New York City
Department of Buildings

51st Street Crane Investigation

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Contents

Tables

Figures

Photographs

a close-up of the failed end of Beam B4 after removal to the NYCOEM warehouse. The top photo is by the crane engineer Peter Stroh IMG-0096.jpg. The bottom photo is provided by Arup.

Photograph 1.13 - Failed pin end of Beam B5 at the 9th floor level as seen immediately following the collapse. Photo provided by the NYCDOB.

Photograph 1.14 - 3rd floor collar assembly following the collapse. The 9th floor collar and 18th floor collar rest atop the 3rd floor collar. Source: New York County District Attorney IMG_0355_085.jpg.

Photograph 1.15 - Fracture of a tie-beam which did not fully fail on the 3rd floor collar. (The fracture of interest is to the right of the tape measure. The damage to the left of the tape measure is a flame-cut (post failure) to assist in the removal of the debris. Photo by Arup.

Photograph 1.16 - View of street side (south) of tower base after failure. The dunnage beam is no longer on the supporting concrete foundation wall. Photo by New York County District Attorney IMG_0327_057.jpg.

Photograph 1.17 - View of building side (north) of tower base after collapse. The dunnage beam has been displaced towards the building and the plywood bearing pads are off of the foundation wall. Lifting of the back tower legs and slippage of the front tower legs is also visible. Photo by New York County District Attorney IMG_379_109.jpg.

Photograph 1.18 - Damage to 9th floor balcony. The imprint in the slab is from Beam 9B. Photo by Arup.

Photograph 1.19 - Damage to 3rd floor balcony. Photo by Arup.

Photograph 1.20 - Minor spalling of concrete in the vicinity of the bolted connections to the 9th floor slab. Photo by Arup.

Photograph 1. 21 – View of the intact tower crane after collapse. Photograph by the crane engineer Peter Stroh, “After” photographs, IMG_066.jpg.

Photograph 1.22 – View of the intact tower crane after collapse. Photograph, from the crane engineer Peter Stroh, “After” photographs IMG_077.jpg.

Photograph 3.1 - Close-up of crane tower shortly before collapse. Two polyester web slings are seen in a choker hitch around the legs of the tower. (Source: photograph by Gary Halby.

Photograph 3.2 - OSHA retained evidence from NYPD GB9. Note the two clips only, with the saddle on the dead end of the wire rope. Photo by Arup.

Photograph 3.3 - OSHA evidence GB14 from NYPD. Note spacing of the U-bolt clips in the lower right of the photograph. Photo by Arup.

Photograph 3.4 - Base of tower crane after collapse. Note the lack of knee bracing at the base, the dunnage beam on which the tower stands and the slippage of the dunnage beam off of the submerged concrete wall foundation. Photo from New York County District Attorney IMG_332_062.jpg.

Photograph 3.5 - View of the west elevation of the collapsed tower collars. The location of visible wire rope attachments to the collars are circled. Note the lack of thimbles in the formed sling. Photo from New York County District Attorney 328_58.jpg.

Photograph 3.6 - View the south and east elevations of the collapsed tower collars. The location of visible wire rope attachments to the collars are circled. Note the lack of thimbles in the formed sling. Photo from the crane engineer, Peter Stroh, IMG_0081.jpg.

Photograph 3.7 - View of tower crane with collar half being lifted into place via polyester web slings (right of photo) and chain falls incorporating polyester web slings choke hitched around the tower legs (lower portion of tower). (Source: photograph by Gary Halby).

Photograph 3.8 - OSHA evidence item 1. Polyester web sling removed from the collapse site. Note the faded label. This was taken from the southwest corner of the 9th floor collar. It was attached to the lifting point via a chain fall. Photo by Arup.

Appendices

NYCDOB List of Standard Evidence

Appendix J

Macroscopic Examination of Crane components

Appendix K

Dr. Tushar Ghosh Report

Executive Summary

Following the collapse of an external self-climbing tower crane at 303 East 51st Street, New York, New York, on March 15, 2008, Ove Arup & Partners, PC (Arup) was hired by the New York City Department of Buildings (NYCDOB) to provide engineering and investigative services.

Immediately prior to the collapse, the crane tower was connected to the building under construction at both the 3rd and 9th floor levels. Each floor-to-tower connection consisted of a structural steel collar assembly surrounding the tower. Three tie-beams were connected via pins to each completed collar. The far end of each tie-beam was welded to a steel base plate, which was in turn anchored by bolts to the reinforced concrete slab of the building, thus fixing the tie-beam to the building. The following Figure 0.1 shows a plan view of this arrangement.

At the time of their installation the tower crane was not yet in service and a mobile crane was used to assist in their installation. The final configuration for the 3rd and 9th floor collars included the use of wire rope slings for vertical support at varying attachment points on the two collars.

At the time of the collapse the team had just “jumped” or extended the height of the tower and was proceeding to install a new collar connection to the 18th floor. The collar had to be erected in two halves and connected together so as to surround the tower. One-half of the collar was lifted into place where it was temporarily suspended from the tower by polyester web slings (each extended with a chain fall) using two attachment points not in conformance with any of the tower crane manufacturer specified four attachment points. Following this, the other half of the collar was similarly lifted into place, suspended by polyester web slings and the two halves were bolted together. Thus four attachment points, which were not in conformance with the eight tower crane manufacturer specified attachment points, were being used to temporarily suspend the completed collar at the time of the tower crane collapse.

As the crane crew began to place the first of its three tie-beams, the collar was suspended from the tower by only four polyester web slings (extended with chain falls). The Manufacturer has specified eight chain blocks to be used for this operation at locations other than those actually used for the polyester web slings. All aforementioned tower crane manufacturer specified locations and numbers of attachment points are per the Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 (Figure 1.13).

At that point, the four polyester web slings failed allowing the collar to fall downwards along the tower. The 18th floor collar struck the 9th floor collar below destroying its connection to the building and allowing both collars to fall. After the 9th floor connection failed, only the level 3 connection and the base friction remained to resist overturning of the tower. This system was further compromised by the deflection of the level 3 collar and significant bending of the connected tie-beams from the impact of the collars above. With lateral restraint remaining at Level 3 and the base only, the tower overturned as its base slid towards the building structure to the north and the crane cab at the top fell to the south causing damage to the adjacent buildings and generating seven fatalities and multiple injuries.

The base friction proved inadequate and the tower overturned as its base slid towards the building structure to the north and the crane cab at the top fell to the south. This is consistent with the design criteria used by the tower crane engineer (Stroh Engineering) since the crane base was not designed to make use of the friction that might have developed between the tower and the foundation as per Figure 1.7 in Chapter 1. For the crane configuration that existed at the time of the accident, the crane engineer’s (Stroh Engineering) calculations and design relied on the capacity of the ties at the 3rd and 9th floor levels to provide the lateral resistance of the crane tower and prevent overturning.

The work undertaken for our investigation included the following:

1. Document review, including documents supplied by NYCDOB and the New York County District Attorney (DA) and the New York City Department of Investigation (DOI).
2. Site visit to the collapse site.
3. Site visits to view components of the tower crane and collar assemblies which had been removed from the site prior to our engagement.
4. Structural analysis of the tower crane, including both static analysis and non-linear dynamic analysis.
5. Structural analysis of the sling system including non-linear dynamic behaviour.
6. Materials and metallurgical testing.
7. Witnessing of sling tests specified by OSHA, review of raw data from those tests and review of OSHA-related correspondences.
8. A review of codes, standards and regulations of relevance to the collapse
9. A review of the permitting process associated with this crane
10. An independent peer review of the crane's support designs connecting the crane tower to the building and to the ground.
11. An independent visual assessment, including microscopic examination, of the polyester slings used for supporting the collar assemblies recovered from the collapse site.
12. Review of the report prepared by the U.S. Department of Labor, Occupational Safety and Health Administration (OSHA) dated September 2008. The report is titled "Investigation of the March 15, 2008 Fatal Tower Crane Collapse at 303 East 51st Street, New York, New York". The report was prepared by Mohammad Ayub, PE, Directorate of Construction at OSHA, Washington, DC.

The study by OSHA, which focused primarily on the slings, was a particularly relevant complement to this investigation. OSHA retained a sling expert to examine the sling remnants and address the failure characteristics of the slings. The sling expert conducted a visual examination of the fractured surfaces of the slings recovered from the accident site. Additionally, the Center for Advanced Technology for Large Structural Systems Research Center at Lehigh University, under the direction of OSHA, conducted actual tests of equivalent new slings to determine failure loads of the slings subject to similar field conditions at the time of the accident. Among other results, the tests indicated that the slings failed at loads significantly lower than their ultimate capacities.

In parallel, Arup retained an expert in polyester materials for industrial applications. The materials expert visually examined the failed surfaces of the slings and the characteristics of the failed polyester fibers using microphotographs taken by Arup. Additionally, Arup reviewed codes and standards related to accepted industry practice in the use of polyester slings as well as the actual usage of the slings at the time of the collapse.

As a result of these studies, it has been concluded that, as stated in the OSHA report, at least one of the slings was "frayed and deteriorated" before its use here and should have been "discarded and not used." In addition, there were other factors associated with the slings that related to the crane failure. These criteria, as determined from the OSHA report, are summarized herein:

- The choice of using polyester slings to suspend the collar was questionable even if the recommended number of slings was used.
- The collar was rigged improperly causing the slings to be choked around the vertical legs of the crane and seated in V-shaped grooves with unprotected/unpadded edges.

- There were no provisions for proper protection of the slings from unpadded edges.
- The number of slings used to support the collar did not meet the crane manufacturer recommendations. Only half of the specified number of slings recommended by the manufacturer was used to support the collar at the 18th floor that initiated the collapse.
- A deteriorated sling was used to suspend the collar. Due to its deteriorated condition there would have been a reduction in the overall capacity of the sling. According to industry standards and OSHA's expert, this sling should have been discarded if proper inspection of the sling was done prior to its use.

Based on our investigation as summarized above, we have therefore identified the following relevant factors in the collapse of the tower crane:

1. The collapse of the tower was initiated when the polyester web slings supporting a steel collar at the 18th floor level failed, allowing the collar to fall.
2. Improper usage of the polyester web slings resulted in the failure of the slings. Specifically:
    - the number of supports provided by the slings did not meet the crane manufacturer's requirements – only half of the required eight supports were used;
    - the positioning of the slings on the collar as installed was inconsistent with the crane manufacturer's instructions;
    - the method of attaching the slings to the tower was not in accordance with accepted industry practice and standards; and,
    - one of the slings had been previously used rendering its condition such that its load capacity would have been reduced.
3. As demonstrated by structural analysis, the failure of one sling in the arrangement as installed used could lead to a sudden and uneven redistribution of load to the remaining three slings. This effect could be sufficient to lead to a sequential failure of the remaining slings, thus allowing the collar to fall from Level 18.
4. As demonstrated by structural analysis, the unanticipated loads arising from the dynamics of the collar falling from Level 18 caused failure of the collar connection at Level 9 which in turn allowed slippage of the dunnage steel at the base of the crane resulting in the overturning of the tower.

Additional observations include:

5. The tie-beam assembly at the 9th floor level was not welded as specified. However, this is not considered to have contributed to the tower collapse. Computer analysis indicates that the welds that connect the tie-in beams to the steel base plates, which in turn are anchored to the building slab, would still have failed if the welds had been properly fabricated per the design. Further, site observations indicated that all three connections of the tie-beams to the collar also failed.
6. Wire ropes, providing vertical support to the 9th and 3rd floor level collars, were improperly rigged from the collars to the crane tower. However, this is not considered to have contributed to the tower collapse. Computer analysis indicates that the wire ropes would still have failed if the connections had been properly executed.
7. A review of the crane engineer's submissions (by Stroh Engineering) was performed. The tower and tower supports were found to be generally well-engineered and designed to industry standards.

Based upon the information supplied and work completed, it is our professional opinion to a reasonable degree of engineering certainty that the cause of collapse of the tower crane on March 15, 2008 was the failure of the polyester web slings due to improper usage.

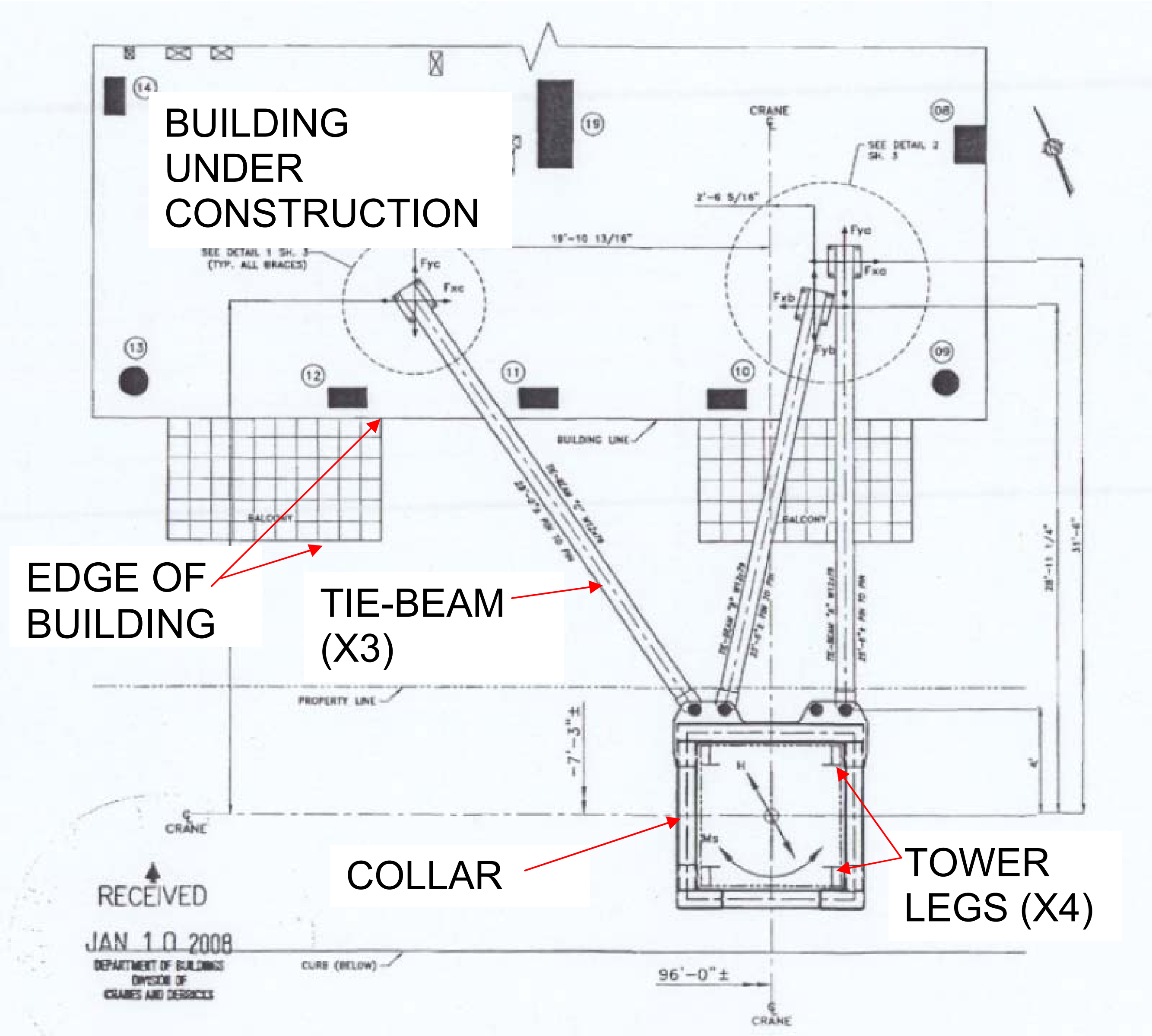


Figure 0.1 - Plan view of typical collar assembly as-designed (Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 Drawing 3 of 4 dated 1/2/08, with clarifying labels by Arup).

1 Introduction

1.1 Background and Description of the Tower Crane

1.1.1 General

303 East 51st Street is a proposed 43-story concrete frame building designed by Garrett Gourlay Architect PLLC on behalf of Kennelly Development Company, L.C. An external self-climbing luffing tower crane, Model M440E, manufactured by Favelle Favco Cranes Pty. Ltd. and supplied by New York Crane, was being used in the construction of the upper levels. The tower crane collapsed on March 15, 2008, resulting in fatalities and damage to the crane, its surroundings and nearby buildings.

From site records reviewed (JCI/Joy Contractors journal), it appears that the tower crane was first passed by inspection for use at 303 East 51st Street on January 29, 2008 by the New York City Department of Buildings (NYCDOB). On February 14, 2008, welding repair of the tie-beam base plates for the 9th floor level tie-beam was performed. By March 11, 2008 the 18th level floor slab was being poured, on which day "steel to tie crane" was also delivered. On March 12, the 18th floor level walls were stripped of their formwork and the 19th floor level deck was framed. Preparations for the tower crane jump, scheduled for Saturday March 15, were also commenced on this day. The 19th floor deck was poured on March 13 and 14.

The crane was not used until the ties at the 3rd and 9th floor were in place.

On March 15, the crane jump commenced at 7:00 am, involving the addition of three sections of the tower. At 12:30 pm, after "all 3 section of tower in place," the collar halves had been lifted, bolted together and suspended from the tower and while the first tie-in beam was being installed, the polyester slings being used to suspend the 18th floor level collar from the tower broke, allowing the collar to fall striking the 9th floor level collar below and severing its connection to the building; the two collars continued to fall impacting the 3rd floor level collar causing significant displacement of the collar and the tower crane collapsed.

It is to be noted that, while the Model M440E tower crane was the crane used at the site, much of the documentation and details reviewed as part of this failure investigation relate to the Model M440D. Information respecting the differences between these two models revealed no substantive difference as it may relate to the tower collapse; see Section 1.1.5 in this Chapter. Hence, both the M440D and M440E tower crane models are referred to herein as the information provided may dictate.

The luffing crane, counterweight assembly, operator's cabin and other components supported by the tower at and above the slewing ring turntable assembly are opined to not have contributed to the tower crane collapse. The following description is therefore limited to the supporting tower and ancillary components. Additional details are presented where needed in the body of the report.

1.1.2 Overall Layout of the Tower

An elevation view of the tower crane the day before the collapse is given in Figure 1.1. The reference datum (i.e., El. 0 ft) is given as the 1st floor. The base of the tower crane was located at an elevation of El. 2'-7" which corresponds to the top of the dunnage steel. The tower consisted of fifteen (15) tower sections the day before the collapse, each section approximately 13'-1" in height. The top of the tower stood at El. 199'-4". The tower was supported laterally by tie-ins to the building at the 3rd and 9th floors at elevations of El. 22'-

11½" and El. 88'-5½", respectively. The total unbraced length between Tie #2 at the 9th floor and the underside of crane platform above the 9th floor is approximately 117 ft.

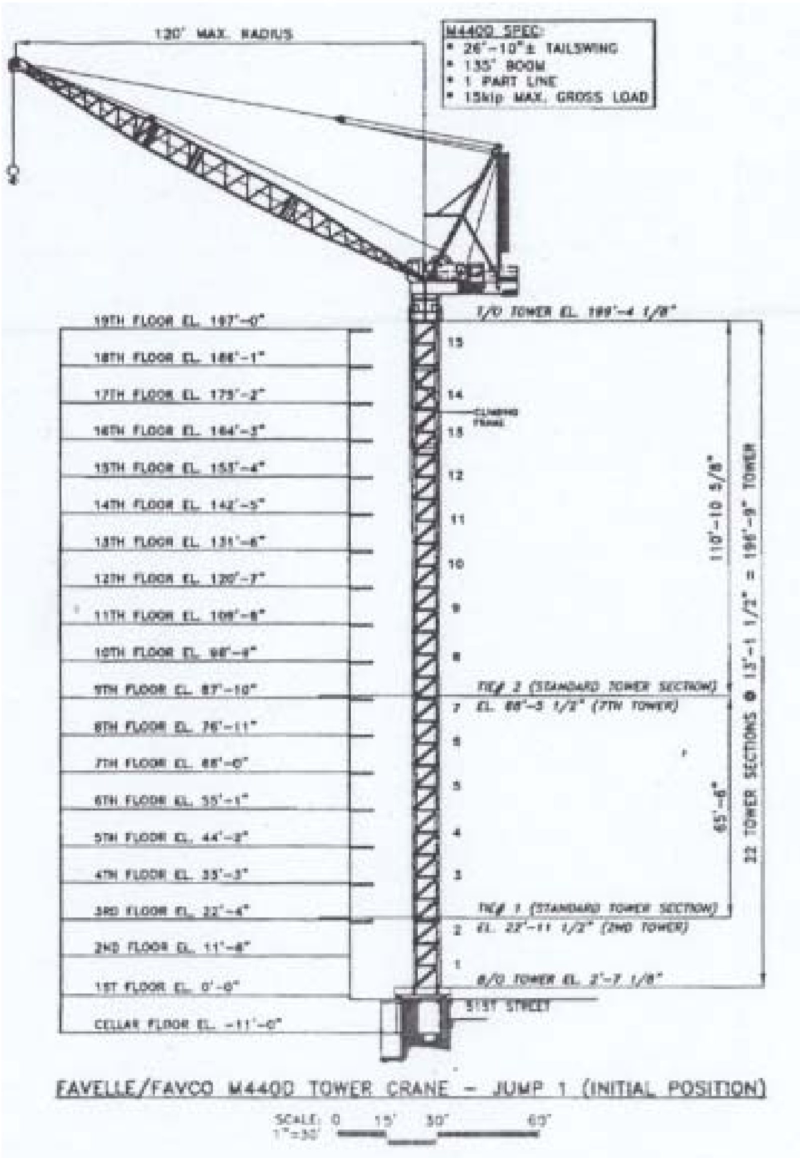


Figure 1.1 - Elevation view of tower crane the day before the collapse (source: Drawing 07-046C-1, drawing 4 of 4 dated 1/2/08, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

Three additional tower sections had been added the day of the collapse, as noted in the Contractor's Daily Report (i.e., JCI/JOY Contractors, Inc., "Project Description of Day's Activities" by Matt Katkocin dated March 15, 2008), to bring the top elevation of the tower to El. 238' 1". The rigging crew had yet to install the lateral support assembly at 18th floor.

1.1.3 Tower Sections

The tower sections of the collapsed crane were 393 Tower Sections as per the approved prototype application. This was verified by:

- Field measurements of the dimensions of the tower sections;
- Field observations of the connectivity of the tower section members; and
- Field measurements of the member sizes and thicknesses.

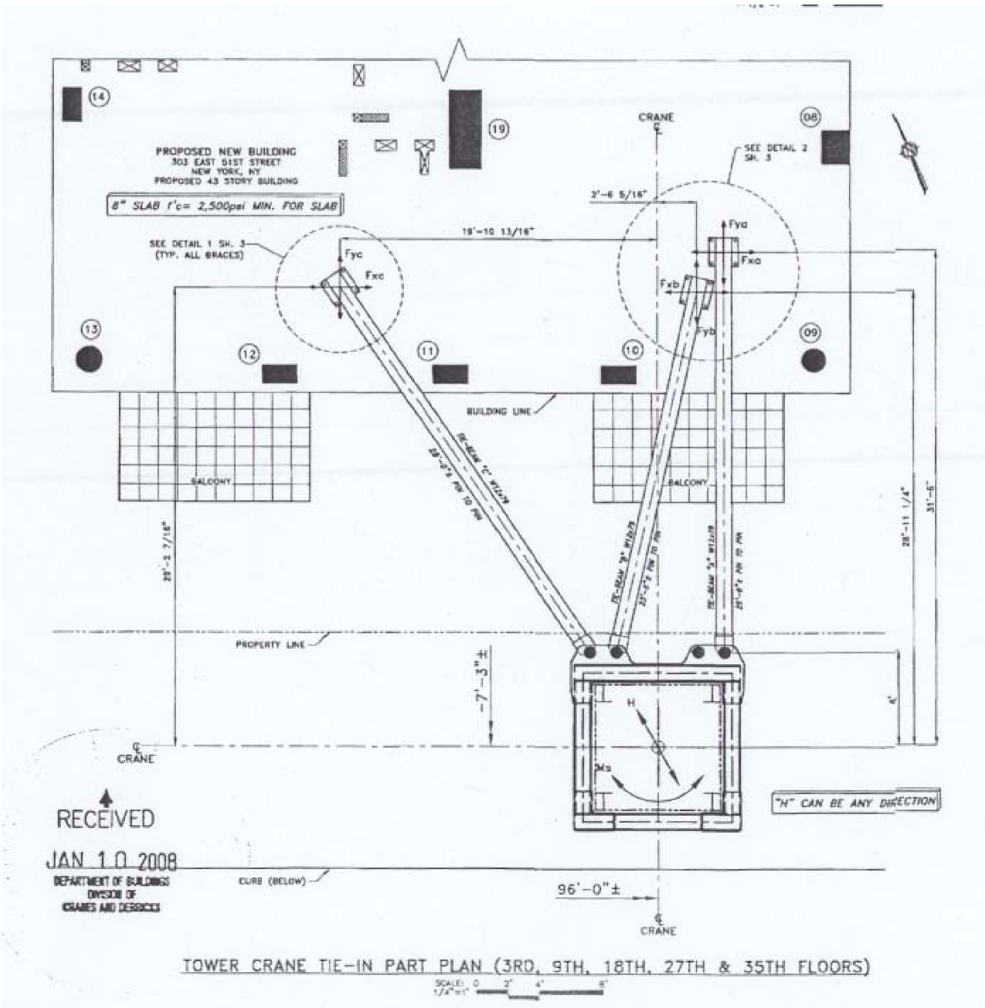


Figure 1.2 - Lateral connection of tower crane to building (source: Drawing 07-046C-1, drawing 3 of 4, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

1.1.4 Tower Support Conditions

1.1.4.1 Tower Building Connections

The tower was supported laterally by collar and tie-in beam assemblies at the 3rd and 9th floors of the building prior to the collapse. A drawing of these tie-in assemblies is given in Figure 1.2. Each tie-in assembly consisted of a steel framed collar (chocked to the tower frame) and three (3) W12x79 tie-beams. The tie-beams were connected to the collar with 3in (0.076m) diameter pins (Note: the pins were not yet connected at the 18th floor collar). At the building end, the tie-beams were connected to the building via steel base plates anchored to the floor slabs of the building, themselves welded to the tie-beams.

Each collar consisted of steel plated I-sections with the flanges aligned vertically with the 2 in (0.050m) thick top and bottom plates and chock housings at each corner. The collars were made in two halves that were bolted together around the tower. The weight of the two collar halves was 11,279lb.

The tower height above the second tie-in assembly was approximately 143 ft when the collar at the third tie-in assembly was being installed.

1.1.4.2 Tower Base (Dunnage Steel)

The base of the tower rested on steel dunnage beams, which were designed to carry the tower loads over an existing Con-Edison vault to the tower foundations. Drawings for the dunnage steel (plan, elevation, sections and details) are given in Figures 1.3 through 1.7. A photograph of the dunnage steel as constructed appears in Photograph 1.1.

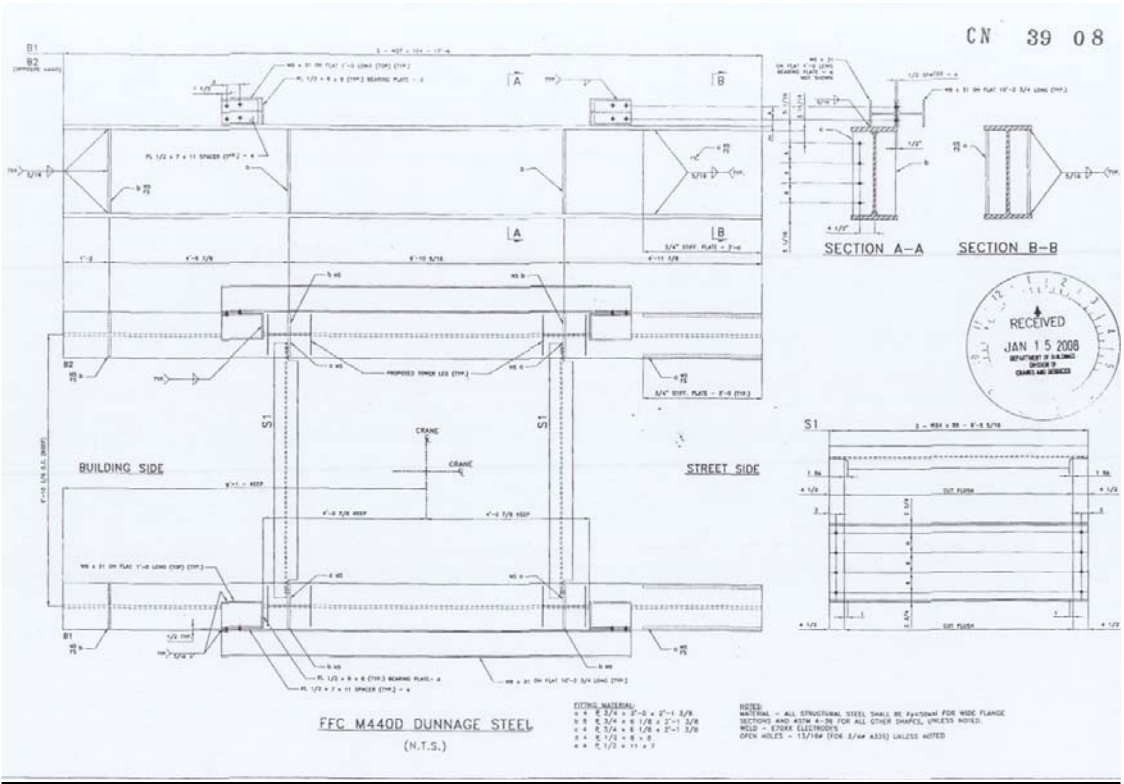


Figure 1.3 - Detailed plan, elevation and sections of dunnage steel (source: Drawing 07-0465-2 dated 12/18/07, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

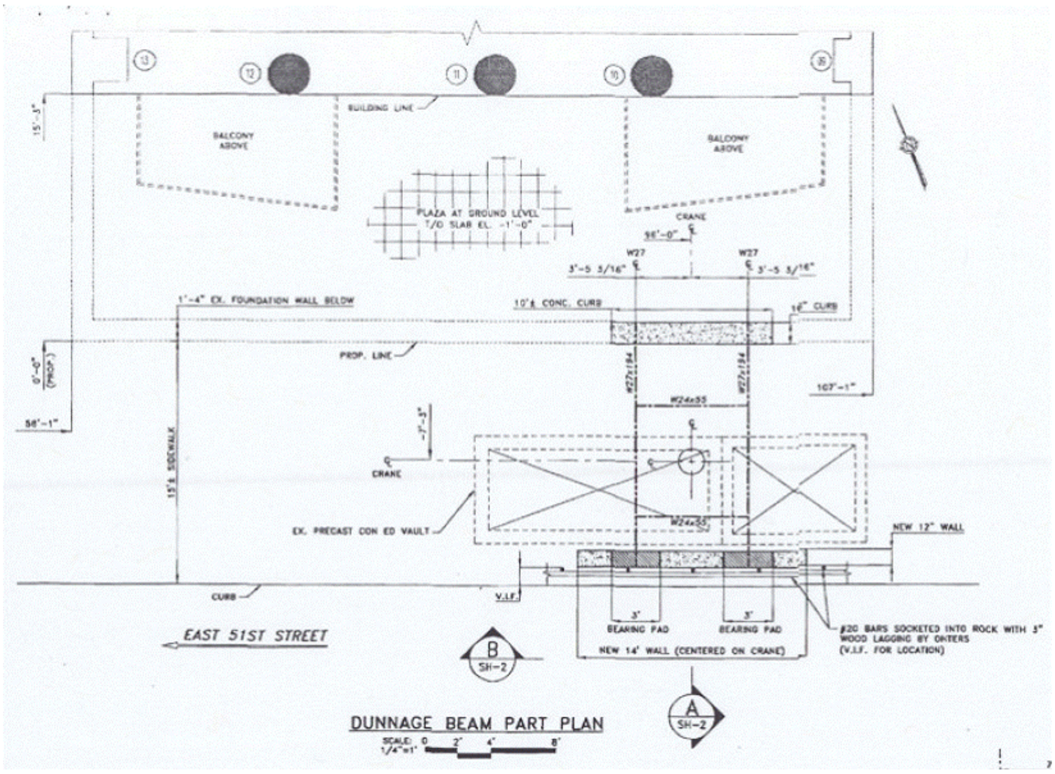


Figure 1.4 - Foundation of tower crane - plan (source: Drawing 07-046C-1, drawing 2 of 4 dated 1/2/08, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

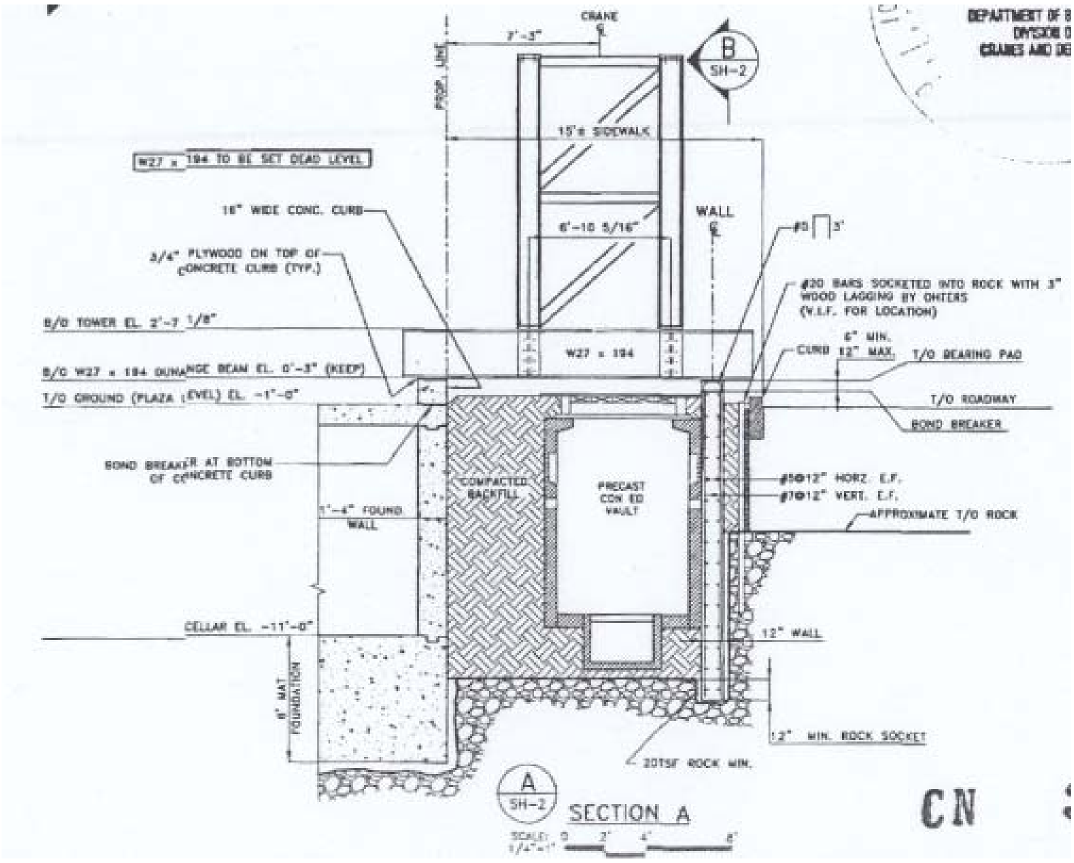


Figure 1.5 - Foundation of tower crane – Section A (source: Drawing 07-0466-1, drawing 2 of 4 dated 1/2/08, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

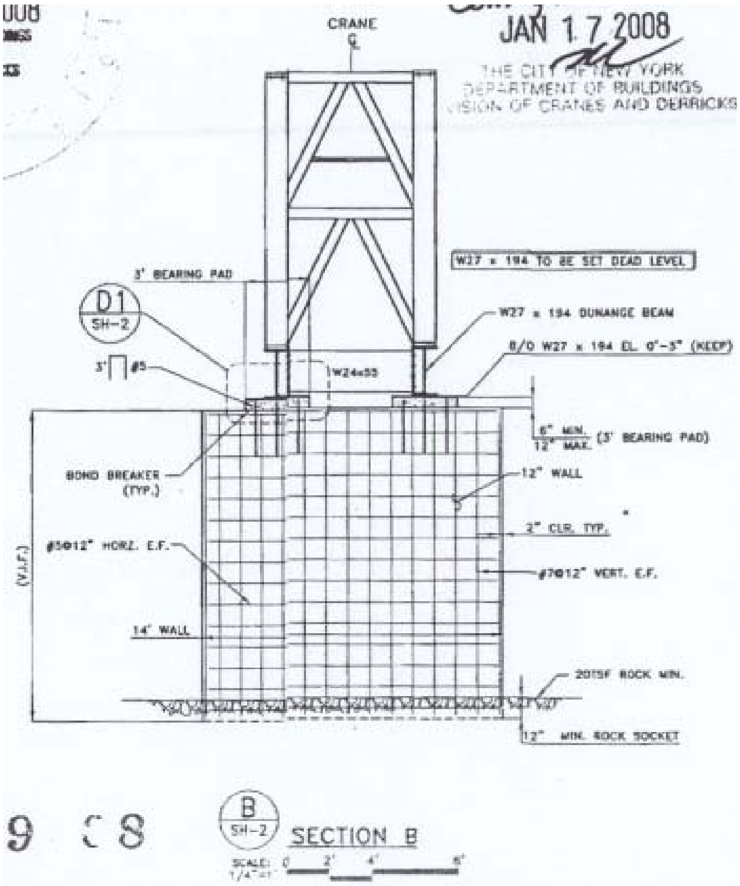


Figure 1.6 - Foundation of tower crane – Section B (source: Drawing 07-0466-1, drawing 2 of 4 dated 1/2/08, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site inspection).

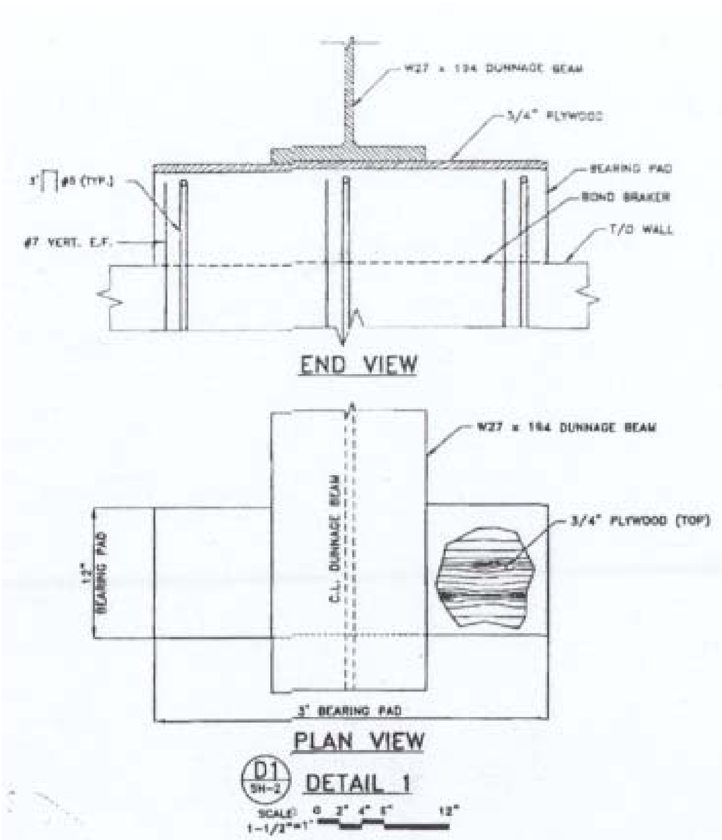


Figure 1.7 - Foundation of tower crane – Detail 1 (source: Drawing 07-046C-1, drawing 2 of 4 dated 1/2/08, submitted by the tower crane engineer (Stroh Engineering) as part of the application for a Certificate of On-Site Inspection).

Photograph 1.1 - Dunnage steel at the base of the tower crane (source: Img_0282 from April 18, 2008 visit to OEM warehouse by Arup). Visible in the photo are a W24x194 dunnage beam with a W24x55 cross-brace and W8x31 sections to create lateral restraint for the tower legs. Note the absence of anchorage for tower legs, which rested in the nooks created by the W8x31's.

Photograph 1.2 - Tower foundations (source: photograph entitled "Manhattan Crane Collapse (03-26-08) 005" from March 26, 2008 visit to the collapse site by Arup). The foundation wall for the building is in the foreground. The foundation support wall, which was constructed specifically for the dunnage beams, is in the background. The Con-Ed vault can be seen between the foundations (in the center of the photo). Note the absence of anchorage for the dunnage steel, as per the design. Also note the absence of significant damage to the foundations (limited spalling evident only).

Photograph 1.3 - Photograph showing conditions at tower base following the collapse (source: New York County District Attorney's Office, Photograph ID No. IMG-0326-056).

Photograph 1.4 – Photograph showing conditions at tower base following the collapse (source: New York County District Attorney's Office, Photograph ID No. IMG-0332-062).

Photograph 1.5 – Photograph showing conditions at tower base following the collapse (source: New York County District Attorney's Office, Photograph ID No. IMG-0372-102).

Photograph 1.6 – Photograph showing the external climbing frame attached to top of tower at the time of the collapse (source: Photos by the New York Police Department, Pictures 117-178, page 3 of 62).

Photograph 1.7 - Photograph showing the position of the crane boom during tie-in assembly, approximately 1 hour before the crane collapse (source: photograph by Gary Halby).

The main dunnage beams (x2) are W24x194 steel sections with W24x55 bracing members (x2) between the main beams. The legs of the tower were restrained in the horizontal direction by W8x31 steel sections, which were welded to the top of the dunnage beams. The tower legs were not anchored to the top of the dunnage steel and as such were free to lift. Note that if the legs lift above the W8 restraints, friction between the top of the dunnage and the tower legs becomes the only restraint against sliding, which did occur during the tower crane collapse as shown in Photographs 1.3 through 1.6.

1.1.4.3 Tower Foundations

The foundations for the tower crane (dunnage steel) consisted of:

- 12 inch thick reinforced concrete support wall taken to bedrock at one end of the main dunnage beams; and
- the reinforced concrete basement wall of the new building, which is also, founded in bedrock, at the other end of the main dunnage beams.

Drawings of the foundation support condition at the base of the tower (dunnage steel) are given in Figures 1.3 through 1.7. The base of the dunnage beams rested on the foundation walls atop of plywood bearing pads (i.e., no anchorage was provided). The design drawings suggest that movement of the dunnage beam is restricted solely by friction between the dunnage beam, plywood and concrete surfaces.

The absence of anchorage can be seen in Photograph 1.2, which was taken at the collapse site on March 26, 2008. Also evident in the photo is the lack of significant damage to the foundation walls.

1.1.5 M440D versus M440E

As discussed later in this report, photographs from the collapse site revealed that the collapsed crane was a Model M440E crane as opposed to a Model M440D crane, the model that was indicated on all documentation related to the permitting process. NYCDOB obtained documentation for the M440E crane from the crane manufacturer, Favelle Favco, after the completion of the static structural modeling. The information was used in the dynamic analysis for the tower stability and collar integrity as presented in Chapter 7.

The documentation for the M440E crane was compared to the documentation on file at NYCDOB for the M440D crane. The M440E model crane was found to be a slightly upgraded version of the M440D model crane. Both cranes are compatible with the 393 model tower sections used at the site. The arrangement of components on the machinery deck was found to vary slightly between the two crane models, however, these differences were not found to significantly alter the forces and overturning moment imposed by the crane at the top of the tower.

It was therefore determined, based on our analysis, that the differences between the M440D and M440E model cranes are immaterial with regard to findings of this investigation.

1.2 Precipitating Event

As part of the crane operating procedures, the height of the crane would be increased periodically using pre-approved established jumping (i.e., climbing) procedures supplied by the tower crane manufacturer. This is summarized in Figure 1.8 below. At the time of the collapse on March 15, 2008, the tower external climbing procedure had been completed.

