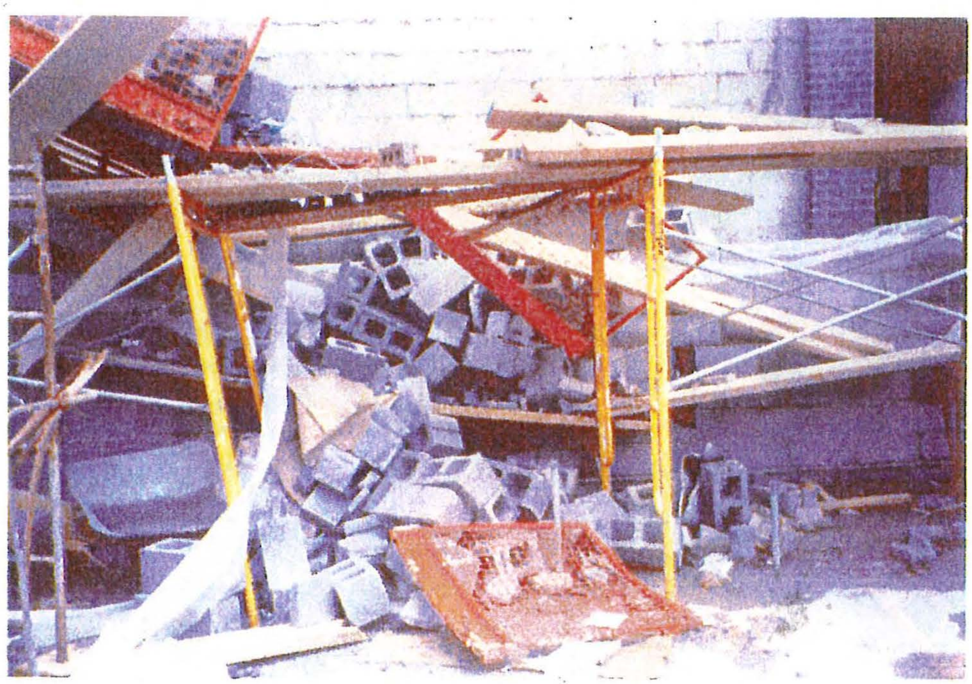


FIGURE 7 (PARTIALLY COLLAPSED PLATFORM ALONG WITH CINDERBLOCKS ON GROUND)

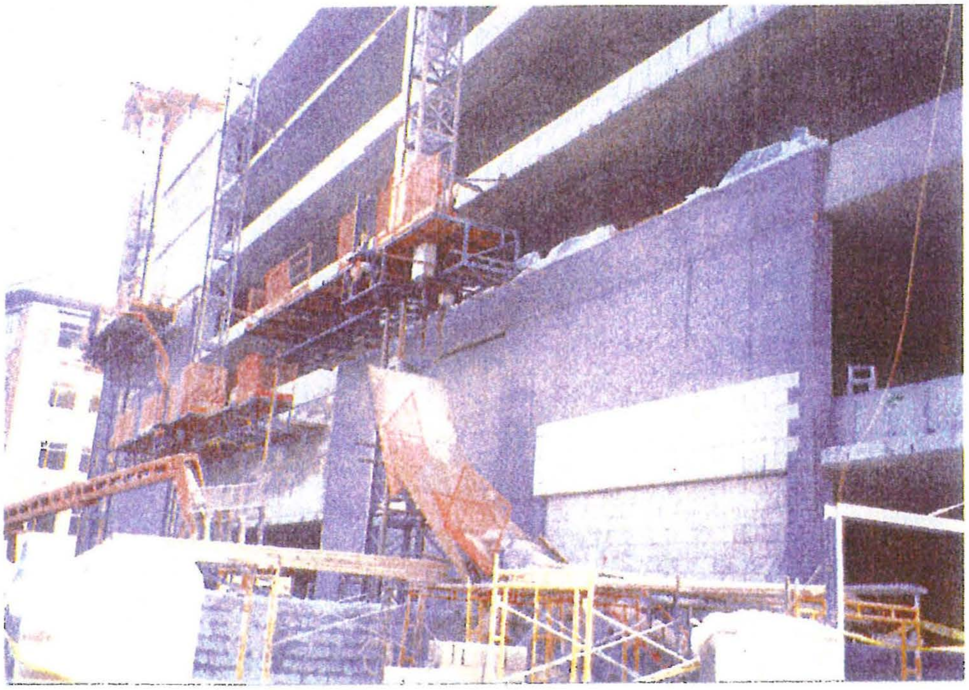


FIGURE 8 (UNCOLLAPSED PLATFORM AT OTHER END OF TOWER)