Following completion of a jump, an external collar would be applied around the tower and tied back to the concrete building via steel beam tie-backs, themselves anchored to the concrete slab. This would take place at various heights as determined by the crane engineer of record, Stroh Engineering. The procedure for attaching the collar and tie-backs was provided by Favelle Favco, as shown in Figure 1.8 below. Immediately prior to the collapse, the crane tower was connected to the building under construction at both the 3rd and 9th floor levels. At the time of their installation the tower crane was not yet in service and a mobile crane was used to assist in their installation. The final configuration for the 3rd and 9th floor collars included the use of wire rope slings for vertical support at varying attachment points on the two collars.

At the 18th floor level, one-half of the collar was lifted into place, temporarily suspended by polyester web slings from the tower using two attachment points not in conformance with any of the tower crane manufacturer specified four attachment points. Following this, the other half of the collar was similarly lifted into place, suspended by polyester web slings and the two halves bolted together. Thus a total of four attachment points which were not in conformance with the eight tower crane manufacturer specified attachment points were being used to temporarily suspend the completed collar at the time of the tower crane collapse. All aforementioned tower crane manufacturer specified locations and numbers of attachment points are per the Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 (Figure 1.13).

At the time of the collapse the team had just “jumped” or extended the height of the tower and was proceeding to install a new collar connection to the 18th floor. As already indicated, the collar had been erected in two halves and connected together so as to surround the tower. The collar was suspended from the tower by four polyester web slings as the crane crew began to place the first of its three tie-beams. Eight chain blocks were specified to be used for this purpose by the manufacturer at locations other than those actually used for the polyester web slings.

It is important to note that, at the time of the collapse, it is reported that the construction crew was in the process of installing the first 18th floor level tie-beam. The weight of the tie-beam, a W12x79 less than 30 feet in length, would not have exceeded 2500lb. The crane boom was in a nearly vertical position. This would have resulted in loads on the crane below design conditions. It is also noted that this beam was later flung through the air impaling and causing damage to an adjacent building.

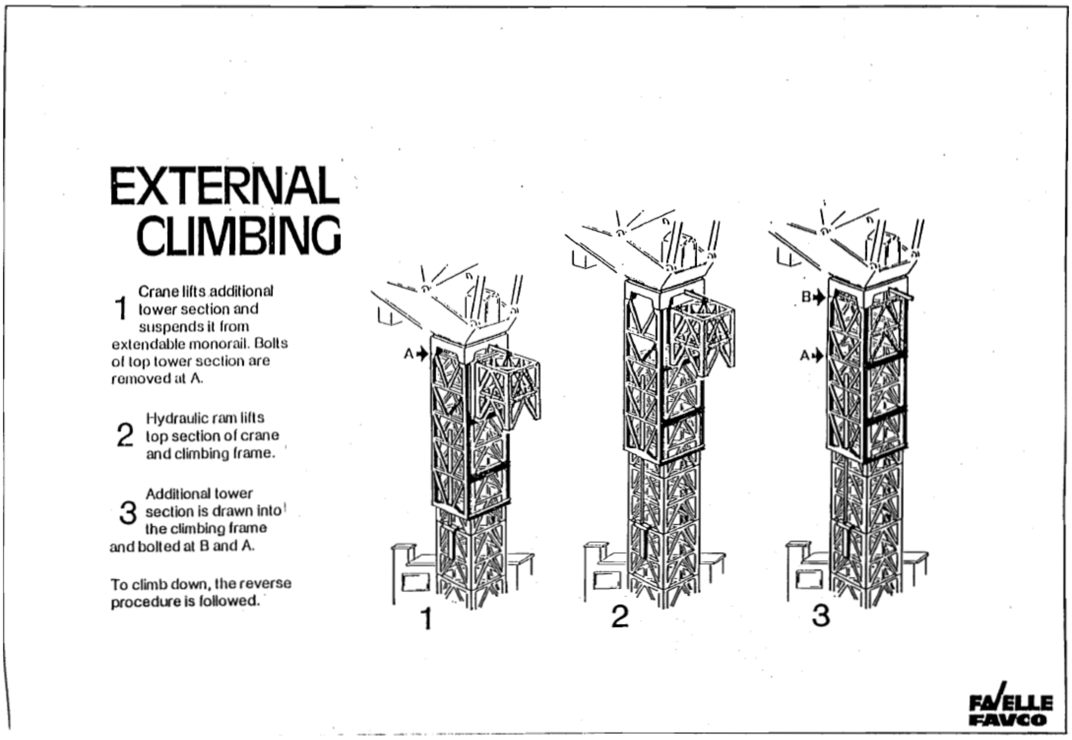


Figure 1.8 - Favelle Favco External Climbing Procedure (source: Favelle Favco Tower Cranes Operating, Maintenance and Parts Manual for a Type M440D tower crane)

The collar/tie erection sequence describes five stages of work. Prior to the collapse, the erection of the collar had been completed through Stage 3 (see Figure 1.9 below, enlarged from Figure 1.13); i.e., the two collar halves had been bolted together and each half was being held in place vertically by two chain-falls. The chain-fall consisted of a hook, winch and chain. The chain-fall was attached to the collar by the hook via the lifting points. Each chain-fall was in turn supported by a polyester sling, attached via another hook, which was choked around a tower leg. A photograph of a recovered hook/winch/chain/hook/polyester sling assembly is provided in Photograph 1.8.

Stage 4 (see Figure 1.10 below, enlarged from Figure 1.13), which involves installation of the “tie-beams” (tie-backs), had begun to the point of positioning the first tie-back into its lug in the collar where it awaited the insertion of its pin.

Photograph 1.8 – Chain-fall and polyester sling assembly recovered from collapse site by Arup.

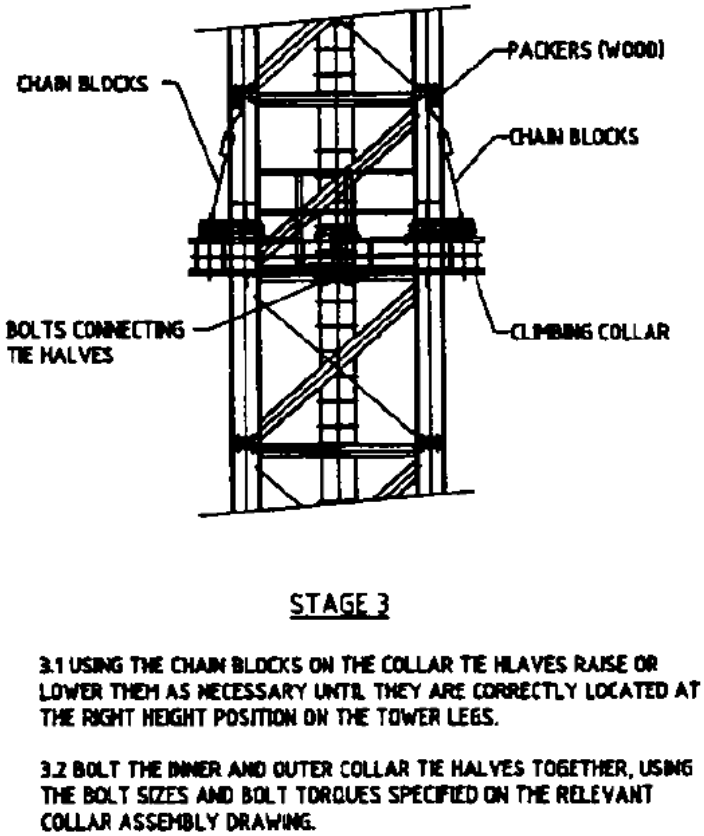


Figure 1.9 - Stage 3 of the external climbing collar/tie-erection sequence (enlarged from Figure 3.3)

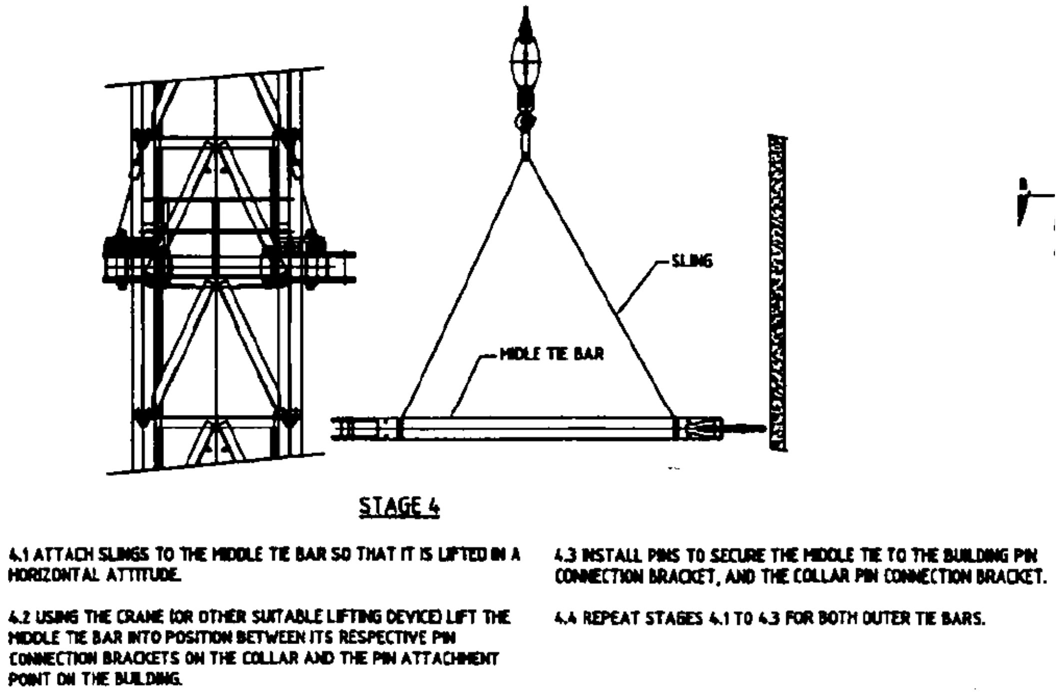


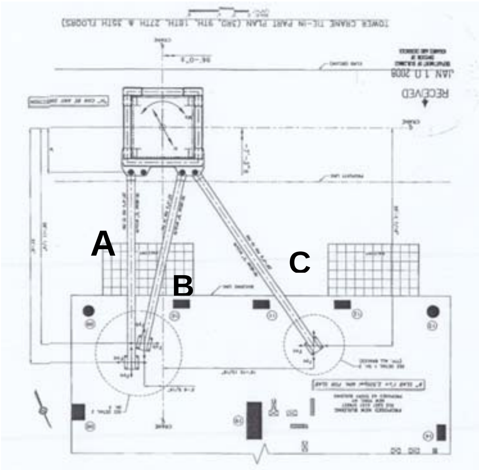
Figure 1.10 - Stage 4 of the external climbing collar/tie-erection sequence (enlarged from Figure 3.3)

It is related by two witnesses interviewed by the New York City Department of Investigations (DOI) and the New York County District Attorney (DA) that during execution of Stage 4 the four polyester slings broke, most likely, starting with the southwest corner sling, and proceeding with the southeast corner and then the two north corners. The breaking of the slings released the collar, allowing it to fall. At the time the polyester slings broke it is understood from the witness statements that a tie-beam was attached to the crane ready to be positioned for attachment to the collar.

Chock blocks, used as shims to remove any slack between the collar and the tower legs, were to be made snug against the tower legs after installation of the horizontal tie-beams as part of Stage 5 in the External Collar/Tie Erection Sequence (see Figure 1.13). The chock blocks at Level 18 would therefore not have been extended at the time the collar fell, assuming the procedures in the collar/tie-erection sequence were followed, but rather would have been retracted into their housings. As such, the chock blocks would have offered no frictional resistance against the legs of the tower at the time the slings broke and minimal frictional resistance as the collar fell.

The collar fell without deviation from the vertical axis of the tower to which its direction of travel was confined, striking the 9th floor collar below. The impact broke the tie-beam connections at the 9th floor. Five (5) points of failure at the 9th floor collar assembly were observed:

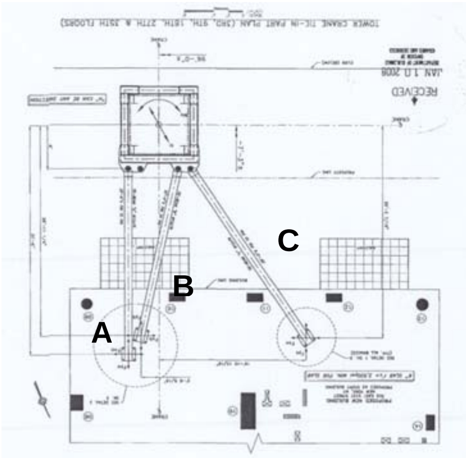
- Two (2) tie-beam to base plate welds (i.e., for base plates “9A” and “9C” as shown in Photographs 1.9 and 1.10);
- Two (2) tie-beam ends near the collar connection (i.e., at the other ends of Beams connected to base plates “B4” and “B5”, as shown in Photographs 1.11 and 1.12); and
- One (1) failure of the tie-beam at the pin connection to the collar (i.e., Beam “9B”, as shown in Photograph 1.13).



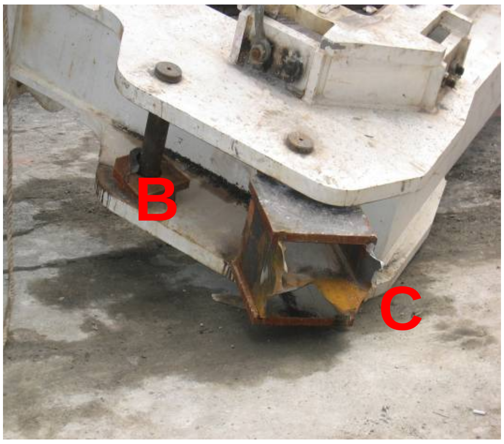
Plan view of typical collar assembly as-designed (Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 Drawing 3 of 4 dated 1/2/08

Photograph 1.9- Failed base plate 9A. Photo provided by the NYCDOB.

Photograph 1.10 – Conditions at 9th Floor: Failed base plate 9C for Beam B5 prior to its removal from the site (the bent beam in the photograph is Beam 9B). Photos provided by the NYCDOB.



Plan view of typical collar assembly as-designed (Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 Drawing 3 of 4 dated 1/2/08



Photograph 1.11 – 9th Floor Collar: Top left corner shows the general layout of tie-beams to the collar. The lower left hand photograph shows the failed end for Beam B5 (labelled "B" in the photograph) prior to and after its removal from the site (the pin end of Beam 9B (labelled "C" in the photograph) sheared off). The lower right hand photograph is the same tie-beam end after removal to the NYCOEM warehouse. Photos by the crane engineer Peter Stroh IMG_0093.jpg and Arup.

9th Floor Collar Assembly

Photograph 1.12 – The upper photograph shows the failed end of 9th floor tie-beam near collar connection (i.e., Beam B4 ("A"), at center of the photograph). The lower photograph is a close-up of the failed end of Beam B4 after removal to the NYCOEM warehouse. The top photo is by the crane engineer Peter Stroh IMG-0096.jpg. The bottom photo is provided by Arup.

Note that tie-beams B4 and B5 were welded to base plates 9A and 9C, respectively, in accordance with identification tags attached to these items by NYCDOB after recovery from the collapse site.

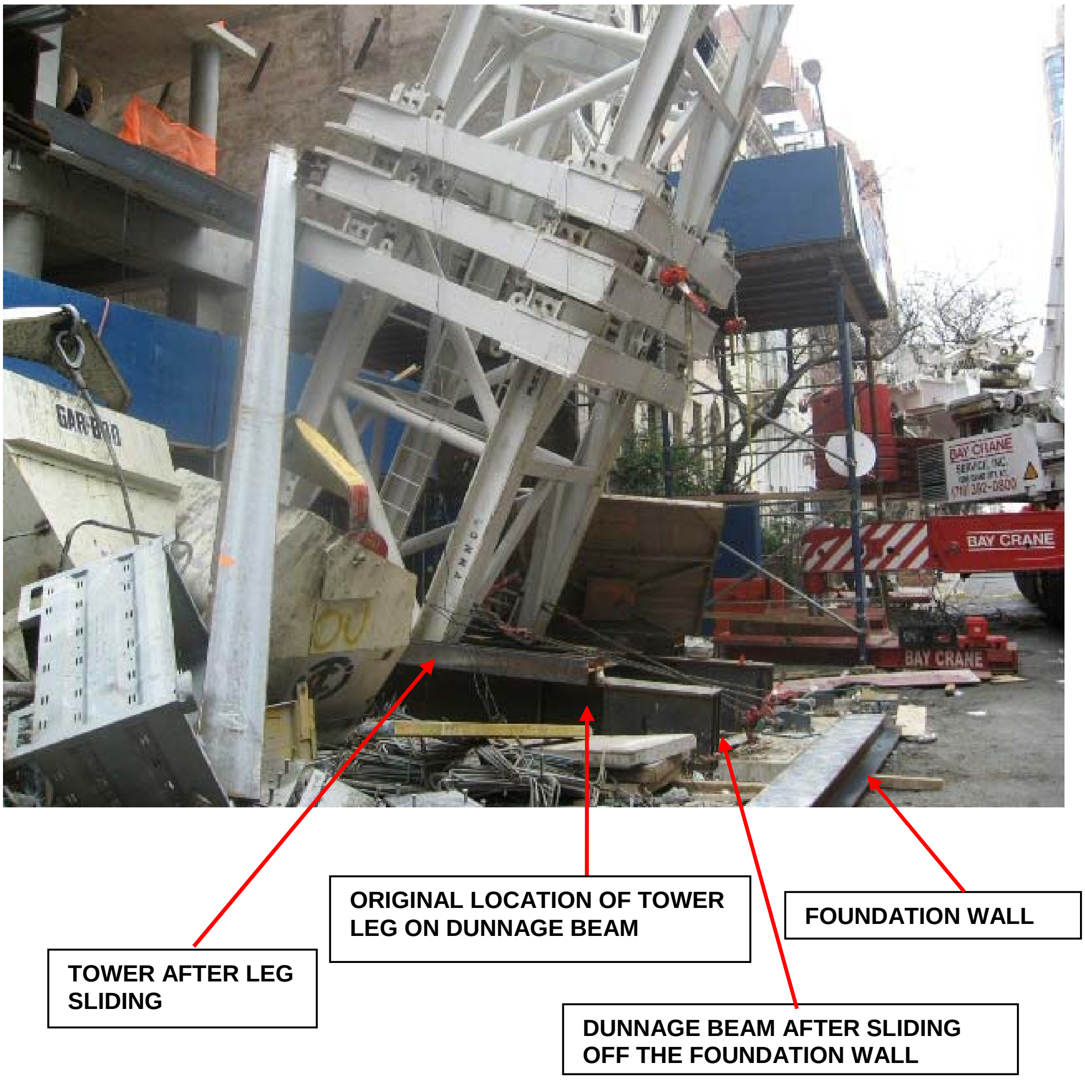
Photograph 1.13 - Failed pin end of Beam B5 at the 9th floor level as seen immediately following the collapse. Photo provided by the NYCDOB.

Following failure of the 9th collar connections, the two collars proceeded downward to strike the 3rd floor collar. While the 3rd floor collar connections were not completely severed, fractures at the collar-end of the tie-beams had commenced and plastic deformation of the beams was observed. A photograph of the 3rd floor collar assembly is given in Photograph 1.14. A view of the fracture of a tie-beam from the 3rd floor collar assembly, which did not fully fail is given in Photograph 1.15.

At some time during the fall of the collars, the supporting dunnage beam slipped off one of its supporting walls (i.e., the one furthest from the building). Additionally, the tower legs at the base of the tower lifted out of the supporting pockets located on the dunnage beam and slid along the dunnage beam. The dunnage beam movement and tower leg slippage can be seen in Photographs 1.16 and 1.17.

Photograph 1.14 - 3rd floor collar assembly following the collapse. The 9th floor collar and 18th floor collar rest atop the 3rd floor collar. Source: New York County District Attorney IMG_0355_085.jpg.

Photograph 1.15 - Fracture of a tie-beam which did not fully fail on the 3rd floor collar. (The fracture of interest is to the right of the tape measure. The damage to the left of the tape measure is a flame-cut (post failure) to assist in the removal of the debris. Photo by Arup.



Photograph 1.16 - View of street side (south) of tower base after failure. The dunnage beam is no longer on the supporting concrete foundation wall. Photo by New York County District Attorney IMG_0327_057.jpg.

Photograph 1.17 - View of building side (north) of tower base after collapse. The dunnage beam has been displaced towards the building and the plywood bearing pads are off of the foundation wall. Lifting of the back tower legs and slippage of the front tower legs is also visible. Photo by New York County District Attorney IMG_379_109.jpg.

The balconies at the 9th and 3rd floors of the building were damaged, as can been seen in Photographs 1.18 and 1.19. No other significant damage to the building itself was apparent. Some minor spalling of concrete was observed in the vicinity of the bolted connections, as can be seen in Photograph 1.20.

Of significant interest was the absence of failure of the tower crane sections themselves beyond that experienced by the impact of the upper sections with the building opposite (see Photographs 1.21 and 1.22 showing the lean of the tower across the street). The photographs also show the extent of the damage to the adjacent building caused by the crane collapse and the impact of the tower crane on these buildings. Figures 1.11 and 1.12 highlight the buildings affected by the collapse of the tower crane.

Despite the dynamic loading, impact against the building and final positioning, the main tower sections did not fail with the exception of the point of impact to the building. Photograph 1.6 shows the top part of the tower after breaking from the lower section and falling to ground level, still intact.

Photograph 1.18 - Damage to 9th floor balcony. The imprint in the slab is from Beam 9B. Photo by Arup.

Photograph 1.19 - Damage to 3rd floor balcony. Photo by Arup.

Photograph 1.20 - Minor spalling of concrete in the vicinity of the bolted connections to the 9th floor slab. Photo by Arup.

Representative views of the tower crane before and after collapse, based upon photograph evidence, are given in Figures 1.11 and 1.12.

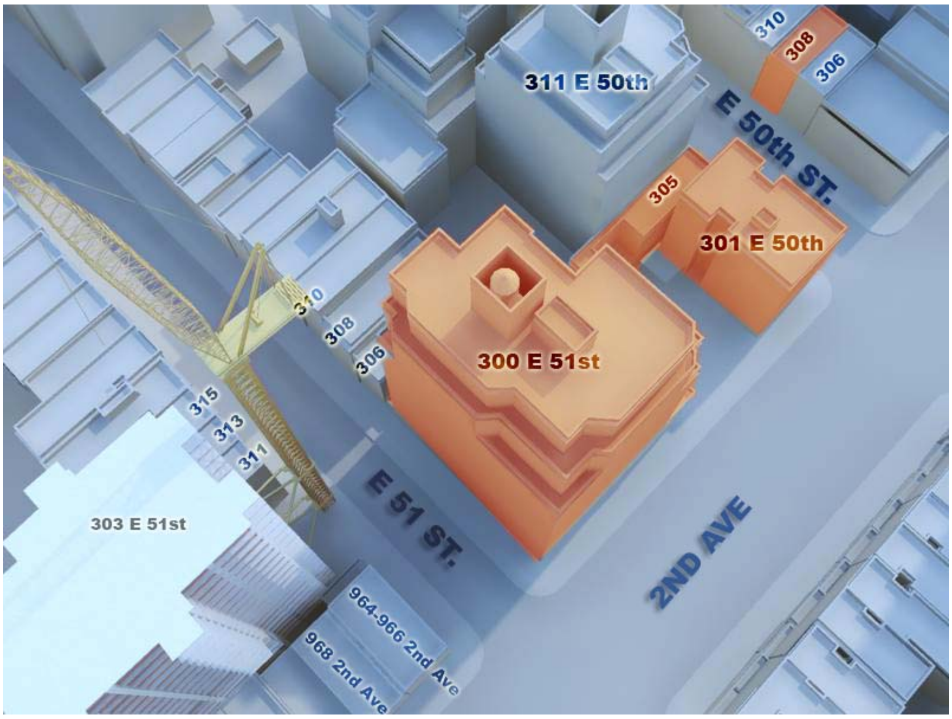


Figure 1.11 - Representation of the tower crane prior to collapse. Prepared by Arup.

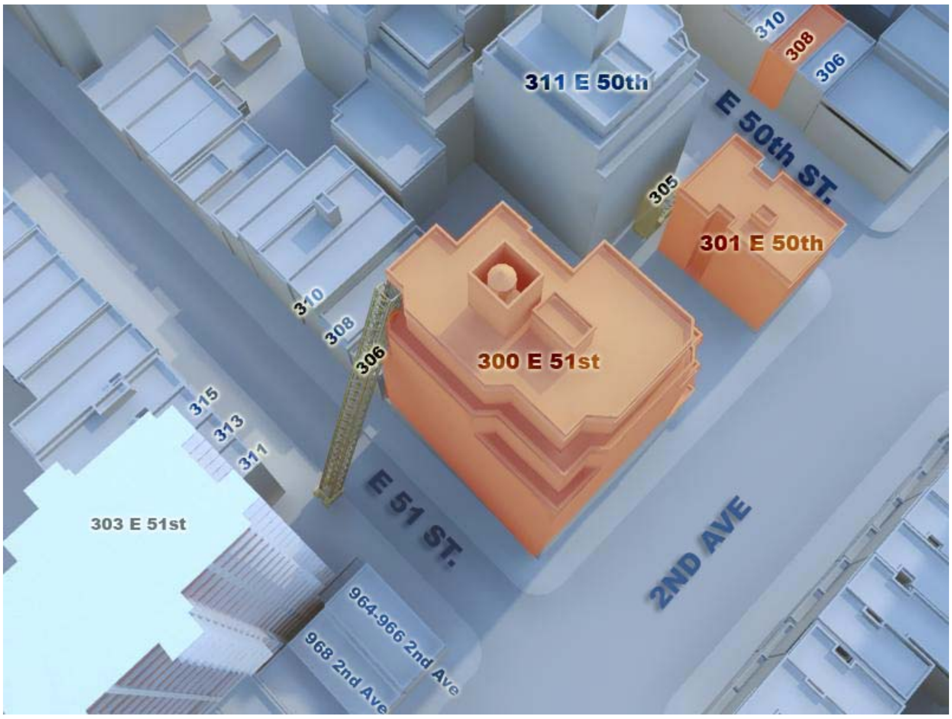
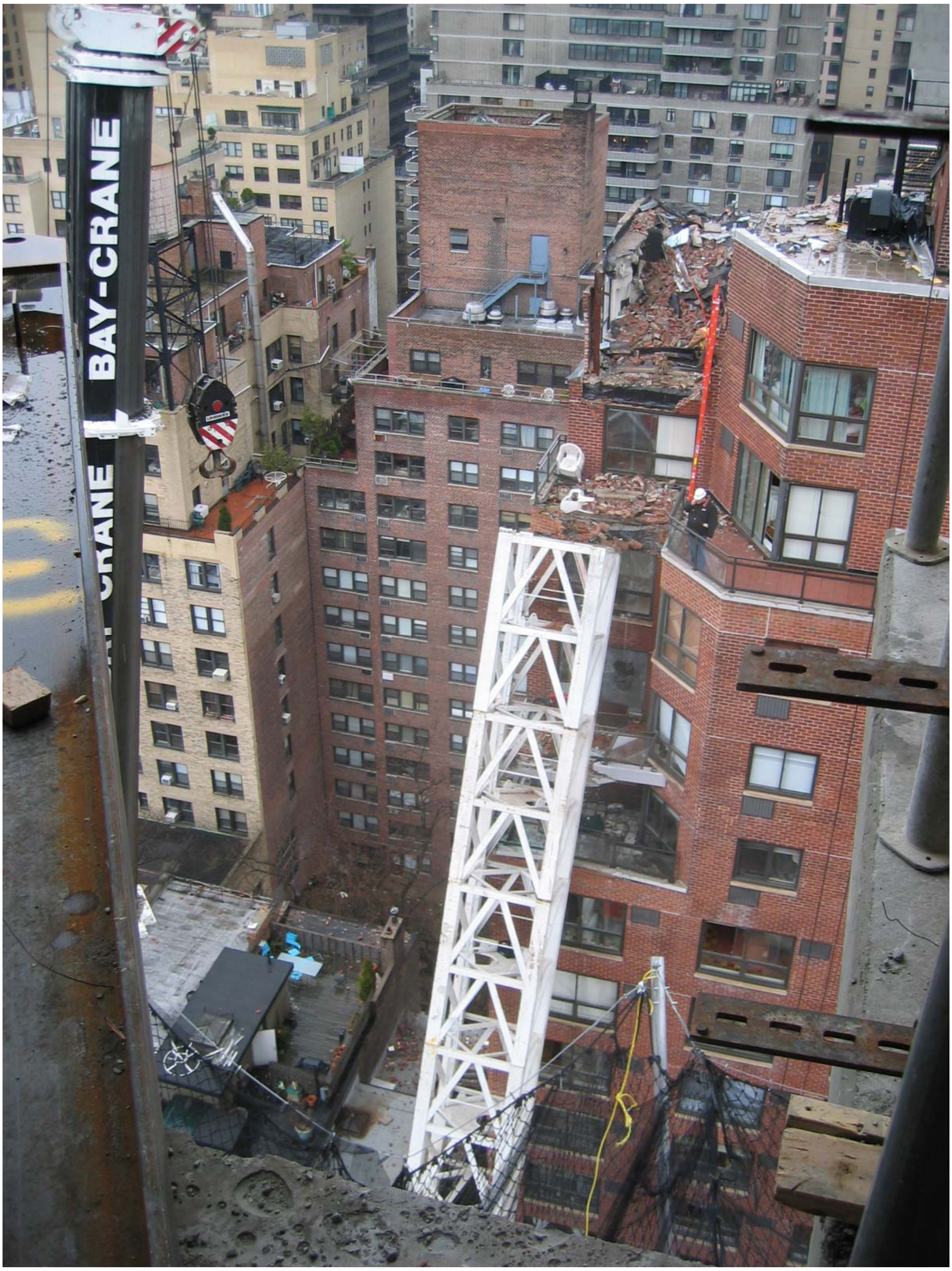


Figure 1.12 - Representation of the tower crane after collapse. Reference Photograph 1.6 with regard to the position of the crane within 305 E 50th St. Prepared by Arup.



Photograph 1. 21 – View of the intact tower crane after collapse. Photograph by the crane engineer Peter Stroh, “After” photographs, IMG_066.jpg.

Photograph 1.22 – View of the intact tower crane after collapse. Photograph, from the crane engineer Peter Stroh, "After" photographs IMG_077.jpg.

1.3 Statement of Engagement and Scope of Work

Ove Arup & Partners PC ("Arup") was engaged to provide engineering and investigative services in connection with the crane collapse.

As part of the investigation, external sub consultants were retained to assist in selected tasks. These included, Dale H. Curtis, PE, Curtis Engineering Corporation, a licensed professional engineer and crane certified agent, the Center for Advanced Technology for Large Structural Systems (ATLSS), a National Engineering Research Center, Lehigh University, Bethlehem, PA., and, Dr. Tushar Ghosh of North Carolina State University College of Textiles, a specialist in polyester materials.

As part of the included scope of work for this investigation, work undertaken has included:

1. Document review, including documents supplied by NYCDOB and the New York County District Attorney (DA) and the New York City Department of Investigation (DOI)
2. Site visit to the collapse site
3. Site visits to view components of the tower crane and collar assemblies, including tower segments, tie-beams and base plates, collar assemblies, chain fall and polyester sling assemblies, wire rope segments and other components recovered from the collapse site
4. Structural analysis of the tower crane, including both static analysis and non-linear dynamic analysis
5. Structural analysis of the sling system including non-linear dynamic behavior.
6. Materials and metallurgical testing
7. Witnessing of sling tests specified by OSHA, review of raw data from those tests and review of the OSHA-related correspondences.
8. A review of codes, standards and regulations of relevance to the collapse
9. A review of the permitting process associated with this crane
10. An independent peer review of the crane's support connecting to the building and to the ground.

1.4 General Approach to the Investigation and Structure of this Report

As indicated previously, Ove Arup & Partners, PC (Arup) was hired by the New York City Department of Buildings (NYCDOB) to provide engineering and investigative services in connection with the crane collapse.

At the time the investigation began the cause of collapse was unknown. Certain component parts were identified as being of especial interest, with particular emphasis initially placed upon the base plate welds connecting the collar tie-beams to the building. Due to the breadth of potential initial causes for the collapse and range of possible contributing factors, attention was initially given to all facets of the tower crane, including design, fabrication, assembly and usage in the initial review of data, as well as potential issues surrounding the cause and consequences of potential weld defects in the base plates.

The approach to the investigation therefore began with identification of failed components through direct observation of the available tower crane components and failure site with a view to establishing priorities for investigation, concentrating on the base plate welds and associated components. This included site visits to the collapse area as well as NYCDOB, NYCDOI, NYDA and eventually OSHA repositories to view the collected physical evidence. Site visits are listed in Chapter 10. A list of items retained by NYCDOB is given in Appendix I. As additional information was obtained either through direct observation, document review or analysis, these priorities were adjusted.

Industry accepted standards were used as the basis for establishing accepted industry practice. Codes and standards selected for review included the New York City Building Code, selected sections of the American Society of Mechanical Engineers Standards B30 Safety Standard for Cableways, Cranes, Derricks, Hoists, Hooks, Jacks, and Slings, American Welding Society Structural Welding Code D1.1, publications of the Web Sling and Tie Down Association, publications of the Wire Rope Technical Board publications and certain military standards on wire rope. These are described in Chapter 3.

Documentation related to the tower crane failure was collected and reviewed. Again, as the range of possible contributory factors to the collapse was initially vast, the review of documentation was similarly broad. As part of the documentation, a review of the permits prepared and submitted was carried out. This is described in Chapter 4.

As hypotheses were proposed as possible explanations of the observed phenomena, it became necessary to formulate experimental studies via computer modeling to test these hypotheses. Results of these analyses needed to be consistent with both the observed phenomena and documentation as well as consistent with related studies underway simultaneously in order to dependably determine causes of failure. Both prioritized areas of interest and failure hypotheses would be adjusted as analysis results and other additional information were obtained.

Computer analyses therefore included both macroscopic studies, encompassing the tower crane structure in its entirety and localized studies involving detailed analyses of smaller components. The global approach was used in the investigations of general stability to the tower and in the development of design criteria for the various components which supported the tower. These were initially limited to static analyses of the tower crane. As the investigation progressed plastic deformation / large displacement / dynamic computer analyses were implemented. Due to the extremely long durations which such computer runs required these were necessarily selective rather than all encompassing. Smaller models investigating particular components, such as welds, tie-beams and polyester web slings were more easily manipulated and adjusted as additional information would inevitably arrive. These analyses are described in detail in the Chapters 6 and 7 of this report.

Theories that encompassed wider domains were then developed to bind these hypotheses surrounding the smaller events into a coherent structure. This in turn assisted in reaching the final conclusions presented within this report.

Within the context of this investigation was the need to maintain and clearly be able to demonstrate objectivity. This included sensitivity analyses of unknown or ill-defined parameters to establish a lack of bias in the selection of data. This practice also allowed for a measure of the reliability of these data to be established. Among parameters which were allowed to vary included initial impact velocity of the 18th floor level collar on the 9th floor level collar, friction coefficients between the dunnage beam and foundation, polyester web sling tensions and base plate fillet weld ductility.

Expertise was sought to supplement knowledge and experience in certain specialized areas. These areas included metallurgy, polyester materials and tower crane practice. Results of these specialized studies are given in Chapters 5 (crane practice), 8 (polyester materials) and 9 (metallurgical examination).

Each element of the investigation, including but not limited to dynamic analysis modeling, development of test protocols, and document review was subjected to peer review to confirm its validity.

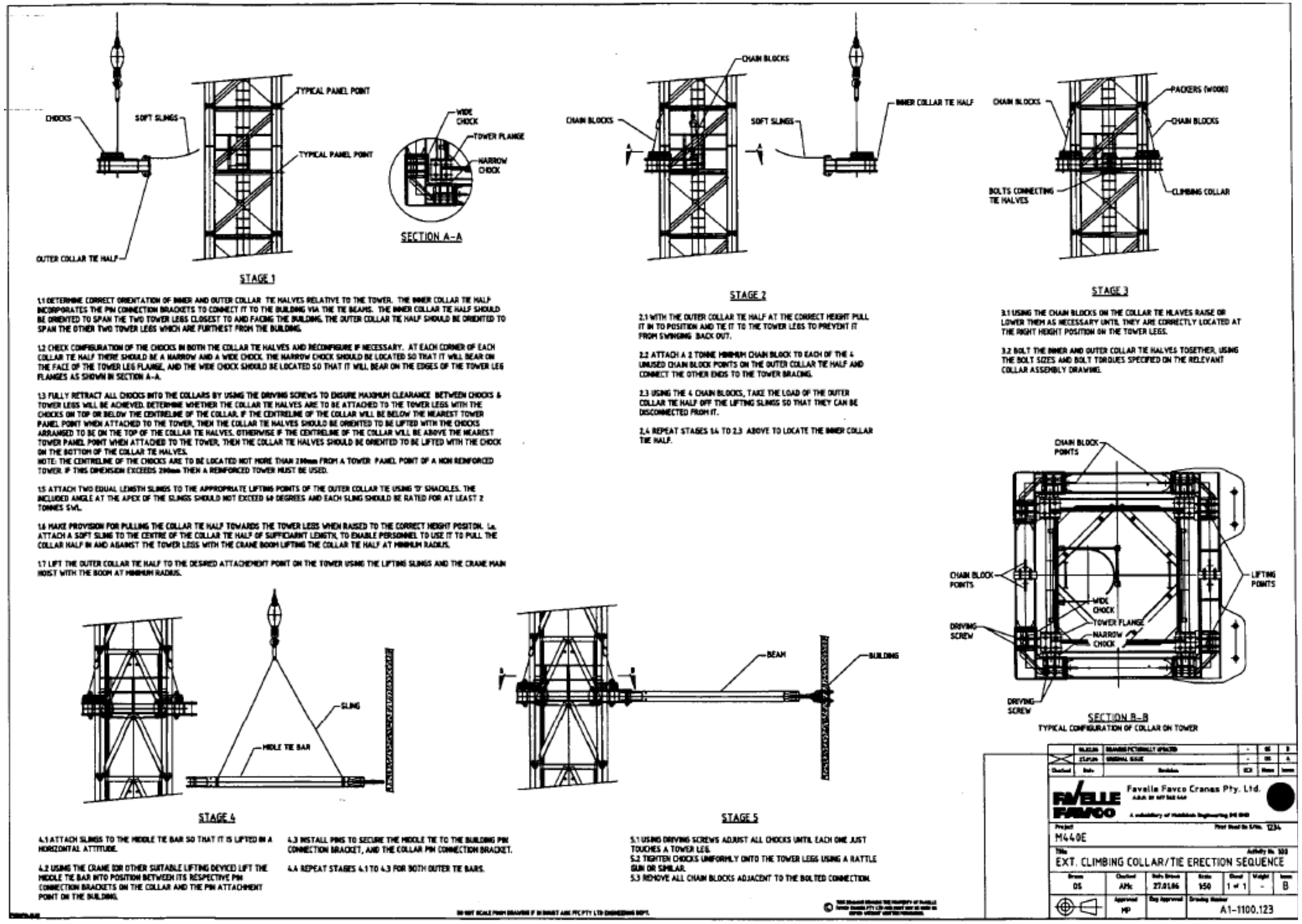
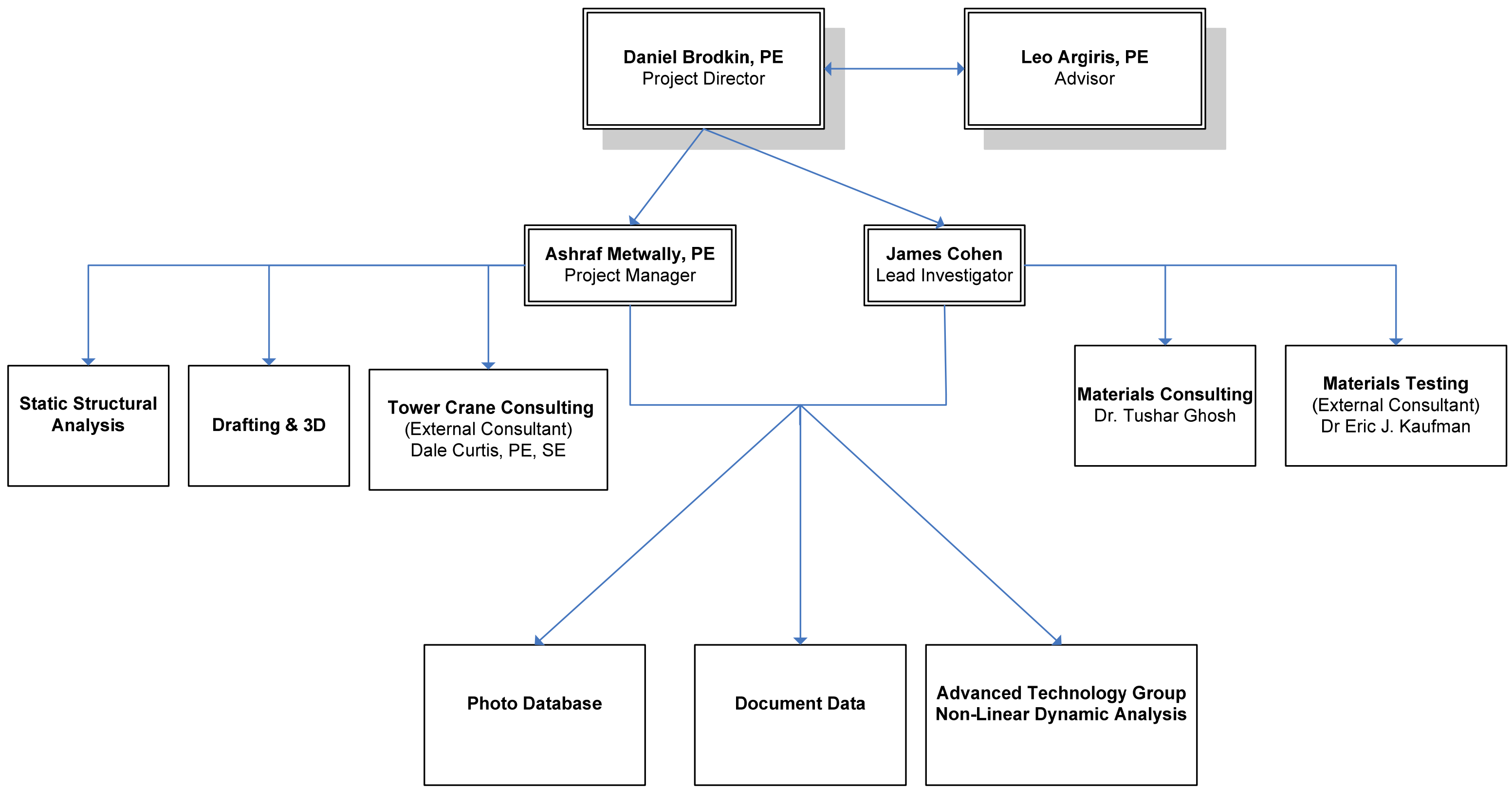


Figure 1.13 - External climbing collar/tie-erection sequence (source: Favelle Favco drawing no A1-1100-123, Rev B dated February 6, 2006)

2 Team


Daniel Brodkin, PE
Project Director
Leo Argiris, PE
Advisor
Ashraf Metwally, PE
Project Manager
James Cohen
Lead Investigator
Static Structural Analysis
Drafting & 3D
Tower Crane Consulting
(External Consultant)
Dale Curtis, PE, SE
Materials Consulting
Dr. Tushar Ghosh
Materials Testing
(External Consultant)
Dr Eric J. Kaufman
Photo Database
Document Data
Advanced Technology Group
Non-Linear Dynamic Analysis

3 Codes and Standards Review

Selected industry codes and standards related to the tower crane collapse were reviewed. For each of these documents, relevant sections have been extracted and are provided below. Extracts were selected based upon several considerations. In some cases, they provide basic background to terminology or usage, while in others; they provide specific performance and other requirements. A brief discussion is given of each clause or section only where these relate to issues of non-compatibility with the specific requirements or recommendations contained. Where requirements or recommendations were in compliance, or items are included for information only, no further comment follows.

Design and installation of tower cranes in New York City are regulated by the NYC Building Code and the standards referenced herein. We note that the standards promulgated by the American Society of Mechanical Engineers and the Web Sling and Tie Down Association recommended standards are not cited in the New York City Building Code. However, the former are prepared under the procedures accredited as meeting the criteria for American National Standards and incorporate the latter documents in these standards. As such, they represent accepted industry practice with regard to the design, manufacture and safe usage of the items contained within those documents and are therefore included in the document review below. The quoted standards materials in sections 3.1.3 to 3.2.2 are owned by their respective promulgators and not the preparer of this report or the City of New York.

3.1 Codes and Standards

The following codes and standards were reviewed as part of the failure investigation:

- New York City Building Code Subchapter Title 27 Subchapter 19 – Safety of Public and Property During Construction Operations
- New York City Building Code Reference Standard RS-19-2 Power Operated Cranes and Derricks
- American Society of Mechanical Engineers B30.26-2004 Rigging Hardware
- American Society of Mechanical Engineers B30.3-2004 Construction Tower Cranes
- American Society of Mechanical Engineers B30.9-2006 Slings
- American Welding Society D1.1:2004 Structural Welding code – Steel, cited by the New York City Building Code
- Web Sling and Tie Down Association WSTDA-WS-1 Recommended Standard Specification for Synthetic Web Slings; 4th Revision 2004
- Occupational Safety & Health Administration 29 CFR Part 1910 Subpart N Section 1910.184 – Slings
- Occupational Safety & Health Administration 29 CFR Part 1926 Subpart N Section 1926.550 – Cranes and Derricks

Note: references to figures within any quoted standard have been removed.

The following excerpts from the above referenced codes and standards were of particular interest with regard to the failure investigation. Note that a permitting review of the New York City Building Code is presented elsewhere in this report and clauses respecting permitting issues are therefore not repeated below.

3.1.1 Title 27 / Subchapter 19 SAFETY OF PUBLIC AND PROPERTY DURING CONSTRUCTION OPERATIONS

ARTICLE 1 GENERAL

§[C26-1900.1] 27-1007 Scope.-The provisions of this subchapter shall govern the conduct of all construction operations with regard to the safety of the public and property. For the purposes of this subchapter, construction operations shall include excavation, erection, alteration, repair, removal and demolition as related to buildings. For regulations relating to the safety of persons employed in such construction operations, the provisions of subchapter ten of the labor law as implemented by the industrial code of the state of New York, rule no. 23, shall apply

…

§[C26-1900.3] 27-1009 General requirements.-

(a) A contractor engaged in building work shall institute and maintain safety measures and provide all equipment or temporary construction necessary to safeguard all persons and property affected by such contractor's operations.

(b) No structure, device, or construction equipment, whether permanent or temporary, including all partly or fully completed elements or sections of the building, shall be loaded in excess of its design capacity.

As discussed elsewhere in this report, the capacity of the polyester web slings was exceeded, resulting in the collapse of the tower.

…

(d) A construction site safety coordinator must be designated and present on a construction site in accordance with department rules and regulations.

§[C26-1900.4] 27-1010 Inspection.- Except for the installation of underpinning and the construction of temporary retaining structures (see section 27-724 of article thirteen of subchapter eleven of this chapter) and for other operations specifically required by the provisions of this subchapter to be inspected by an engineer or an architect, inspection of operations for compliance with the provisions of this subchapter may be performed by, or under, the authority of the person superintending the work. Unless required by the provisions of this subchapter, inspection and test reports relating to operations within the scope of this subchapter need not be filed.

Based upon documents reviewed, it appears that William Rapetti had full responsibility for all inspecting operations with regard to the erection of the tower collars and tie-beam assemblies, including rigging.

§[C26-1900.5] 27-1011 Sizes and stresses of materials.-

(a) Sizes.-All sizes and dimensions prescribed in this subchapter are minimum requirements. Lumber sizes are nominal or commercial except where stated otherwise.

(b) Stresses.-Except where sizes are specifically prescribed in this subchapter, temporary equipment and constructions shall be designed so that the allowable stress values prescribed in subchapter ten of this subchapter are not exceeded.

§[C26-1900.6] 27-1012 Inspection.- Any construction equipment or device, except hand tools, that would affect the public safety when operated shall be inspected by the person superintending the work or by his or her designated representative before using the equipment or device on a specific job. Such inspection shall be carefully made, and every defect or unsafe condition shall be corrected before use is permitted. Any unsafe equipment or device shall be made safe immediately or removed from the site. Periodic inspection procedures shall be instituted during construction operations, and a record of inspections shall be kept at the site for the duration of the work.

Based upon documents reviewed, it appears that William Rapetti had full responsibility for all inspecting operations with regard to the erection of the tower collars and tie-beam assemblies, including rigging.

…

§[C26-1900.9] 27-1015 Design.-Whenever design is required by the provisions of this subchapter, such design shall be executed by, or under, the supervision of an engineer or an architect who shall cause his or her seal and signature to be affixed to any drawings or specifications that may be required for the work. All such documents shall be kept at the site for inspection by the commissioner for the duration of the job.

Based upon documents reviewed, the crane engineer (Stroh Engineering) had responsibility for the design of the ancillary components, including the tie-beams, tie-beam connections and foundations and accordingly these drawings and associated calculations were signed and sealed by the crane engineer (Stroh Engineering).

…

§[C26-1909.2] 27-1055 Rigging, rope, chains, and their appurtenances and fittings.-

…

(b) Wire rope or cable.-

…

(2) Wire cable shall not be used under the following conditions:

a. When it is knotted or kinked.

b. When more than ten percent of the total wires are broken in any lay, a lay being that distance measured along the cable in which one strand makes a complete revolution around the cable axis.

c. When the wires on the crown of the strands are worn down or rusted to less than sixty percent of their original cross-sectional area.

d. When any combination of broken wires, rust, or abrasion has reduced the strength of the cable to eighty percent or less of its original strength.

(3) At least four turns of the cable shall remain on the hoist drum at all times.

(4) Wire cable fastenings shall conform to the provisions of article eleven of subchapter ten of this chapter, and shall consist of zinc-filled sockets, wedge sockets with at least one cable clip above the socket, thimble and splice connections, or thimble and cable [clips].

(5) Where cable clips are used, the minimum number shall conform to the following:

Diameter of wire rope No. of clips

Up to and incl. 3/4 in………………... 3

From 3/4 in. up to and incl. 1 in…….. 4

From 1 in. up to and incl. 1 1/4 in…… 5

From 1 1/4 in. up to and incl. 2 1/2 in.. 6

(6) Clip spacing shall be at least six times the diameter of the cable, and the "U" part of the clip shall be placed over the short end of the cable. After the rope is in service and while it is under tension, the nuts on the clips shall be retightened.

As noted in the section below on ASME B30.26 Rigging Hardware, wire ropes were used to provide vertical support to the collars after completion of the tie-beam assembly. These were attached to the collars via the chain block points which were designated by Favelle Favco for the chain blocks. The "U" part of the clip for the wire rope U-bolt clip saddles were placed both on the live (i.e., long) and dead (i.e., short) ends of the wire ropes, and, additionally, were often not spaced with the requisite minimum spacing. The installation of the wire rope was therefore non-compliant with the requirements cited above.

…

(e) Fittings.-

(1) All wire rope fittings, including sockets, thimbles, clips, blocks, shackles, etc. shall be of the standard size, diameter, and grooving to fit the size of and to develop the breaking load capacity of the rope on which they are to be installed.

(2) Hooks, shackles, or other fittings deformed due to wear, over-stress, or other cause shall not be used.

(3) Safety hooks or open type hooks with wire mousings shall be used where loads may be accidentally unhooked.

…

(g) Slings.-

(1) Blocks or heavy padding shall be used at corners of the load to protect the sling from sharp bending.

As noted in the section below on ASME B30.26 Rigging Hardware, this was not done.

(2) When lifting a load with multiple slings, the slings shall be so arranged as to equalize the load between the slings.

(3) The ends of slings made of wire or fibre shall be properly spliced to form the eyes. Eyes for wire rope shall be formed using thimbles.

As noted in the section below on ASME B30.26 Rigging Hardware, this was not done.

(4) Wire rope slings shall be frequently inspected and lubricated.

…

§[C26-1909.4] 27-1057 Testing inspection, approval and use of power operated cranes, derricks and cableways

…

(h) Special requirements for cranes and derricks.- The construction, installation, inspection, maintenance and use of power operated cranes and derricks shall be in conformance with reference standard RS 19-2.

3.1.2 NEW YORK CITY BUILDING CODE REFERENCE STANDARD RS-19-2 Power Operated Cranes and Derricks

1.0 Scope.-This standard applies to the construction, installation, inspection, maintenance and use of power operated cranes and derricks used for hoisting and/or rigging purposes; or used for the construction, alteration, demolition, excavation and maintenance purposes, including highways or sewers; or used for the installation of piles; or used for the hoisting or lowering of any article on the outside of any building or structure.

…

2.0 Definitions.-

2.1 ACCESSORY.-A secondary part of assembly of parts which contributes to the overall function and usefulness of a machine.

2.2 APPOINTED.-Assigned specific responsibilities by the employer or by the employers representative.

…

2.18 COUNTERWEIGHT.-Weight used to supplement the weight of the machine in providing stability for lifting working loads.

2.19 CRANE.-A power operated machine for lifting or lowering a load and moving it horizontally which utilizes wire rope and in which the hoisting mechanism is an integral part of the machine.

…

…

2.23.1 ENGINEER.-The word engineer as used in these regulations shall mean a licensed professional engineer except that the certifications for matters relating to crane design may be made by an engineer licensed by any state or foreign jurisdiction or upon proof, to the satisfaction of the commissioner, of his professional competence.

…

2.32 LOAD RATINGS.-Maximum loads that may be lifted by a crane or derrick at various angles and positions as approved by the department.

…

2.36 ROPE.-Refers to wire rope unless otherwise specified.

…

2.39 STANDING (GUY) ROPE.-A supporting rope which maintains a constant distance between the points of attachment to the two components connected by the rope.

2.40 STRUCTURAL COMPETENCE.-The ability of the machine and its component to withstand the stresses imposed by applied loads.

2.41 SUPERSTRUCTURE. The rotating upper frame structure of the machine and the operating machinery mounted thereon.

…

2.44.2 TOWER CRANE.-A crane in which a boom, swinging jib or other structural member is mounted upon a vertical mast or tower.

…

11.0 Welded Construction.-

11.1 Welding of structural members of cranes and derricks enumerated in 1 and 2 of 3.1.1 shall conform to the recommended practices of the American Welding Society as outlined in specifications for welded highway and railway bridges AWS D2.0-66 or other recognized standards or pursuant to RS 19-2.

11.2 For welding of steels not covered by AWS D2.0- 66, for all cranes manufactured after April 1, 1970 and all welding performed after that date shall conform to recognized standards.

11.3 The commissioner may require such welds to be tested as he deems necessary.

A review of AWS D1.1 is given below with respect to the tie-beam connections. Documentation respecting testing of these welds has not been reviewed. It is understood that testing was not required.

…

13.0 Load Ratings Where Structural Competence Governs Lifting Performance.-

13.1 Load rating at some radii for mobile cranes and load ratings for climber, tower cranes and derricks are governed by structural competence. Therefore, the limitation on crane loading must be such that no structural member is overstressed, and load rating charts shall be subject to this limitation.

13.2 Load Rating Chart.-

…

13.2.3 Tower and climber cranes.-A substantial, durable and clearly legible rating chart shall be provided with each tower and climber crane and securely affixed in the cab. The chart shall include load ratings approved by the department for specific lengths of components, counterweights, swing, and radii.

…

15.0 Inspection Required by Owner for Cranes and Derricks.-

15.1 Certification and inspections required.-The owner of a crane or derrick when applying for a certificate of approval in accordance with 3.0 shall certify that all applicable regulations regarding inspection and maintenance will be complied with. All inspections required by the owner shall be performed only by appointed personnel. The inspections shall be performed to provide information requested in a department supplied chart and all deficiencies shall be corrected. No record of *information not required by such chart shall be required to be maintained in writing.*

15.2 Inspection classification.-Inspection procedure for cranes and derricks in regular service is divided into two general classifications based upon the intervals at which inspection should be performed. The intervals in turn are dependent upon the nature of the critical components of the crane or derrick and the degrees of their exposure to wear, deterioration or malfunction. The two general classifications are herein designated as "frequent" and "periodic" with respective intervals between inspections as defined below:

15.2.1 Frequent inspection.-Daily to monthly intervals.

15.2.2 Periodic inspection.-1 to 12 month intervals or as specifically recommended by the manufacturer.

**15.3 Frequent inspection.-Items such as the following shall be inspected for defects at intervals as defined in 15.2.1 or as specifically indicated, including observation during operation for any defects which might appear between regular inspections. Any defects revealed by inspection shall be corrected. Where such defects constitute a safety hazard, the crane or derrick shall not be operated until such defects are corrected.*

**Local Law 50-1973*

15.3.1 All control mechanisms for maladjustment interfering with proper operation.-Daily.

15.3.2 All control mechanisms for excessive wear of components and contamination by lubricants or other foreign matter.

15.3.3 All safety devices for malfunction.

15.3.4 Deterioration or leakage in air or hydraulic systems.-Daily

**15.3.5 Crane or derrick hooks with deformations or cracks.-Refer to 17.3.3(c).*

**Local Law 50-1973*

15.3.6 Rope reeving for non-compliance with crane or derrick manufacturer's recommendations.

15.3.7 Electrical apparatus for malfunctioning, signs of excessive deterioration, dirt, moisture accumulation, weatherproofing and grounding.

15.3.8 Tension in derrick guys.-Daily.

15.3.9 Plumb of derrick mast.

15.3.10 Hoist brakes, clutches and operating levers.- Check daily for proper functioning before beginning operations.

It is noted that inspection by the owner of chain falls supporting the collars for self-erecting tower cranes is not required.

**15.4 Periodic inspections of cranes and derricks.-*

Complete inspections of the crane or derrick shall be performed at intervals as generally defined in 15.2.2, depending upon its activity, severity of service, and environment, or as required by 15.5.1 or 15.5.2. These inspections shall include the requirements of 15.0, and in addition, items specifically indicated below. Any *defects revealed by inspection shall be corrected. Where such defects constitute a safety hazard the crane or derrick shall not be operated until such defects are corrected.*

**Local Law 50-1973*

15.4.1 Deformed, cracked or corroded members in the crane or derrick structure and boom.

15.4.2 Loose bolts or rivets.

15.4.3 Cracked or worn sheaves and drums.

15.4.4 Worn, cracked or distorted parts such as pins, bearings, shafts, gears, rollers and locking devices.

15.4.5 Excessive wear on brake and clutch system parts, linings, pawls and ratchets.

15.4.6 Load, boom angle and other indicators over their full range, for any significant inaccuracies.

15.4.7 Gasoline, diesel, electric or other power plants for improper performance or non-compliance with safety requirements.

15.4.8 Excessive wear of chain drive sprockets and excessive chain stretch.

15.4.9 Crane or derrick hooks.-Magnetic particle or other suitable crack detecting inspection should be performed at least once each year by an inspection agency retained by the owner and approved by the department. Certified inspection reports are to be made available to the department upon request.

15.4.10 Travel steering, braking and locking devices, for malfunction.

15.4.11 Excessively worn or damaged tires.

15.4.12 Derrick gudgeon pin for cracks, wear and distortion each time the derrick is to be erected.

15.4.13 Foundation or supports shall be inspected for continued ability to sustain the imposed loads.

It is noted that inspection by the owner of chain falls supporting the collars for self-erecting tower cranes is not required.

…

18.0 Rope Inspection, Replacement and Maintenance by Owner for Cranes and Derricks.-

18.1 Inspection.-

**18.1.1 All ropes in continuous service should be visually inspected once every working day. A thorough inspection of all ropes in use shall be made at least once a month. All inspections shall be performed by an appointed person. Any deterioration resulting in measurable loss of original strength, such as described below shall be carefully noted and determination made as to whether further use of the rope would constitute a safety hazard:*

(a) Reduction of rope diameter below nominal diameter due to loss of core support, internal or external corrosion or wear of outside wires.

(b) A number of broken outside wires and the degree of distribution of concentration of such broken wires.

(c) Worn outside wires, "birdcaging," or kinks.

(d) Corroded or broken wires at end connections.

(e) Corroded, cracked, bent, worn or improperly applied end connections.

Wire ropes were used to provide vertical support to the collars after completion of the tie-beam assembly. These were attached to the collars via the chain block points which were designated by Favelle Favco for the chain blocks. Observations of the removed artifacts indicate that wire rope clips were improperly located and applied. A fuller discussion is presented in the section on ASME B30.26 Rigging Hardware, below.

(f) Severe kinking, crushing, cutting or unstranding.

**Local Law 50-1973*

…

18.3 Rope Maintenance.-

18.3.1 Rope shall be stored to prevent damage or deterioration.

18.3.2 Unreeling or uncoiling of rope shall be done as recommended by the rope manufacturer and with extreme care to avoid kinking or inducing a twist.

18.3.3 Before cutting a rope, seizings shall be placed on each side of the place where the rope is to be cut to prevent unlaying of the strands. On preformed rope, one seizing on each side of the cut is required. On nonpreformed ropes of 7/8 inch diameter or smaller, two seizings on each side of the cut are required, and for non-preformed rope of one inch diameter or larger, three seizings on each side of the cut are required.

18.3.4 During installation care shall be observed to avoid dragging of the rope in dirt or around objects which will scrape, nick, crush, or induce sharp bends in it.

As noted in the section on ASME B30.26 Rigging Hardware, presented below, the method by which the wire rope was rigged to the collars would necessarily induce nicking, crushing and/or sharp bends. Specifically, the wire rope installation involved threading the rope through the lugs provided on the collar, thus forming a sling at each lug. The lug eyeholes were fabricated without rounding the hole edges. No thimble insert was provided by which to maintain the sling shape and to protect the wire rope form bearing on the unprotected/unpadded edges of the eyeholes. It is unknown how the wire rope was attached to the tower.

18.3.5 Rope should be maintained in a well lubricated condition. It is important that lubricant applied as part of a maintenance program shall be compatible with the original lubricant and to this end the rope manufacturer should be consulted. Those sections of rope which are located over sheaves or otherwise hidden during inspection and maintenance procedures require special attention when lubricating rope. The object of rope lubrication is to reduce internal friction and to prevent corrosion. Periodic field lubrication is particularly important for non-rotating rope.

…

3.1.3 American Society of Mechanical Engineers B30.26-2004 Rigging Hardware – Text (quoted material in italics, this section)

SECTION 26-0.1: SCOPE

Chapter 26-0 Scope, Definitions, and References

Volume B30.26 includes provisions that apply to the construction, installation, operation, inspection, and maintenance of detachable rigging hardware used for lifting purposes in conjunction with equipment described in other volumes of the B30 Standard. This hardware includes shackles, links, rings, swivels, turnbuckles, eyebolts, hoist rings, wire rope clips, wedge sockets, and rigging blocks. Use of the same hardware for purposes other than lifting is excluded from the provisions of this Volume.

As part of the collar installation, chain blocks are specified to be installed, by which the collar is lifted into final position. The rigging which was used was subsequently detached from the collars and replacement rigging installed. Consequently, the provisions of this standard apply to the rigging hardware used for the installation of the collars. This is with specific reference to the chain/winch/polyester web slings which were used during the installation of the collars.

SECTION 26-0.2: DEFINITIONS

<selected>

abnormal operating conditions: environmental conditions that are unfavorable, harmful, or detrimental to or for the operation of a piece of detachable hardware, such as excessively high or low ambient temperatures; exposure to weather; corrosive fumes; dust laden or moisture laden atmospheres; and hazardous locations.

angle of loading: the acute angle between horizontal and the leg of the rigging, often referred to as the horizontal angle…

design factor: ratio between nominal or minimum breaking strength and rated load of the rigging hardware.

designated person: a person who is selected or assigned by the employer or representative as being competent to perform specific duties...

hitch, choker: a method of rigging a sling in which the sling is passed around the load, then through one loop eye, end fitting, or other device with the other loop-eye or end fitting attached to the lifting device.

in-line loading: condition where the load is applied through the centerline of the rigging hardware at the intended bearing points.

jaw: a U-shaped load bearing connection, designed for use with a removable pin

line pull: the tension load in a rope entering a rigging block

live end: the section of wire rope that is tensioned under load…

proof load: the specific load applied in performance of the proof tests.

proof test: a nondestructive load test made to a specific multiple of the rated load of the rigging hardware.

qualified person: a person who, by possession of a recognized degree in an applicable field or certificate of professional standing, or who, by extensive knowledge, training, and experience, has successfully demonstrated the ability to solve or resolve problems relating to the subject matter and work…

rated load: the maximum allowable working load established by the rigging hardware manufacturer. The terms "rated capacity" and "working load limit" are commonly used to describe rated load.

saddle: the base of a wire rope clip

shackle: a U-shaped load-bearing connector designed to be used with a removable pin.

shock load: any condition which causes a momentary increase in the forces in a load-supporting component beyond the weight of the actual load being lifted.

sling: an assembly used for lifting when connected to a lifting mechanism. The upper portion is connected to the lifting mechanism and the lower supports the load, as described in the chapters of this Volume…

wire rope clip: a fitting for clamping two parts of wire rope of the same diameter to each other by compressing the wire ropes between a saddle and a U-bolt or between two saddles.

u-bolt type: wire rope clip using one saddle and a U-bolt.

double saddle type: wire rope clip using two saddles.

With regard to the tower, at the time of loading, no known abnormal operating conditions were present. Weather records indicate no unusual or extreme events as per Figure 3.1.

LCD Daily Form Page 1 of 2

QUALITY CONTROLLED LOCAL CLIMATOLOGICAL DATA
(may be updated)
NOAA, National Climatic Data Center
Month: 03/2008

Station Location: CENTRAL PARK (94728)
NEW YORK , NY
Lat. 40.783 Lon. -73.967
Elevation(Ground): 156 ft. above sea level

Date	Temperature (Fahrenheit) Max.	Min.	Avg.	Dep From Normal	Avg. Dew pt.	Avg Wet Bulb	Degree Days Base 65 Degrees Heating	Cooling	Sun Sunrise LST	Sunset LST	Significant Weather	Snow/Ice on Ground(In) 1200 UTC Depth	1800 UTC Water Equiv	Precipitation (In) 2400 LST Snow Fall	2400 LST Water Equiv	Pressure (inches of Hg) Avg. Station	Avg. Sea Level	Wind: Speed=mph Dir=tens of degrees Resultant Speed	Res Dir	Avg. Speed	max 5-second Speed	Dir	max 2-minute Speed	Dir	Date
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
01	46	33	40	2	26	34	25	0	0630	1747	RA BR UP	0	M	T	0.14	29.82	29.95	5.8	25	9.2	32	270	23	270	01
02	43	28	36	-3	13	28	29	0	0629	1748		0	M	0.0	0.00	30.07	30.22	8.3	29	9.1	30	310	21	310	02
03	55	36	46	7	28	39	19	0	0627	1749		0	M	0.0	0.00	30.01	30.16	3.4	16	6.1	23	230	16	200	03
04	63*	45	54*	15	46	49	11	0	0626	1750	RA BR	0	M	0.0	0.19	29.75	29.89	1.7	18	6.8	25	210	16	220	04
05	57	37	47	8	37	43	18	0	0624	1751	RA BR	0	M	0.0	0.30	29.47	29.69	6.2	23	10.4	35	160	22	170	05
06	49	34	42	3	27	35	23	0	0622	1752		0	M	0.0	0.00	30.03	30.18	0.8	14	5.5	15	150	12	160	06
07	45	36	41	1	36	39	24	0	0621	1753	RA BR	0	M	0.0	1.21	30.00	30.08	7.1	06	7.8	21	070	16	060	07
08	57	36	47	7	42	45	18	0	0619	1755	RA FG+ FG BR	0	M	0.0	0.75	29.38	29.53	2.3	20	8.1	48	240	32	250	08
09	42	30	36	-4	12	28	29	0	0618	1756		0	M	0.0	0.00	29.83	30.05	11.3	27	12.1	46	260	26	260	09
10	45	27*	36*	-4	15	29	29	0	0616	1757		0	M	0.0	0.00	30.17	30.31	4.6	28	7.5	26	290	17	290	10
11	49	35	42	1	25	34	23	0	0615	1758		0	M	0.0	0.00	29.95	30.05	0.5	14	5.4	21	170	14	190	11
12	47	35	41	0	26	36	24	0	0613	1759		0	M	0.0	0.00	29.59	29.74	6.9	28	9.1	35	270	23	280	12
13	45	31	38	-3	24	33	27	0	0611	1800		0	M	0.0	0.00	29.81	29.95	3.1	16	5.4	21	150	15	160	13
14	56	40	48	6	35	41	17	0	0610	1801	RA HZ	0	M	0.0	0.02	29.65	29.78	4.6	06	6.2	30	070	22	060	14
15	56	41	49	7	36	42	16	0	0608	1802	RA BR	0	M	0.0	0.11	29.54	29.71	4.7	04	8.4	26	060	21	060	15
16	47	35	41	-1	30	37	24	0	0606	1803	RA	0	M	0.0	0.04	29.80	29.99	3.8	34	7.7	31	300	18	290	16
17	50	30	40	-3	11	30	25	0	0605	1804		0	M	0.0	0.00	30.26	30.43	5.1	35	8.4	28	270	18	290	17
18	44	35	40	-3	20	33	25	0	0603	1805		0	M	0.0	0.00	30.26	30.38	3.4	13	7.6	21	140	15	160	18
19	53	41	47	4	42	44	18	0	0601	1807	RA BR	0	M	0.0	0.95	29.71	29.76	3.0	13	6.1	18	160	14	160	19
20	58	36	47	3	29	39	18	0	0560	1808	RA BR	0	M	0.0	0.22	29.41	29.60	11.2	29	12.4	41	290	26	300	20
21	45	33	39	-5	11	30	26	0	0558	1809		0	M	0.0	0.00	29.83	29.99	11.1	28	12.6	35	300	23	300	21
22	50	34	42	-2	4	30	23	0	0556	1810		0	M	0.0	0.00	29.79	29.93	4.7	29	7.2	20	290	14	300	22
23	48	30	39	-6	8	29	26	0	0555	1811		0	M	0.0	0.00	29.96	30.13	3.1	30	5.9	18	280	14	290	23
24	50	33	42	-3	14	32	23	0	0553	1812		0	M	0.0	0.00	30.06	30.22	3.6	03	6.4	21	020	14	040	24
25	48	31	40	-5	16	31	25	0	0551	1813		0	M	0.0	0.00	30.09	30.23	3.1	15	8.1	25	170	18	160	25
26	61	43	52	6	27	41	13	0	0550	1814		0	M	0.0	0.00	29.86	30.00	3.7	24	7.0	29	210	18	270	26
27	49	44	47	1	29	39	18	0	0548	1815	RA	0	M	0.0	T	29.91	30.04	0.0	23	2.4	14	080	10	110	27
28	48	38	43	-3	35	40	22	0	0547	1816	RA	0	M	0.0	0.01	29.72	29.89	3.6	02	7.9	23	290	16	290	28
29	47	33	40	-6	12	30	25	0	0545	1817		0	M	0.0	0.00	30.16	30.36	6.5	31	9.1	25	270	17	290	29
30	49	28	39	-8	11	29	26	0	0543	1818		0	M	0.0	0.00	30.51	30.66	2.6	07	6.3	18	060	14	050	30
31	57	37	47	0	40	44	18	0	0542	1819	RA BR	0	M	0.0	0.14	30.32	30.42	5.1	15	7.1	23	180	15	210	31
	50.3	35.0	42.7		24.7	35.9	22.2	0.0	<-----Monthly Averages | Totals----->				M	M	4.08	29.90	30.04	1.7	29	7.7	<Monthly Average				
	0.5	-0.1	0.2	<-----------Departure From Normal----------->									-0.29												

Degree Days Monthly Season to Date
Total Departure Total Departure
Heating: 687 -8 3997 -242
Cooling: 0 -2 0 -2

Greatest 24-hr Precipitation: 1.93s Date: 07-08
Greatest 24-hr Snowfall: M Date: M
Greatest Snow Depth: M Date: M

Sea Level Pressure Date Time (LST)
Maximum 30.71 30 0919
Minimum 29.19 08 1831

Number of Days with -------->
Max Temp >=90: 0
Max Temp <=32: 0
Thunderstorms : 0
Min Temp <=32: 8
Min Temp <=0 : 0
Heavy Fog : 1
Precipitation >=.01 inch: 12
Precipitation >=.10 inch:
Snowfall >=1.0 inch : 0

* EXTREME FOR THE MONTH - LAST OCCURRENCE IF MORE THAN ONE.

Data Version: VER2

http://cdo.ncdc.noaa.gov/qclcd/QCLCD 4/23/2008

Figure 3.1 - Weather records for Central Park, New York, NY for March 2008. National Climactic Data Center.

- At the time of the collapse, there did not appear to be any specifically named "designated person" with regard to the rigging, collar installation or tie-beam installation, which position was assumed by William Rapetti by virtue of his Tower Crane Rigger license. It is noted that although he is listed as "Master Rigger" by JCI, information obtained from the NYCDOB website shows him licensed as a Tower Crane Rigger (see Appendix A). As the tower crane rigger, Mr. Rapetti was the "qualified person" who, by extensive knowledge, training, and experience, had successfully demonstrated the ability to solve or resolve problems relating to the subject matter and work as indicated by his license.
- Also, as indicated in Section 4.1.2 of this report, the "Master Rigger" license requires "at least 5 years' practical experience in the hoisting and rigging business" and, specifically, "knowledge of and be able to explain the risks incident to such business and precautions to be taken in connection therewith, safe loads and computation thereof, types of rigging, size and strength of ropes, cables, blocks, poles, derricks, sheerlegs and other tools used in connection with such business." This is not an explicit requirement for the Tower Crane Rigger, who is required to "have erected or dismantled, as part of a team, eight or more tower and/or climber cranes of which at least three erections and dismantlings of such cranes shall be under his supervision, and/or oversee the safety and code requirements for the same."

During the collar installation Mr. Rapetti was present on the adjacent floor slab of the building under construction; however, in this position he was unable to view all operations underway as he indicated in his statement to the NYDOI investigators. During the work Mr. Rapetti indicated that he contacted Peter Stroh, the engineer of record regarding certain measurements; however, Mr. Stroh was not at the site during these operations.

Polyester web slings were attached to the crane tower via a choker hitch. An extract from a photograph taken shortly before the collapse, shown in Photograph 3.1 below, clearly shows the choker hitch around the legs of the tower for two of the slings.

Photograph 3.1 - Close-up of crane tower shortly before collapse. Two polyester web slings are seen in a choker hitch around the legs of the tower. (Source: photograph by Gary Halby.

Chapter 26-3 Compression Hardware - Selection, Use, and Maintenance

SECTION 26-3.0: SCOPE

This Chapter applies to compression hardware including forged wire rope clips and wedge sockets.

SECTION 26-3.1: TYPES, MATERIALS, AND ASSEMBLY

26-3.1.1 Types

(a) Wire rope clip types covered are U-bolt and double saddle.

26-3.1.2 Materials

(a) Wire rope clip materials shall be of sufficient strength such that failure of the wire rope will occur before failure of the wire rope clip at the temperatures that the manufacturer has specified for use. Saddles shall be forged steel…

Installation and Loading

GENERAL NOTE: Correct number of clips for wire rope size shall be used.

NOTES:

(1) correct turnback length should be used

(2) correct orientation of saddle on live end shall be observed

(3) correct spacing of clips should be used

(4) correct torque on nuts shall be applied

26-3.1.4 Assembly - Wire Rope Clips

(a) Before installing a wire rope clip on plastic coated or plastic impregnated wire rope, consult the wire rope clip manufacturer, wire rope manufacturer, or a qualified person.

(b) For U-bolt clips used to create end terminations, the saddle shall be placed on the live end of the wire rope, with the U-bolt on the dead end side.

(c) At least the minimum number of clips as recommended by the manufacturer or a qualified person shall be used.

(d) The spacing and turn-back should be as recommended by the manufacturer or a qualified person.

(e) The wire rope clip shall be tightened to the torque recommended by the manufacturer or a qualified person.

(f) After assembly, the connection shall be loaded to at least the expected working load. After unloading, wire rope clips shall then be re-tightened to the torque recommended by the manufacturer or a qualified person…

Wire ropes were used to provide vertical support to the collars after completion of the tie-beam assembly. These were attached to the collars via the chain block points which were designated by Favelle Favco for the chain blocks. These locations are shown circled in red in Figure 3.2; however, based upon the number of wire ropes retrieved and photographs showing wire rope still dangling from the collars following the collapse, it appears that not all points were used. Actual selected points and criteria regarding their selection could not be established.

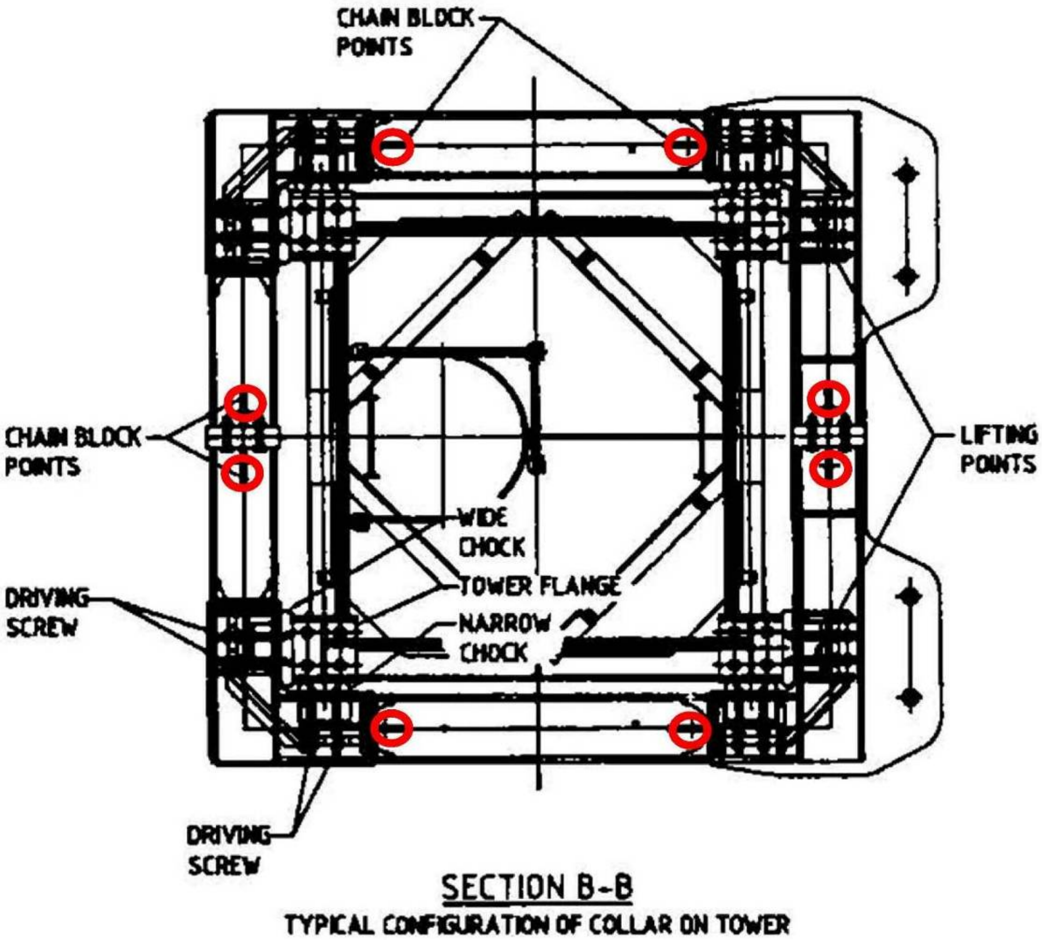


Figure 3.2 - Chain block lifting points; locations are shown circled in red. Taken from Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123.

Wire rope U-bolt clip saddles were placed both on the live and dead ends of the wire ropes. An example can be seen in Photograph 3.2 taken of one of the wire ropes retained by OSHA.

Photograph 3.2 - OSHA retained evidence from NYPD GB9. Note the two clips only, with the saddle on the dead end of the wire rope. Photo by Arup.

> *SECTION 26-3.2: DESIGN FACTOR*
>
> *Due to the nature of the design and use, wire rope clips and wedge sockets do not have a conventional design factor. Wire rope clips and wedge sockets shall be designed to have an 80% minimum connection efficiency based on the wire rope published minimum breaking force with which they are used.*

Design criteria for the selection of the attachment points could not be established. There are stated load ratings, including a 2 ton safe working load (SWL), given on the External Climbing Collar/Tie Erection Sequence drawing A1-1100.123, for the lifting slings (Stage 1, clause 1.5), and a 2 ton capacity for chain blocks (Stage 2, clause 2.2) on the same drawing. The latter value is not stated as a safe, rated, nominal or maximum capacity. There is no similar statement given regarding the use of wire rope, nor are wire ropes explicitly mentioned. See Figures 3.3, 3.4 and 3.5. Note also there is a reference to “soft sling” when pulling the half collar into position for the chain blocks (Stage 2, clause 1.6).

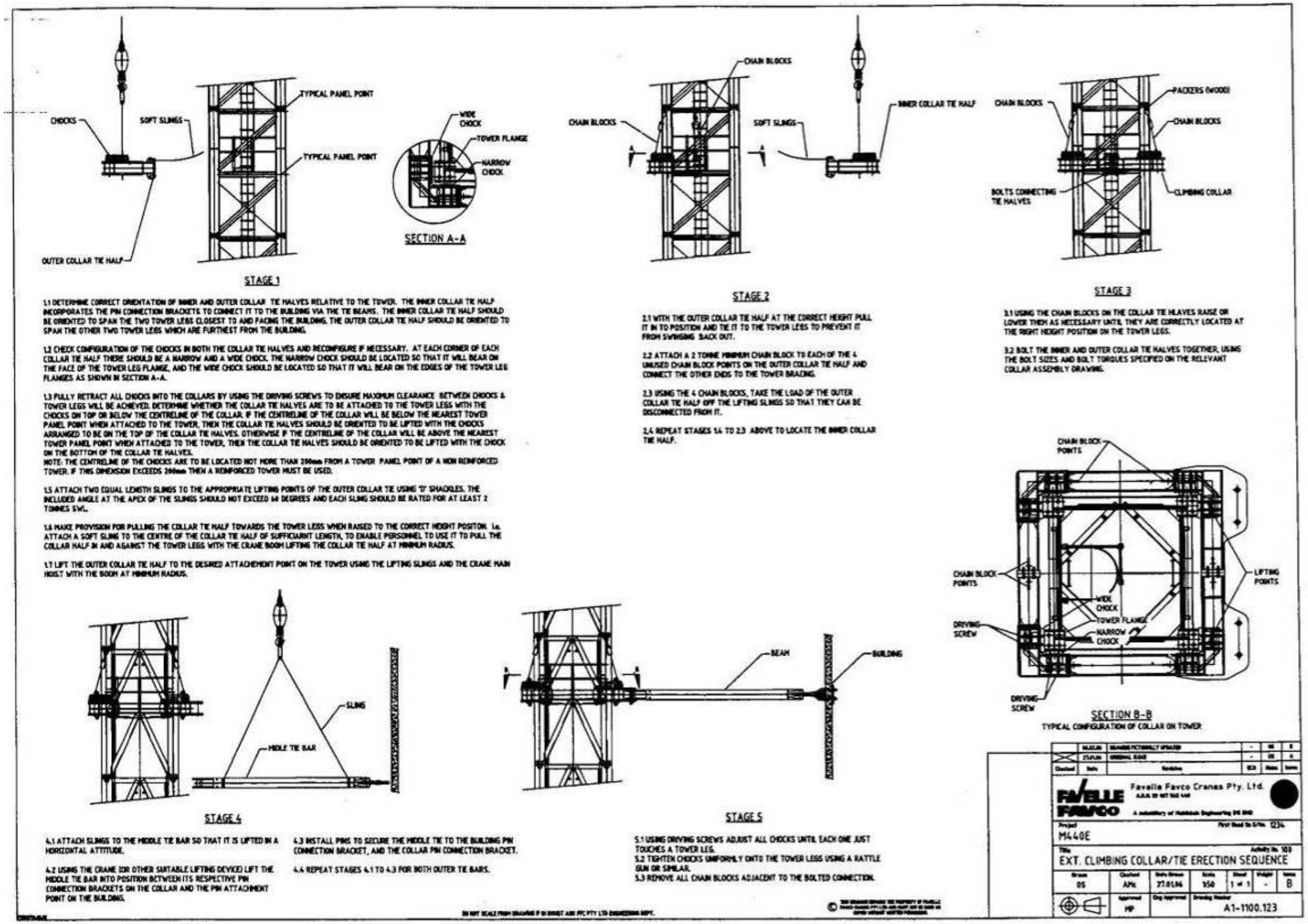


Figure 3.3 - Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123.

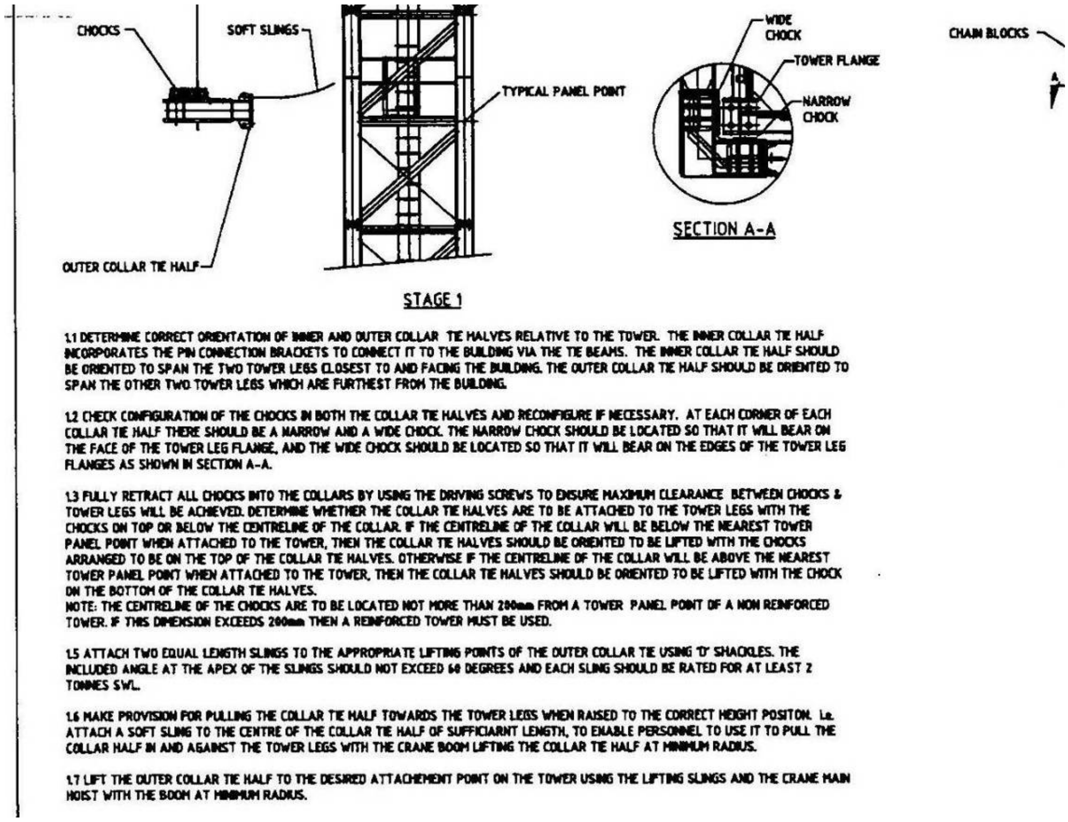


Figure 3.4 - Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 Stage 1 details. Note clauses 1.5 and 1.6 references to 2 ton SWL for each lifting sling and a soft sling for pulling the half collar manually to the tower legs.