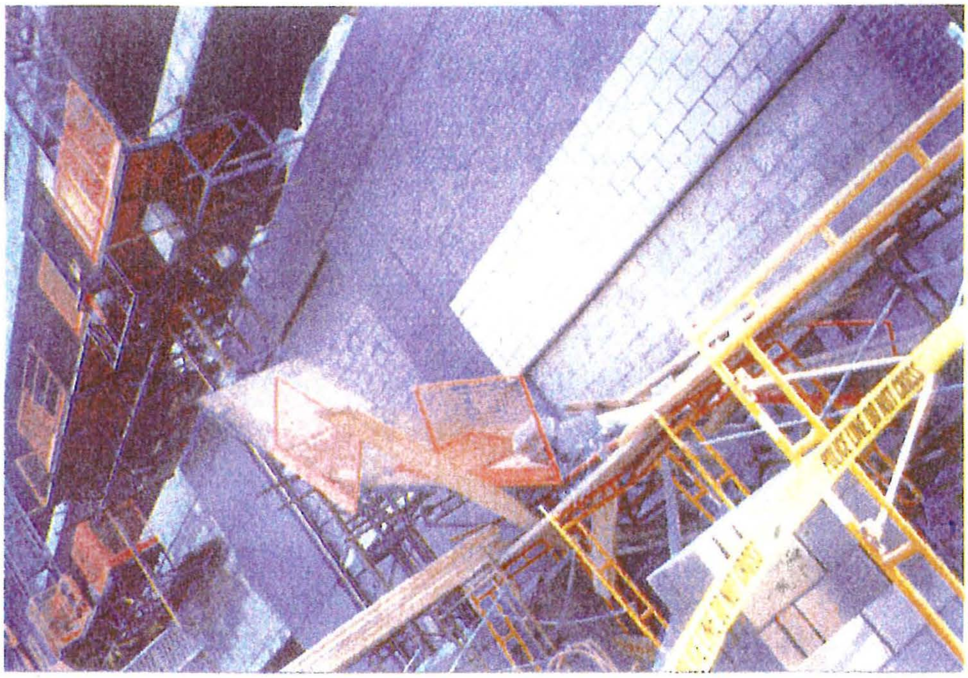


FIGURE 9 (UNCOLLAPSED PLATFORM AT OTHER END OF TOWER

APPENDIX A

ANALYSIS

INDEX

Attachments: 1, 2, 3, & 4

PURPOSE: To evaluate the partially collapsed mast climbing platform framing for the imposed loading.

SOURCES OF DATA AND REFERENCES:

1. Field information collected during inspection of the failed platform
2. AISC's Manual of Steel Construction, LRFD, First Edition
3. Structural Engineering Handbook by Gaylord & Gaylord, © 1968 by McGraw Hill
4. Design of Welded Steel Structures by Blodgett, June 1966 by James F. Lincoln Arc Welding Foundation

ATTACHMENT:

1.	STAAD III Model	(1 sheet)
2.	STAAD III Analysis	(12 sheets)
3.	FAX from CSHO to M. Ayub	(2 sheets)
4.	Loading distribution chart	(1 sheet)

ASSUMPTIONS:

1. Member sizes are obtained from field inspection of the failed platform.
2. All top and bottom chord members are considered to be TS 1 ½" x 1 ½" x ⅛".
3. All vertical and diagonal members are considered to be TS 1 ⅛" x 1 ⅛ "x ⅛".
4. The yield strength of steel material is considered to be 46 ksi.
5. In calculating slenderness ratio of the top chord and diagonal member, an effective length factor K = 1.0 is considered.
6. Actual resulting loads are compared against failure loads. Using failure loads, interaction value is calculated for combined axial and flexure loads.
7. American Institute of Steel Construction's (AISC) Load and Resistance Factor design (LRFD) method is applied for design. Load and resistance factors of 1.0 are considered
8. The platform is comprised of 20 foot long fixed platform supported in the center by tower. At one end of the fixed platform, 3-5 feet long platform sections are added. At other end of the platform, 4-5foot long platform sections are added. The total length of the platform is 55-foot. The combined weight of 5340 pounds consisting of cinderblocks, bricks and 3-men are considered on the last four sections of the platform. The 5340-pound load is considered uniformly distributed on the 20-foot long section. Only first section closest to the fixed platform is considered for evaluation. Reactions due to the loads placed over the other three sections are applied on the first section.

MEMBER PROPERTIES								
MEMBER	WIDTH	DEPTH	Tw	AREA	IZ, Iy	J(Ix)	Rx, Ry	Z
TS	b	d	t					
	(IN)	(IN)	(IN)	(IN^2)	(IN^4)	(IN^4)	(IN)	(IN^3)
1 1/2X 1 1/2 X 0.112	1.5	1.5	0.112	0.621824	0.200962	0.378	0.56849	0.324362
1 1/2X 1 1/2 X 0.1	1.5	1.5	0.1	0.56	0.183867	0.3375	0.573004	0.2945
1 1/8X 1 1/8 X 0.110	1.125	1.125	0.11	0.4466	0.077584	0.156621	0.416798	0.170653
1 1/8X 1 1/8 X 0.1	1.125	1.125	0.1	0.41	0.072476	0.142383	0.420441	0.158094
1 1/2X 1 1/2 X 0.125	1.5	1.5	0.125	0.6875	0.218424	0.421875	0.563656	0.355469
1 1/8X 1 1/8 X 0.125	1.125	1.125	0.125	0.5	0.084635	0.177979	0.411425	0.188477
1 1/4X 1 1/4 X 1/8	1.25	1.25	0.125	0.5625	0.120117	0.244141	0.462106	0.238281
1 5/8X 1 5/8 X 1/8	1.625	1.625	0.125	0.75	0.283203	0.536377	0.614495	0.422852

SUMMARY OF STRESSES ON CRITICAL HORIZONTAL BOTTOM MEMBER OF THE PLATFORM					
Member Size	Loads			Allowable	Remarks
TS 1 ½ X 1 ½ X 1/8" (Nominal wall thickness)	P_{axial} = 19.34 kips (Compression Load)	$P_{ultimate}$ = 16.74 kips	P_{axial} / $P_{ultimate}$ = 1.1553	P_{axial} / $P_{ultimate}$ = 1.000	Beam is overloaded by more than 15% in axial failure load.
	M_X = 1.54 In-Kips; M_y = 0.32 In-Kips	Mpx = Mpy 16.33"k	P_{axial} / $P_{ultimate}$ + (M_X + M_y) / Mp X (8/9) = 1.2565	P_{axial} / $P_{ultimate}$ + (M_X + M_y) / Mp X (8/9) = 1.00	Beam is overloaded by more than 25% in failure load of interaction of axial load and flexural moments.
TS 1 ½ X 1 ½ X 1/8" (actuall wall thickness)	P_{axial} = 19.34 kips (Compression Load)	$P_{ultimate}$ = 13.94 kips	P_{axial} / $P_{ultimate}$ = 1.383	P_{axial} / $P_{ultimate}$ = 1.000	Beam is overloaded by more than 38% in axial failure load.
	M_X = 1.54 In-Kips; M_y = 0.32 In-Kips	Mpx = Mpy 16.33"k	P_{axial} / $P_{ultimate}$ + (M_X + M_y) / Mp X (8/9) = 1.4842	P_{axial} / $P_{ultimate}$ + (M_X + M_y) / Mp X (8/9) = 1.00	Beam is overloaded by more than 48% in failure load of interaction of axial load and flexural moments.