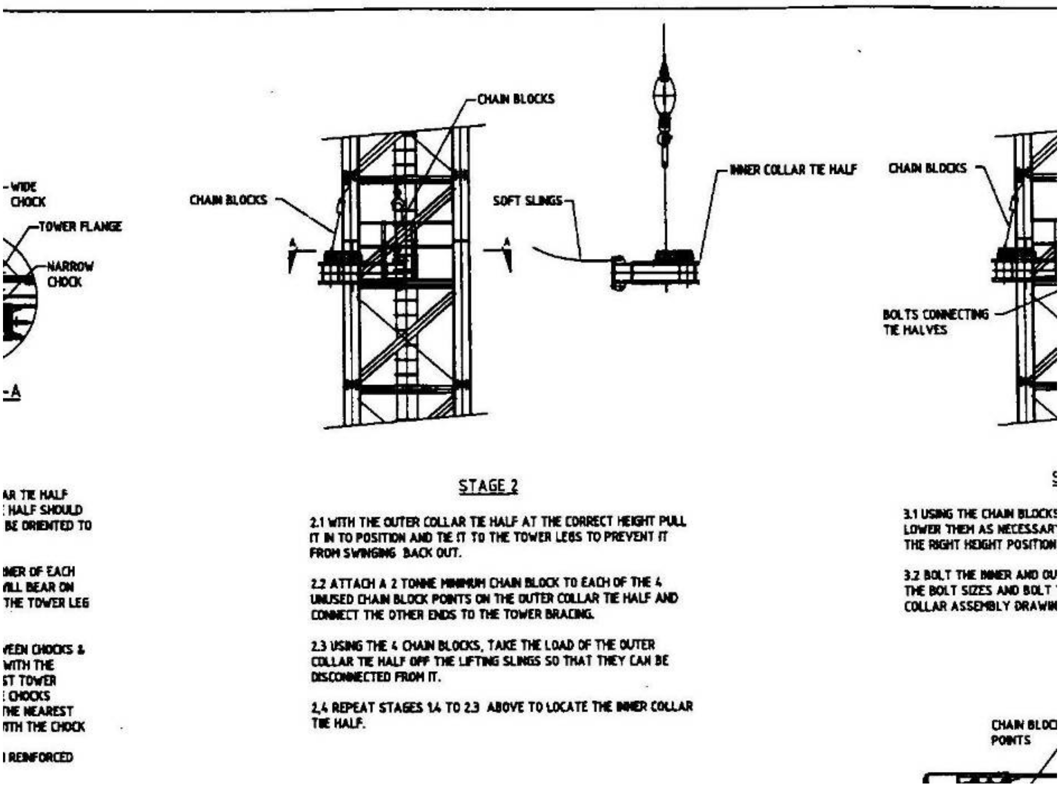


Figure 3.5 - Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 Stage 2 details. Note clause 2.2 reference to a "2 ton minimum chain block." There is no indication that this is a SWL, rated capacity or other.

SECTION 26-3.3: RATED LOADS

The rated load for wire rope assemblies using compression hardware is based on the following factors:

(a) wire rope minimum breaking force

(b) 80% minimum connection efficiency

(c) design factor of the wire rope application

As already noted, there is no specified load given for the use of wire ropes.

SECTION 26-3.4: PROOF TEST

26-3.4.1 Proof Test Requirements

(a) Compression hardware is not required to be proof tested unless specified by the purchaser.

(b) If required, the proof test shall be applied to the wedge socket or the connection made by the wire rope clips after the assembly is complete.

(c) After proof testing, wire rope clips on a finished assembly shall be re-tightened to the torque recommended by the wire rope clip manufacturer or a qualified person.

(d) If proof tested, compression hardware shall be inspected after the test for the conditions stated in para. 26-3.8.4.

There was no requirement to proof test the wire rope clip connections reviewed.

26-3.4.2 Proof Load Requirements

The proof load shall be a minimum of 40%, but not exceed 50%, of the wire rope minimum breaking force unless approved by the compression hardware manufacturer or a qualified person.

As already stated, there was no requirement to proof test the wire rope clip connections reviewed.

SECTION 26-3.5: IDENTIFICATION

26-3.5.1 Wire Rope Clip Saddle Identification

Each new wire rope clip saddle shall have forged or die stamped markings by the manufacturer to show

(a) name or trademark of manufacturer

(b) size…

26-3.5.3 Maintenance of Identification

Compression hardware identification should be maintained by the user so as to be legible throughout the life of the hardware…

These markings were not observed on the items viewed; however, it is noted that failure of the wire rope clips themselves were not observed.

SECTION 26-3.7: TRAINING

Compression hardware users shall be trained in the selection, inspection, cautions to personnel, effects of environment, and rigging practices as covered by this Chapter.

There was no documentation reviewed regarding either personnel responsible for use of the wire ropes or any requisite training or qualifications for the use of the wire ropes.

SECTION 26-3.8: INSPECTION, REPAIR, AND REMOVAL

26-3.8.1 Initial Inspection

Prior to use, all new, altered, modified, or repaired compression hardware shall be inspected by a designated person to verify compliance with the applicable provisions of this Chapter. Written records are not required.

No designated person has been identified.

26-3.8.2 Frequent Inspection

(a) A visual inspection shall be performed by the user or other designated person each day before the compression hardware is used. Semi-permanent and inaccessible locations where frequent inspections are not feasible shall have periodic inspections performed….

(c) Written records are not required.

There was no documentation reviewed, including site diaries and field notes, suggesting that any inspection of the compression hardware was undertaken.

26-3.8.3 Periodic Inspection

(a) A complete inspection of the compression hardware shall be performed by a designated person. The compression hardware shall be examined for conditions such as those listed in para. 26-3.8.4 and a determination made as to whether they constitute a hazard.

(b) Periodic lnspecfion Frequency. Periodic inspection intervals shall not exceed one year. The frequency of periodic inspections should be based on

(1) frequency of compression hardware use

(2) severity of service conditions

(3) nature of lifts being made

(4) experience gained on the service life of compression hardware used in similar circumstances

(5) Guidelines for the time intervals are

(a) normal service - yearly

(b) severe service - monthly to quarterly

(c) special service - as recommended by a qualified person

(c) Written records are not required.

As already stated, no designated person has been identified. There was no documentation reviewed, including site diaries and field notes, suggesting that any inspection of the compression hardware was undertaken.

26-3.8.4 Removal Criteria

Compression hardware shall be removed from service if conditions such as the following are visible and shall only be returned to service when approved by a qualified person:

(a) missing or illegible identification

(b) indications of heat damage including weld spatter or arc strikes

(c) excessive pitting or corrosion

(d) bent, twisted, distorted, stretched, elongated, cracked, or broken components

(e) excessive nicks or gouges

(f) a 10% reduction of the original or catalog dimension at any point

(g) evidence of unauthorized welding

(h) unauthorized replacement components

(i) insufficient number of wire rope clips

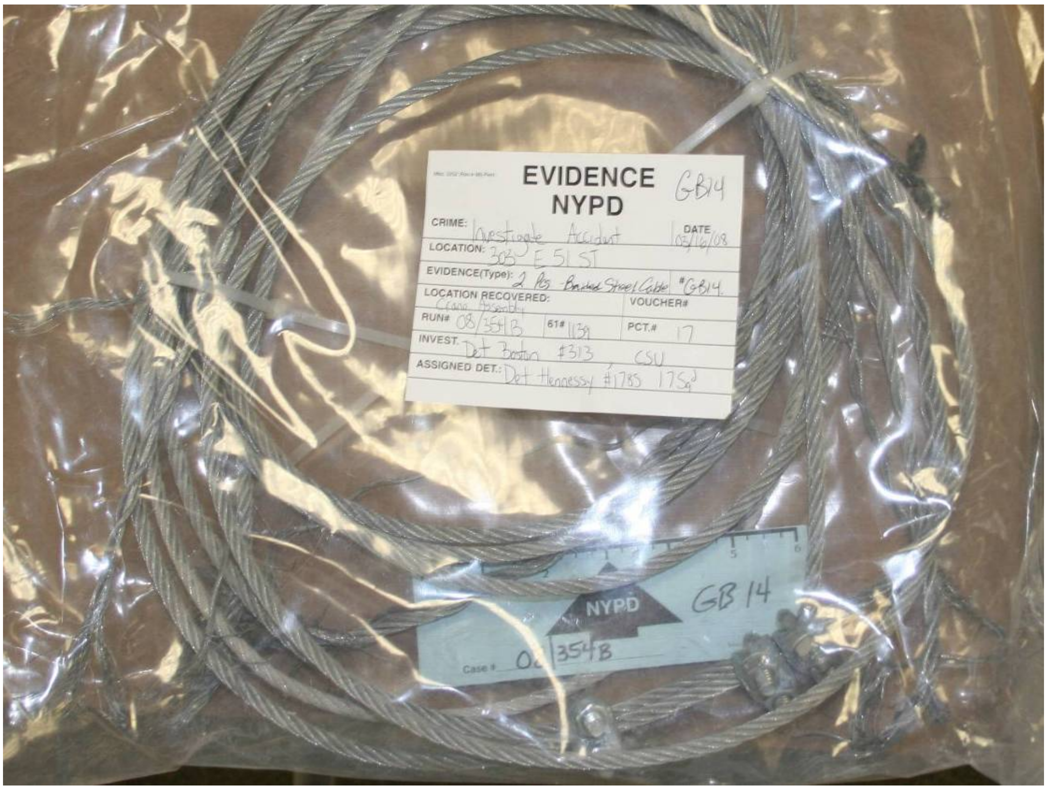
(j) improperly tightened wire rope clips

(k) indications of damaged wire rope

(l) indications of wire rope slippage

(m) improper assembly or other conditions, including visible damage, that cause doubt as to continued use…

Notwithstanding the lack of documentation and photographic records for the installation of the wire ropes, observations of the removed artefacts indicate that wire rope clips were improperly located and applied. See Photograph 3.3 below – clearly showing improper spacing of the clips.



Photograph 3.3 - OSHA evidence GB14 from NYPD. Note spacing of the U-bolt clips in the lower right of the photograph. Photo by Arup.

26-3.9.4 Rigging Practices

26-3.9.4.1 Wire Rope Clips

(a) Assemble wire rope clips in accordance with para. 26-3.1.4.

(b) Wire rope clips should not be in contact with the load or any obstruction during the lift.

(c) Shock loading should be avoided.

(d) Rigging using wire rope clips should not be dragged on an abrasive surface.

(e) When wire rope clips are applied to join two lengths of wire rope in an in-line splice, the requirements of para. 26-3.1.4 shall be followed (see Fig. 10).

(f) The use of wire rope clips to fabricate slings is generally prohibited. See ASME B30.9 for specific exceptions…

The wire ropes, as evidenced in the post-collapse photographs, were used as slings on the collars, in violation of the requirements contained in this clause. See the discussion regarding ASME B30.9, Section 3.1.5 of this Chapter.

3.1.4 American Society of Mechanical Engineers B30.3-2004 Construction Tower Cranes (quoted material in italics, this section)

From “General”

…Some of the provisions of this Standard require compliance with information found in manuals or other documents supplied by the manufacturer with the equipment. This information includes recommendations, requirements, and instructions (e.g., "the reeving shall be checked for compliance with the recommendations of the manufacturer").

Compliance with the provisions should not preclude the possibility of consulting a qualified person. This is true particularly when: the equipment has been altered, repaired, or modified; the manuals or documents supplied by the manufacturer are no longer available; or the manufacturer or a successor is no longer in business and the manuals are no longer available. However, the purpose of consulting a qualified person shall not be to avoid contacting the manufacturer and using the information supplied by the manufacturer…”

In this instance it appears that Peter Stroh assumed the role of “qualified person,” as Mr. Stroh’s designs modified and augmented the information supplied by Favelle Favco. See Chapter 5.

SECTION I: SCOPE

This Standard applies to the construction, installation, operation, inspection, and maintenance of jacks; power operated cranes, monorails, and crane runways; power operated and manually operated derricks and hoists; lifting devices, hooks, and slings; and cableways…

As the crane collapse involved a construction crane, this standard is applicable.

SECTION II: PURPOSE

This Standard is designed to:

(a) guard against and minimize injury to workers, and otherwise provide for the protection of life, limb, and property by prescribing safety requirements;

(b) provide direction to owners, employers, supervisors, and others concerned with, or responsible for, its application; and

(c) guide governments and other regulatory bodies in the development, promulgation, and enforcement of appropriate safety directives…

CONSTRUCTION TOWER CRANES

Chapter 3-0 Scope, Definitions, and References

SECTION 3-0.1: SCOPE OF B30.3

Within the general scope as defined in Section I, B30.3 applies to construction tower cranes, powered by electric motors or internal combustion engines, and any variations thereof which retain the same fundamental characteristics. The scope includes cranes of the above type that adjust operating radius by means of a boom luffing mechanism, or by means of a trolley traversing a horizontal boom, or by means of a combination of the two. Construction tower cranes may be mounted on fixed or traveling bases. Additional mounting means may include *arrangements that permit the crane to climb in the structure being built, or that permit increasing the tower height as the structure rises and utilizing braces attached to the host structure as needed…*

The M440E tower crane used on this project was an external climbing tower crane with luffing boom. The provisions of this Section would therefore apply directly to this crane.

SECTION 3-0.2: DEFINITIONS

3-0.2.1 Types of Cranes

(a) By Type of Application

construction tower crane: a hammerhead, luffing, or other type of tower crane that is regularly assembled and disassembled for use at various sites. It is usually characterized by provisions to facilitate erection and dismantling and may include features to permit climbing or telescoping…

(b) By Type of Boom

…luffing tower crane: a crane with a boom pinned to the superstructure at its inner end and containing load hoisting tackle at its outer end, and with a hoist mechanism to raise or lower the boom in a vertical plane to change load radius.

(c) By Support Arrangement

braced or guyed tower crane: a tower crane with braces or guys attached to the tower (mast) to permit the crane to be erected to greater than the maximum free-standing height…

(d) By Ability to Travel

fixed-base tower crane: a free-standing, braced, or guyed tower crane that is mounted on a foundation or structural frame and does not travel…

The M440E is a fixed-base self-erecting braced external climbing luffing tower crane. As defined in this section, the method by which the tower "does not travel" is not specified.

3-0.2.2 General (04)

accessory: a secondary part or assembly of parts that contributes to the overall function and usefulness of a machine…

The collar and tie-beams would therefore be defined as accessories.

base, anchor bolt: a crane base that is bolted to a footing

The tower base was not bolted to a footing.

base, expendable: for static-mounted cranes, a style of bottom mast section or member that is cast into a concrete footing block; all or part of this component is lost to future installations

The tower base was not expendable. Both the dunnage beams and tower base were retained. The concrete footing, however, is lost to future installations as this was cast permanently into the ground.

base, knee brace: a crane base that uses diagonal members to spread the loading

There were no knee braces used to spread the loading.

> *base, tower crane: a mounting accessory to secure the bottom of the tower (mast) to a foundation, structural frame, or travel base.*

The dunnage beams would constitute the tower crane base for the 51st St. tower crane. The method of securing was direct bearing of the tower legs to the sides of the dunnage beam. The dunnage beam resisted lateral loads through frictional forces transmitted through plywood "bearing pads" through to concrete walls cast into the ground.

A view of the foundation base and dunnage beam is given in Photograph 3.4 below.

Photograph 3.4 - Base of tower crane after collapse. Note the lack of knee bracing at the base, the dunnage beam on which the tower stands and the slippage of the dunnage beam off of the submerged concrete wall foundation. Photo from New York County District Attorney IMG_332_062.jpg.

brace, tower: a structural attachment placed between a crane tower and an adjacent structure to pass loads to the adjacent structure and permit the crane to be erected to greater than free-standing height..

The collar/tie-in beam assembly constituted the tower brace.

climbing: for free-standing, braced, or guyed cranes, the process whereby the height of the tower (mast) is increased by adding sections at the top; for internal climbing cranes, the process whereby the entire crane is raised on or within a structure which is under construction as the height of that structure increases

climbing frame: for free-standing, braced, or guyed cranes, a structural frame supporting the superstructure which surrounds the tower (mast) and contains arrangements to raise the frame and superstructure of the crane for insertion of an additional tower section; for internal climbing cranes, a frame used to transmit operational and climbing reactions to the host building frame…

counter jib (counterweight jib): a horizontal member of a crane on which the counterweights and usually the hoisting machinery are mounted.

counterweight: weights added to a crane superstructure to create additional stability or to counter the effects of the lifted load; they rotate with the crane as it swings.

crane: in this volume, the use of the word crane refers to construction tower cranes, which are lifting machines consisting of a tower (mast) with a superstructure that rotates and includes a load boom (jib) and, on some cranes, a counter jib extending in the opposite direction to the load boom (jib)…

designated person: a person selected or assigned by the employer or the employer's representative as being competent to perform specific duties.

free-standing height: that height of a crane which is supported by the tower (mast) alone without assistance from braces, guys, or other means…

qualified person: a person who, by possession of a recognized degree in an applicable field or a certificate of professional standing, or by extensive knowledge, training, and experience, has successfully demonstrated the ability to solve or resolve problems relating to the subject matter and work…

Peter Stroh, as a licensed engineer with specific experience in crane design and William Rapetti, a licensed tower crane rigger with the City of New York, would both be presumed to meet the requirements of a qualified person within the limits of their training and experience.

rope: refers to wire rope unless otherwise specified…

tower (mast): a vertical structural frame consisting of columns and bracing capable of supporting a superstructure with its working and dynamic loads and transmitting them to the supporting surface or structure…

Chapter 3-1 Erection and Dismantling, Characteristics, and Construction

SECTION 3-1.1: SITE PREPARATION AND ERECTION

3-1.1.1 Crane Supports

(a) All load bearing foundations, supports, and rail tracks shall be constructed or installed to support the crane loads and to transmit them to the soil or other support medium. In addition to supporting vertical load, foundations and supports,

rail supports excepted, should be designed to provide a moment resisting overturning equal to a minimum of 150% of the maximum crane overturning moment. This requirement may be met by means of structural anchors or ballast…

It is noted elsewhere that, as this standard is not referenced by the New York City Building Code, that this provision is not required to be met by the tower foundation supplied. Neither ballast nor anchors were included in the foundation design.

(f) The crane manufacturer or a qualified person shall provide maximum resulting loads at the base of the crane, or wheel loads, for use in design of the supports [see para. 3-1.4.1(a)(1)].

These were supplied by Peter Stroh, PE, who was also the designer for the tower base.

3-1.1.2 General Erection and Dismantling

Requirements

(a) When cranes are erected/dismantled, written instructions by the manufacturer or qualified person and a list of the weights of each sub-assembly to be erected/dismantled shall be at the site.

These were provided in the Favelle Favco tower crane manual kept in the cabin of the crane.

(b) Erection and dismantling shall be performed under the supervision of a qualified person.

With specific regard to the collars and tie-beam installations, the qualified person was William Rapetti.

(c) Procedures shall be established before erection/dismantling work commences to implement the erection/dismantling instructions and to adapt them to the particular needs of the site. The need for temporary or permanent guying or bracing should be considered…

It is unknown if the erection instructions for the collar and tie-beam assembly were available on site. Any actions undertaken for their implementation are likewise unknown at this time. Documentation indicating the use of temporary or permanent guying or bracing beyond that indicated in the Favelle Favco drawing referenced above has not been reviewed.

3-1.1.4 Guyed or Braced Cranes

(a) A bracing schedule should be prepared in advance of the installation. The schedule should indicate each level at which the crane is to be braced, the maximum tower height to be erected above each brace, and the maximum height to which construction can be carried before the crane must be increased in height.

This was undertaken by Peter Stroh. See the section on permitting.

(b) The vertical spacing between the braces and the free standing height of the crane above the topmost brace shall be in accordance with the manufacturer's or a qualified person's recommendations; wind conditions of the geographic area per ANSI/ASCE 7 should be considered.

This is discussed in Chapter 5. While the maximum unsupported height was temporarily in excess of that specified by the manufacturer, this was done under the aegis of Peter Stroh, in accordance with the terms of this clause.

> *(c) Braces should be designed and anchored to the bracing structure to resist the forces given by the manufacturer or by a qualified person when applied in any horizontal direction…*

This was undertaken by Peter Stroh and calculations submitted. See the section on permitting. This is also discussed in Chapter 5 and the designs found to be in general accordance with accepted practice.

> *3-1.1.5 Climbing Cranes*
>
> *(a) A climbing- schedule should be prepared in advance of the installation. The schedule should indicate each level at which the crane will be mounted for operation giving the locations of supports for both horizontal and vertical crane reactions and the maximum height to which construction can be carried before the crane must climb again.*
>
> *(b) The means of transferring horizontal and vertical crane reactions to the host structure shall be reviewed by a qualified person. Wind effects for the height of crane mounting should be considered.*
>
> *…*
>
> *(d) When climbing cranes are used, the integrity of the host structure shall be reviewed by a qualified person, for the effects of crane, load, and wind forces at each level of the structure…*

These were undertaken by Peter Stroh and calculations submitted. See the section on permitting. This is also discussed in Chapter 5 and the designs found to be in general accordance with accepted practice.

> *(h) Written climbing instructions should be kept at the site and all climbing operations shall be performed under the supervision of a qualified person…*

Although the Favelle Favco tower crane manual containing jumping and other erection procedures was on site, it is unknown if the erection instructions for the collar and tie-beam assembly were likewise available.

> *3-1.1.6 Preoperation Tests*
>
> *(a) When cranes are erected, and after each climbing operation, before placing the crane in service, all functional motions, motion limiting devices, and brakes shall be tested for operation.*

See items contained in the permitting section of this report, Chapter 4.

> *(b) When cranes are erected, load-limiting devices shall be tested for proper setting and operation before the crane is placed in service.*
>
> *(c) After erection, the structural support or foundation to which the crane base is attached shall be tested before placing the crane in service. The test shall be conducted with the rated load placed at maximum radius permitted by site conditions...*

Documentation indicating that any tests on the foundation were carried out to “prove” design have not been reviewed beyond any indicated in the permitting section of this report.

…

(f) The order in which tests of a newly erected crane are to be performed is as follows:

(1) functional motion tests without load;

(2) functional motion tests at rated load (for other than traveling cranes, this may be combined with tests of supports);

(3) tests of supports per clause (c).

During functional motion tests, the crane supports shall be checked. Any observed displacement is reason to refrain from continuing testing until an evaluation is made by a qualified person.

Documentation reflecting completion of the initial functional motion tests have not been reviewed because the review of such documentation is not required by this standard.

SECTION 3-1.2: LOAD RATINGS

…

3-1.2.2 Load Ratings Where Structural Competence Governs Lifting Performance

(a) For each stipulated operating radius, the manufacturer or a qualified person shall ascertain that the crane is capable of supporting rated loads without stresses exceeding predetermined acceptable values. Dynamic effects associated with hoisting and slewing shall be considered and wind shall be taken in the least favorable direction and at the maximum in-service velocity, as specified by the manufacturer or a qualified person.

(b) Under any condition of loading, stresses may also be affected by boom (jib) length, counterweight arrangement, tower (mast) height and arrangement, hoist line reeving, and hoisting speed range. Therefore, the manufacturer or a qualified person shall evaluate structural competence for the least favorable configuration and operating conditions covered by given load ratings…

(d) In addition to the above, the following stipulations shall apply to proof of competence determinations:

(1) the crane is mounted level, except as in para. 3-1.2.2(c), but for those cranes that exhibit significant elastic deformation due to dead, live, wind, or dynamic loads, the effects of such deformations shall be taken into account;

(2) lifting attachments which are a permanent part of the crane in its working condition shall be considered part of the load for rating calculation purposes whether or not such attachments are part of published load ratings…

Calculations supporting the tower crane design were prepared by Favelle Favco and submitted by Peter Stroh. See the permitting section of this report, Chapter 4.

The following extract details the documentation required to be provided with each crane. This would appear to be a requirement of the tower crane manufacturer and tower crane supplier. The documentation supplied by the manufacturer is unknown. With regard to the tower crane supplier, it is unknown if the tower crane manual found on site and other documentation contained in the permitting documents (see the section on permitting

Chapter 4) reflect the actual documents provided to the contractor. These items are therefore enumerated below for information only.

Section 3-1.4: DOCUMENTATION

3-1.4.1

Each crane shall be provided with informational literature written in English including, but not limited to, the following.

(a) Installation preparation instructions which should provide:

(1) vertical and horizontal forces and torsional and overturning moments applicable to each recommended configuration; the data should indicate whether governing forces are due to in-service or out-of-service winds, the applicable wind velocity, and whether the wind has been taken perpendicular or diagonal to the tower (mast); for traveling cranes, the data can be stated in terms of wheel or bogie loads;

(2) data or tower (mast) height limitations based on several wind velocity levels for out-of-service conditions;…

(5) the minimum distance between horizontal reaction support levels for internal climbing cranes;

(6) locations where tower (mast) sections have sufficient strength for internal climbing wedging and external climbing collar installation…

(8) anchorage arrangements for cranes to be installed on fixed bases;

(9) crane dimensional data.

These were generally found to have been provided through the permitting process.

(b) Erection and dismantling instructions which should provide:

(1) weight and dimensions for components and subassemblies;

(2) recommended lifting attachment points;

(3) center of gravity location for nonuniform components and subassemblies;

Center of gravity information was not generally reviewed.

(4) the method and recommended sequence of assembly and disassembly of components and subassemblies; warnings should be given alerting erection personnel when member strength or stability requires particular methods or sequencing;

(5) details, including diagrams where necessary, of critical component connections describing and identifying bolts, pins, and other parts needed, the method of assembling the joint, the torque or tension to be applied to prestressed (traction) bolts, the point in time in the erection process for applying torque or tension, and the means for retaining pins, etc.;…

(c) Operating instructions, limitations, and precautions.

(d) Maintenance requirements and recommendations including identification of those members or locations that should be periodically observed, or tested, for the purpose of detecting the onset of metal fatigue, the loosening of prestressed (traction) bolts, or wear affecting the ability of the crane to support rated loads.

(e) Repair recommendations including advice on welding procedures. The type of metal used for load sustaining members shall be identified (see para. 3-1.18.5).

(f) Design characteristics affecting safety, such as:

(1) location, proper settings and adjustments, and functioning of limiting and indicating devices;

(2) high and low ambient temperature limitations;

(3) permitted variations in electrical supply and circuit parameters;

(4) location and required settings of hydraulic or pneumatic pressure relief valves and locations of points where circuit pressures can be checked (see para. 3-1.18.9).

(5) limitations on service life of load bearing members and mechanisms including recommendations of frequency of inspection as a function of severity of service;…

SECTION 3-1.13: REEVING ACCESSORIES

(a) Eye splices shall be made in a manner recommended by the rope or crane manufacturer and rope thimbles should be used in the eye.

With regard to the wire rope used on the collars, it appears that thimbles were not used in the wire rope. As the rope was threaded through the chain block points on the collars, thimbles, unless split, could not have been installed.

(b) Wire rope clips shall be drop-forged steel of the single saddle (U-bolt) or double saddle type clip. Malleable cast iron clips shall not be used. For spacing, number of clips, and torque values, refer to the clip manufacturer's recommendation. Wire rope clips attached with U-bolts shall have the U-bolt over the dead end of the rope and the live rope resting in the clip saddle. Clips shall be tightened evenly to the recommended torque. After the initial load is applied to the rope, the clip nuts shall be retightened to the recommended torque to compensate for any decrease in rope diameter caused by the load. Rope clip nuts should be retightened periodically to compensate for any further decrease in rope diameter during usage…

As indicated in the section on ASME B30.3, U-bolt compression fittings were used to form the wire rope slings used through the chain block points on the collar. Clip saddles appear to have been randomly located with respect to their positioning over the dead or live ends of the wire rope. Additionally, spacing of the U-bolt clips appears to have been uncontrolled, as their orientation in the as-retrieved condition shows no discernible pattern.

3-1.18.5 Welded Construction

Welding procedures and welding operator qualifications for use in repair or alteration of load sustaining members shall be in accordance with ANSI/AWS D14.3 or ANSI/AWS D1.l. Where special steels or other materials are used, the manufacturer or a qualified person shall provide welding procedure instructions. The type of metal used for load sustaining members shall be identified by the manufacturer [see para. 3-1.4.l(e)]…

The documentation reviewed did not include welder qualifications. With regard to the tie-beam base plates, welding procedures neither complied with AWS D1.1 nor with the fabrication drawings, as the fillet welds did not meet the minimum size requirements for the plate sizes joined. As note in the results of the macroscopic examination by ATLSS, the weld leg sizes were generally 3/16 in., as compared to 5/16 in. being the minimum required by AWS. See Section 3.1.6 of this report for further information on the welds.

Chapter 3-2 Inspection, Testing, and Maintenance

SECTION 3-2.1: INSPECTION

3-2.1.1 General

The manufacturer shall furnish operation and maintenance information [see paras. 3-1.4.l(c) to (f)].

3-2.1.2 Inspection Classification

(a) Initial Inspection. Prior to initial use, all new, reinstalled, altered, or modified construction tower cranes shall be inspected by a qualified person to verify compliance with the applicable provisions of this volume.

This appears to have been carried out. See the section in this report on permitting, Chapter 4.

(b) Regular Inspection. Inspection procedures for cranes in regular service are divided into two general classifications based on the intervals at which inspection should be performed. The intervals in turn are dependent upon the nature of the critical components of the crane and the degree of their exposure to wear, deterioration, or malfunction. The two general classifications are designated as frequent and periodic with respective intervals between inspection as defined below.

(1) Frequent Inspection. Visual examination by the operator or other designated person with records not required:

(a) light service - monthly;

(b) normal service - weekly to monthly;

(c) heavy service - daily to weekly.

(2) Periodic Inspection. Visual inspection by an appointed person at 1 to 12 month intervals or as specifically recommended by the manufacturer or by a qualified person. Records shall be kept of apparent external conditions to provide a basis for continuing evaluation…

See the section on permitting for a review of the crane inspections, Chapter 4.

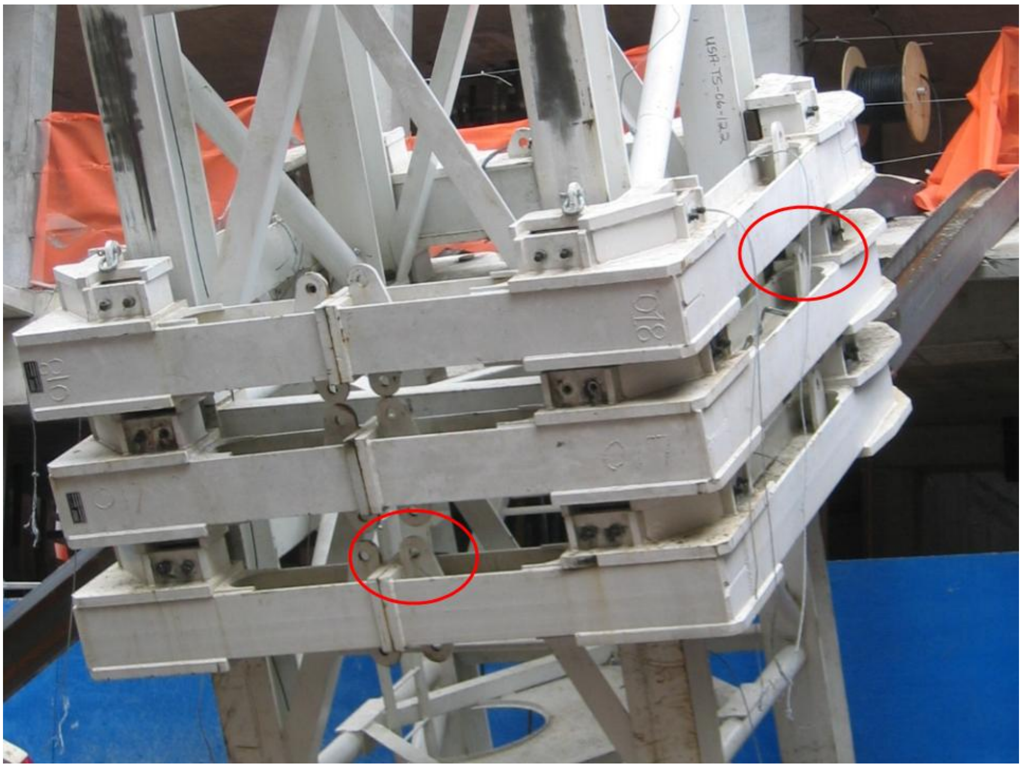
3.1.5 American Society of Mechanical Engineers B30.9-2006 Slings (quoted material in italics, this section)

SECTION 9-0.1: SCOPE OF ASME B30.9

Volume B30.9 includes provisions that apply to the fabrication, attachment, use, inspection, and maintenance of slings used for lifting purposes, used in conjunction with equipment described in other volumes of the B30 Standard, except as restricted in B30.12 and B30.23. Slings fabricated from alloy steel chain, wire rope, metal mesh, synthetic fiber rope, synthetic webbing, and synthetic fiber yarns in a protective cover(s) are addressed…

As part of the collar installation, wire ropes were implemented at completion of the tie-beam installation, as evidenced by post-collapse photographs as shown in Photographs 3.5 and 3.6.

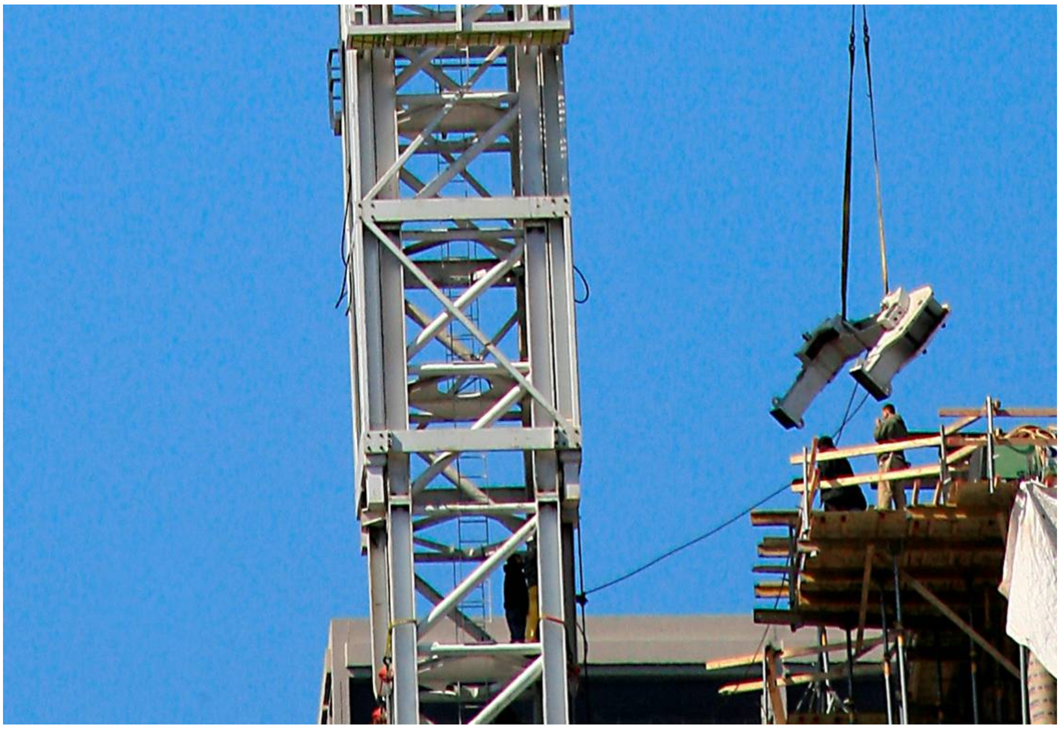
Photograph 3.5 - View of the west elevation of the collapsed tower collars. The location of visible wire rope attachments to the collars are circled. Note the lack of thimbles in the formed sling. Photo from New York County District Attorney 328_58.jpg.



Photograph 3.6 - View the south and east elevations of the collapsed tower collars. The location of visible wire rope attachments to the collars are circled. Note the lack of thimbles in the formed sling. Photo from the crane engineer, Peter Stroh, IMG_0081.jpg.

As can be seen in the photographs, the ropes were attached directly to the lugs on the collars. As previously mentioned, the specific lugs to which they were attached are not consistent with Favelle Favco's recommendations regarding the location of the four chain block attachment points which were to remain on the collars after completion of the collar / tie-beam installation. See Figure 3.2, 3.3, 3.4 and 3.5 for extracts from the Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123.

Polyester web slings were used both for lifting the collars into place as well as for securing them temporarily in a vertical position while the two collar halves were bolted together and positioned to their final vertical position using the chain falls. Photograph 3.7 shows the collar half being lifted into place as well as the chain falls with the polyester web slings choke hitched around the legs.



Photograph 3.7 - View of tower crane with collar half being lifted into place via polyester web slings (right of photo) and chain falls incorporating polyester web slings choke hitched around the tower legs (lower portion of tower). (Source: photograph by Gary Halby).

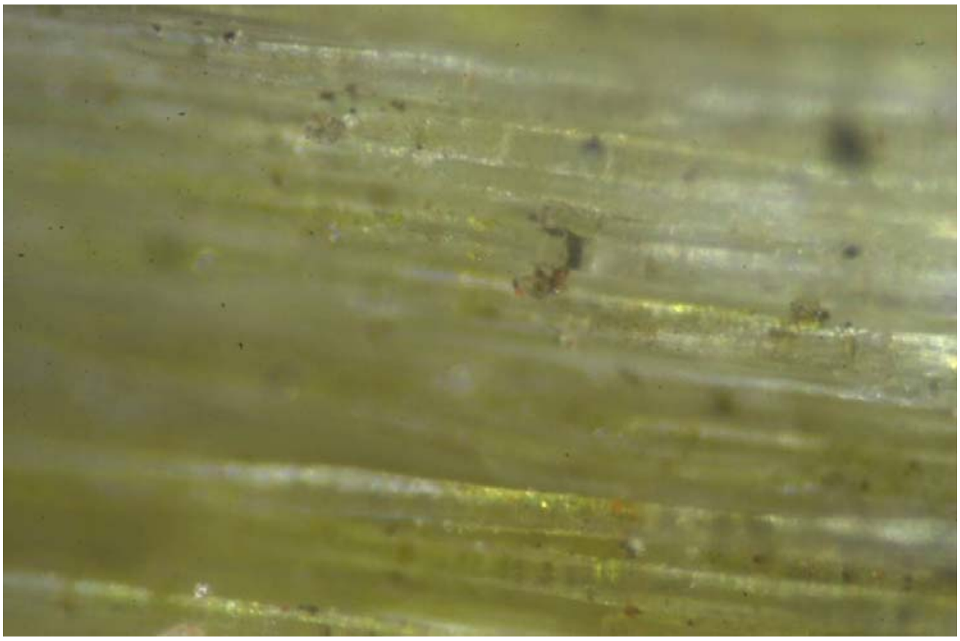
SECTION 9-0.2: DEFINITIONS

abnormal operating conditions: environmental conditions that are unfavorable, harmful, or detrimental to or for the operation of a sling, such as excessively high or low ambient temperatures; exposure to weather; corrosive fumes; dust-laden or moisture-laden atmospheres; and hazardous locations.

At the time of the collapse the environmental conditions under which the slings were used were not "abnormal" as the weather records indicate. However, over time there would be exposure to weather including water and ultraviolet rays, possible temperature extremes and other hazards particularly for the polyester web slings. There is evidence that at least one polyester web sling was previously exposed to long periods of ultraviolet radiation sufficient to bleach both the outer fibers of the polyester web sling and the manufacturer's label. See Photograph 3.8, 3.9 and 3.10 below in which the faded label and the change in polyester fiber color can be easily observed.

Photograph 3.8 - OSHA evidence item 1. Polyester web sling removed from the collapse site. Note the faded label. This was taken from the southwest corner of the 9th floor collar. It was attached to the lifting point via a chain fall. Photo by Arup.

Photograph 3.9 - OSHA evidence item 1. Polyester web sling removed from the collapse site. This was taken from the southwest corner of the 9th floor collar. Note the varying degrees of fading of the webbing coinciding with the relative degree of potential exposure to ultraviolet rays. Magnification is approximately 2.5x. Photo by Arup.



Photograph 3.10 - Microphotograph of OSHA evidence item 1, polyester web sling taken from the southwest corner lifting point of the 9th floor collar. Note the loss in color of many of the fibers. Magnification is approximately 200x. Photo by Arup.

Photograph 3.11 - OSHA evidence item 1. Polyester web sling removed from the collapse site. This was taken from the southwest corner of the 9th floor collar. Note both the melted and cut fibers within the melted region. Magnification is approximately 10x. Photo by Arup.

abrasion: the mechanical wearing of a surface resulting from frictional contact with other materials or objects.

There is evidence that the polyester web slings were exposed to frictional contact at their bearing points arising from the unprotected/unpadded edges of the tower legs around which they were choke hitched. Photograph 3.11 is a close-up of one such location, in which the melt of the polyester from friction generated heat and cut fibers can all be observed. Additional information is contained in Chapter 8 on Polyester Slings.

angle of choke: angle formed in a sling body as it passes through the choking eye or fittings.

Based upon the tests carried out at the ATLSS test center, it is unlikely that the angle of choke would have been greater than 120 deg.

angle of loading: the acute angle between horizontal and the leg of the rigging, often referred to as horizontal angle.

assembly: a synonym for sling. See sling.

authorized: approved by a duly constituted administrative or regulatory authority.

body (sling): that part of a sling between the eyes, end fittings, or loop eyes...

cable-laid rope: a cable composed of six wire ropes laid as strands around a wire rope core.

cable-laid rope sling, mechanical joint: a wire rope sling made from a cable-laid wire rope with eyes fabricated by swaging one of more metal sleeves over the rope junction…

Wire rope was used for maintaining the vertical position of the collars after completion of the tie-beam installation. These were attached to the collar by threading the rope through the chain block attachment points as indicated in the Section 3.1.4 on ASME B30.3 in this Chapter. A sling was then formed by connecting the “dead” end of the rope to the “live” end of the rope via U-bolt clips.

D/d ratio: the ratio between the curvature taken by the sling, D, and the diameter of the component rope, d.

For a wire rope sling, this is normally controlled by use of a thimble inserted into the eye of the sling. As suggested by the lack of thimbles retrieved from the debris and photographs taken of the slings after the collapse, this was not done. As the wire rope needed to be threaded through the chain block attachment point, use of a thimble would have been impractical although it is possible that an open pattern thimble could also have been inserted around the chain block attachment, thereby creating an acceptable D/d ratio (where D is the minimum diameter of the sling formed by bending the wire rope and “d” is the diameter of the wire rope). A photo of the chain block attachment is given in Photograph 3.12 below. The unprotected/unpadded edge of the attachment is clearly visible. The chart which follows, Figure 3.6, taken from the Wire Rope Sling Users Manual published by the Wire Rope Technical Board provides information regarding the reduction in capacity arising from unacceptable D/d ratios.

Photograph 3.12 - Collar C2-A chain block attachment point. Note the unprotected edges on which the wire rope sling would have been bearing (typical for all chain block and lifting attachments for all collars).. Photo by Arup.

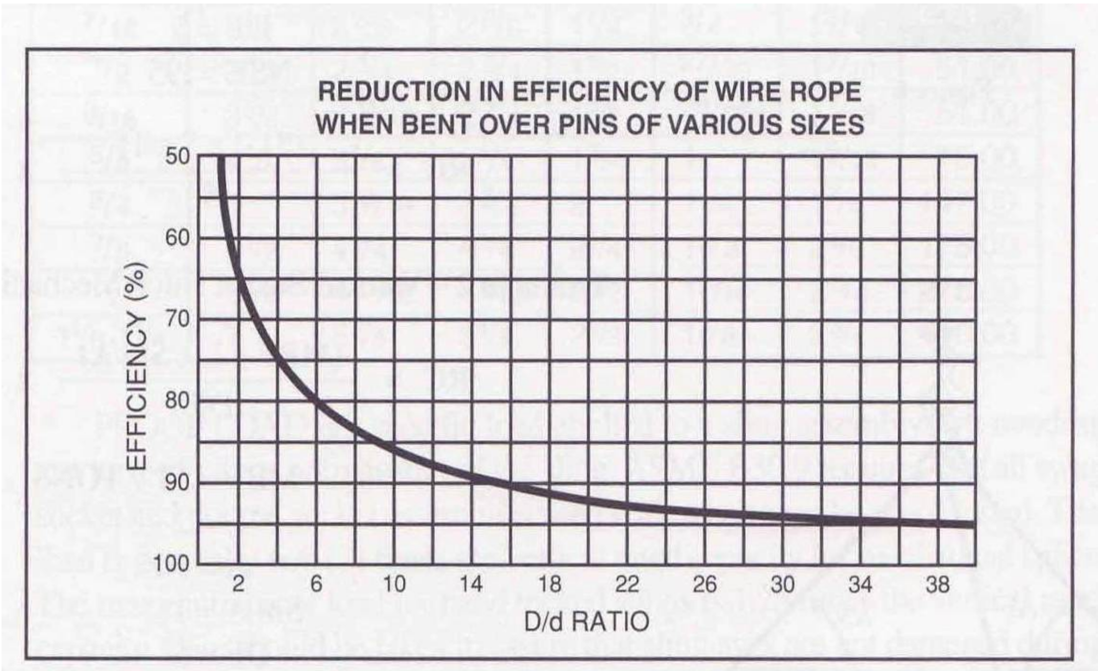


Figure 3.6 - Efficient reduction of wire rope when bent over pins of various sizes. Extracted from the Wire Rope Sling Users Manual third edition.

design factor: ratio between nominal or minimum breaking strength and rated load of the sling.

designated person: selected or assigned by the employer or employer's representative as being competent to perform specific duties.

Based upon the documentation reviewed, it is unknown who, if anyone, was selected as the designated person for use of the polyester web and wire rope slings.

end fitting: terminal hardware on the end of a sling. See sling…

eye opening: the opening in the end of a sling for the attachment of the hook, shackle, or other lifting device or the load itself…

fabrication efficiency: the sling assembly strength, as a percentage of the material strength prior to fabrication.

fitting: hardware on the end of a sling…

hitch (hitched): a method of rigging (attaching) a sling temporarily to a load or object for the purpose of lifting…

hitch, choker: a method of rigging a sling in which the sling is passed around the load, then through one loop eye, end fitting, or other device, with the other loop eye or end fitting attached to the lifting device. This hitch can be done with a sliding choker hook or similar device.

The polyester web slings were attached to the tower via a choker hitch.

hitch, vertical: a method of rigging a sling in which the load is attached to the loop eye or end fitting at one end of the sling and the loop eye or end fitting at the other end is attached to the lifting device. Any hitch less than 5 deg from the vertical may be considered a vertical hitch.

Angles calculated for the polyester web slings indicate that the angle to the horizontal was approximately 9 degrees and would therefore not have been considered a vertical hitch at the collar attachment point. The angle of the wire rope sling to the attachment points is unknown.

horizontal angle: the acute angle between the horizontal plane and the leg of the rigging, also known as the angle of loading.

length, sling: the distance between the extreme bearing points of the sling.

Based upon measurements of the chainfall/polyester web assembly to the breaking point from the OSHA evidence, this length is approximately 104 inches. The length of the polyester web sling from its attachment to the chain to the bearing point at the back of the tower leg was measured to be approximately 48 (OSHA Evidence C Items no. 1, 12 and 13) and 58 (Item no. 2) inches, from a total polyester web length, eye bearing to eye bearing (pull to pull) of 72 inches.

single-leg slings without endfittings: measured from pull to pull or from bearing to bearing of eyes.

single-leg slings with end fittings: measured from pull to pull of end fittings or eyes…

loop eye (web sling): a length of webbing that has been folded back upon itself, forming an opening, and joined to the sling body to form a bearing surface.

ply: a layer of load bearing webbing used in a web sling assembly.

The body of the polyester web slings used was 2-ply.

proof load: the specific load applied in performance of the proof tests.

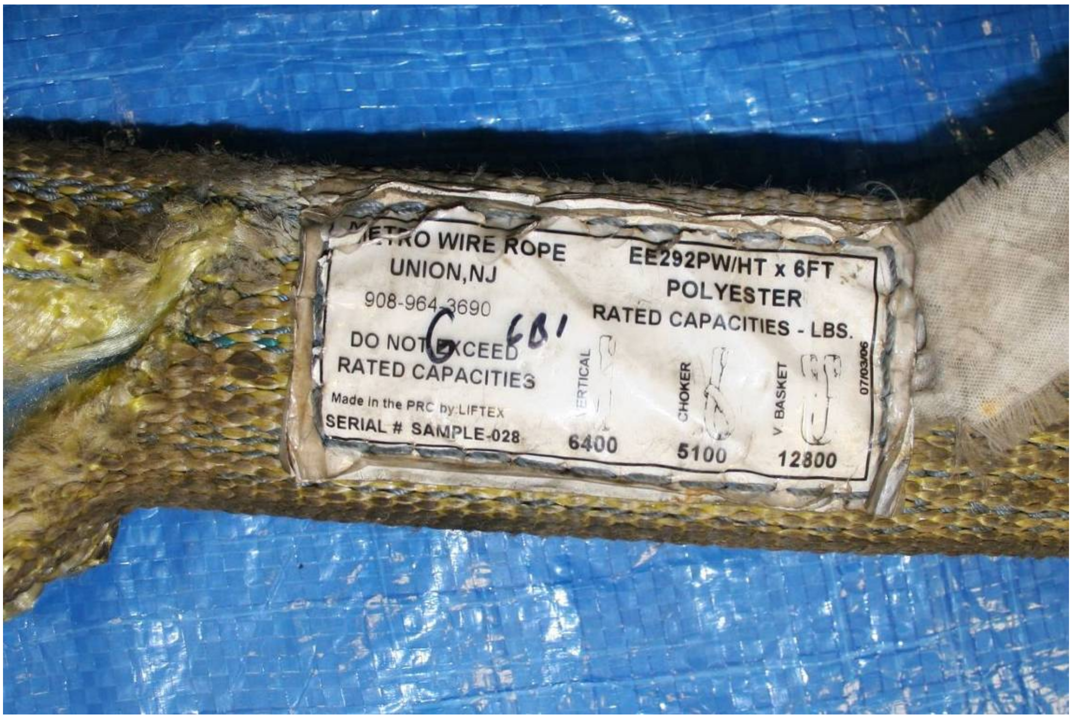
proof test: a nondestructive load test made to a specific multiple of the rated load of the sling.

qualified person: a person who, by possession of a recognized degree or certificate of professional standing in an applicable field, or who, by extensive knowledge, training, and experience, has successfully demonstrated the ability to solve or resolve problems relating to the subject matter and work.

With regard to use of the polyester web and wire rope slings, the denoted tower crane rigger, William Rapetti, would have been the qualified person. With regard to the design of the collar tie-beams, the qualified person would have been Peter Stroh, engineer of record.

rated load: the maximum allowable working load established by the sling manufacturer. The terms "rated capacity" and "working load limit" are commonly used to describe rated load…

The rated loads were clearly marked on the polyester web slings. Two different polyester web sling manufacturers have been identified as Lift-All Co., Inc. (OSHA Evidence Items no. 2, 12 and 13) and Liftex Corporation (OSHA Item no. 1). A photo of each label is given in the following three photographs.



Photograph 3.13 - OSHA Evidence Item no. 1, view of label. Photo by Arup.

Photograph 3.14 - OSHA Evidence Item no. 2, view of label. Photo by Arup.

Photograph 3.15 - OSHA Items Evidence no. 12 (top) and 13 (bottom), view of label. Photo by Arup.

selvage edge: the woven or knitted edge of synthetic webbing so formed as to prevent raveling.

shock load: any condition of rapid lift, sudden shifting of load, or arrest of a falling load.

sling: an assembly to be used for lifting when connected to a lifting mechanism. The upper portion of the sling is connected to the lifting mechanism and the lower supports the load, as described in this Volume…

The polyester web slings were attached to chain falls incorporating a winch and chain assembly. This was used for final positioning of the collars and, as such, would have therefore have been "connected to a lifting mechanism" per this definition. See Photograph 1.8 for an example assembly. The method of tensioning and installing the wire rope slings is unknown; however, per the Favelle Favco installation instructions, their use should have been part of the general installation and final placement of the collars had these instructions been followed, notwithstanding that chain blocks rather than wire ropes would have been used. This definition would therefore also be applicable to the use of the wire rope as "slings."

sling manufacturer (fabricator): a person or company assembling or fabricating sling components into their final form. The sling manufacturer and the manufacturer of the sling material may or may not be identical.

While the manufacturers of the polyester web slings were clearly known and labelled on the slings, the source of the wire rope for the wire rope slings is unknown.

sling service

normal: service that involves use of loads within the rated load.

severe: service that involves normal service coupled with abnormal operating conditions.

special or infrequent: service that involves operation, other than normal or severe, which is approved by a qualified person…

Use of the slings outside of "normal" service would therefore be permitted, subject to approval by a "qualified person." In this case, the "qualified person" would be William Rapetti, as he was the tower crane rigger for the collar assembly and installation.

splice (web sling): that part of a sling that is lapped and secured to become an integral part of the sling.

assembly splice (web sling): any splice that joins two or more parts of the sling without bearing any of the applied load.

load bearing splice (web sling): that part of a sling that is lapped and secured to become an integral load bearing part of the sling…

The eyes of the polyester web slings would therefore constitute "load bearing splices".

splice, turn back or returned loop (wire rope): mechanical splice in which the rope is looped back on itself and secured with one or more metal sleeves. This method requires special fittings, techniques, and equipment to produce an end termination to meet the requirements of this Volume.

The wire rope was looped back on itself and fitted with U-bolt clips so as to achieve a mechanical splice.

strand laid rope: a wire rope made with strands (usually six to eight) formed around a fiber core, wire strand core, or independent wire rope core (IWRC).

The wire rope used in the installation of the collars was a 7x19 ¼ inch diameter wire rope, also known as a small bore, small diameter or aircraft cable, typically used for specialty applications

> *strength (wire rope and structural strand), minimum breaking: load at which a new and unused wire rope or structural strand could be expected to break when loaded to destruction in direct tension…*
>
> *Chapter 9-1 Alloy Steel Chain Slings: Selection, Use, and Maintenance*

While steel chains were used to connect the polyester web slings to the chain winch and thereby to the collars, they have not been a focus of attention in this investigation and the provisions regarding their use are therefore note included in this report at this time.

> *Chapter 9-2 Wire Rope Slings: Selection, Use, and Maintenance*
>
> *SECTION 9-2.0: SCOPE*
>
> *Chapter 9-2 includes provisions that apply to wire rope slings.*
>
> *SECTION 9-2.1: TRAINING*
>
> *Wire rope sling users shall be trained in the selection, inspection, cautions to personnel, effects of environment, and rigging practices as covered in this Chapter.*

Beyond the certification for William Rapetti it is not known if any other personnel had been trained in the use of wire rope per the provisions of this standard.

> *SECTION 9-2.2: MATERIALS AND COMPONENTS*
>
> *(06) 9-2.2.1 Wire Rope Material*
>
> *The wire rope shall be manufactured and tested in accordance with ASTM A 1023-02 and ASTM A 586.*
>
> *(a) Only new or unused wire rope shall be used for fabricating slings covered in this Chapter.*
>
> *(b) Only regular-lay wire rope shall be used for fabricating slings covered in this Chapter.*
>
> *(c) Rotation resistant wire rope shall not be used for fabricating slings covered in this Chapter.*

These provisions were met by the use of new 7x19 ¼ inch aviation cable.

> *9-2.2.2 Components*
>
> *…*
>
> *(d) Rigging hardware, when employed, shall meet the requirements of ASME B30.26.*
>
> *9-2.2.3 Other Materials and Components*
>
> *Wire ropes and components, other than those listed in paras. 9-2.2.1 and 9-2.2.2, may be employed. When such materials are employed, the sling manufacturer or a qualified person shall provide specific data. These slings shall comply with all other requirements of this Chapter.*

SECTION 9-2.3: FABRICATION AND CONFIGURATIONS

9-2.3.1 Fabrication

Methods of fabrication include hand splicing, turnback eye, return loop or flemish eye mechanical splicing, and poured or swaged socketing.

(a Wire rope clips shall not be used to fabricate wire rope slings except where the application of slings prevents the use of prefabricated slings and where the specific application is designed by a qualified person.

(1) Wire rope clips, if employed, shall be installed and maintained in accordance with the recommendations of the clip manufacturer or a qualified person, or in accordance with the provisions of ASME B30.26.

(2) Malleable cast iron clips shall not be used to fabricate slings.

Wire rope clips were used to fabricate the wire rope slings. No documentation that has been reviewed indicated that this had been designed by a qualified person nor that their installation was maintained. Use of U-bolt clips for wire rope slings would therefore have been a violation of this standard.

(b) Knots shall not be used to fabricate slings,

(c) The diameter and width of the bearing surface of the fitting can affect the strength of the sling. The sling manufacturer's recommendation should be followed when fittings are used with the sling.

(d) Other fabrication methods not covered by this Chapter shall be rated in accordance with the recommendation of the sling manufacturer or a qualified person, and shall conform to all other provisions of this Chapter.

It does not appear from a review of the documentation that the wire rope fabrication method was rated by the sling manufacturer or a qualified person for this particular use.

9-2.3.2 Configurations

(a) Single-leg slings, two-leg, three-leg, and four-leg bridle slings, used in vertical, choker, and basket hitches are covered by this Chapter.

(b) Slings made of rope with 6 x 19 and 6 x 36 classification and cable laid slings shall have a minimum clear length of rope 10 times the rope diameter between splices, sleeves, or end fittings, unless approved by a qualified person…

SECTION 9-2.4: DESIGN FACTOR

The design factor for wire rope slings shall be a minimum of 5.

The minimum breaking strength for a galvanized 7x19 ¼ inch small diameter cable is given by ASTM A1023-07, as 7000 lb. Prior to further reductions in the rated capacity arising from the type of hitch, angle of location, diameter of curvature and fabrication capacity, the maximum possible rated load would therefore have been 1400 lb. The required minimum safe working load ("SWL") is specified by Favelle Favco as 2 tonnes per permanent chain block, or 4,480 lb.

SECTION 9-2.5: RATED LOAD

The term rated capacity is commonly used to describe rated load.

9-2.5.1

These rated loads are based on the following factors:

(a) material strength(s)

(b) design factor

(c) type of hitch

(d) angle of loading

(e) diameter of curvature over which the sling is used (D / d)

(f) fabrication efficiency…

9-2.5.4

Rated loads for slings used in a choker hitch shall conform to the values shown…provided that the angle of choke is 120 deg or greater…

It is unknown how the wire rope sling was attached to the tower.

9-2.5.5

Rated loads for angles of choke less than 120 deg shall be determined by using the values in Fig. 7, the sling manufacturer, or a qualified person.

9-2.5.6

Other materials and configurations not covered by this Chapter shall be rated in accordance with the recommendation of the sling manufacturer or a qualified person and shall conform to all other provisions of this Chapter.

9-2.5.7

When components of the sling have a lower rated load than the wire rope with which it is being used, the sling shall be identified with a rated load consistent with the lowest load rating of any of the components.

Information regarding the U-bolt clips was not reviewed; however, this is not considered to be critical in the failure investigation.

SECTION 9-2.6: PROOF TEST REQUIREMENTS

9-2.6.1

(a) Prior to initial use, all new swaged sockets, poured sockets, turnback eyes, mechanical joint grommets, and endless wire rope slings shall be proof tested by the sling manufacturer or a qualified person...

The wire rope sling incorporates a turnback eye formed by mechanical attachment of U-bolt clips. There was no documentation reviewed indicating this was proof tested.

9-2.6.2 Proof Load Requirements

(a) For single- or multiple-leg slings and endless slings, each leg shall be proof loaded to the following load requirements based on fabrication method. In no case shall the proof load exceed 50% of the component ropes' or structural strands' minimum breaking strength.

(1) Mechanical Splice Slings. The proof load shall be a minimum of 2 times the single-leg vertical hitch rated load…

It does not appear from the document review that this was done.

(b) The proof load for components (fittings) attached to single legs shall be the same as the requirement for single-leg slings in para. 9-2.6.2(a)…

SECTION 9-2.7: SLING IDENTIFICATION

9-2.7.1 ldentification Requirements

Each sling shall be marked to show

(a) name or trademark of manufacturer

(b) rated loads for the type(s) of hitch(es) used and the angle upon which it is based

(c) diameter or size

(d) number of legs, if more than one

9-2.7.2 Initial Sling ldentification

Sling identification shall be done by the sling manufacturer.

9-2.7.3 Maintenance of Sling ldentification

Sling identification should be maintained by the user so as to be legible during the life of the sling.

9-2.7.4 Replacement of Sling ldentification

Replacement of the sling identification shall be considered a repair as specified in paras. 9-2.9.5(a) and (b). Additional proof testing is not required.

Wire rope slings were field fabricated. Manufacturer's identification would therefore not have been applicable.

SECTION 9-2.8: EFFECTS OF ENVIRONMENT

9-2.8.1 Temperature

…

(b) When IWRC wire rope slings are to be used at temperatures above 400°F (204°C) or below -40°F (-4O0C), the sling manufacturer should be consulted.

9-2.8.2 Chemically Active Environments

The strength of wire rope slings can be degraded by chemically active environments. This includes exposure to chemicals in the form of solids, liquids, gases, vapors, or fumes. The sling manufacturer or qualified person should be consulted before slings are used in chemically active environments.

There was no documentation or other evidence reviewed suggesting that either temperature or environmental limitations were exceeded.

SECTION 9-2.9: INSPECTION, REMOVAL, AND REPAIR

9-2.9.1 Initial Inspection

Prior to use, all new, altered, modified, or repaired slings shall be inspected by a designated person to verify compliance with the applicable provisions of this Chapter.

9-2.9.2 Frequent Inspection

(a) A visual inspection for damage shall be performed by the user or other designated person each day or shift the sling is used.

(b) Conditions such as those listed in para. 9-2.9.4 or any other condition that may result in a hazard shall cause the sling to be removed from service. Slings shall not be returned to service until approved by a qualified person.

(c) Written records are not required for frequent inspections.

It is unknown if any inspections of the wire rope or slings were ever undertaken.

9-2.9.3 Periodic Inspection

(a) A complete inspection for damage to the sling shall be periodically performed by a designated person. Inspection shall be conducted on the entire length including splices, end attachments, and fittings. The sling shall be examined for conditions such as those listed in para. 9-2.9.4 and a determination made as to whether they constitute a hazard.

(b) Periodic Inspection Frequency. Periodic inspection intervals shall not exceed 1 year. The frequency of periodic inspections should be based on

(1) frequency of sling use

(2) severity of service conditions

(3) nature of lifts being made

(4) experience gained on the service life of slings used in similar circumstances

(c) Guidelines for the time intervals are

(1) normal service - yearly

(2) severe service - monthly to quarterly

(3) special service - as recommended by a qualified person

(d) A written record of the most recent periodic inspection shall be maintained.

An initial inspection record was not included in the documentation reviewed.

9-2.9.4 Removal Criteria

A wire rope sling shall be removed from service if conditions such as the following are present:

(a) missing or illegible sling identification (see Section 9-2.7)

(b) broken wires

(1) for strand-laid and single-part slings, ten randomly distributed broken wires in one rope lay, or five b broken wires in one strand in one rope lay

(2) for cable-laid slings, 20 broken wires per lay…

(c) severe localized abrasion or scraping

(d) kinking, crushing, birdcaging, or any other damage resulting in damage to the rope structure

(e) evidence of heat damage

(f) end attachments that are cracked, deformed, or worn to the extent that the strength of the sling is substantially affected

(8) severe corrosion of the rope, end attachments, or fittings

(h) for hooks, removal criteria as stated in ASME B30.10

(i) for rigging hardware, removal criteria as stated in ASME B30.26

(j) other conditions, including visible damage, that cause doubt as to the continued use of the sling

Document and evidence review has not suggested that any of the above conditions were present at the time of the accident; however, the unprotected/unpadded edge of the wire rope sling within the collar chain block attachment point would have been likely to have cause localized damage, though this may have been hidden from view (see below).

9-2.9.5 Repair

(a) Slings shall be repaired only by the sling manufacturer or a qualified person.

(b) A repaired sling shall be marked to identify the repairing agency per Section 9-2.7.

(c) End attachments and fittings used for sling repair shall comply with the provisions of this Chapter.

(d) Repair of hooks shall comply with ASME 830.10.

(e) The wire rope used in the sling shall not be repaired.

(f) Repairs to wire rope slings shall be restricted to end attachments and fittings.

(8) Modifications or alterations to end attachments or fittings shall be considered as repairs and shall conform to all other provisions of this Chapter.

(h) All repairs shall comply with the proof test requirements of Section 9-2.6.

It is unknown if slings were ever repaired.

SECTION 9-2.10: OPERATING PRACTICES

9-2.10.1 Sling Selection

(a) Slings that appear to be damaged shall not be used unless inspected and accepted as usable under Section 9-2.9.

(b) Slings having suitable characteristics for the type of load, hitch, and environment shall be selected in accordance with the requirements of Sections 9-2.5 and 9-2.8.

(c) The rated load of the sling shall not be exceeded…

As noted earlier, the maximum rated capacity of a galvanized 7x19 ¼ inch small diameter cable as specified in ASTM A1023-07 after allowing for a design factor only, is 1,400 lb. Additional reductions in the rated capacity could arise due to the type of hitch, angle of location, diameter of curvature and fabrication capacity. The required minimum safe working load (“SWL”) is specified by Favelle Favco as 2 tonnes per permanent chain block, or 4,480 lb. As the collar had an estimated weight of approximately 11,000 lb., the load per wire rope, assuming an even distribution over four ropes, would have been 2,750 lb. This is in excess of the maximum rated capacity.

(g) When D/d ratios smaller than those cited in the tables are necessary, the rated load of the sling shall be decreased. Consult the sling manufacturer for specific data or refer to the WRTB Wire Rope Sling User's Manual.

(h) The fitting shall be of the proper shape and size to ensure that it seats properly in the hook or lifting device…

See comments above regarding the use of thimbles. The D/d limitations were not met.

9-2.10.3 Effects of Environment

(a) Slings should be stored in an area where they will not be subjected to mechanical damage, corrosive action, moisture, extreme temperatures, or kinking (see Section 9-2.8).

It is unknown how the wire rope was stored on site.

(b) When used at or in contact with extreme temperatures, the guidance provided in Section 9-2.8 shall be followed…

9-2.10.4 Rigging Practices

(a) Slings shall be shortened or adjusted only by methods approved by the sling manufacturer or a qualified person.

(b) Slings shall not be shortened or lengthened by knotting, twisting, or by wire rope clips.

Wire rope U-bolt clips were used in forming the slings.

(c) The sling shall be hitched in a manner providing control of the load.

(d) Slings in contact with edges, corners, or protrusions should be protected with a material of sufficient strength, thickness, and construction to prevent damage to the sling.

There is no evidence that protection was provided to the wire rope slings where these were in contact with the chain block attachment points. See a sample location in the Photograph 3.16 below. It is also unknown in what manner the load was controlled, as there appears to have been no provision for assessing the tension in the wire rope.

Photograph 3.16 - Close-up of 3rd floor level collar, west elevation, south corner, after collapse, showing the lack of evident protection or thimbles in the formation of the wire rope sling. Photo from New York County District Attorney 328_58.jpg.

(e) Shock loading should be avoided.

(f) Loads should not be rested on the sling.

(g) Slings should not be pulled from under a load when the load is resting on the sling.

(h) Twisting and kinking shall be avoided.

(i) During lifting, with or without load, personnel shall be alert for possible snagging…

(l) Slings should not be dragged on the floor or over an abrasive surface.

(m) In a choker hitch, the choke point should only be on the sling body, not on a splice or fitting.

(n) In a choker hitch, an angle of choke less than 120 deg should not be used without reducing the rated load (see para. 9-2.5.5).

(0) Slings should not be constricted, bunched, or pinched by the load, hook, or any fitting.

(p) The load applied to the hook should be centered in the base (bowl) of the hook to prevent point loading on the hook, unless the hook is designed for point loading.

(q) An object in the eye of a sling should not be wider than one half the length of the eye…

(s) Slings made with wire rope clips shall not be used as a choker hitch…

It is unknown how the wire ropes were attached to the tower; e.g., a choker hitch may have been used. Documentation and evidence review does not suggest that the remaining provisions were not met.

Chapter 9-5 Synthetic Webbing Slings: Selection, Use, and Maintenance

SECTION 9-5.0: SCOPE

Chapter 9-5 includes provisions that apply to synthetic webbing slings.

As noted earlier in this section and elsewhere in this report, polyester web slings were used for providing temporary vertical support and to vertically position the collar halves before and after assembly at their designated levels on the tower crane. The provisions of this section of the standard will therefore apply.

SECTION 9-5.1: TRAINING -

Synthetic webbing sling users shall be trained in the selection, inspection, cautions to personnel, effects of the environment, and rigging practices as covered by this Chapter.

Beyond the certification for William Rapetti it is not known if any other personnel had been trained in the use of wire rope per the provisions of this standard.

SECTION 9-5.2: MATERIALS AND COMPONENTS

9-5.2.1 Webbing

The synthetic webbing shall be manufactured and tested in accordance with WSTDA-WB-I.

9-5.2.2 Thread

The thread used in the fabrication of synthetic webbing slings shall be manufactured and tested in accordance with WSTDA-TH-I.

Both Lift-All Co., Inc. and Liftex Corporation, the two manufacturers of the polyester web slings used for the 51st St crane collar installation, are understood to be members in good standing with the Web Sling and Tie Down Association ("WSTDA"). Additionally, information published by Lift-All Co. indicates compliance with both ASME and WSTDA standards. Similar statements could not be found for Liftex Corporation.

9-5.2.3 Coatings

Finishes and coatings shall be compatible with the other components and not impair the performance of the sling.

9-5.2.4 Components

(a) Fittings shall be manufactured to ensure that the rated load shall be at least the same as the synthetic webbing sling.

(b) Fittings shall have sufficient strength to sustain twice the rated load of the sling without visible permanent deformation.

(c) All surfaces of fittings shall be cleanly finished and sharp edges removed.

(d) Hooks, when employed, shall meet the requirements of ASME B30.26.

(e) Rigging hardware, when employed, shall meet the requirements of ASME B30.26.

9-5.2.5 Other Materials

Synthetic webbings other than those listed in paras. 9-5.2.1 and 9-5.2.4 may be employed. When such materials are employed, the sling manufacturer or a qualified person shall provide specific data. These slings shall comply with all other requirements of this Chapter.

SECTION 9-5.3: FABRICATION AND CONFIGURATIONS

9-5.3.1 Fabrication

(a) Stitching shall be the method for fabricating synthetic webbing slings.

(b) The thread shall be the same yarn type as the sling webbing.

(c) The diameter and width of the bearing surface of the fitting can affect the strength of the sling. The sling manufacturer's recommendation should be followed when fittings are used with the sling.

9-5.3.2 Configurations

(a) Single-leg slings, two-leg, three-leg, and four-leg bridle slings, used in vertical, choker, and basket hitches are covered by this Chapter.

(b) One-ply, two-ply, and four-ply slings are covered by this Chapter…

SECTION 9-5.4: DESIGN FACTOR

The design factor for synthetic webbing slings shall be a minimum of 5.

Based upon this clause, the rated capacities as given in on the polyester web sling labels provided suggest minimum breaking capacities for the polyester web slings of 25,500 lb. for the Liftex Corporation sling and 25,000 lb. for the Lift-All Co. slings after allowing for a choker hitch assembly, but prior to considerations of diameter of curvature, angle of loading and angle of hitch.

SECTION 9-5.5: RATED LOAD

The term rated capacity is commonly used to describe rated load.

9-5.5.1

These rated loads are based on the following factors:

(a) material strength(s)

(b) design factor

(c) type of hitch

(d) angle of loading

(e) diameter of curvature over which the sling is used

(f) fabrication efficiency…

9-5.5.3

Horizontal sling angles less than 30 deg shall not be used except as recommended by the sling manufacturer or a qualified person…

Calculations suggest that the horizontal angle of loading based upon the angle of the collar to the polyester web sling was approximately 81 degrees. However the angle of the choke hitch to the pull of the sling had a resulting horizontal angle of load less than 30 degrees. See Section 8.2.2 of Chapter 8.

9-5.5.4

Rated loads for slings used in a choker hitch shall conform to the values shown in Tables 21 through 25, provided that the angle of choke is 120 deg or greater.

The angle of choke in the as-installed condition is unknown; however, tests at ATLSS suggest that the angle of choke would not have been greater than 120 degrees. See Section 8.2.2 of Chapter 8.

9-5.5.5

Rated loads for angles of choke less than 120 deg shall be determined by using the values in Fig. 19, the sling manufacturer, or a qualified person.

…

9-5.5.7

When components of the sling have a lower rated load than the synthetic webbing with which it is being used, the sling shall be identified with a rated load consistent with the lowest load rating of any of the components.

SECTION 9-5.6: PROOF TEST REQUIREMENTS

9-5.6.1

(a) Prior to initial use, all synthetic webbing slings incorporating previously used or welded fittings and all repaired slings shall be proof tested by the sling manufacturer or a qualified person.

(b) All other new synthetic webbing slings and fittings are not required to be proof tested unless specified by the purchaser.

Visual inspection of the polyester web slings and the OSHA report on the tower collapse indicates that at least one sling had been in service previously. There was no documentation reviewed indicating that any proof testing was undertaken.

9-5.6.2 Proof Load Requirements

(a) For single- or multiple-leg slings and endless slings, each leg shall be proof loaded to 2 times the single-leg vertical hitch rated load.

(b) The proof load for fittings attached to single legs shall be a minimum of 2 times the single-leg vertical hitch rated load…

SECTION 9-5.7: SLING IDENTIFICATION

9-5.7.1 Identification Requirements

Each sling shall be marked to show

(a) name or trademark of manufacturer

(b) manufacturer's code or stock number

(c) rated loads for the type(s) of hitch(es) used and the angle upon which it is based

(d) type of synthetic web material

(e) number of legs, if more than one

9-5.7.2 Initial Sling Identification

Sling identification shall be done by the sling manufacturer.

9-5.7.3 Maintenance of Sling Identification

Sling identification should be maintained by the user so as to be legible during the life of the sling.

9-5.7.4 Replacement of Sling Identification

Replacement of the sling identification shall be considered a repair as specified in paras. 9-5.9.5(a) and (b). Additional proof testing is not required.

Labels containing the above requisite information were all present on the polyester web slings following the collapse, as observed in the earlier photographs. However, the label which presumably had provided the safety information for the Liftex Corporation sling was bleached or otherwise damaged and illegible.

SECTION 9-5.8: EFFECTS OF ENVIRONMENT

9-5.8.1 Temperature

Polyester and nylon webbing slings shall not be used in contact with an object or at temperatures in excess of 194 F (90 C) or below -40 F (-40 C).

…

9-5.8.3 Sunlight and Ultraviolet Light

The strength of synthetic webbing slings is degraded by exposure to sunlight or ultraviolet light. The sling manufacturer or qualified person should be consulted for additional retirement or inspection requirements. For additional degradation information, see WSTDA-UV-Sling-2003.

As noted earlier, the Liftex Corporation was visibly bleached, indicating prolonged exposure to ultraviolet light. Close-up views and microphotographs also substantiate a range of color in fibers which would have had varying degrees of shielding due to overlying layers (see earlier Photographs 3.8, 3.9 AND 3.10).

SECTION 9-5.9: INSPECTION, REMOVAL, AND REPAIR

9-5.9.1 Initial Inspection

Prior to use, all new, altered, modified, or repaired slings shall be inspected by a designated person to verify compliance with the applicable provisions of this Chapter.

9-5.9.2 Frequent Inspection

(a) A visual inspection for damage shall be performed by the user or other designated person each day or shift the sling is used.

(b) Conditions such as those listed in para. 9-5.9.4 or any other condition that may result in a hazard shall cause the sling to be removed from service. Slings shall not be returned to service until approved by a qualified person.

(c) Written records are not required for frequent inspections.

No documentation, including site reports and field notes, were reviewed suggesting that any inspections were undertaken. However, a statement by William Rapetti as provided in the witness statement taken by the NYDOI investigators suggests that, according to Rapetti, he inspected these polyester web slings before they were used on 3/15/08. It is also claimed in his statement that all of the slings were manufactured by Lift-All Co. which was not per the OSHA evidence observed.

> *9-5.9.3 Periodic Inspection*
>
> *(a) A complete inspection for damage to the sling shall be periodically performed by a designated person. Each sling and component shall be examined individually, taking care to expose and examine all surfaces. The sling shall be examined for conditions such as those listed in para. 9-5.9.4 and a determination made as to whether they constitute a hazard.*
>
> *(b) Periodic Inspection Frequency. Periodic inspection intervals shall not exceed 1 year. The frequency of periodic inspections should be based on*
>
> *(1) frequency of sling use*
>
> *(2) severity of service conditions*
>
> *(3) nature of lifts being made*
>
> *(4) experience gained on the service life of slings used in similar circumstances*
>
> *(c) Guidelines for the time intervals are*
>
> *(1) normal service - yearly*
>
> *(2) severe service - monthly to quarterly*
>
> *(3) special service - as recommended by a qualified person*
>
> *(d) A written record of the most recent periodic inspection shall be maintained.*

No written records were reviewed.

> *9-5.9.4 Removal Criteria*
>
> *A synthetic webbing sling shall be removed from service if conditions such as the following are present:*
>
> *(a) missing or illegible sling identification (see Section 9-5.7)*

It has been noted that a label on the Liftex sling was illegible. While it is presumed this contained safety information, this has not been confirmed.

> *(b) acid or caustic burns*
>
> *(c) melting or charring of any part of the sling*

The failed polyester web slings all exhibit some degree of melt. See Photograph 3.17 below for another example. In this photograph cut fibers as well as melted fibers can be seen along the central cut. However, based upon observations during the full-scale tests undertaken by ATLSS and observations by the polyester materials and the sling experts, it is concluded that this was a result of the installation and failure, not a pre-existing condition. Additional information is provided in Chapter 8, Polyester Slings.

Photograph 3.17 - Close-up of both cut and melted fibers for OSHA item no. 2. Magnification approximately 9x.. Photo by Arup.

(d) holes, tears, cuts, or snags

As previously suggested, the failed polyester web slings all exhibit some degree of cut at regular intervals. See the previous Photograph 3.17. Based upon the testing and observations mentioned above, it is concluded that this was a result of the installation and failure, not a pre-existing condition.

The Liftex Corporation polyester web sling, however, exhibits other damage not consistent with the pattern of damage to the other three polyester web slings, remote from the areas which may have been in bearing against the tower leg. Combined with the bleached fibers, this would be strongly indicative of damage to the sling unrelated to the tower failure. As the slings have been protected against further damage since the collapse it is concluded that this damage was pre-existing. See Photograph 3.18 for sample damage to the Liftex Corporation, OSHA item no. 1. While the all polyester web slings show some cross-width diagonally oriented damage at approximately 12-inch intervals, corresponding to the tower leg dimensions, the Liftex sling shows other damage throughout its length, both along the edges and within the width.

Photograph 3.18 - Liftex polyester web sling (OSHA item no. 1). Damage can be seen along the edges, as well as wear within the width of the sling, at uneven intervals, unlike the other polyester web slings where damage occurs at approximately 12 inch intervals. Photo by Arup.

(e) broken or worn stitching in load bearing splices

(f) excessive abrasive wear

(g) knots in any part of the sling

(h) discoloration and brittle or stiff areas on any part of the sling, which may mean chemical or ultraviolet/sunlight damage

(i) fittings that are pitted, corroded, cracked, bent, twisted, gouged, or broken

(j) for hooks, removal criteria as stated in ASME B30.10

(k) for rigging hardware, removal criteria as stated in ASME B30.26

(l) other conditions, including visible damage, that cause doubt as to the continued use of the sling…

Per the comments and photograph above, the Liftex Corporation sling, OSHA item no. 1, exhibits abrasive wear, discoloration and visible damage strongly indicative of prior use. As such, under the provisions of this clause the polyester web sling should have been removed from service prior to use.

SECTION 9-5.10: OPERATING PRACTICES

9-5.10.1 Sling Selection

(a) Slings that appear to be damaged shall not be used unless inspected and accepted as usable under Section 9-5.9…

See comments to the standard section 9-5.9.4 on Removal Criteria, above.

(c) The rated load of the sling shall not be exceeded…

See comments on rated loads below.

(f) Rated loads cited in this chapter are based on pin diameters shown in WSTDA-WS-1. Pin diameters smaller than these may reduce the rated load of the sling.

(g) The fitting shall be of the proper shape and size to ensure that it is seated properly in the hook or lifting device…

9-5.10.3 Effects of Environment

(a) Slings should be stored in an area where they will not be subjected to mechanical, chemical, or ultraviolet damage or extreme temperatures (see Section 9-5.8).

It is evident that the Liftex Corporation polyester web sling had been subjected to ultraviolet light resulting in an unknown extent of damage to the sling, as previously discussed.

(b) When used at or in contact with extreme temperatures the guidance provided in Section 9-5.8 shall be followed.

(c) When extensive exposure to sunlight or ultraviolet light is experienced by nylon or polyester webbing slings, the sling manufacturer should be consulted for recommended inspection procedure.

It does not appear from William Rapetti's statement or from documentation reviewed that this was done.

9-5.10.4 Rigging Practices

(a) Slings shall be shortened or adjusted only by methods approved by the sling manufacturer or a qualified person.

(b) Slings shall not be shortened or lengthened by knotting or twisting.

(c) The sling shall be hitched in a manner providing control of the load.

There is no indication that the loads being applied to the polyester web or wire rope slings were known with any degree of certainty. This is corroborated for the polyester web slings in the OSHA report.

(d) Slings in contact with edges, corners, protrusions, or abrasive surfaces shall be protected with a material of sufficient strength, thickness, and construction to prevent damage.

This was not done for either the wire rope slings or for the polyester slings.

(e) Shock loading should be avoided.

As discussed in the dynamic analysis section of this report, Chapter 7, the slings were all subjected to shock loading; however, this was after the initial failure of at least one polyester sling at the 18th floor level collar.

(f) Loads should not be rested on the sling.

(g) Slings should not be pulled from under a load when the load is resting on the sling.

(h) Twisting shall be avoided.

As the polyester slings were choke hitched around the tower legs, bunched into the V-groove formed by the intersection of the tower diagonals to the tower legs, with a vertical angle of approximately 9 degrees necessary to connect the sling to the collar, twisting could not have been avoided.

> *(i) During lifting, with or without load, personnel shall be alert for possible snagging.*
>
> *(j) When using multiple basket or choker hitches, the load should be rigged to prevent the sling from slipping or sliding along the load….*

The melt of the polyester web slings at the failure lines is strongly indicative of sliding of the polyester against the tower leg steel edge.

> *(m) In a choker hitch, the choke point should only be on the sling body, not on a splice or fitting.*

It is unknown how the polyester choker hitches were arranged on the tower.

> *(n) In a choker hitch, an angle of choke less than 120 deg should not be used without reducing the rated load (see para. 9-5.5.5).*

Based upon the ATLSS test results, it is unlikely that the choker hitch could have made more than a 120 degree angle of choke. See Figure 3.7 for definition of Angle of Choke.

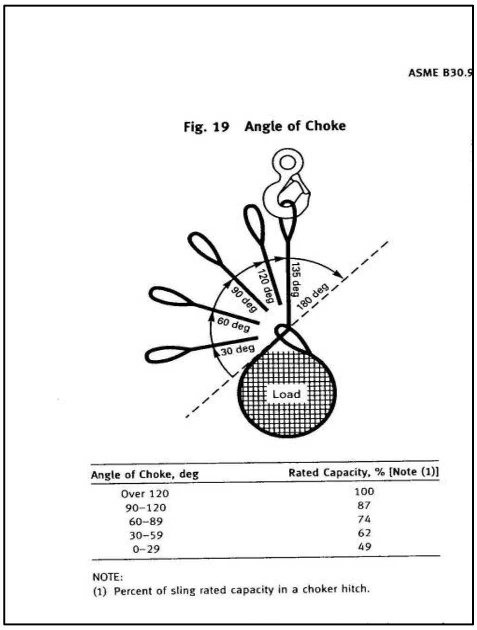

ASME B30.9

Fig. 19 Angle of Choke

Angle of Choke, deg	Rated Capacity, % [Note (1)]
Over 120	100
90–120	87
60–89	74
30–59	62
0–29	49

NOTE:
(1) Percent of sling rated capacity in a choker hitch.

Figure 3.7 - Definition of Angle of Choke and Associated Capacities, from ASME B30.9-2006

> *(o) Slings should not be constricted, bunched, or pinched by the load, hook, or any fitting.*

As the polyester web sling was choke hitched around the tower crane leg in the V-groove formed by the junction of the tower diagonal and leg, the sling, under load, would have necessarily been constricted, bunched and pinched. See Photograph 3.19 below.

Photograph 3.19 - ATLSS test setup prior to testing. Note that, as installed around the tower leg in the V-groove formed between the diagonal and the leg, the sling will necessarily be constricted, bunched and pinched by the load. Photo provided by OSHA.

(p) The load applied to the hook should be centered in the base (bowl) of the hook to prevent point loading on the hook, unless the hook is designed for point loading.

(9) An object in the eye of a sling should not be wider than one-third the length of the eye…

3.1.6 American Welding Society - AWS D1.1/D1.1M:2004 Structural Welding Code-Steel 19th Edition

Interest in the AWS D1.1 is limited to the issue of the fillet welds used for the tie-beam base plate. Macroscopic examination results by the ATLSS test center is contained in the Appendix J

1.1 Scope

1. General Requirements

This code contains the requirements for fabricating and erecting welded steel structures. When this code is stipulated in contract documents, conformance with all provisions of the code shall be required, except for those provisions that the Engineer (see 1.4.1) or contract documents specifically modifies or exempts…

1.2 Limitations

The code is not intended to be used for the following:

(1) Steels with a minimum specified yield strength greater than 100 ksi [690 MPa]

(2) Steels less than 1/8 in [3 mm] thick. When base metals thinner than 1/8 in [3 mm] thick are to be welded, the requirements of AWS D1.3, Structural Welding Code-Sheet Steel, should apply. When used in conjunction with AWS D 1.3, conformance with the applicable provisions of this code shall be required.

(3) Pressure vessels or pressure piping

(4) Base metals other than carbon or low-alloy steels. AWS D1.6, Structural Welding Code-Stainless Steel, should be used for welding stainless steel structures. Whenever contract documents specify AWS D 1.1 for welding stainless steel, the requirements of AWS Dl .6 should apply…

1.4 Responsibilities

1.4.1 Engineer's Responsibilities. The Engineer shall be responsible for the development of the contract documents that govern products or structural assemblies produced under this code. The Engineer may add to, delete from, or otherwise modify, the requirements of this code to meet the particular requirements of a specific structure. All requirements that modify this code shall be incorporated into contract documents. The Engineer shall determine the suitability of all joint details to be used in a welded assembly.

The Engineer shall specify in contract documents, as necessary, and as applicable, the following:

(1) Code requirements that are applicable only when specified by the Engineer.

(2) All additional NDT that is not specifically addressed in the code.

(3) Verification inspection, when required by the Engineer.

(4) Weld acceptance criteria other than that specified in Section 6.