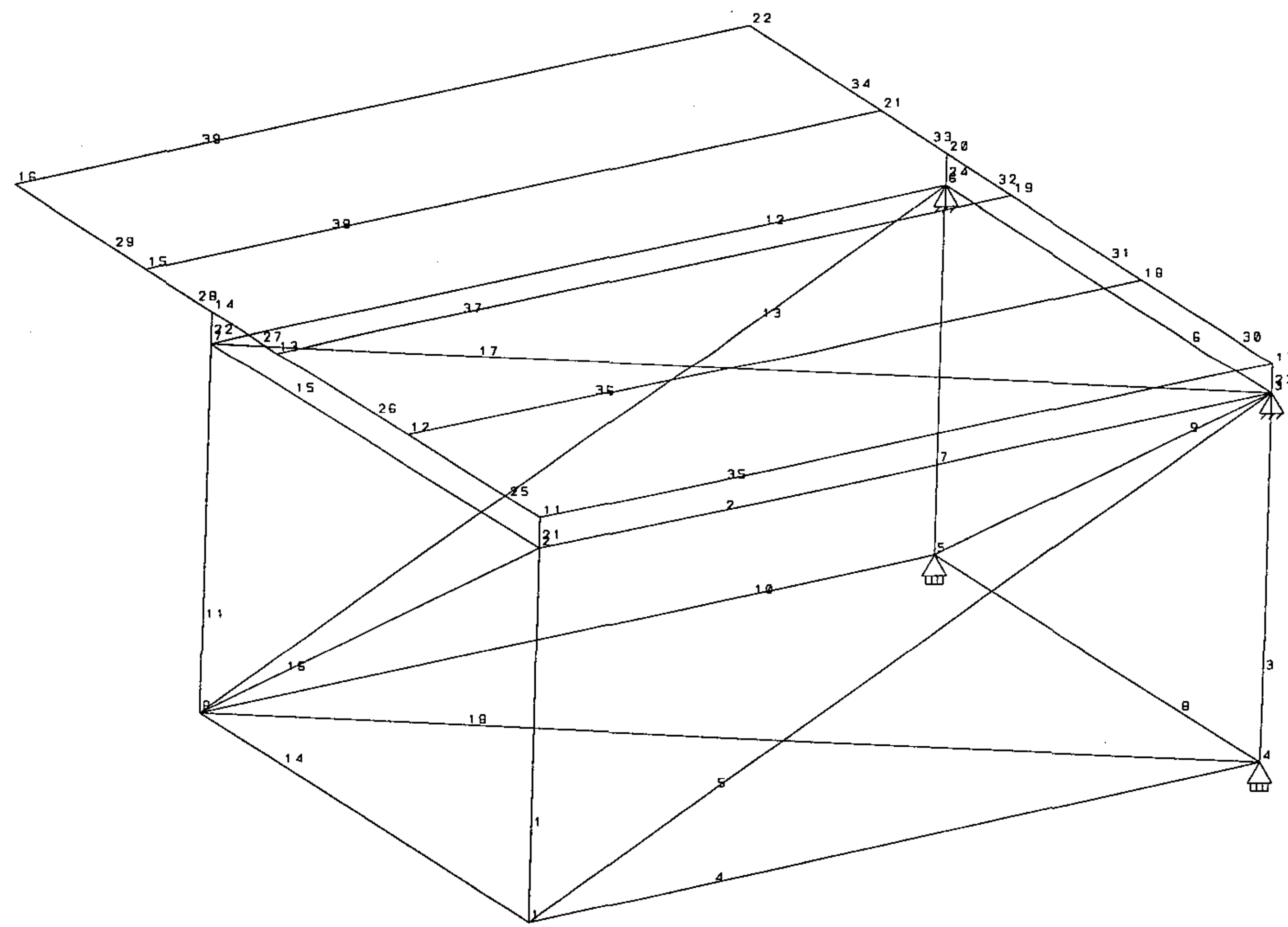
User ID: Research Engineers, Inc.

MN/ELEM

STRUCTURE DATA

TYPE = SPACE

NJ =	20
NM =	37
NE =	0
NS =	0
NRJ=	4
NL =	4
XMAX=	54.5
YMAX=	27.5
ZMAX=	63.8



J=20,M=37

UNIT INC KIP

S T A A D P O S T - P L O T (REV: 22.3a)   DATE: NOV 30, 2001

TITLE: "UDL IN MID PLATFORM; TW=0.125"; REV.R2T

ATTACHMENT 1, PAGE 1 OF 1