(5) CVN toughness criteria for weld metal, base metal, and/or HAZ when required.

(6) For nontubular applications, whether the structure is statically or cyclically loaded.

(7) All additional requirements that are not specifically addressed in the code.

(8) For OEM applications, the responsibilities of the parties involved.

The fabrication drawings prepared by the crane engineer (Stroh Engineering) are given in the Figures 3.8 and 3.9 below. Minimal information is provided beyond the weld electrode, size and type of weld. NDT, verification inspection, weld acceptance criteria are all unspecified.

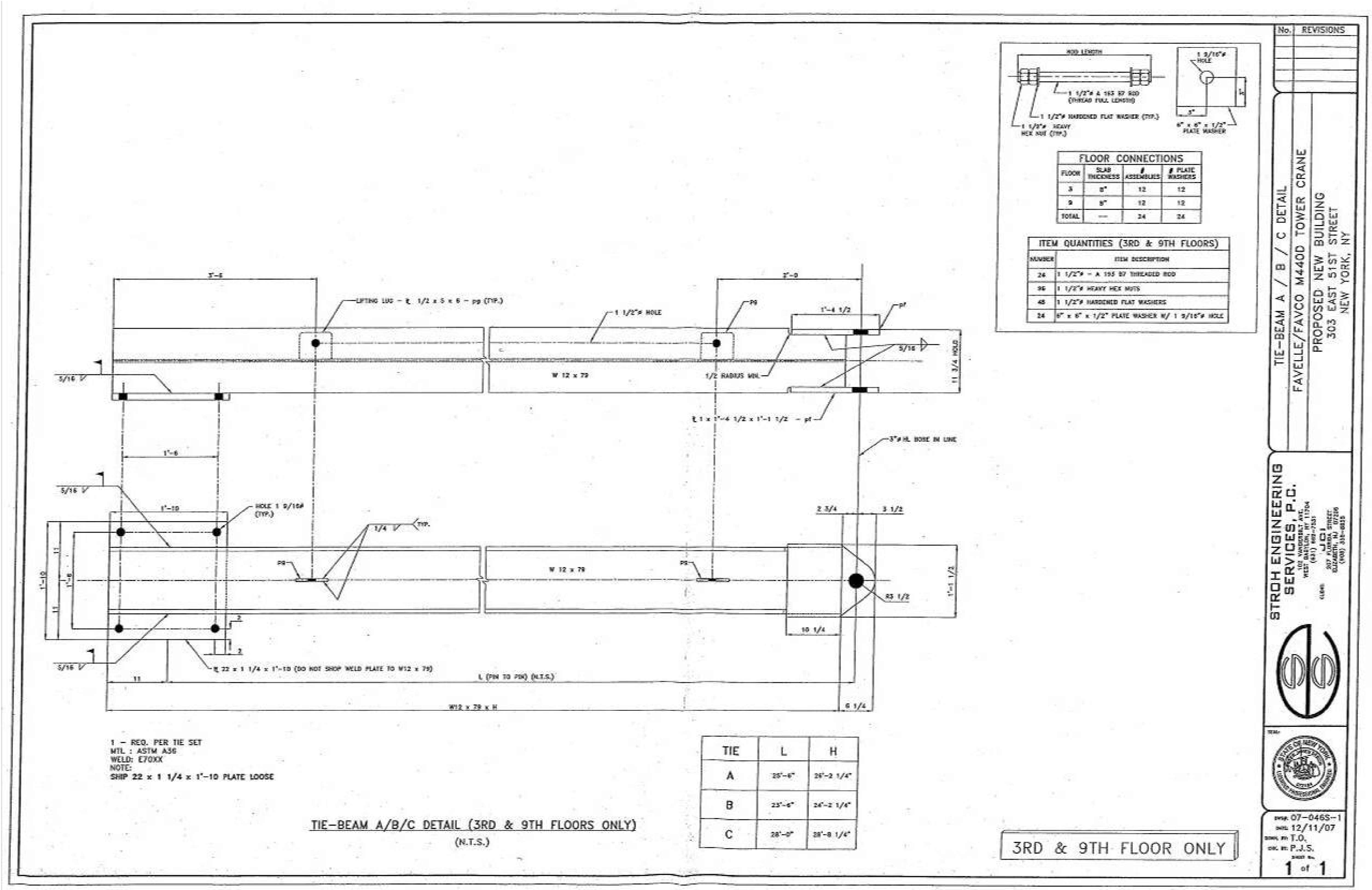


Figure 3.8 - Drawing 07-046S-1, Tie-Beam A/B/C Detail, prepared by the crane engineer Stroh Engineering Services, PC

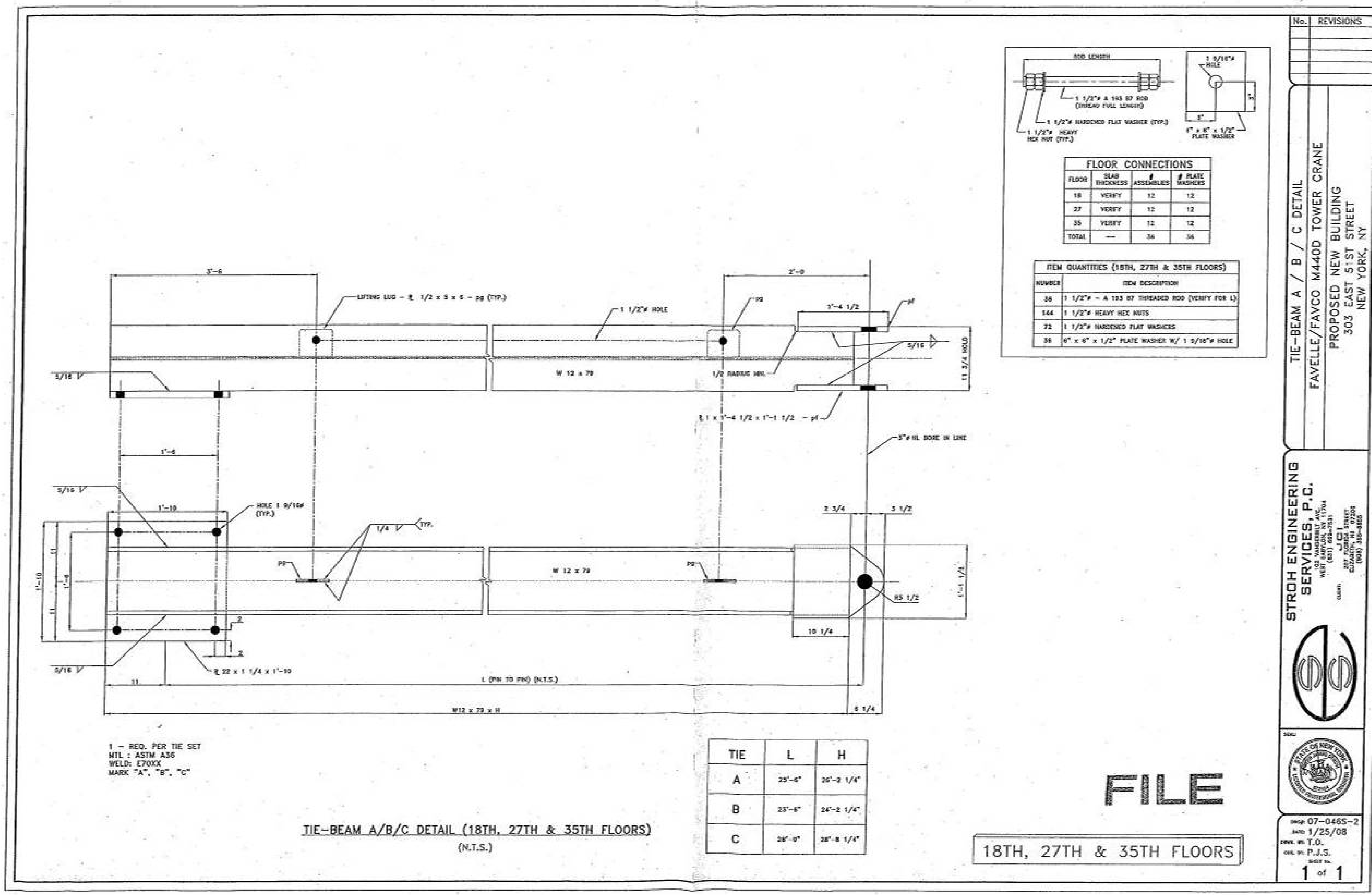


Figure 3.9 - Drawing 07-046S-2, Tie-Beam A/B/C Detail, prepared by the crane engineer Stroh Engineering Services, PC

Photograph 3.20 - Tie-Beam 9A (9th floor level collar, formerly located at the east side of the collar). Collar end. Note the lack of clearance between the end of the failed fillet weld and the radiused cut in the beam flange (at arrow location). Photo by Arup.

Note that there is a specification in Drawing 07-046S-2, Tie-Beam A/B/C Detail, prepared by the crane engineer Stroh Engineering Services, PC, shown in Figures 3.8 and 3.9 to radius the end of the cut flange with a ½ inch minimum radius cut. An example of this fabrication is shown in Photograph 3.20. The beam is specified and was supplied as a W12X79. Two points of reference give a sense of scale to this photograph. First, the beam has a measured flange thickness of 0.74 inches. Second, while the depth of cut is unspecified on the drawings above, it would need to be approximately 1 inch to accommodate the plate (missing in the photograph). It is therefore evident that, while the radius appears to be approximately ½ inch as specified, the clearance which was intended to be provided via the radius is significantly less than ½ inch. Photograph 3.21 below shows a similar connection of an unfailed tie-beam from the 3rd floor level collar.

Photograph 3.21 - Tie-Beam from the 3rd floor level. Collar end. Note the lack of clearance between the end of the failed fillet weld and the radiused cut in the beam flange as well as the excessive cut for the bottom plate. The top plate appears to have been fabricated as specified. Photo by Arup.

The fabrication drawings shown above reference 5/16 inch fillet welds connecting the base plates to the tie-beams at the building end of the tie-beams. It has been noted elsewhere that two of the base plates at the 9th floor level failed as a result of the impact of the 18th floor level collar. Photograph 3.22 below shows a typical measurement of the failed weld on the base plates. The measurement, taken on base plate 9C is 7 mm, or 0.275 inch. This is just under the specified 5/16 inch and represents approximately 11.8% less leg size than specified. Photograph 3.23,- which follows is a similar measurement on the same plate at another location. The latter fillet weld measured as 5 mm, or 0.197 inch. This is approximately 37.0% under the specified 5/16 inch weld. Note that both locations were randomly selected. Observations of base plates 9A and 9C indicated these would be typical measurements.

Photograph 3.22 - Fillet weld measurement of base plate 9C. Measurement is shown to be 7 mm, just under 9/32 inch which is 11.8% under the specified 5/16 inch. Photo by Arup.

Photograph 3.23 - Fillet weld measurement of base plate 9C. Measurement is shown to be 5 mm, just over 3/16 inch, which is 37.0% under the specified 5/16 inch. Photo by Arup.

1.4.2 Contractor's Responsibilities. The Contractor shall be responsible for WPSs, qualification of welding personnel, the Contractor's inspection, and performing work in conformance with the requirements of this code and contract documents.

As noted in the immediately preceding photographs the contract documents; i.e., drawings, were neither followed with respect to the collar end pin connection welds nor with respect to the base plate / tie-beam connection fillet welds.

1.4.3 Inspector's Responsibilities

1.4.3.1 Contractor Inspection. Contractor inspection shall be supplied by the Contractor and shall be performed as necessary to ensure that materials and workmanship meet the requirements of the contract documents.

1.4.3.2 Verification Inspection. The Engineer shall determine if Verification Inspection shall be performed. Responsibilities for Verification Inspection shall be established between the Engineer and the Verification Inspector.

No documentation has been reviewed indicating that verification inspection of the tie-beam welds was to be performed. There has also been no documentation reviewed indicating that contractor inspection of the welds had been undertaken to ensure that materials and workmanship met the requirements of the contract documents, which they did not meet.

1.5 Approval

All references to the need for approval shall be interpreted to mean approval by the Building Commissioner or the Engineer…

It is unknown who, if anybody, had approved the tie-beam welds.

2. Design of Welded Connections

2.0 Scope of Section 2

This section covers requirements for design of welded connections. It is divided into four parts as follows:

Part A-Common Requirements for Design of Welded Connections (Nontubular and Tubular Members)

Part B-Specific Requirements for Design of Nontubular Connections (Statically or Cyclically Loaded). The requirements shall apply in addition to the requirements of Part A.

Part C-Specific Requirements for Design of Nontubular Connections (Cyclically Loaded). When applicable, the requirements shall apply in addition to the requirements of Parts A and B…

Part A - Common Requirements for Design of Welded Connections (Nontubular and Tubular Members)

2.1 Scope of Part A

This part contains requirements applicable to the design of all welded connections of nontubular and tubular structures, independent of loading.

2.2 Contract Plans and Specifications

2.2.1 Plan and Drawing Information. Complete information regarding base metal specification designation (see 3.3 and 4.7.3) location, type, size, and extent of all welds shall be clearly shown on the contract plans and specifications, hereinafter referred to as the contract documents. If the Engineer requires specific welds to be performed in the field, they shall be designated in the contract documents. The fabrication and erection drawings, hereinafter referred to as the shop drawings, shall clearly distinguish between shop and field welds.

These requirements appear to have been met on the fabrication drawings.

2.2.2 Notch Toughness Requirements. If notch toughness of welded joints is required, the Engineer shall specify the minimum absorbed energy with the corresponding test temperature for the filler metal classification to be used, or the Engineer shall specify that the WPSs be qualified with CVN tests. If WPSs with CVN tests are required, the Engineer shall specify the minimum absorbed energy, the test temperature and whether the required CVN test performance is to be in the weld metal, or both in the weld metal and the HAZ (see 4.1.1.3 and Annex 111).

No specification respecting for notch toughness has been reviewed.

2.2.3 Specific Welding Requirements. The Engineer, in the contract documents, and the Contractor, in the shop drawings, shall indicate those joints or groups of joints in which the Engineer or Contractor require a specific assembly order, welding sequence, welding technique or other special precautions.

Specific instructions regarding field welding of the 3rd and 9th floor level tie-beams is indicated on the fabrication drawings.

2.2.4 Weld Size and Length. Contract design drawings shall specify the effective weld length and, for PJP groove welds, the required weld size "(E)." For fillet welds and skewed T-joints, the following shall be provided on the contract documents.

(1) For fillet welds between parts with surfaces meeting at an angle between 80° and 100°, contract documents shall specify the fillet weld leg size…

Fabrication drawing information provides the fillet weld leg size.

2.2.5.3 Symbols…

2.3.2 Fillet Welds

2.3.2.1 Effective Length (Straight). The effective length of a straight fillet weld shall be the overall length of the full size fillet, including end returns. No reduction in effective length shall be assumed in design calculations to allow for the start or stop crater of the weld…

2.3.2.3 Minimum Length. The minimum length of a fillet weld shall be at least four times the nominal size, or the effective size of the weld shall be considered not to exceed 25% of its effective length.

2.3.2.4 Intermittent Fillet Welds (Minimum Length). The minimum length of segments of an intermittent fillet weld shall be 1-1/2 in. [38 mm].

2.3.2.5 Maximum Effective Length. For end-loaded fillet welds with a length up to 100 times the leg dimension, it is allowed to take the effective length equal to the actual length. When the length of end-loaded fillet welds exceeds 100 but not more than 300 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction coefficient P.

where

p = reduction coefficient

L = actual length of end-loaded weld, in. [mm]

w = weld leg size, in. [mm]

When the length exceeds 300 times the leg size, the effective length shall be taken as 180 times the leg size.

2.3.2.6 Calculation of Effective Throat. For fillet welds between parts meeting at angles between 80" and 100" the effective throat shall be taken as the shortest distance from the joint root to the weld face of a 90° diagrammatic weld…

2.3.2.8 Minimum Size. The minimum size fillet weld shall not be smaller than the size required to transmit the applied load nor that provided in 5.14…

2.3.2.10 Effective Area of Fillet Welds. The effective area shall be the effective weld length multiplied by the effective throat…

With the exception of minimum fillet weld size, the foregoing requirements appear to have been met by the actual fabrication of the tie-beam welds as well as on the fabrication drawings.

3.9 Fillet Weld Requirements

See Table 5.8 for minimum fillet weld sizes. <figure given below>

3.9.1 Details (Nontubular). See Table below for the limitations for prequalified fillet welds. <provides maximum allowable fillet sizes>

Table 5.8
Minimum Fillet Weld Sizes (see 5.14)

Base-Metal Thickness (T)[1]		Minimum Size of Fillet Weld[2]	
in.	mm	in.	mm
T ≤ 1/4	T ≤ 6	1/8 (Note 3)	3 (Note 3)
1/4 < T ≤ 1/2	6 < T ≤ 12	3/16	5
1/2 < T ≤ 3/4	12 < T ≤ 20	1/4	6
3/4 < T	20 < T	5/16	8

Notes:
1. For non-low-hydrogen processes without preheat calculated in conformance with 3.5.2, T equals thickness of the thicker part joined; single-pass welds shall be used.
   For non-low-hydrogen processes using procedures established to prevent cracking in conformance with 3.5.2 and for low-hydrogen processes, T equals thickness of the thinner part joined; single-pass requirement shall not apply.
2. Except that the weld size need not exceed the thickness of the thinner part joined.
3. Minimum size for cyclically loaded structures shall be 3/16 in. [5 mm].

The plates welded to the pin end of the tie-beams are specified on the fabrication drawing to be 1 inch. The base plate thickness at the building end of the tie-beam is specified on the fabrication drawing to be 1-1/4 inch. Based upon the above table from AWS D1.1, the minimum allowable size of fillet weld would therefore be 5/16 inch. This minimum requirement was not met by the actual fabricated fillet weld, which was randomly measured to be 5 and 7 mm (approximately 3/16 to 9/32 in.). Macroscopic examination of the welds by ATLSS (see Chapter 9) confirmed that the majority of the welds were 3/16 in.

3.2 Other Documents

3.2.1 Web Sling &Tie Down Association WSTDA-WS- 1 Recommended Standard Specification For Synthetic Web Slings (quoted material in italics, this section)

Much of this standard is incorporated in the 3.1.5 American Society of Mechanical Engineers B30.9-2006 Slings. A selection of relevant material only is therefore presented for discussion at the end of this section and elsewhere in the report.

…

Section 1.3 BASIC SLING TYPES

…

1.3.3 TYPE III - Web sling made with a flat loop eye on each end with loop eye opening on same plane as sling body. This type of sling is sometimes called a flat eye and eye, eye and eye, or double eye sling.

1.3.4 TYPE IV - Web Sling made with both loop eyes formed as in Type III, except that the loop eyes are turned to form a loop eye, which is at a right angle to the plane of the sling body. This type of sling is commonly referred to as a twisted eye sling…

….

CHAPTER 2.0 CONSTRUCTION OF WEB SLINGS

Section 2.1 PURPOSE

2.1.1 This chapter provides an outline of materials and construction characteristics of synthetic web slings.

Section 2.2 WEBBING

2.2.1 The webbing shall be certifiable to tensile strength, have uniform thickness and width, and have selvedges.

….

2.2.5 Class 5 webbing, either loom state or treated, shall have a minimum breaking strength of 6800 pounds p a inch of width112 1.43 kilograms per millimeter of width.

Class 7 webbing, either loom state or treated, shall have a minimum breaking strength of 9800 pounds per inch of width I 175 kilograms per millimeter of width.

…

Section 2.8 DESIGN FACTOR

2.8.1 The design factor for new synthetic web slings with or without fittings shall be a minimum of five (5) when tested in accordance with Section 3.

…

Section 2.10 RATED CAPACITIES

<this section references multiple tables providing rated capacities for various types and configurations of web slings>

…

2.1 0.3 Rated Capacity Determination - The formula for determining the rated capacity of a new web sling is as follows:

Where:

RC=CTS x FE / 5

RC = Rated Capacity

CTS = Certified Tensile Strength of Webbing

FE = Fabrication Efficiency

5 = Design Factor of 5

2.10.4 The choker hitch capacity shall be rated at a maximum of 80% of the vertical capacity…

2.10.6 Rated capacities are affected by angle of lift (sling to load) measured from the horizontal when used with multi-legged slings or choker basket hitches.

To determine the actual sling capacity at a given angle of lift, multiply the original sling rating by the appropriate loss factor determined from the table…

2.10.7 For web slings used in a choker hitch, rated capacities in Tables are for an angle of choke of 120 degrees or greater for the angle formed in the web sling body as it passes through the choking eye….

…

CHAPTER 5.0 OPERATING PRACTICES FOR WEB SLINGS

Section 5.1 PURPOSE

5.1.1 The purpose of this chapter is to provide guidelines for the qualified person responsible for web sling selection, rigging, inspection and use.

Section 5.2 MECHANICAL CONSIDERATIONS

5.2.1 Determine weight of the load. The weight of the load shall be within the rated capacity of the web sling.

5.2.2 Select a web sling having suitable characteristics for the type of load, hitch and environment.

5.2.3 Web slings shall not be loaded in excess of the rated capacity shown on the attached identification tag. Consideration shall be given to the sling to load angle which affects rated capacity.

…

5.2.7 Web slings shall always be protected from being cut or damaged by comers, edges, protrusions or abrasive surfaces with protection sufficient for the intended purpose.

…

5.2.12 Web slings that appear to be damaged shall not be used unless inspected and accepted as usable under Section 5.3, 5.4, and 5.5.

5.2.13 The web sling shall be hitched in a manner providing control of the load…

5.2.17 Shock loading should be avoided.

…

5.2.22 Web slings shall be long enough so that the rated capacity (Working Load Limit) is adequate when the sling to load angle is taken into consideration (See 2.10)

5.2.23 Only web slings with legible identification tags shall be used.

….

5.2.25 Web slings shall not be constricted or bunched between the ears of a clevis or shackle…

Sect ion 5.3 ENVIRONMENTAL CONSIDERATIONS

…

5.3.5 Environments in which synthetic web slings are continuously exposed to ultraviolet light can affect the strength of synthetic web slings in varying degrees ranging from slight to total degradation.

CAUTION: Degradation can take place without visible indications.

a. Factors, which affect the degree of strength loss, are:

1. Length of time of continuous exposure

2. Web sling construction and design

3. Other environmental factors such as weather conditions and geographic location

….

c. Some visual indications of sunlight or ultra-violet degradation are:

1. Bleaching out of web sling color

2. Increased stiffness of web sling material

3. Surface abrasion in areas not normally in contact with the load

d. Proof Testing Warning: Slings used in environments where they are subject to continuous exposure to sunlight or ultra-violet light shall be proof tested to twice the rated capacity semi-annually, or more frequently depending on severity of exposure.

…

Section 5.5 REMOVAL FROM SERVICE

….

5.5.2 A web sling shall be removed from service if any of the following are visible:

a. If sling rated capacity or sling material identification is missing or not readable

…

c. Melting, charring or weld spatters on any part of the web sling

d. Holes, tears, cuts, snags or embedded particles

e. Broken or worn stitching in load bearing splices

f. Excessive abrasive wear

….

i. Any other visible damage that causes doubt as to the strength of the sling

Section 5.6 INSPECTION RECORDS

5.6.1 Written inspection records, utilizing the identification for each sling as established by the user, should be kept on file for all web slings. These records should show a description of the sling and its condition on each periodic inspection.

….

The Web Sling & Tie Down Association (WSTDA) is a technical association dedicated to the development and promotion of voluntary recommended standards and associated reference materials. These “standards” are not American National Institute of Standards (“ANSI”) accredited and, as such, are therefore voluntary, although they may represent industry best practice.

Members of the WSTDA include manufacturers and suppliers of synthetic web slings and tie downs, polyester roundslings, synthetic webbing, fibers, thread and related components. The WSTDA-WS-1 applies to synthetic web slings used for general lifting purposes. Among other purposes, the document provides rated capacity(s) and advice on the use, maintenance and inspection of these slings.

The recommendations contained in the WSTDA-WS-1 recommended standard (see above) are largely adopted in the ASME B30.9 standard without modification. Tabular values for a Class 7, 2-ply, 2-inch webbing slings are also unchanged.

Six types of web slings are defined in the document. Of interest are the following which reflect the slings used for the tower crane collar chain falls:

TYPE III - Web sling made with a flat loop eye on each end with loop eye opening on same plane as sling body. See Photograph 3.24 showing one of the slings used to temporarily support the collars during erection recovered from the collapsed crane.

Photograph 3.24 - OSHA Evidence Item #4A, being the sling taken from a north leg. Photo by Arup.

Class 5 webbing, either loom state or treated, shall have a minimum breaking strength of 6800 pounds per inch of width. Class 5 nylon or polyester webbing is to contain an external black identification marker clearly visible and woven in at least one edge. Class 7 webbing, either loom state or treated, shall have minimum breaking strength of 9800 pounds per inch of width and contains no similar marker.

Polyester web slings as found did not contain a black identification marker, suggesting that the polyester webbing was a Class 7. As the polyester webbing was 2 inches wide and 2-ply, the minimum breaking strength would therefore be:

2 x 2 x 9800 = 39200 lb

The design factor for new synthetic web slings with or without fittings is stated in the standard to be a minimum of five. When determining the rated capacity of a sling, the following formula is to be used:

RC=CTS x FE/5

Where: RC = Rated Capacity

CTS = Certified Tensile Strength of Webbing

FE = Fabrication Efficiency

5 = Design Factor of 5

The rated capacity for the slings would therefore be

39200 x FE / 5 = 7840 x FE lb.

The choker hitch, the method of attachment used for the polyester web slings to the tower, is to have a rated capacity of a maximum of 80% of the vertical capacity. It is noted in the standard that the assumed angle of choke is 120 degrees at the point of choke. These rated capacities are further reduced by the angle of the polyester web sling to the load. At 80 degrees, the loss factor is only 0.985 and for 70 degrees 0.940. However, at 40 degrees the loss factor is 0.574. See Chapter 8, Section 8.2 for additional discussion.

Hence for an angle to the load of 80 degrees, the maximum rated capacity for the polyester web slings in a choker hitch would be

7840 x 0.80 x FE = 6272 lb.

Rated loads for a Class 7, 2-ply, 2-inch Type III, synthetic webbing sling in a choker hitch is given in the WSTDA recommended standard Table 7A as 4960 lb. suggesting an industry standard accepted value of 0.80 for the fabrication efficiency.

Slings observed have a stated rated capacity of 5100 lb and 5000 lb in a choker hitch for the Liftex Corporation and Lift-All Co., Inc. slings, respectively.

Losses can be compounded for a choker hitch, where the end of the sling is passed around the object to be lifted and then through the open eye of the other end of the sling. Section 2.10.7 provides a loss factor for angles less than 120 degrees. For an angle of choke between 90 and 120 degrees the capacity factor is 0.87, for 60-89 degrees it is 0.74 and for 30-59 degrees it is 0.62.

With regard to markers other than the Class type, polyester webbing receives a blue marker. Webbing manufacturer identification markers are also provided and can be located either in the binder or between the surface plies. A marker to identify the synthetic webs ling manufacturer is to be located inside a splice.

The recommended standard also provides guidelines in web sling selection, rigging, inspection and use are also provided. Among those pertinent to the tower failure are the following:

- Web slings shall always be protected from being cut or damaged by corners, edges, protrusions or abrasive surfaces with protection sufficient for the intended purpose.
- Web slings should not be dragged on the floor or over abrasive surfaces
- Web slings that appear to be damaged shall not be used unless inspected and accepted as usable.
- Shock loading should be avoided.
- Environments in which synthetic web slings are continuously exposed to ultraviolet light can affect the strength of synthetic web slings in varying degrees ranging from slight to total degradation. This can take place without visible indications. Factors affecting the degree of strength loss, are:
  - Length of time of continuous exposure
  - Web sling construction and design
  - Other environmental factors such as weather conditions and geographic location
- Some visual indications of sunlight or ultra-violet degradation are:
  - Bleaching out of web sling color
  - Increased stiffness of web sling material
  - Surface abrasion in areas not normally in contact with the load

Various levels of inspection are described.

Types of inspection include initial inspection, frequent inspection and periodic inspection. The frequency of inspection should be based on the frequency of web sling use and the severity of service conditions, combined with experience gained on the service life of web slings used in similar applications. It is also recommended that inspections be conducted at least annually

There are various visible indications for retiring a sling provided. Among these are the following:

- If sling rated capacity or sling material identification is missing or not readable
- Holes, tears, cuts, snags or embedded particles
- Excessive abrasive wear
- Any visible damage that causes doubt as to the strength of the sling

Finally, in section, it is stated that:

“Written inspection records, utilizing the identification for each sling as established by the user, should be kept on file for all web slings. These records should show a description of the sling and its condition on each periodic inspection.”

Such records have not been made available for review.

3.2.2 OSHA Regulations

Occupational Safety & Health Administration ("OSHA") regulations pertaining to Tower Cranes and to Slings were reviewed as part of this investigation. Per http://osha.gov/oshinfo/mission.html, OSHA's role is

> *"…to promote the safety and health of America's working men and women by setting and enforcing standards; providing training, outreach and education; establishing partnerships; and encouraging continual process improvement in workplace safety and health."*

As stated in http://osha.gov/as/opa/osha-faq.html, OSHA's mission is *"to prevent work-related injuries, illnesses, and deaths."*

OSHA's top inspection priorities are

> *"…reports of imminent dangers-accidents about to happen; second are fatalities or accidents serious enough to send three or more employees to the hospital. Third are employee complaints. Referrals from other government agencies are fourth. Fifth are targeted inspections-such as the Site Specific Targeting Program, which focuses on employers that report high injury and illness rates, and special emphasis programs that zero in on hazardous work such as trenching or equipment such as mechanical power presses. Follow-up inspections are the final priority."*

Penalties for violating an OSHA standard depend *"upon how likely the violation is to result in serious harm to employees."*

OSHA regulations therefore are focused on workplace safety, not on engineering design. Those regulations which were found to be pertinent to the investigation of the tower crane collapse are given below. As these are largely contained in the earlier referenced codes and standards, they are presented for information without additional commentary. OSHA undertook a review of the compliance of these regulations with regard to the collapse of the tower crane. The issue of compliance is presented in a report issued by OSHA to which Arup defers.

3.2.2.1 OSHA 29 CFR Part 1910 Subpart N Section 1910.184 - Slings

> *1910.184(a)*
>
> *Scope. This section applies to slings used in conjunction with other material handling equipment for the movement of material by hoisting, in employments covered by this part. The types of slings covered are those made from alloy steel chain, wire rope, metal mesh, natural or synthetic fiber rope (conventional three strand construction), and synthetic web (nylon, polyester, and polypropylene).*
>
> *1910.184(b)*
>
> *Definitions.*
>
> *Angle of loading is the inclination of a leg or branch of a sling measured from the horizontal or vertical plane as shown in Fig. N-184-5; provided that an angle*

of loading of five degrees or less from the vertical may be considered a vertical angle of loading.

…

Cable laid rope is a wire rope composed of six wire ropes wrapped around a fiber or wire rope core.

Cable laid rope sling-mechanical joint is a wire rope sling made from a cable laid rope with eyes fabricated by pressing or swaging one or more metal sleeves over the rope junction.

Choker hitch is a sling configuration with one end of the sling passing under the load and through an end attachment, handle or eye on the other end of the sling.

…

Designated means selected or assigned by the employer or the employer's representative as being qualified to perform specific duties.

Equivalent entity is a person or organization (including an employer) which, by possession of equipment, technical knowledge and skills, can perform with equal competence the same repairs and tests as the person or organization with which it is equated.

…

Hitch is a sling configuration whereby the sling is fastened to an object or load, either directly to it or around it.

…

Proof load is the load applied in performance of a proof test.

Proof test is a nondestructive tension test performed by the sling manufacturer or an equivalent entity to verify construction and workmanship of a sling.

Rated capacity or working load limit is the maximum working load permitted by the provisions of this section.

…

Selvage edge is the finished edge of synthetic webbing designed to prevent unraveling.

Sling is an assembly which connects the load to the material handling equipment.

Sling manufacturer is a person or organization that assembles sling components into their final form for sale to users.

…

Strand laid rope is a wire rope made with strands (usually six or eight) wrapped around a fiber core, wire strand core, or independent wire rope core (IWRC).

Vertical hitch is a method of supporting a load by a single, vertical part or leg of the sling.

Safe operating practices. Whenever any sling is used, the following practices shall be observed:

1910.184(c)(1)

Slings that are damaged or defective shall not be used.

1910.184(c)(2)

Slings shall not be shortened with knots or bolts or other makeshift devices.

1910.184(c)(3)

Sling legs shall not be kinked.

1910.184(c)(4)

Slings shall not be loaded in excess of their rated capacities.

1910.184(c)(5)

Slings used in a basket hitch shall have the loads balanced to prevent slippage.

1910.184(c)(6)

Slings shall be securely attached to their loads.

1910.184(c)(7)

Slings shall be padded or protected from the sharp edges of their loads.

1910.184(c)(8)

Suspended loads shall be kept clear of all obstructions.

1910.184(c)(9)

All employees shall be kept clear of loads about to be lifted and of suspended loads.

1910.184(c)(10)

Hands or fingers shall not be placed between the sling and its load while the sling is being tightened around the load.

1910.184(c)(11)

Shock loading is prohibited.

1910.184(c)(12)

A sling shall not be pulled from under a load when the load is resting on the sling.

1910.184(d)

Inspections. Each day before being used, the sling and all fastenings and attachments shall be inspected for damage or defects by a competent person designated by the employer. Additional inspections shall be performed during sling use, where service conditions warrant. Damaged or defective slings shall be immediately removed from service.

…

1910.184(f)

Wire rope slings.

1910.184(f)(1)

Sling use. Wire rope slings shall not be used with loads in excess of the rated capacities shown in Tables N-184-3 through N-184-14. Slings not included in these tables shall be used only in accordance with the manufacturer's recommendations.

1910.184(f)(2)

Minimum sling lengths.

1910.184(f)(2)(i)

Cable laid and 6x19 and 6x37 slings shall have a minimum clear length of wire rope 10 times the component rope diameter between splices, sleeves or end fittings.

1910.184(f)(2)(ii)

Braided slings shall have a minimum clear length of wire rope 40 times the component rope diameter between the loops or end fittings.

1910.184(f)(2)(iii)

Cable laid grommets, strand laid grommets and endless slings shall have a minimum circumferential length of 96 times their body diameter.

1910.184(f)(3)

Safe operating temperatures. Fiber core wire rope slings of all grades shall be permanently removed from service if they are exposed to temperatures in excess of 200 deg. F. When non fiber core wire rope slings of any grade are used at temperatures above 400 deg. F or below minus 60 deg. F, recommendations of the sling manufacturer regarding use at that temperature shall be followed.

1910.184(f)(4)

End attachments.

1910.184(f)(4)(i)

Welding of end attachments, except covers to thimbles, shall be performed prior to the assembly of the sling.

1910.184(f)(4)(ii)

All welded end attachments shall not be used unless proof tested by the manufacturer or equivalent entity at twice their rated capacity prior to initial use. The employer shall retain a certificate of the proof test, and make it available for examination.

1910.184(f)(5)

Removal from service. Wire rope slings shall be immediately removed from service if any of the following conditions are present:

1910.184(f)(5)(i)

Slings. - 1910.184 Page 21 of 40 Ten randomly distributed broken wires in one rope lay, or five broken wires in one strand in one rope lay.

1910.184(f)(5)(ii)

Wear or scraping of one-third the original diameter of outside individual wires.

1910.184(f)(5)(iii)

Kinking, crushing, bird caging or any other damage resulting in distortion of the wire rope structure.

1910.184(f)(5)(iv)

Evidence of heat damage.

1910.184(f)(5)(v)

End attachments that are cracked, deformed or worn.

1910.184(f)(5)(vi)

Hooks that have been opened more than 15 percent of the normal throat opening measured at the narrowest point or twisted more than 10 degrees from the plane of the unbent hook.

1910.184(f)(5)(vii)

Corrosion of the rope or end attachments.

…

1910.184(i)

Synthetic web slings --

1910.184(i)(1)

Sling identification. Each sling shall be marked or coded to show the rated capacities for each type of hitch and type of synthetic web material.

1910.184(i)(2)

Webbing. Synthetic webbing shall be of uniform thickness and width and selvage edges

shall not be split from the webbing's width.

1910.184(i)(3)

Fittings. Fittings shall be:

1910.184(i)(3)(i)

Of a minimum breaking strength equal to that of the sling; and

1910.184(i)(3)(ii)

Free of all sharp edges that could in any way damage the webbing.

1910.184(i)(4)

Attachment of end fittings to webbing and formation of eyes. Stitching shall be the only method used to attach end fittings to webbing and to form eyes. The thread shall be in an even pattern and contain a sufficient number of stitches to develop the full breaking strength of the sling.

1910.184(i)(5)

Sling use. Synthetic web slings illustrated in Fig. N-184-6 shall not be used with loads in excess of the rated capacities specified in Tables N-184-20 through N-184-22. Slings not included in these tables shall be used only in accordance with the manufacturer's recommendations.

1910.184(i)(6)

Environmental conditions. When synthetic web slings are used, the following precautions shall be taken:

1910.184(i)(6)(i)

Nylon web slings shall not be used where fumes, vapors, sprays, mists or liquids of acids or phenolics are present.

1910.184(i)(6)(ii)

Polyester and polypropylene web slings shall not be used where fumes, vapors, sprays, mists or liquids of caustics are present.

1910.184(i)(6)(iii)

Web slings with aluminum fittings shall not be used where fumes, vapors, sprays, mists or liquids of caustics are present.

1910.184(i)(7)

Safe operating temperatures. Synthetic web slings of polyester and nylon shall not be used at temperatures in excess of 180 deg. F. Polypropylene web slings shall not be used at temperatures in excess of 200 deg. F.

1910.184(i)(8)

Repairs.

1910.184(i)(8)(i)

Synthetic web slings which are repaired shall not be used unless repaired by a sling manufacturer or an equivalent entity.

1910.184(i)(8)(ii)

Each repaired sling shall be proof tested by the manufacturer or equivalent entity to twice the rated capacity prior to its return to service. The employer shall retain a certificate of the proof test and make it available for examination.

1910.184(i)(8)(iii)

Slings, including webbing and fittings, which have been repaired in a temporary manner shall not be used.

1910.184(i)(9)

Removal from service. Synthetic web slings shall be immediately removed from service if any of the following conditions are present:

1910.184(i)(9)(i)

Acid or caustic burns;

1910.184(i)(9)(ii)

Melting or charring of any part of the sling surface;

1910.184(i)(9)(iii)

Snags, punctures, tears or cuts;

1910.184(i)(9)(iv)

Broken or worn stitches; or

1910.184(i)(9)(v)

Distortion of fittings.

OSHA 29 CFR Part 1926 Subpart N Section 1926.550 – Cranes and Derricks

Clauses of relevance to Tower Cranes are given below.

1926.550(a)

General requirements.

1926.550(a)(1)

The employer shall comply with the manufacturer's specifications and limitations applicable to the operation of any and all cranes and derricks. Where manufacturer's specifications are not available, the limitations assigned to the equipment shall be based on the determinations of a qualified engineer competent in this field and such determinations will be appropriately documented and recorded. Attachments used with cranes shall not exceed the capacity, rating, or scope recommended by the manufacturer.

1926.550(a)(2)

Rated load capacities, and recommended operating speeds, special hazard warnings, or instruction, shall be conspicuously posted on all equipment. Instructions or warnings shall be visible to the operator while he is at his control station.

1926.550(a)(3)

[Reserved]

1926.550(a)(4)

Hand signals to crane and derrick operators shall be those prescribed by the applicable ANSI standard for the type of crane in use. An illustration of the signals shall be posted at the job site.

1926.550(a)(5)

The employer shall designate a competent person who shall inspect all machinery and equipment prior to each use, and during use, to make sure it is in safe operating condition.

Any deficiencies shall be repaired, or defective parts replaced, before continued use.

1926.550(a)(6)

A thorough, annual inspection of the hoisting machinery shall be made by a competent person, or by a government or private agency recognized by the U.S. Department of Labor. The employer shall maintain a record of the dates and results of inspections for each hoisting machine and piece of equipment.

1926.550(a)(7)

Wire rope shall be taken out of service when any of the following conditions exist:

1926.550(a)(7)(i)

In running ropes, six randomly distributed broken wires in one lay or three broken wires in one strand in one lay;

1926.550(a)(7)(ii)

Wear of one-third the original diameter of outside individual wires. Kinking, crushing, bird caging, or any other damage resulting in distortion of the rope structure;

1926.550(a)(7)(iii)

Evidence of any heat damage from any cause;

…

1926.550(a)(7)(iv)

Reductions from nominal diameter of more than one-sixty-fourth inch for diameters up to and including five-sixteenths inch, one-thirty-second inch for diameters three-eighths inch to and including one-half inch, three-sixty-fourths inch for diameters nine-sixteenths inch to and including three-fourths inch, one-sixteenth inch for diameters seven-eighths inch to 1 1/8 inches inclusive, three-thirty-seconds inch for diameters 1 1/4 to 1 1/2 inches inclusive;

1926.550(a)(7)(v)

In standing ropes, more than two broken wires in one lay in sections beyond end connections or more than one broken wire at an end connection.

1926.550(a)(7)(vi)

Wire rope safety factors shall be in accordance with American National Standards Institute B 30.5-1968 or SAE J959-1966.

…

1926.550(a)(16)

No modifications or additions which affect the capacity or safe operation of the equipment shall be made by the employer without the manufacturer's written approval. If such modifications or changes are made, the capacity, operation, and maintenance instruction plates, tags, or decals, shall be changed accordingly. In no case shall the original safety factor of the equipment be reduced.

…

1926.550(g)

Crane or derrick suspended personnel platforms -

…

1926.550(g)(7)(i)

Hoisting of employees while the crane is traveling is prohibited, except for portal, tower and locomotive cranes, or where the employer demonstrates that there is no less hazardous way to perform the work.

4 Permitting Review

This Section of the report:

- Identifies the New York City Building Code (NYCBC) requirements for licensure of crane operators and riggers and for permitting of cranes; and
- Compares the records on file at the New York City Department of Buildings (NYCDOB) to the previously identified licensing/permitting requirements.

This Section of the report does not provide a thorough assessment of the quality of the information (designs) that were submitted; this will be addressed elsewhere in this report, specifically in Section 5. Rather, the purpose of this Section is to confirm whether or not all of the required information was submitted as per the relevant Codes and Standards to obtain the necessary licenses and permits.

4.1 Licensing of Hoisting Machine Operator and Rigger

The following sections of the New York City Building Code pertain to licensure of Hoisting Machine Operators (HMOs) and Riggers:

- Title 26, Subchapter 2, Article 5 – Hoisting Machine Operator's License
- Title 26, Subchapter 2, Article 6 – Rigger License

The above sections of the Code are addressed in the sections that follow.

4.1.1 Hoisting Machine Operator's License (Title 26, Subchapter 2, Article 5)

A hoisting machine operator's (HMO) license is required for the person(s) operating "power-operated hoisting machine used for hoisting purposes…except power operated scaffolds and window washing machines".

Three classes of licensure currently exist:

- Class A: Basic HMO License
- Mobile Cranes: Maximum of 200-ft of total boom with unlimited lifting capacity
- Tower Cranes: Total boom of 200-ft or less (does not include mast)
- Class B: Unlimited HMO License
- Class C: Special HMO License
- Class C1: Wheel mounted crane 200-ft of total boom and maximum lifting capacity of 50 tons.
- Class C2: Truck mounted crane with 200-ft of total boom and maximum lifting capacity of 50 tons
- Class C3: Truck mounted crane with 135-ft of total boom and maximum lifting capacity of 3 tons.

Either a Class A or a Class B license would have been required to lawfully operate the tower crane on the site, which had a boom length of 135-ft. The crane operator, Wayne Bleidner, had a valid Class B HMO license at the time of the collapse (i.e., License No. 005499, valid until August 31, 2008). A copy of the license information on-file at the NYCDOB is given in Appendix A.

4.1.2 Rigger's License (Title 26, Subchapter 2, Article 6 & Appendix A, Chapter 25)

A rigger's license is required to hoist or lower any article on the outside of a building, with the following exceptions:

- Signs under supervision of a licensed sign hanger
- Building material or equipment, other than boilers or tanks, in the course of construction or alteration.

There are two classifications of rigger licenses defined in Article 6;

- Master Rigger – Unlimited lifting capacity
- Special Rigger – Limited to 1200 lbs.
- There is also a specialist rigger defined in Appendix A, Chapter 25;
- Tower Crane Rigger

In accordance with Article 6 and Chapter 25 and specifically 26-172 of Article 6; the erection or dismantling of a Tower Crane must be performed or supervised by either a Master Rigger or a Tower Crane Rigger.

The requirements for a Master Rigger's license are defined in 26-176 and are as follows:

- Satisfactory proof establishing that the applicant has had at least 5 years' practical experience in the hoisting and rigging business;
- The applicant shall also have knowledge of and be able to explain the risks incident to such business and precautions to be taken in connection therewith, safe loads and computation thereof, types of rigging, size and strength of ropes, cables, blocks, poles, derricks, sheerlegs and other tools used in connection with such business.

In addition, the Department of Buildings has confirmed that applicants are required to pass a test to become licensed as a Master Rigger.

The requirements for a Tower Crane Rigger's license are defined in Appendix A, Chapter 25 and are as follows:

- Be able to read, write and speak the English language.
- Be able to interpret structural and erection drawings.
- Have at least five years of supervisory experience within the last 10 years in the planning and execution of the erection or dismantling of tower and climber cranes; or for a period of five years within the last 10 years, an applicant shall have erected or dismantled, as part of a team, eight or more tower and/or climber cranes of which at least three erections and dismantlings of such cranes shall be under his supervision, and/or oversee the safety and code requirements for the same.
- Applicants shall be required to have passed a written and practical examination no more than one year prior to the application filing date. Prior to being eligible to take such examination, an applicant must submit satisfactory evidence that: (1) the applicant has at least five years of supervisor experience in the planning and execution of the erecting and dismantling of tower or climber cranes; or (2) the applicant has at least five years of practical experience working as part of a team

erecting and dismantling tower cranes and has participated in at least eight such erections or dismantlings.

- Licenses issued under the above stated rules may be renewed annually without examination.
- Failure to renew this license annually shall require an examination or re-examination as appropriate. Renewal applications shall be submitted between 30 and 60 days prior to the expiration date of the license.

The rigger, William Rapetti, had a valid Tower Crane Rigger's license at the time of the collapse (i.e., License No. 000021, valid until July 31, 2008). A copy of the license information on-file at the NYCDOB is given in Appendix A.

4.2 Permitting of Crane

The following sections of the New York City Building Code (NYCBC) pertain to permitting of cranes:

- Title 27, Subchapter 19, article 10 – Material Handling and Hoisting Equipment
- Reference Standard RS 19-2

There are effectively three steps to permitting a crane for operation in New York City:

- Certificate of Approval
- Certificate of Operation
- Certificate of On-Site Inspection

These steps are outlined in 27-1057 and more specific requirements are given in RS 19-2.

RS 19-2 was amended in 2006, therefore the Certificate of Approval process followed the old requirements and the Certificate of Operation and Certificate of On-Site Inspection processes followed the newer requirements.

This review of each step is based on the requirements of the RS19-2 that was in effect at the time.

The specific requirements for each step are a little confusing as the terminology is not consistent and the various steps and sub steps are also known by other common names. The Department of Buildings has produced further documentation to clarify.

The three steps are explained in more detail in the Sections that follow.

4.2.1 Certificate of Approval

The process for obtaining a Certificate of Approval is described in 27-1057(b). The specific requirements are given in Section 3.1 of RS 19-2 and are addressed later in this Section (see 3.2.1.2).

The Certificate of Approval is also commonly known as 'Prototype Approval' or 'Prototype Application' or 'Prototype Number'. This step includes the review of the crane manufacturer's engineering and testing for compliance with code requirements and to establish acceptable boom lengths and safe working loads for the crane (i.e., load capacity charts). If the process is completed satisfactorily, approval is given. A prototype number is assigned to the application when it is received by the DOB. This is for reference and tracking purposes. The issuance of a prototype number does not mean approval has been given. The Certificate of Approval process need only be completed once for a particular make & model of crane (or derrick); subsequent cranes of the same make & model can simply reference the approved Prototype Number. However, if a previously approved model of crane is modified or altered to increase the boom length, jibs or any extensions to the boom beyond the maximum approved length, or when the load ratings are increased beyond the maximum approved ratings, then an amendment shall be filed with the Department of Buildings.

Administratively, the Certificate of Approval application is made using form CD-1.

4.2.1.1 Review of Certificate of Approval Process for the Collapsed Crane

The Certificate of Approval application for the collapsed crane was originally submitted by Mr. Peter Stroh, who at the time was in the employ of MRA Engineering of Hempstead, New York. The application was submitted on behalf of Favelle Favco Cranes (USA) Inc. of Harlingen, Texas for an M440-D tower crane, which is a diesel/hydraulic luffing tower crane with a lattice boom.

The original application is dated November 13, 1998 and stamped “Received” by the NYCDOB Cranes & Derricks Division the same day. A receipt for $4,000.00 was issued on November 13, 1998 by the Department of Buildings cashier (Invoice No. 60185880).

The cover letter that accompanies the original application states that the application covers lattice booms ranging from 105-ft to 226-ft in length (i.e., 105-ft (32m); 120-ft (36.6m); 135-ft (41.2m); 150-ft (45.8m); 165-ft (50.4m); 180-ft (55m); 196-ft (59.6m); 210-ft (64m); 226-ft (68.8m)). The letter indicates that details of a fixed length boom extension will be submitted in the future.

The prototype application form (i.e., NYCDOB official form) and the payment receipt identify the prototype number for the crane as “P331”. The form quotes the tower sections as having a depth of 8’-11 1/8”, a width of 8’-10 5/8” and a height of 157’-6”, which is consistent with a 441 tower section or possibly a 442 tower section but not a 393 tower section or a TG1900 tower section. Details of the 393 tower were submitted at a later date.

The cover letter states that the application will be filed with several tower options: (tower section numbers) 441, 442, 393 and TG1900. The letter explains that the details of tower sections 441 and 442 are submitted at present and that the details of tower sections 393 and TG1900 will be submitted at a later date. The details of sections 393 and TG1900 were subsequently submitted by Peter Stroh under the cover of a letter dated December 3, 1998.

The NYCDOB C&D Division responded to the original application by means of an “Objection Sheet” dated December 11, 1998. The objection sheet consisted of the following objections:

1) Provide a copy of (Australian Standard) AS-1418 part 1994 and AS-1418 Part 4-1988.
2) Provide information for boom, mast (tower) member size, cross section area, elastic properties.

3) Submit a list of chemical component of steel compared to ASTM Standard. Use of foreign steel must be approved by the Department as per Article 5.0 of RS-19.
4) Approved safety device required for tower crane as per Article 19.
5) Provide a sketch showing the center-of-gravity of boom, mast, machinery deck slew mount.
6) Provide wind force calculation, wind area for upper tower crane, tower.
7) Provide manual calculation to verify the boom computer output (in-service and out-of-service).
8) Provide information of 10-ft jib boom (calculations and drawings).
9) Check connection bolt between tower section while under tension and shear (out-of-service per 1.6.3. of AISC).

The response to the objections consisted of the following:

- Peter Stroh submitted calculations and sketches for the boom extension (i.e., “jib boom” as per Objection (8) above), as promised in the cover letter that accompanied his original November 1998 application. The calculations submitted included design checks for combined stresses (i.e., treated as a beam column as required by RS-19-2 Article 4.0).
- Peter Stroh submitted an itemized response to the above objections on December 7, 1999. A comparison of the responses to the objections is as follows (refer to original object numbers above):

1) The letter states that AS 1418.1 (1994) and AS 1418.4 (1988) are submitted as requested. (Note: This is likely since these standards were on file with NYCDOB C&D and have been re-produced by NYCDOB C&D for this investigation.)
2) The letter states the boom and tower member sizes and section properties are listed with the analysis for the appropriate crane component. The letter refers the reader to the appropriate section of the original submittal for this information.
3) A copy of the steel composition and mechanical properties was included with the letter.
4) The letter informs that the crane is equipped with a Robway/FFC RCI 3100 Crane Safety System and that this system has been filed with MEA for approval.
5) The letter informs that the center-of-gravity information requested will be provided with the filing for the tower crane erection; since the components can be sub-assembled, the erection plan is dependent on site conditions. (In other words, this is an issue for the Certificate of On-Site Inspection as opposed to the Certificate of Approval.)
6) The attachments to the letter include wind area calculations for the tower sections and the upper crane, however, wind force calculations are not provided as requested. It is noted that wind force calculations were submitted as part of the subsequent Certificate of Operation process.
7) The attachments to the letter include a manual verification of the boom analysis software output.

8) The letter refers to the previous letter dated February 10, 1999 for information on the 10-foot boom extension.

9) The attachments to the letter include calculations for combined shear and tension on the tower bolts (i.e., for both in-service and out-of-service loads for each of the four proposed tower types: 441, 442, 393 and TG1900).

The tower wind area calculations (Objection (6) above) are of interest. The calculations reference the following drawings for the various tower sections submitted under the application:

- Tower 441 - Reference Drawing A3-7150.077
- Tower 442 - Reference Drawing A3-7150.101
- Tower 393 - Reference Drawing A1-7150.141
- Tower TG 1900 – Reference Drawing 76-A-12580

The above referenced drawings were not submitted as part of the original application on file at NYCDOB. Drawings were provided for Tower 441 and 442, however, the drawing numbers do not match those quoted above (i.e., A1-7150.070 Issue A (2 pages) was provided for Tower 441 and A1-7150.015 Issue F (1 page) was provided for Tower 442). The original application does not contain any drawings for Tower 393 or TG 1900, however, information on these tower sections was provided under cover of a letter from Peter Stroh dated December 3, 1998. NYCDOB obtained Drawing A1-7150.141 for Tower 393, the tower section type involved in the collapse, during the course of this investigation. These submitted drawings were adequate for the Prototype approval.

The responses to the objections were deemed acceptable and the Favelle/Favco Tower Crane Model No. 440-D was approved for use in New York City on March 3, 2000 by Leo Y. Lee, Director, Cranes & Derricks. Approval was based on the above quoted boom lengths, load capacity charts and four (4) tower types. The confirmation of approval and Prototype Number (P331) was annotated and dated by DOB on the approved load capacity charts.

4.2.1.2 Compliance with RS-19-2 Requirements for Certificate of Approval

A comparison of the relevant requirements of RS-19-2 to the approved application (i.e., original submission plus responses to objections) is given in Table 4.1 below:

Table 4.1 - Comparison of the Relevant Requirements of RS-19-2 to the approved application. (Note: Reference Standard RS-19-2 has been revised since the Certificate of Approval was issued for the crane model in question (i.e., revised 2006). This table addresses the requirements that were in place at the time the crane was approved as opposed to the requirements in the most recent version of the Reference Standard.) Prepared by Arup.

RS-19-2 Para. No.	Requirement	Compliance
3.1.1.1.a	Design calculations for boom or mast	Yes. Analysis provided for both the boom (for all boom lengths considered) and the mast.[1] Analysis is also provided for the tower sections[2] and the slew

RS-19-2 Para. No.	Requirement	Compliance
		mount.
3.1.1.1.b	Design calculations for jib or extensions	Yes.
3.1.1.1.c	Design calculations for gantries	Not applicable to this crane.
3.1.1.1.d.	Design calculations for counterweight supports and attachments	Yes. Analysis provided for the machinery deck, which supports the counterweights.
3.1.1.1.e.	Design calculations for rope.	Yes. Analysis provided for the main hoist, luff ropes and pendent ropes.
3.1.1.1.f.	Design calculations for overturning stability.[3]	Design calculations are provided to calculate the overturning moment at the underside of the slew mount.[4] Design calculations are also provided to calculate overturning moments at the base of freestanding towers of various heights (i.e., 9 to 12 tower sections tall).[5]
3.1.1.2	Load test reports where required under this reference standard.	The approval of the prototype submission was based on the design calculations and tower crane manufacturer's information. As mentioned at the start of this Section, load tests are typically not performed until the crane is erected at a production site.
3.1.1.3	A material specification statement certifying the type of steel used in the construction of items 3.1.1.1.a through 3.1.1.1.e.	Yes. (This was provided in response to an NYCDOB objection.)
3.1.1.4	A list of standards used in design and testing.	Yes. (Australian Standards were provided in response to an NYCDOB objection.)
4.0	The boom shall be designed as a beam-column, subject simultaneously to axial force, torsion, and bending.	Yes. The calculations considered simultaneous axial force, bending and torsion.
4.0 (continued)	Stress induced in the boom by its movement and due to wind during	Yes. The calculations considered wind loadings. The calculations considered both

RS-19-2 Para. No.	Requirement	Compliance
	normal operation should be considered.	"in-service" and "out-of-service" loadings.
5.0	All steel used in the construction of a crane or derrick not specifically accepted for use by the New York City Building Code, or an appropriate reference standard thereof, must be approved by the commissioner.	Yes. The specifications for the steel used for the tower crane in question were submitted in response to a NYCDOB objection and subsequently accepted by NYCDOB.
16.1.1.	In addition to prototype tests and quality control measures, each new production crane and derrick shall be tested by the manufacturer to the extent necessary to insure compliance with the operational requirements of this section, including functions such as the following: (a) Load hoisting and lowering mechanisms; (b) Boom hoisting and lowering mechanisms; (c) Swinging mechanism; (d) Travelling mechanism; (e) Safety devices.	Load charts were provided but no other evidence of load tests was submitted. As mentioned at the start of this Section, load tests are typically not performed until the crane is erected at a production site.
16.1.3.	Operational test and production test results certified by the manufacturer or a licensed professional engineer shall be made available to the department with each application for a certificate of approval for a crane or derrick in accordance with 3.0.	Load charts were provided but no other evidence of load tests was submitted. As mentioned at the start of this Section, load tests are typically not performed until the crane is erected at a production site.

Notes to Table Above:

1) RS-19-2 refers to the tower sections as the "mast", whereas the crane manufacturer uses the term "mast" to describe the A-frame at the top of the machine deck. The term in Table 4.1 refers to the A-frame

2) Analysis has been provided for all four tower sections considered for the application: 441, 442, 393 and TG 1900. The analysis for the 393 and TG1900 tower sections were not submitted with the original November 1998 application but rather arrived under the cover of a letter from Peter Stroh (MRA Engineering) to Kwan Kuo (NYCDOB C&D) dated December 3, 1998. The measurements given in the application for a 393 tower crane (i.e., member size, chord length, section height) are consistent with measurements performed on tower sections in the field by Arup during site visits to the crane yard in South Kearny, NJ on April 25, 2008 and to the OEM warehouse on July 10, 2008.

3) The requirement to provide design calculations for overturning stability as part of the application for the Certificate of Approval is often not practical, as it is not possible to calculate the (global) overturning stability of a crane without knowing its support conditions, which are site dependent and therefore the dominion of the Certificate of On-Site Inspection.

4) The overturning moments that were calculated for the slew mount were not used to determine the overturning stability per say, but rather to analyze the slew mount itself. (The slew mount was designed for a moment of 551.65 mT (3,990 kip-ft), which is based on the maximum dynamic loading for "a load case 11.8 tonnes (26 kips) for a 40.0 m (131.2 ft) radius and a 59.6 m (195.5') boom.

5) The overturning moments at the base of the freestanding crane were calculated to check the structural capacity of the tower sections, not to determine the overturning stability of the structure. The crane manufacturer's submission provides foundation reactions (i.e., normal force, shear force, overturning moment and maximum tension/compression at a corner) for freestanding towers of limited heights (i.e., maximum tower heights of 36 m to 44 m (118 ft to 144 ft) depending on tower section type). Foundation reactions are not given for laterally supported cranes (i.e., attached to adjacent structures) because this information is dependent on the number and location of lateral support ties, which is a variable for the crane engineer's site-specific design (by Stroh Engineering).

4.2.1.3 Summary of Certificate of Approval Review

The Certificate of Approval application process generally addresses the requirements of the relevant local codes and standards (i.e., NYCBC Title 72, Subchapter 19, Article 10 and Reference Standard RS-19-2). Furthermore, NYCDOB reviewers appear to have performed a reasonably thorough review of the application and actively sought to enforce the local codes and standards.

A valid Certificate of Approval (Prototype Number P331) and approved load charts were issued on March 3, 2000 in general accordance with the code requirements.

4.2.2 Certificate of Operation

The Certificate of Operation covers the process of determining if a specific crane is in a safe working condition. The process for obtaining a Certificate of Operation is defined in 27-1057(c). This requires that full approval for a Certificate of Operation is only given after inspections and tests have confirmed that the crane is in a safe operating condition. However, this was written with regard to mobile cranes. For tower cranes it is not possible or practical to satisfactorily inspect or test the crane until it has arrived at a job site. The Department of Buildings has therefore developed an adapted two stage process for issuing Certificates of Operation to tower cranes. This is explained in the following paragraphs.

The first stage is purely an administrative process. The crane is registered with New York City and issued a Certificate of Operation number. This is also commonly known as the 'CD number' or 'Crane Device number'. This CD number must be renewed annually and the annual application for renewal must be accompanied by the owner's inspection and maintenance records. However, even with the certificate of operation granted, the crane cannot be used because it has not been tested and inspected by the Department of Buildings. This is done as part of the second stage.

The second stage is the Temporary Certificate of Approval and Operation (TCAO). This is a separate and independent process from the previous 'Certificate of Approval'. When a tower crane is brought on site to be used for a specific job, its components are inspected and a load test performed. Inspections include both unassembled and assembled inspections. If these are found to be acceptable, the crane is issued with a TCAO. The Certificate of Operation then becomes valid and the crane is safe for use in New York City.

Typically, the inspections that are required for the Certificate of Operation stage are performed around the same time as the inspections required for the Certificate of On-Site Inspection.

Administratively the Certificate of Operation and TCAO process is followed using form CD-2.

The majority of the information on this form is relevant to the first administration stage and is completed first. The section relating to the TCAO stage is completed once that stage has been satisfactorily addressed.

The Certificate of Operation references the approved prototype number, which links the individual crane to the approved load charts and to the acceptable limits of configuration (i.e., mast, boom and jib lengths, etc.) for that particular model of crane. No change to a crane or derrick not provided for in the Certificate of Operation may be made until the owner obtains a new Certificate of Operation.

The detailed requirements of the Certificate of Operation are covered in RS19-2. However due to the different terminology adopted by the code and the Departments own adapted two stage process, there is some confusion over exactly what is required and when. It is further complicated, as some of the inspections performed as part of the TCAO are similar to the requirements of the final step which is the Certificate of On-Site Inspection. The DOB have developed checklists for inspectors and other documents to clarify the process but they do not clarify exactly what is inspected under the TCAO stage or exactly what is inspected under the later Certificate of On-Site Inspection.

Documents and interviews with the NYCDOB officials have clarified some of the requirements and reviewed materials.

In accordance with 27-1057(c)(3), if the owner applies for renewal of a Certificate of Operation within not more than sixty or less than thirty days prior to the date of expiration of his or her certificate, such owner may continue to use his or her crane or derrick (with a valid application) until the department performs an inspection and either grants or denies approval of the application.

4.2.2.1 Review of Certificate of Operation – Stage 1 (Administration)

The Certificate of Operation on-file at the NYCDOB for the collapsed crane is for a Favelle Favco Model 440D tower crane, model year 2007. The CD number and Serial Number (SN) are given as #3774 and #1371, respectively, which are consistent with other documentation on file at NYCDOB for this crane. However the actual crane model appears to have been 440E not the 440D submitted. The differences are discussed in Section 4.3. The prototype number is given as P331, which matches the prototype number on the approved prototype application, as discussed in the preceding Section of this report.

The Certificate of Operation application was submitted by Sal Isola, General Manager, New York Crane & Equipment Corporation (Brooklyn, NY) on February 15, 2007. The application is stamped "received" by NYCDOB on February 16, 2007. The top right corner of the document indicates a fee of $3,000.00 was received on February 23, 2007 (Invoice No. 60599811). The expiration date for the Certificate of Operation is indicated as February 16, 2008, one year from the stamped "received" date.

An application for renewal was made by Sal Isola General Manager, New York Crane & Equipment Corporation (Brooklyn, NY). This was dated as February 1, 2008 but is stamped as "received" by NYCDOB on March 14, 2008, the day before the collapse. There is no indication of receipt of payment. In addition there is no indication of the additional inspection and maintenance records that are required for renewal by RS 19-2.

The administration stage of the Certificate of Operation had therefore expired and the renewal process was incomplete.

4.2.2.2 Review of Certificate of Operation – Stage 2 (TCAO)

The TCAO stage (and hence the full Certificate of Operations approval) appears to have been completed on February 1st 2008. This is the date of the TCAO stamp that is on the foot of the Certificate of Operation form. However as the Certificate of Operation administration stage was not renewed, the TCAO (and hence the Certificate of Operation) expired on February 16th 2008.

It appears however that regular inspections and tests were performed before and after the expiration of the Certificate of Approval. These are described in the following paragraphs.

The crane was inspected during the erection process. NYCDOB inspection records are available for the following inspections:

- Crane inspection report dated January 17, 2008 for the unassembled inspection of four (4) tower sections.
- Crane inspection report dated January 18, 2008 for the unassembled inspection of four (4) tower sections.
- Crane inspection report dated January 19, 2008 for the unassembled inspection of the four boom sections.
- Crane inspection report dated January 29, 2008 for the assembled load test of the crane. The boom is listed as 135 ft in length, which is consistent with the sum of lengths of the four boom sections inspected on January 19 as per above. The mast is listed as 145-ft in length, which is generally consistent with the calculated height of eleven (11) 393 tower sections (i.e., 11 x 13'-1" = 143'-11"), which is the number of tower sections known to be erected by January 29. However, as per above, (unassembled) inspection reports have been produced for only eight (8) of these tower sections.
- Crane inspection report dated February 11, 2008 for the unassembled inspection of two (2) tower sections.
- Crane inspection report dated March 3, 2008 for the unassembled inspection of two (2) tower sections.

- Crane inspection report dated March 14, 2008 for the unassembled inspection of three (3) tower sections.

The NYCDOB inspection and testing procedures pay particular attention to the suitability of welds of latticed elements (e.g., tower sections and booms), requiring that welds of these sections be tested annually, although these requirements/procedures do not appear to be documented in either the NYCBC or other regulations. The on-site unassembled inspections performed by NYCDOB inspectors generally consist of a comparison of the identification number on the unassembled sections to the identification numbers given in weld testing reports (i.e., to confirm that the sections to be installed have been tested). The sections must be delivered to site the day of the inspection. The on-site inspection is rendered invalid if the section is subsequently removed from the site.

The weld testing of tower and boom sections used at the 303 E51st Street site was performed by Crane Inspection Services, Ltd. (CIS) of Pocono Lake, PA. The testing consisted of magnetic particle inspections (MPI) of "all butt welds, anchor points and welds on the lacing" (quote from CIS inspection reports). The tests are reported to have been performed using a white line portable magnetic particle machine and using dry powder and prod methods of inspection. The testing is reported to have been performed in accordance with New York City Building Code specifications. The testing reports refer to the CD number of the tower crane used at 303 East 51st Street (i.e., CD #3774) and describe the tested tower sections as being 13'-1" in height having "I-beam chords with both tubular and angle lacing". This description is consistent with the 393 tower sections erected at the site. The test reports refer to the boom sections as having tubular (round) chords and lacing, which again is consistent with the boom erected at the site.

A summary of inspection and testing dates for tower and boom sections of the collapsed crane is given in Table 4.2 below.

Table 4.2 - Inspection and testing dates for tower and boom sections of the collapsed crane. Prepared by Arup.

Section of collapsed crane[1]	Date of unassembled inspection by NYCDOB	Date of weld testing by CIS
Tower Sections		
USA-TS-02011	1/17/08	1/10/08
USA-TS-02012	1/18/08	1/10/08
USA-TS-05-47	1/18/08	1/10/08, 7/2/07
USA-TS-05-55	3/14/08	2/27/07, 7/19/07
USA-TS-05-74	1/18/08	1/10/08
USA-TS-06-86	1/18/08	1/10/08, 7/19/08
USA-TS-06-90	3/14/08	7/19/07
USA-TS-06-92	2/11/08	7/2/07
USA-TS-06-104	3/14/08	7/19/07
USA-TS-06-107	Unknown[1,2]	1/10/08
USA-TS-06-120	Unknown[1,2]	1/10/08
USA-TS-06-122	1/17/08	1/10/08
USA-TS-06-126	Unknown[1,2]	1/10/08
USA-TS-06-128	1/17/08	1/10/08
USA-TS-06-139	1/17/08	1/10/08
USA-TS-07-73	2/11/08	9/18/07
USA-TS-07-74	3/3/08	9/18/07

Section of collapsed crane[1]	Date of unassembled inspection by NYCDOB	Date of weld testing by CIS
USA-TS-07-75	3/3/08	9/18/07
Boom Sections		
1371-B1 (45 ft heel)	1/19/08	1/3/08
1371-B4 (30 ft)	1/19/08	1/3/08
1371-B6 (15 ft)[3]	1/19/08	1/3/08
1371-B8 (45 ft head)	1/19/08	1/3/08

<u>Notes to Table Above:</u>

1) Sections that could not be identified from NYCDOB inspection reports were identified from a list of crane components held at the crane supplier's storage yard in South Kearny, New Jersey (source: memo from Wilson, Elser, Moskowitz, Edelman & Dicker, LLP dated April 14, 2008).

2) Unassembled inspection reports have not been produced by NYCDOB for these sections. However, these sections did not fail and the lack of any report is not considered significant to the collapse.

3) The CIS report notes that boom section 1371-B6 has a length of 30 feet, which does not agree with the length of 15 feet from the NYCDOB unassembled inspection report. Furthermore, the CN number quoted for the boom on the January 19 unassembled inspection report is "40/08", which differs from the CN number quoted on the assembled inspection report and throughout the crane documentation on file at NYCDOB (i.e., CN 39/08). However, these discrepancies are not considered significant to the collapse.

In addition to testing of the tower and boom sections, the CIS report dated January 3, 2008 (i.e., Project #101, Crane CD #3774, Client's #1371) states that the following items were also inspected and found to be free of defect:

- an 8'-3" tip extension marked 1371-TE;
- an "A" frame marked 1371-AF;
- a turntable marked 1371-TT; and
- an external jumping frame marked USA-TSC-06-007-C & D.

The specifics of these inspections are not indicated. The requirements of inspections for these components, like the requirements of inspections for tower and boom sections, are not documented in either the NYCBC or other, as confirmed with NYCDOB.

It should be noted that the external climbing frame from the collapsed crane (held at the crane supplier's storage yard in South Kearny New Jersey) was marked TSC-06-007-A & B (source: memo from Wilson, Elser, Moskowitz, Edelman & Dicker, LLP dated April 14, 2008), which differs from the external jumping frame indicated above. However, this discrepancy is not considered significant to the collapse.

NYC DOB do not typically inspect the internal climbing frame. They rely on the inspection by the third party which in this case would be CIS.

4.2.2.3 Compliance with RS-19-2 Requirements for Certificate of Operation

A comparison of the relevant requirements of RS-19-2 to the approved Certificate of Operation application is given in Table 4.3 below.

Table 4.3 - Comparison of the relevant requirements of RS-19-2 to the documentation on-file at NYCDOB. (Note: This comparison is based on the 2006 revision of Reference Standard RS-19-2, which was the governing version of the Standard at the time of the application. Prepared by Arup.

RS-19-2 Para. No.	Requirement	Compliance
10.0.	Application for renewal of a Certificate of Operation shall be accompanied by inspection and maintenance records in accordance with 15.1 and 18.1. Upon approval of the application, a new certificate of operation shall be issued after a satisfactory inspection by a department inspector. (Note: Paragraph 17.1 also requires that detailed maintenance records be made available to the department. Note also that this only applies for renewal. It is not required for the original)	An application for renewal was initiated but there is no evidence that inspection and maintenance records were submitted as required. The requirement for a "satisfactory inspection by a department inspector" is listed in its broad sense. The records indicate that all of the tower and boom sections were tested and inspected prior to erection. Records also indicate that the turntable, A-frame, boom extension and external climbing frame were inspected, although the serial number for the climbing frame on the collapsed crane did not match the serial number on the inspection report. [1]
15.0 – Inspections	Inspection procedure for cranes and derricks in regular service is divided into two general classifications: frequent inspections and periodic inspections. "Frequent" is defined as daily to one month. "Periodic" is defined as 1 to 12 months.	Not applicable. The inspections in Section 15 refer to inspections required of the owner as opposed to inspections required of DOB inspectors. As such, the requirements of Section 15 are not applicable to obtaining a Certificate of Operation (with the exception that inspection and maintenance records must be submitted as part of the application for a Certificate of Operation).
19.1.2	Safety Devices	See below. Page numbers below refer to the page numbers in the "Crane Manufacturers Maintenance Manual" for the M440D model tower crane.
	a. Warning light activated at 100% allowable overturning moment.	Visual alarm provided. (pg. 30)
	b. Acoustic signal activated at 105% allowable overturning	Audible alarm provided. (pg.30)

RS-19-2 Para. No.	Requirement	Compliance
	moment.	
	c. Automatic stop if 110% allowable overturning moment is reached.	Yes. (pg.31)
	d. Automatic stop if load exceeds maximum rated load in high gear.	Yes, has automatic stop.
	e. Automatic stop if load exceeds maximum rated load in intermediate gear.	Yes, has automatic stop.
	f. Automatic stop if load exceeds maximum rated load in low gear.	Yes, has automatic stop.
	g. Pre-deceleration before top position of hook.	Yes. (pg. 32)
	h. Limit switch for top position of hook.	Yes. (pg. 32)
	i. Pre-deceleration before low position of hook.	Yes. (pg. 32)
	j. Limit switch for trolley travelling out	Yes. Limits on boom luffing (pg. 31 and 32)
	k. Limit switch for trolley travelling in.	Yes. Limits on boom luffing (pg. 31 and 32)
	l. Acceleration limits on hoist movement.	Yes, has high speed prevention.
	m. Acceleration limits on the swing movement.	Yes, has high speed prevention.
	n. Acceleration limits on the trolley (boom) movement.	Yes, has high speed prevention.
	o. Dead man control on both joy sticks in the box.	Yes. (pg. 8)

<u>Notes to Table Above:</u>

1) This inspection and testing is typically performed in the presence of NYCDOB inspectors once the crane has been assembled at site. This was the case for the crane at the E51st Street site, which received an assembled inspection by NYCDOB inspectors on January 29, 2008.

4.2.2.4 Compliance with Additional NYCBC Requirements

Title 27, Subchapter 19, Article 10 of the New York City Building Code (i.e., Material Handling and Hoisting Equipment) contains additional requirements related to cranes.

These are given in Table 4.4 below, along with a comparison of the requirements to the information supplied with the application submission.

Table 4.4 - Comparison of Title 27, Subchapter 19, Article 10 requirements to the application submission. Prepared by Arup.

27-19-10 Para. No.	Requirement	Compliance
27-1055(a)	Hoisting line – only wire rope shall be used with power driven hoisting machinery.	Yes.
27-1055(b)(1)	All hoisting cable shall be at least one-half inch diameter plow steel grade.	Yes. From crane manufacturer's calculations: Main hoist is 1-1/4" wire rope. Luff rope is 1-1/4" wire rope. Pendant ropes are 1-7/8" wire ropes.

4.2.2.5 Summary of Certificate of Operation Review

It appears that the original Certificate of Operation for the crane had recently expired (i.e., by one month) at the time of the collapse. According to 27-1057(c)(3) of the NYCBC, if the owner applies for renewal of a Certificate of Operation within no more than sixty or less than thirty days prior to the date of expiration, he may continue to use the crane until the Department grants or denies the new Certificate. However, there is no evidence to indicate that such a renewal was in-progress.

Conformance with the inspection and testing requirements of the NYCBC (including RS-19-2) and the inspection and testing requirements of NYCDOB was reviewed. Deviations from these requirements were found to include the following:

- Maintenance and inspection records for the crane were not submitted, or at least do not appear to be on file (the exception is inspection and testing reports submitted from CIS for the tower and boom sections, the A-frame, the turntable and the boom extension). However, as mentioned in 4.2.2.3 these are only required for an application for renewal so would not be required for the original application.
- Unassembled inspection reports are missing for three tower sections (i.e., the 9th, 10th and 11th tower sections from the bottom, which were referenced in inspection report dated January 29, 2008). However, CIS testing records indicate that the required weld testing was performed for these sections and they did not fail. The lack of any inspection report for these sections is not considered significant to the collapse.
- An application for renewal was made and was noted as received by DOB on March 14 2008, the day before the collapse. However this appears to have been incomplete and had not been approved. Therefore, it would seem the crane was operating without a valid Certificate of Operation on the day of the collapse. However, this is not considered to have contributed to the collapse

4.2.3 Certificate of On-Site Inspection

The Certificate of On-Site Inspection pertains to the erection of an approved crane (as per preceding sections) at a specific job site. It is effectively a work permit for the crane to work at a specific location to do a particular job. Certificates of On-Site Inspection are addressed in C27-1057(d) [C26-1909.4(d)] of the New York City Building Code (NYCBC). The written procedure is as follows:

- The owner or their representative (usually the crane engineer) files an application on the appropriate department form with the fee as stipulated in 26-215 of title twenty-six of the administrative code. The applicant is required to specify the date when the equipment will be at the job site for use, but not less than three (3) working days from the date the application is filed. The application must contain the information as set forth in RS-19-2 and RS-19-3.
- Upon approval of the application, a copy of the approval is given to the applicant. The approval should say that the equipment shall not be operated prior to the date indicated, which again shall not be less than three (3) working days from the filing of the application, unless otherwise allowed by Reference Standard RS-19-2.
- Upon inspection by the department and a finding of satisfactory compliance, the approval shall be deemed to be a Certificate of On-Site Inspection.
- It is unlawful to operate the equipment before the specified date unless it has been inspected and found to be satisfactory by the department. If the equipment has not been inspected by the department on or before the said date, then the equipment may only be operated after inspection provided that the conditions and statements contained in the approved application are complied with.

The Sections of this report that follow provide an overview of the information submitted with the Certificate of On-Site Inspection application and a comparison of this information to the requirements of C27-1057(d) of the New York City Building Code and Reference Standard RS-19-2. (Note: Reference Standard RS-19-3 is for Cableways and therefore is not applicable to this investigation.)

4.2.3.1 Review of Certificate of On-Site Inspection for the Collapsed Crane

The application for the Certificate of On-Site Inspection was filed by the crane engineer, Peter Stroh of Stroh Engineering Services, PC (Babylon, NY), on behalf of the equipment user, Anthony Lorenzo of Joy Contractors Inc. (Elizabeth, NJ). The application, which is dated January 8, 2008, was received by the NYCDOB on January 10, 2008. The application consisted of drawings, calculations and a certification that the crane engineer had visited the site to inspect the foundation conditions and had designed the foundation accordingly.

Objections to the application were raised by the NYCDOB reviewer on January 14, 2008. Peter Stroh responded to these objections the same day, submitting additional drawings and calculations to address the objections. Peter Stroh’s response was received by the NYCDOB on the following day (January 15, 2008).

The application was approved on January 17, 2008, contingent on the crane receiving an approved Certificate of Approval and Operation (TCAO). This contingency was met on or about January 29, 2008, the date the crane passed its assembled inspection. The approved application was signed off by the NYCDOB inspector on January 29, 2008. As such, a valid Certificate of On-Site Inspection was in place at the time of the collapse.

4.2.3.2 Compliance with RS-19-2 Requirements

A comparison of the requirements of RS-19-2 to the approved application is given in Table 4.5 below.

Table 4.5 - Comparison of the relevant requirements of RS-19-2 to the approved application. (Note: This comparison is based on the 2006 revision of Reference Standard RS-19-2, which was the governing version of the Standard at the time the application was made.) Prepared by Arup.

RS-19-2 Para. No.	Requirement	Compliance
8.1.1	The application shall be accompanied by plans showing proposed locations of the crane or derrick, pertinent features of the site such as assumed soil bearing values, ground elevations and slopes, vaults or other subsurface structures, supporting platforms or structures, and the swing of the crane or derrick.	Yes. Refer to Drawings 07-046C-1, Drawings 1 and 2 of 4, by Peter Stroh dated 1/2/08, which were approved by NYCDOB on 1/17/08.
8.1.1 (continued)	A document shall be submitted, signed by a licensed engineer or registered architect, which shall include the following information where the crane or derrick is to be supported by soil:	See 8.1.1.a. through 8.1.1.e. below:
8.1.1.a.	That he has inspected the soil at the proposed location or locations of the crane or derrick.	Yes. Refer to letter submitted by Peter Stroh dated 1/9/08 (received by NYCDOB 1/10/08).
8.1.1.b.	His estimate of the soil bearing value.	Not applicable. The crane engineer (Stroh Engineering) designed a foundation to sound rock.
8.1.1.c.	That he has explored the existence of any sheeting or retaining walls supporting soil adjoining any excavation which may be affected and certifies as to its adequacy.	Yes. The crane rests on the new building's foundation wall. The adequacy of the wall was checked. Refer to page 8 of Peter Stroh's calculations dated 1/2/08.
8.1.1.d.	If the crane or derrick is to be on the street, that he has explored the existence of vaults or other subsurface structures, which could impair the bearing value of the street or sidewalk.	Yes. The crane engineer (Stroh Engineering) investigated and devised a foundation system that bridges over a Con-Edison vault. Refer to Drawing 07-046C-1, 2 of 4 submitted by Peter Stroh on 1/2/08.
8.1.1.e	That the load imposed upon the soil by the crane or derrick, including supporting platform, does not exceed such bearing value under any condition of loading.	Yes. Refer to Calculations by Peter Stroh dated 1/16/08 in response to NYCDOB objections from 1/14/08.

RS-19-2 Para. No.	Requirement	Compliance
8.2.1.	Where the crane or derrick is supported by a building or structure, the statement by the licensed engineer or registered architect referred to in 8.1.1 shall include the means of supporting and bracing the equipment.	Yes. Refer to Drawings 07-046C-1, Drawings 3 and 4 of 4, by Peter Stroh dated 1/2/08, which were approved by NYCDOB on 1/17/08. Also refer to Calculations by Peter Stroh also dated 1/2/08.
8.2.1. (continued)	The swing of the crane or derrick shall be shown on the plans to insure clearance during operation.	Yes. Refer to Drawing 07-046C-1, Drawing 1 of 4, by Peter Stroh dated 1/2/08, which was approved by NYCDOB on 1/17/08.
8.2.1. (continued)	Computations shall be submitted with the application showing all reactions imposed on the structure by the crane or derrick, including those due to impact and wind. Such computations shall verify that the stability of the building or structure will not be impaired when the crane or derrick is in operation and that no structural members will be overstressed due to forces induced by the Crane or derrick.	Yes. Refer to Calculations submitted by Peter Stroh dated 1/2/08. Calculations include predicted reactions imposed on the structure by the crane, including those due to wind (but not impact). The Calculations include checks that the slabs of the building will not fail from horizontal loads imposed by the crane ties.
8.2.2.	Concrete structures. -If the structure is a concrete structure, test reports of the compression strength of the concrete shall be submitted to insure that the concrete supports of the crane or derrick have developed sufficient strength to support the crane or derrick before it is installed. The means for establishing concrete strength before imposing crane or derrick loads upon the structure shall be indicated on the application.	The means for establishing concrete strength before imposing crane loads was not indicated on the application. However, the Calculations submitted by Peter Stroh on 1/2/08 indicate on page 46 that the concrete supports for the crane will have sufficient strength based on the 7-day compressive strength of the concrete. As such, it is unlikely this would be an issue.
8.2.3	All anchorages for cranes and derricks shall be approved by an appointed person.	The appointed person for approving the anchorages is not indicated on the application.

4.2.3.3 Summary of Certificate of On-Site Inspection Review

The approved Certificate of On-Site Inspection generally addresses the requirements of the NYCBC and Reference Standard RS-19-2. NYCDOB reviewers appear to have performed an adequate review of the application and actively sought to enforce the code requirements.

One notable exception is the requirement given in RS-19-2, Paragraph 8.2.3., which requires all anchorages for cranes and derricks be approved by an appointed person. The appointed person is not indicated on the application. However, it is not clear if this is a requirement for the application.

4.3 Model M440E versus Model M440D

Photographs from the collapse site revealed that the collapsed crane was a Model M440E tower crane as opposed to a Model M440D tower crane, the model that was indicated on all documentation related to the permitting process. Photographs of the M440E crane being removed from the collapse site are given in Appendix B.

NYCDOB obtained documentation for the M440E tower crane from the crane manufacturer, Favelle Favco, following the collapse. The documentation for the M440E tower crane was compared to the documentation on file at NYCDOB for the M440D tower crane. The findings of this comparison include:

- The Model M440E tower crane is a slightly upgraded version of the Model M440D tower crane;
- Both models are diesel-driven luffing-boom tower cranes;
- Both models are compatible with the 393 Tower Sections that were used;
- The Model M440E has a more advanced electronic safe load Indicator and controller system than the M440D;
- The arrangement of components on the machinery deck varies slightly between the two cranes; however, these differences do not significantly alter the overturning moment imposed by the crane on the slewing ring (turn table).

The differences between the M440D and the M440E models are not considered to have played a role in the collapse.

4.4 Discussion & Conclusions

The permitting process defined in the relevant codes is complicated and does not specifically and completely address the requirements for tower cranes for some of the processes. In particular; the code requirements for obtaining the Certificate of Operation for tower cranes are confusing and not well documented. It is therefore difficult to fully confirm compliance with the permitting requirements.

It is however possible to identify the following principal conclusions;

1. The licenses for the Hoisting Machine Operator and Rigger were valid and the licensing process appears to have been in compliance with code requirements.
2. The Certificate of Approval for the crane was valid and the permitting process appears to have been in compliance with code requirements.
3. The Certificate of Operation had expired and therefore was not valid. An application for renewal had been submitted but was incomplete and had not been approved. However, this is not considered to have contributed to the collapse.
4. The Certificate of On-Site Inspection was valid and the permitting process appears to have been in compliance with code requirements.
5. The actual crane used was a model 440E not 440D as listed on the submitted documents. The differences in the models are not significant and not considered to have played a role in the collapse.

4.5 References

1. New York City Department of Buildings. “Cranes and Derricks 101” (MS PowerPoint slides used for presentations to the construction industry). Bethany Klein (Executive Director, Cranes & Derricks) and Michael Carbone (Chief Inspector, Cranes & Derricks), July 18, 2007.
2. Building Code of the City of New York as of October 1, 2004 including relevant updates through June 30, 2008.

5 Peer Review of Tower Crane On-Site Design

5.1 Introduction

This chapter presents an independent engineering review by a Crane Certified Agent (licensed in California #CA-51), Dale H. Curtis, P.E. (the reviewer). Mr. Curtis has been an active California Crane Certified Agent since 1974 specializing in large mobile cranes and tower cranes. His engineering experience of tower crane foundations and tie-in systems dates back to 1992 for some 28 different models of tower cranes in 8 western states. In addition, he is an active member in the Crane Certification Association of America based in Vancouver, Washington.

5.2 Scope

Under instruction from Arup, the scope of this engineering review was limited to review of the engineering package submitted to the City of New York by Stroh Engineering Services, P.C. as prepared by Peter J. Stroh, president of his firm and a licensed professional engineer in the State of New York (the crane engineer). The application package was received January 10, 2008 by the Department of Buildings, Division of Cranes and Derricks and assigned CN Number CN 39/08 and CD #3774. The application package was approved January 17, 2008 contingent upon receipt of a valid Certificate of Operation.

5.3 Review & Analysis

The application package included 4 sheets dated 1/2/08 of building reactions generated using Favelle Favco software for the 4 jumps and some 34 sheets dated 1/2/08 of computer-generated brace and building loads. The 34 pages of computer printout on the crane engineer (Stroh Engineering) sheets appear to have values taken from the 4 sheets dated 1/2/08.

The reviewer (Dale Curtis) prepared 4 independent calculation sheets to analyze the forces on the tower crane and resulting forces in the tie-in struts for both in-service and storm wind conditions for comparison with the original submitted engineering package. The calculations also included a design check for the W12x79 tie-in struts and their welded connections to the steel base plates (building slabs). The calculations were prepared using the tower crane manufacturer's data for base reactions and for external climbing limitations for the Model M440D (such data is usually part of the Operations Manual). Attached Appendix C shows this important data along with a blank format which can be used for climbing and tie-in analysis.

The independent calculations are included in the attached Appendix D. The forces in each of the 3 tie-in struts are tabulated for the 105' height above the top secured tie-in and for a typical 98' back-span. Sheet 3 of the reviewer's calculations provides a comparison of the independently calculated forces with those from the crane engineer's application package (Stroh Engineering).

5.4 Discussion

One factor controlling the tie-in spacing for tower cranes is the minimum permissible backspan distance. The term "backspan" is described as the vertical distance between upper-most secured and tight tie-ins to the building. Tower crane engineers may specify that lower tie-ins be loosened in order to increase the backspan distance. A greater backspan distance results in lower horizontal reactions at the upper-most secured collar. If the backspan distance is too small, the upper-most collar reaction forces will be much greater against the mast. For this reason, tower crane manufacturers specify minimum heights (backspans) between secured top ties.

Mr. Stroh's Drawing No. 07-046C-1 Sheet 4 of 4 indicates Jumps #1 through #4 having backspan distances between upper Tie-ins of 66 ft, 98 ft, 98 ft and 85 ft from bottom to top, respectively. As required, these exceed the manufacturer’s minimum allowable backspan of 52 ft (16 m) for a tower of similar dimensions to the one used at the site (i.e., refer to Appendix E, External Climbing, Tower Type 392).

Another factor controlling the tie-in spacing for in-service operation is the manufacturer's permissible tower height above the top brace. The aforementioned Drawing (Sheet 4 of 4) shows maximum heights of 111 ft, 105 ft, 111 ft and 103 ft of the mast above the upper-most tie-ins for jumps 1 through 4, respectively. The larger figures slightly exceed the manufacturer’s permissible maximum height above the top tie-in of 105’ (32 m) for a tower of similar dimensions to the one used at the site (i.e., refer to Appendix C, External Climbing, Tower Type 392).

The tower height above Tie-in #2 was approximately 143 ft when the collar at Tie-in #3 was being installed. This was determined by adding the proposed backspan distance between Tie-in #2 and Tie-in #3 (i.e., 98 ft) to the height of the climbing cage (taken to be 39 ft plus an additional 6 ft for the ram). In order to install an upper tie-in collar, the tower sections may sometimes exceed the unbraced height limitation for a short duration when a weather forecast predicts wind velocity less than the upper limit of 20 mph for climbing operations as stated in the ASME/ANSI Standard B 30.3 for construction tower cranes. Nonetheless, such a free-standing temporary height above a secured tie-in has not been analyzed by Mr. Stroh.

Favelle Favco's manual specifies 105 ft upper height for an anticipated storm wind of 94.5 mph and for an in-operation wind of 45 mph. With the adjustment of the maximum storm wind to an "approved" 90 mph (for New York City), the allowed upper height above a secured Tie-in would be 117 ft as per the reviewer’s independent calculations in Appendix F.

The reviewer was unable to locate in the FAVCO computer 34-page printout any utilization of the in-service slewing moment of 325,000 ft-lbs as tabulated for the “Tower Crane Maximum Reaction Forces” on the crane engineer (Stroh Engineering) Drawing entitled “Tie-in Part Plan and Details” (i.e., Drawing No.07-046C-1 Sheet No. 3 of 4 dated January 2, 2008). It is possible the Favco computer program may not have included the slewing moment resulting in significantly lower calculated strut forces for the operating mode.

The independent calculations included checking the stress levels of the tie-struts for the higher forces determined by the reviewer and for the welds to the connection plates and the through-bolts at the concrete floor slab. The tie-in beam stresses and welds were found to be within acceptable levels. The 4 bolts through the floor were found to possibly be marginal.

The foundation base beams and concrete appear to be well engineered. NYC Department of Buildings, Cranes and Derricks Division, on January 14, 2008, had properly requested additional information on the bearing pad design and an explanation for the apparent lack of provision to prevent lateral movement at the bottom. On January 14 or 15, 2008, Mr. Stroh resubmitted his revised base frame details showing the restraining W 8 x 31 beams on both sides of the tower crane base, the added web stiffeners and the boxed-beam side plates on the street-side end of the large W 27 x 194 beams. Mr. Stroh stated in his letter, dated January 14, 2008, that the lateral loads are taken in the 2 ties above and not by the crane steel dunnage frame at the base. The bottom of the steel frame is not positively attached to the concrete other than by friction due to vertical loads from above. The reviewer accepted this explanation as had the NYCDOB staff. However, the reviewer observed that such a base frame is preferred to be positively restrained at the concrete below to prevent lateral movement during disassembly of the tower sections when only the lowest tie is attached to the completed building.

Mr. Stroh's four large drawings appear to be well detailed in showing certain Favelle Favco tie-in forces (although on the low side) and the location of the tower crane relative to the building. His requirement for padeyes on each steel strut is an additional safety requirement. Double nuts on the threaded rods are in compliance with industry best practice.

5.5 Review Summary

The crane engineer's design submission (by Stroh Engineering) is for the most part in accordance with industry "best practice", save for some noteworthy departures as noted above. More specifically:

- Adequate documentation of the design (i.e., drawings and calculations) was provided;
- The tower crane make/model that was selected; i.e., the M440E, was appropriate for the job;
- The support and erection scheme for the tower crane (i.e., collars, horizontal ties, base support conditions, etc.) were appropriate, however the magnitude of the height of free standing tower in the temporary condition (prior to installation of additional tie-ins) was not justified in the calculations provided;
- The calculations address industry standard load-cases/loading scenarios and are generally accurate with the major exception that slewing moments do not appear to have been considered for the in-operation mode. However, the reviewer determined that the design of the tower's attachments had sufficient robustness to resist these slewing moments.
- The design drawings clearly show the details of welds required to connect the tie-beams to the steel base plates. Furthermore, these welds are sufficient as designed to withstand the anticipated lateral loads, both those calculated by the crane engineer (Stroh Engineering) and those calculated by the reviewer. It is considered beyond the expectations of "best practice" to design these welds for impact loads of a magnitude similar to that which is believed to have occurred at the site the day of the collapse (i.e. a 12,000 lb. weight falling a distance of 100 feet).

6 Non-Linear Static Structural Analysis of Tower

6.1 Introduction

A non-linear static structural model of the crane tower was created to assess the stability of the tower structure against elastic buckling between supports (or above the highest collar support) as well as the overall stability of the tower against overturning. Two lateral support configurations were considered. The first including the base and the collars at levels 3 and 9 were used to confirm the design. The second eliminated the level 9 collar to represent the final condition on site.

The modeling was performed using the structural software package GSA by Oasys. The analysis actually couples a linear static analysis for calculating reaction forces with a nonlinear modal buckling analysis. The latter employs a stiffness matrix adjustment to account for the effect of axial load on bending stiffness (i.e. “geometric” stiffness). It then performs an Eigenvalue analysis to identify buckling load factors and mode shapes

6.2 Model Input

This Section describes the input parameters to the GSA model and provides discussion as required.

6.2.1 Overall Layout of the Tower

A description of the tower layout is provided in Chapter 1, Section 1.1 Background and Description of the Tower Crane.

A two-dimensional image of the GSA model of the overall crane tower is given in Figure 6.1. The model consists of eighteen (18) tower sections, as the tower existed just prior to the collapse. Details of the tower sections, assumed support conditions and applied loadings are discussed in the Sections that follow.

6.2.2 Tower Sections

The tower sections of the collapsed crane were 393 Tower Sections as per the approved prototype application. A three-dimensional image of a tower section from the GSA model is given in Figure 6.2.

6.2.3 Tower Support Conditions at the Collapse Site

A general description of the tower support conditions that existed at the site is also given in Chapter 1, Section 1.1 Background and Description of the Tower Crane. An account of how these support conditions were addressed in the static structural model is presented in the section that follows.

6.2.4 Tower Support Conditions Assumed in the Static Structural Model

This section describes how the tower support conditions, as discussed in the previous section, were addressed in the static structural model.

6.2.4.1 Tie-ins

The building tie-ins, where present, were modeled as vertical “rollers” in the static structural model (i.e., restraint in both directions of the lateral plane, but no restraint in the vertical plane). This is considered to be a reasonable assumption since the steel collars of the tie-in assembly are chocked to the tower legs, thereby preventing lateral movement of the collar relative to the tower legs, while conversely the collars could be expected to slide up or down relative to the tower legs if subjected to a large enough force. The clamping force of the chock blocks to the tower legs is unknown; however, the installation drawing (see Figure 3.4) directs the installer to "Tighten chocks uniformly onto the tower legs using a rattle gun or similar." No information regarding the actual clamping force or treatment of the faying surfaces is provided.

6.2.4.2 Tower base and foundations

The tower base cannot accommodate an uplift force because it is not anchored to the top of the dunnage steel. Similarly, the tower base cannot resist lateral forces that exceed the frictional resistance between the tower legs and the top of the dunnage steel (if the legs lift above their restraints as noted above), or between the bottom of the dunnage steel and the plywood pads.

The process required therefore is to carry out the analysis in two stages. In the first stage, each of the four tower legs at the base of the tower is initially considered to have a pinned connection to the top of the dunnage steel (i.e., restrained in the vertical direction and both horizontal directions). Where the resulting reactions indicate uplift or a shear force exceeding the friction capacity, restraint in the appropriate direction is released and the model is re-analyzed with the revised supports. Note that this is a standard approach for this type of analysis.

6.2.5 Forces and Moments Imposed at the Top of the Tower

The top of the crane; i.e., the portion of the crane above and including the slewing ring/turn table assembly, which sits atop the tower, was modeled implicitly through the application of a force and a moment at the top of the tower. The force, which was determined by a summation of the weight of all crane components above the tower, was divided evenly among the four tower legs. The moment, which was determined by a summation of moments for all crane components about the centerline of the tower, was applied as a tension/compression couple to the tower legs.

6.2.5.1 Force and moment for various boom angles

Calculations of the force and moment created by the crane at the top of the tower for various boom angles, with no load on the lifting hook, are given in Tables 6.1 through Table 6.8. Each table corresponds to a different boom angle. The boom angles used for the calculations were selected to match the boom angles used in the calculations submitted by the crane manufacturer as part of their approved prototype application for the Model 440D crane. The boom angles range from 85.9 degrees to 16.4 degrees, the maximum and minimum boom angle permitted, respectively.

A comparison of the moment summation between tables reveals the influence of the boom angle on the moment created at the top of the tower. The greater the boom angle to the horizontal, the greater the moment in the direction towards the back (i.e., counterweight side) of the crane.

6.2.5.2 Load on lifting hook

The moment created at the top of the crane also depends upon the weight carried by the lifting hook. Intuitively, an increase in the weight carried by the lifting hook would increase the moment towards the boom side of the crane.

The calculations for the force and moment at the top of the tower, as presented in the preceding tables, assume that the lifting hook carries no weight. This assumption is consistent with witness statements, which indicate the lifting hook was attached to a tie-beam that was resting on (i.e., supported by) the 18th floor slab and as such would not impose a significant load on the lifting hook.

6.2.5.3 Counterweight

The moment created at the top of the crane also depends upon the number and position of the counterweights. The calculations for the force and moment at the top of the tower, as presented in the preceding tables, assume a total counterweight of 83,800 lbs (38,000 kg). This is consistent with information provided by the crane manufacturer for a M440D model crane with a 41.2 m (135 ft) boom (i.e., the length of the boom on the collapsed crane) as part of the approved prototype application. The counterweights were assumed to be fixed in location as per the crane manufacturers drawings submitted with the approved prototype application (e.g., Drawing No. A1-2000-066 titled “Machinery Deck Assembly” Rev. A dated July 26, 1994).

6.2.5.4 Load from external climbing frame

The external climbing frame, which was used for ‘jumping’ the crane, was attached to the tower at the time of the collapse, as can be seen in Photograph 1.6. As such, the external climbing frame was modeled as a dead load on the top of the tower.

6.2.5.5 Load factors

Load factors were not applied in the calculation of forces and moments. Service level loads are appropriate for buckling analysis.

6.2.5.6 M440D versus M440E

As discussed earlier in this report, photographs from the collapse site revealed that the collapsed crane was a Model M440E crane as opposed to a Model M440D crane, the model that was indicated on all documentation related to the permitting process.

As already mentioned, the arrangement of components on the machinery deck was found to vary slightly between the two crane models, however, these differences were not found to significantly alter the forces and overturning moment imposed by the crane at the top of the tower.

To illustrate, compare the overturning moment created by an M440D model crane with a designated counterweight of 83,800 lbs (38,000 kg), as given in Table 6.1, with the overturning moment created by an M440E model crane with a designated counterweight of 86,700 lbs (39,400 kg), as given in Appendix E. Both calculations assume a boom angle of 85.9 degrees and no load on the lifting hook, which is considered to be the condition at the time of collapse, as will be discussed in greater detail in subsequent sections. The overturning moment for the M440D model crane was calculated to be 2,850 kip-ft towards the counterweight side, which is similar to the overturning moment calculated for the M440E model crane of 3,090 kip-ft, also towards the counterweight side (i.e., an increase of less than 10%, which is not considered to be significant for the purposes of the modeling). The total weight at the base of two different models of tower crane was also found to be similar: 434.5 kips for the M440D model crane versus 457.5 kips for the M440E model crane (i.e., an increase of approximately 5%, which is also not considered to be significant for the purposes of this analysis).

In conclusion, the differences between the M440D and M440E model cranes do not significantly alter the findings of the static structural modeling as presented in the sections that follow.

6.3 Stage 1 Analysis

The following two sets of initial analyses were performed with the GSA model to gain a better understanding of the stability of the crane tower structure under different lateral support conditions:

- Analysis assuming lateral restraint at the 3rd and 9th floors
- Analysis assuming lateral restraint at the 3rd floor but not the 9th floor,

The first set of initial analyses is intended to model the condition that existed immediately prior to the collapse. The second set of initial analyses is intended to model the condition that existed immediately after the destruction of the tie-in at the 9th floor, assuming the tie-in at the 3rd floor is maintained.

6.3.1 Input Assumptions for Stage 1 Analysis

As mentioned in the previous Section, all analyses assume the presence of 83,800 lbs (38,000 kg) of counterweight at the back of the crane and no load on the lifting hook. As demonstrated by the moment calculations in Tables 6.1 through 6.8, this situation will always result in an overturning moment towards the counterweight side (i.e., back) of the crane, regardless of boom angle. As such, the maximum moment generated at the top of the crane, assuming no load on the lifting hook, occurs when the boom is at its maximum angle to the horizontal (i.e., 85.9 degrees). Discounting the potential contribution of the 18th floor level tie-beam would therefore result in a conservative analysis, if, indeed, the tie-beam weight had been actually supported by the crane prior to collapse. It is unknown to what extent the tie-beam was being supported at the time of the collapse, if at all.

All analyses assume the crane is perpendicular to the tower, with the boom side of the crane nearest to the building and the counterweight side of the crane furthest from the building. This is consistent with the operations of the crane at the time of the collapse. As such, the moment from the crane is applied evenly to both sets of tower legs, where each set of tower legs consists of a leg on the boom side plus a leg on the counterweight side.

All initial analyses assume all four legs of the tower being pinned to the top of the dunnage steel. As discussed in a preceding section, the restraints are revised based on the support reactions and the model is analyzed again.

6.3.2 Stage 1 Analysis Output

The results of the initial analyses are presented in Tables 6.9 and 6.10.

The first four columns in each of these Tables (i.e., “boom angle”, “boom radius”, “calculated moment at slew ring” and “T/C couple applied in GSA”) contain identical information related to the overturning moment for various boom angles. This information was taken from Tables 6.1 through 6.8 for boom angles ranging from 85.90 degrees to 16.9 degrees.

The next four columns in Tables 6.9 and 6.10 indicate the vertical reactions at the base of the tower. A positive value indicates compression, whereas a negative value indicates tension (i.e., uplift).

Following the vertical reactions, the next two columns in the Tables indicate the buckling factor (or load factor) determined from a buckling analysis of the tower. A buckling factor of one (1) would be indicative of an elastic buckling failure of the tower between supports or above the uppermost collar support. A buckling factor of less than about five (5) would be indicative of a significant tendency towards buckling requiring further analysis.

Finally, the bottom half of Tables 6.9 and 6.10 present the horizontal reactions at the base of the crane for each of the four tower legs.

6.3.3 Discussion of Stage 1 Analysis Results

Discussion of the results for the two sets of analyses is as follows:

6.3.3.1 Lateral restraint at 3rd and 9th floors

The Stage 1 results summarized in Table 6.9 demonstrate that the vertical base reactions are all compressive (positive). They also indicate small horizontal base reactions. In fact, the total base reaction considering all four legs (nodes) is roughly 2% of the total vertical reaction. It is reasonable to assume that the base friction capacity is ample to resist this load. As neither uplift nor sliding is predicted, a stage 2 analysis, with changed support condition, is not required.

As neither uplift nor sliding is predicted, the model predicts that the tower is stable against overturning. As the buckling factors exceed 8, the model also predicts that the tower is stable against elastic buckling between supports and above the upper collar support. These findings are logical since the model approximates a condition that was designed for as well as one that was actually in use prior to the jumping operation. The model therefore represents another confirmation of the general stability of the design.

6.3.3.2 Lateral restraint at 3rd floor only

Stage 1 results summarized in Table 6.10 demonstrate that the vertical base reactions below the legs nearest the building, i.e. “boom” side, are tensile. A stage 2 analysis is therefore required with the vertical restraints at these legs (nodes 3 and 4) released. Although the total horizontal reaction increased to about 10% of the total vertical reaction, the friction capacity could be expected to exceed this value, so no correction to the horizontal restraints is required for the stage 2 analysis.

6.4 Stage 2 Analysis

The stage 1 analysis determined the need to revise the base support conditions for the model with the level 9 restraint eliminated.

6.4.1 Assumptions

The assumptions used for the additional analysis are outlined in the sections that follow.

6.4.1.1 Base support

The front legs of the tower (i.e., the legs furthest from the building) were taken to be pinned to the top of the dunnage steel and the back legs of the tower were assumed to be unrestrained. This reflects the type of restraint successfully provided by the W8 sections in the final position of the tower after collapse.

6.4.1.2 Crane configuration

The additional analysis focused on the crane configuration that was most likely to have existed at the time of the collapse: a boom angle of 85.9 degrees with no load on the lifting hook and the crane perpendicular to the building with the boom nearest the building and the counterweights furthest away. The “most likely” crane configuration was determined based upon the following sources of information:

- A photograph of the crane taken an hour before the collapse, as given in Photograph 1.7;
- A review of the manufacturer's collar/tie erection sequence drawing (i.e., Drawing No. A11100.123 Rev B), Figures 3.3, 3.4 and 3.5 which indicate the crane would have been using a minimal radius to move the tie-beams into position; and
- Input from an independent crane engineer, Dale Curtis (i.e., the crane engineer who performed the peer review of the original crane engineer's design by Stroh Engineering).

6.4.1.3 3rd floor lateral restraint

As with the initial analyses, the restraint at the 3rd floor is considered to be vertical "roller".

6.4.1.4 9th floor lateral restraint

It was assumed there was no lateral restraint at the 9th floor.

6.4.1.5 Element properties

As with the initial analyses, all finite elements in the GSA model are linear elastic.

6.4.2 Discussion of Stage 2 Analysis

The stage 2 analysis results summarized in Table 6.11 and 6.12 demonstrate that the two remaining vertical base reactions are compressive (positive). Although the total base reaction has increased to about 18% of the vertical reaction, the friction capacity could be expected to exceed this value. As neither sliding nor uplift of the two legs still in contact with the supports is predicted, the calculated reactions are consistent with the assumptions and an additional third stage is not required.

As the buckling factor exceeds 5.0, the model predicts that the model is stable against elastic buckling. As sliding is not predicted, the model concludes that the structure is stable against overturning. This is not consistent with field observations, thus substantiating that the significance of the dynamic effects on the tower, and is discussed further in the next section.

6.5 Summary of Static Analysis Results

The static structural modeling of the crane tower has provided a better understanding of the overall static stability of the tower when subject to various forces and moments under different lateral support conditions.

These analyses demonstrate that the tower is stable against elastic buckling between collars and above the uppermost collar. This is consistent with field observations that showed no damage to the tower segments themselves other than a connection failure near the top.

The analyses also demonstrate that, although the legs nearest the building would start to lift up, they would not slide. As such, an overturning failure would not be predicted. This is inconsistent with observations. However, it is clear that the static analysis is unable to capture key aspects of the event, including, in particular, the dynamic effects of the linear and angular momentum of the crane. These aspects of the tower's structural response are discussed in the section on Dynamic Analysis.

Table 6.1 - Calculation of load and moment above the slewing ring and load below the slewing ring for a boom angle of 85.9 degrees. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			3.70	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	0.980	0.74	1	1,492.83	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	0.658	1.56	1	2,116.31	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	0.329	2.05	1	1,649.60	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	0.968	2.70	1	7,238.61	
Bridle	27	1	213	213.00			0.74	1	157.57	
2- Fall Hook Block	28	1	1141	1,141.00			3.70	1	4,221.70	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-394,285.83**	-394.3 ton*m
										(2,852.5 kip-ft)

BELOW SLEW RING:						
Tower 393	1	18	4900	88,200.00	194.4	kips
Climbing Cylinder	2	2	1500	3,000.00		
Side Panel	3	2	4973	9,946.00		
Tie	4	2	907	1,814.00		
Climbing Support	5	2	332	664.00		
Access Platform	6	1	199	199.00		
			Sum (kg)	**103,823.00**	228.9	kips
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips

Table 6.2 - Calculations of load and moment above the slewing ring and load below the slewing ring for a boom angle of 72.4. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			13.20	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	4.142	2.32	1	4,684.25	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	2.782	5.78	1	7,853.82	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	1.391	7.87	1	6,327.26	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	4.092	10.61	1	28,448.51	
Bridle	27	1	213	213.00			2.32	1	494.42	
2- Fall Hook Block	28	1	1141	1,141.00			13.20	1	15,061.20	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-348,292.99**	-348.3 ton*m
										(2,519.7 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.3 - Calculation of load and moment above the slewing ring and load below the slewing ring for a boom angle of 70.0 degrees. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			15.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	4.686	2.59	1	5,232.35	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	3.147	6.51	1	8,839.18	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	1.573	8.87	1	7,130.60	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	4.629	11.97	1	32,091.12	
Bridle	27	1	213	213.00			2.59	1	552.27	
2- Fall Hook Block	28	1	1141	1,141.00			15.00	1	17,115.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-340,201.92**	-340.2 ton*m
										(2,461.2 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.4 - Calculation of load and moment above the slewing ring and load below the slewing ring for a boom angle 62.1. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			20.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	6.411	3.46	1	6,972.83	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	4.305	8.81	1	11,968.21	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	2.152	12.04	1	9,681.63	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	6.332	16.28	1	43,658.25	
Bridle	27	1	213	213.00			3.46	1	735.98	
2- Fall Hook Block	28	1	1141	1,141.00			20.00	1	22,820.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-315,325.54**	-315.3 ton*m
										(2,281.2 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.5 - Calculations of load and moment above the slewing ring and below the slewing ring for a boom angle of 53.9. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			25.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	8.072	4.29	1	8,649.14	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	5.421	11.03	1	14,981.85	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	2.710	15.10	1	12,138.59	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	7.974	20.44	1	54,798.83	
Bridle	27	1	213	213.00			4.29	1	912.92	
2- Fall Hook Block	28	1	1141	1,141.00			25.00	1	28,525.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-291,156.12**	-291.2 ton*m
										(2,106.4 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.6 - Calculations of load and moment above the slewing ring and load below the slewing ring for a boom angle of 44.7 degrees. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			30.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	9.738	5.12	1	10,330.09	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	6.539	13.26	1	18,003.86	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	3.270	18.16	1	14,602.37	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	9.619	24.61	1	65,970.32	
Bridle	27	1	213	213.00			5.12	1	1,090.34	
2- Fall Hook Block	28	1	1141	1,141.00			30.00	1	34,230.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-266,935.46**	-266.9 ton*m
										(1,931.1 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.7 - Calculations of load and moment above the slewing ring and load below the slewing ring for a boom angle of 33.5. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			35.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	11.424	5.96	1	12,031.55	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	7.672	15.51	1	21,062.73	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	3.836	21.26	1	17,096.19	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	11.285	28.82	1	77,278.08	
Bridle	27	1	213	213.00			5.96	1	1,269.93	
2- Fall Hook Block	28	1	1141	1,141.00			35.00	1	39,935.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-242,488.96**	-242.5 ton*m
										(1,754.3 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.8 - Calculations of load and moment above the slewing ring and load below the slewing ring for a boom angle of 16.9 degrees. Prepared by Arup.

Description	Item	Qnty	Weight (kg)	W Sub (kg)	L, m	projection length	X (m)	dynamic factor	M=W*X (kg*m)	
ABOVE SLEW RING:										
ADDED LOAD:				0.00			40.00	1	0.00	
Slew Mount	12	1	5050	5,050.00			0.00	1	0.00	
Monorail Beam & Trolley	13	1	1695	1,695.00			0.00	1	0.00	
Machinery Deck	14	1	12500	12,500.00			-2.69	1	-33,625.00	
Cabin	15	1	1200	1,200.00			1.30	1	1,560.00	
Cabin Support	16	1	300	300.00			1.30	1	390.00	
Counterweights	17	5	7600	38,000.00			-7.36	1	-279,680.00	
Mast & Sheaves	18	1	5765	5,765.00			-4.70	1	-27,095.50	
Mast Ladder	19	1	305	305.00			-6.50	1	-1,982.50	
Mast Platform	20	1	208	208.00			-6.50	1	-1,352.00	
Boom Buffer	21	1	271	271.00			-1.75	1	-474.25	
Slew Ring	22	1	1950	1,950.00			0.00	1	0.00	
Bottom Boom Section- 13.7m	23	1	2018	2,018.00	13.7	13.108	6.80	1	13,730.82	
Boom Extension Section- 9.2m	24	1	1358	1,358.00	9.2	8.803	17.76	1	24,117.66	
Boom Extension Section- 4.6m	25	1	804	804.00	4.6	4.401	24.36	1	19,586.81	
Top Boom Section- 13.7m	26	1	2681	2,681.00	13.533	12.949	33.04	1	88,571.27	
Bridle	27	1	213	213.00			6.80	1	1,449.29	
2- Fall Hook Block	28	1	1141	1,141.00			40.00	1	45,640.00	
Powerpack/ Winches	29	1	13459	13,459.00			-4.70	1	-63,257.30	
Hoist Rope	30	1	4343	4,343.00			-1.30	1	-5,645.90	
Split Deck - Front	31	1	7763	do not count						
Split Deck - Back	32	1	8631	do not count						
			Sum (kg)	**93,261.00**	205.6	kips			**-218,066.61**	-218.1 ton*m
										(1,577.6 kip-ft)
BELOW SLEW RING:										
Tower 393	1	18	4900	88,200.00	194.4	kips				
Climbing Cylinder	2	2	1500	3,000.00						
Side Panel	3	2	4973	9,946.00						
Tie	4	2	907	1,814.00						
Climbing Support	5	2	332	664.00						
Access Platform	6	1	199	199.00						
			Sum (kg)	**103,823.00**	228.9	kips				
			TOTAL WEIGHT AT BASE:	**197,084.00**	434.5	kips				

Table 6.9 - Results of GSA Stage 1 analyses for 238'-1" tower with lateral restraint at the 3rd (EL 22'- 11 ½") and 9th (El. 88'-5 ½") floors in addition to the base. Prepared by Arup.

	BOOM ANGLE* (DEG)	BOOM RADIUS* (FT)	CALCULATED MOMENT AT SLEW RING** (KIP-FT)	T/C COUPLE APPLIED IN GSA† (KIPS)	VERTICAL REACTIONS AT BASE (KIPS)***				BUCKLING FACTOR
					CWT SIDE		BOOM SIDE		Mode 1
					node1	node2	node3	node4	
1	85.9	12.1	-2852.5	207.3	99.9	105.6	115.0	92.3	8.8
2	72.4	43.3	-2519.7	183.1	99.5	106.2	114.5	92.7	8.8
3	70	49.2	-2461.2	178.9	99.4	106.3	114.4	92.8	8.8
4	62.1	65.6	-2281.2	165.8	99.2	106.6	114.1	93.1	8.9
5	53.9	82.0	-2106.4	153.1	98.9	106.8	113.8	93.3	8.9
6	44.7	98.4	-1931.1	140.3	98.7	107.1	113.5	93.5	8.9
7	33.5	114.8	-1754.3	127.5	98.5	107.4	113.2	93.8	8.9
8	16.9	131.2	-1577.6	114.7	98.2	107.7	112.9	94.0	8.9

	HORIZONTAL REACTIONS AT BASE (KIPS)											
	CWT SIDE						BOOM SIDE					
	node1			node2			node3			node4		
	X	Y	Resultant	X	Y	Resultant	X	Y	Resultant	X	Y	Resultant
1	0.17	0.15	0.23	-0.90	5.10	5.18	-0.11	-2.24	2.24	-1.18	-0.16	1.19
2	0.23	0.15	0.27	-0.85	4.94	5.01	-0.05	-2.40	2.40	-1.12	-0.16	1.13
3	0.24	0.15	0.28	-0.83	4.91	4.98	-0.04	-2.43	2.43	-1.11	-0.16	1.12
4	0.27	0.15	0.31	-0.80	4.82	4.89	-0.01	-2.52	2.52	-1.08	-0.16	1.09
5	0.31	0.15	0.34	-0.77	4.73	4.79	0.02	-2.61	2.61	-1.05	-0.16	1.06
6	0.34	0.15	0.37	-0.74	4.64	4.70	0.05	-2.70	2.70	-1.02	-0.16	1.03
7	0.37	0.15	0.40	-0.71	4.55	4.61	0.08	-2.79	2.79	-0.99	-0.16	1.00
8	0.40	0.15	0.43	-0.68	4.46	4.51	0.11	-2.88	2.88	-0.96	-0.16	0.97

Assumptions:
1. There is no weight on hook.
2. There is 83,800 lbs of counterweight.
3. External climber is attached to tower.
4. Tower is restrained against lateral movement at 3rd and 9th floors.
5. Crane is aligned orthagonally with tower.

*** Ref. FAVCO calculations**
**** Positive Moment is towards boom, Negative moment is towards counterweight**
***** Positive indicates compression at base, negative indicates uplift**
† Determined by dividing the "calculated moment at slew ring" by the distance between tower legs, then dividing by two (for two sets of legs)

Table 6.10 - - Results of GSA analyses for 238'-1 tower with lateral restraint at the 3rd floor in addition to the base (i.e. no restraint at 9th floor). Prepared by Arup.

	BOOM ANGLE* (DEG)	BOOM RADIUS* (FT)	CALCULATED MOMENT AT SLEW RING** (KIP-FT)	T/C COUPLE APPLIED IN GSA† (KIPS)	VERTICAL REACTIONS AT BASE (KIPS)*** CWT SIDE node1	CWT SIDE node2	BOOM SIDE node3	BOOM SIDE node4	BUCKLING FACTOR Mode 1
1	85.9	12.1	-2852.5	207.3	249.1	258.3	-36.2	-58.3	5.1
2	72.4	43.3	-2519.7	183.1	231.1	241.1	-19.0	-40.4	5.2
3	70	49.2	-2461.2	178.9	228.0	238.1	-16.0	-37.2	5.2
4	62.1	65.6	-2281.2	165.8	218.3	228.8	-6.7	-27.5	5.2
5	53.9	82.0	-2106.4	153.1	208.9	219.8	2.3	-18.1	5.2
6	44.7	98.4	-1931.1	140.3	199.4	210.7	11.4	-8.7	5.2
7	33.5	114.8	-1754.3	127.5	189.9	201.6	20.5	0.9	5.2
8	16.9	131.2	-1577.6	114.7	180.4	192.5	29.6	10.4	5.2

	HORIZONTAL REACTIONS AT BASE (KIPS)											
	CWT SIDE						BOOM SIDE					
	node1			node2			node3			node4		
	X	Y	Resultant	X	Y	Resultant	X	Y	Resultant	X	Y	Resultant
1	1.83	0.05	1.83	-0.35	-17.01	17.01	0.37	-24.33	24.33	0.4	-0.25	0.47
2	1.7	0.06	1.70	-0.35	-14.6	14.60	0.37	-21.92	21.92	0.27	-0.24	0.36
3	1.68	0.06	1.68	-0.35	-14.18	14.18	0.37	-21.49	21.49	0.25	-0.24	0.35
4	1.61	0.07	1.61	-0.35	-12.87	12.87	0.37	-20.19	20.19	0.18	-0.23	0.29
5	1.54	0.08	1.54	-0.35	-11.61	11.62	0.36	-18.92	18.92	0.11	-0.23	0.25
6	1.47	0.08	1.47	-0.36	-10.34	10.35	0.36	-17.65	17.65	0.04	-0.22	0.22
7	1.4	0.09	1.40	-0.36	-9.05	9.06	0.36	-16.37	16.37	-0.03	-0.21	0.21
8	1.33	0.1	1.33	-0.36	-7.77	7.78	0.36	-15.09	15.09	-0.1	-0.21	0.23

Assumptions:
1. There is no weight on hook.
2. There is 83,800 lbs of counterweight.
3. External climber is attached to tower.
4. Tower is restrained against lateral movement at 3rd floor.
5. Crane is aligned orthagonally with tower.

*** Ref. FAVCO calculations**
**** Positive Moment is towards boom, Negative moment is towards counterweight**
***** Positive indicates compression at base, negative indicates uplift**
† Determined by dividing the "calculated moment at slew ring" by the distance between tower legs, then dividing by two (for two sets of legs)

Table 6.11 - Results of additional static structural modeling with the 3rd floor lateral restraint at its design elevation (EL.22 ft.). Prepared by Arup.

Analysis Case	Displacement at top	Elastic Stability	Forces at Base				Forces at Collar	
			front leg 1		front leg 2		front leg 1	front leg 2
			y	Z	y	z	y	y
	[inch]	[-]	[kip]	[kip]	[kip]	[kip]	[kip]	[kip]
Linear Static	30.50	n/a	-63.28	172.70	-9.85	240.20	51.47	21.40
Static P-Delta	40.23	25% increase on linear static displacements	-108.90	148.40	-14.63	264.50	87.28	36.20
Dynamic Relaxation	41.20	yes	-108.90	147.70	-14.65	265.10	87.56	36.26
Modal Buckling	n/a	lowest load factor = 5.1	n/a	n/a	n/a	n/a	n/a	n/a

Results are from the model:

Instability check_angle 85.9_1_GSRelax_02.gwb

Table 6.12 - Results of GSA Stage 2 analyses for 238'-1 tower with lateral restraint at the 3rd floor in addition to the base (i.e. no restraint at 9th floor). Prepared by Arup.

	BOOM ANGLE* (DEG)	BOOM RADIUS* (FT)	CALCULATED MOMENT AT SLEW RING** (KIP-FT)	T/C COUPLE APPLIED IN GSA† (KIPS)	VERTICAL REACTIONS AT BASE (KIPS)***				BUCKLING FACTOR
					CWT SIDE		BOOM SIDE		Mode 1
					node1	node2	node3	node4	
1	85.9	12.1	-2852.5	207.3	172.7	240.2	0	0	5.1

	HORIZONTAL REACTION AT BASE (KIPS)			
	CWT SIDE		BOOM SIDE	
	node1	node2	node3	node4
	Y	Y	Y	Y
1	63.3	9.85	0	0

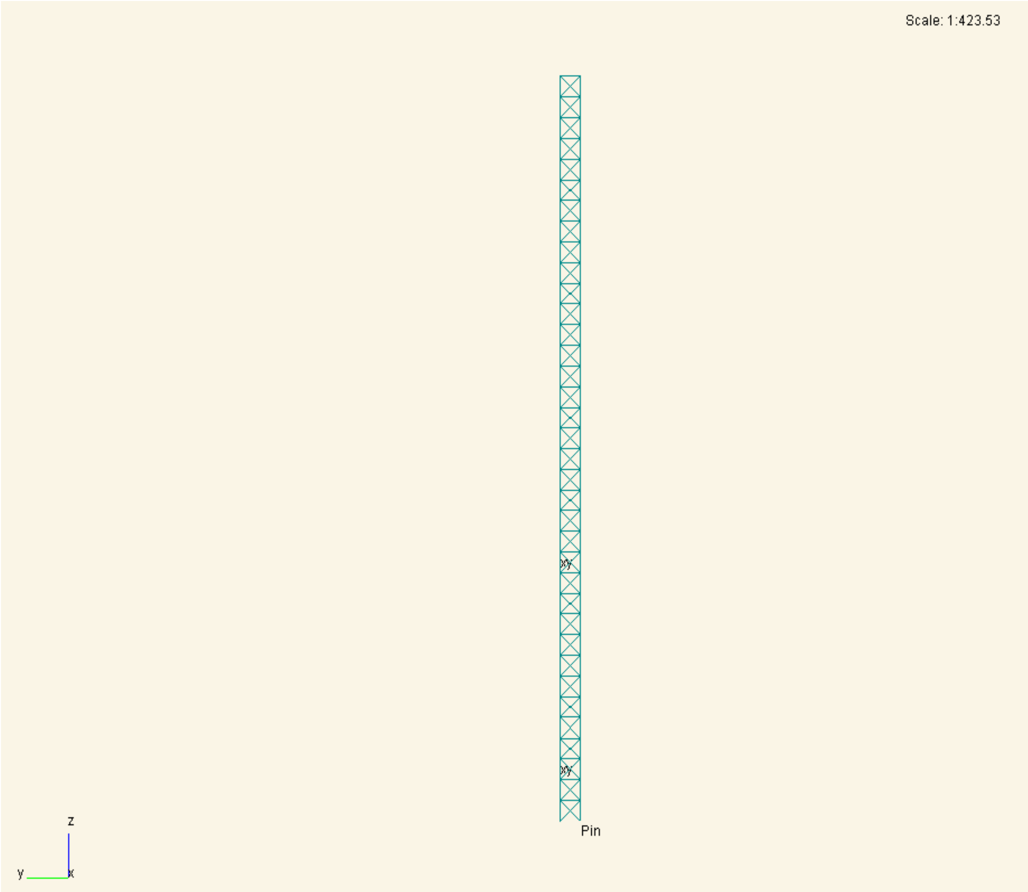


Figure 6.1 - Two-dimensional image of the GSA model of the crane tower. Pin supports (at the base of the four tower legs) and roller supports (at the 3rd floor and 9th floor tie-ins) were added or removed from the model to suit the specific analysis performed as described in this Section of the report. Prepared by Arup.

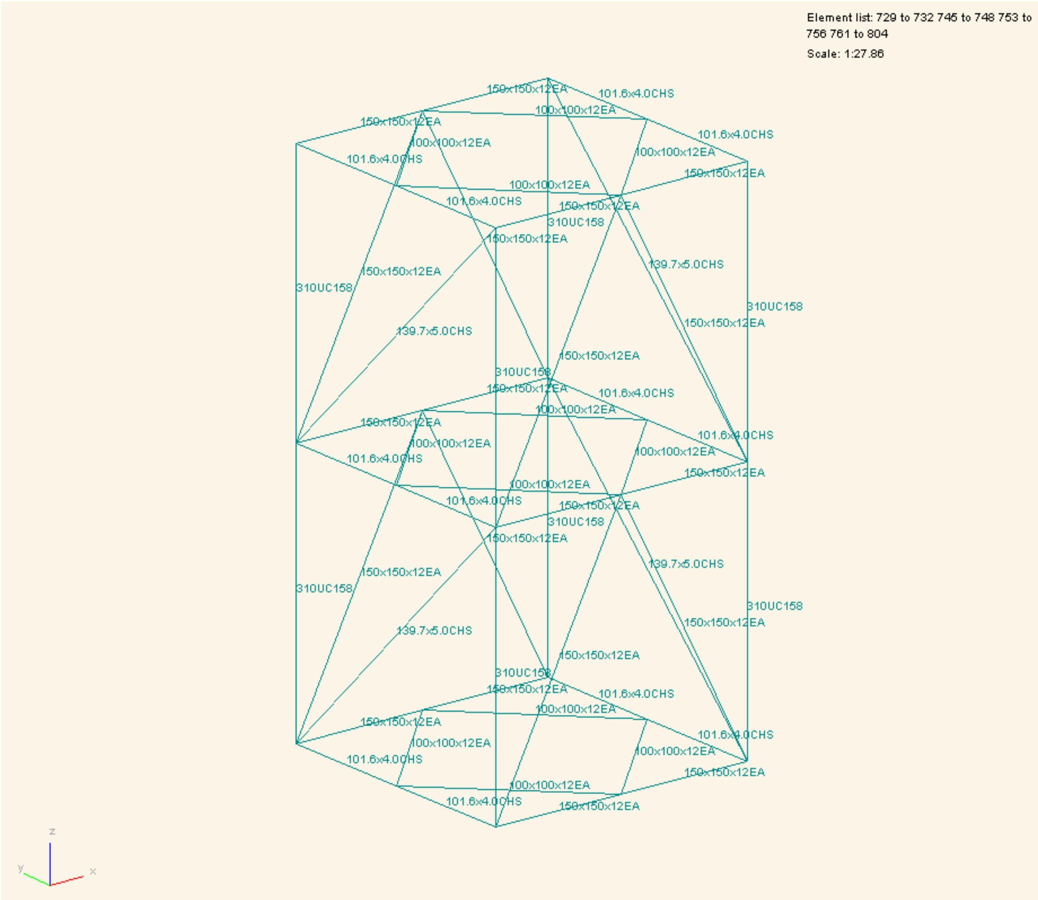


Figure 6.2 - Three-dimensional image of a 393 tower sections from the GSA model. Prepared by Arup.

7 Dynamic Analysis: Collar Integrity and Tower Stability

7.1 Introduction

This chapter summarizes two distinct nonlinear dynamic analyses used to model key events in the collapse sequence.

The first of these is a non-linear dynamic finite element analysis of the impact on the 9th floor collar assembly and presents the results from the analysis. Specifically, the model simulates the impact of the 18th floor collar onto the 9th floor collar in the as-designed configuration based on currently available information. The modeling was based on a combination of drawings, on-site measurement of components and photographic evidence.

In addition to the 9th floor collar impact analyses, a study into the post impact global stability of the overall tower was undertaken. To assess this the model was analyzed for an increased duration in order to establish whether the dynamic effects of the impact on the crane and tower were enough to cause instability after the 9th floor collar connection was destroyed

To facilitate analysis, material properties, loads and dimensions were input using metric units. In the discussion which follows the input values using metric units are therefore provided alongside the original data using English units.

7.2 Supplied Data

The input data included the following items:

1) The following drawings have been taken from ref [1] and ref [4], and can be found in Appendix F:
    a. Tower crane elevations and jump schedule (Figure F1.1)
    b. Collar and tie-beam part plan and details (Figure F1.2)
    c. Tie beam A, B & C Detail (Figure F1.3)
    d. Dunnage beam shop drawing (Figure F1.4)
    e. Crane truss (prototype) (Figure F1.6)
2) Drawing of 9th floor plan (Figure F1.5, Appendix F).
3) Photographs of the failed assembly pieces and of the connections to the building.
4) Sketches of the failed collar assembly pieces with notable measurements including overall dimensions and thicknesses.
5) Tower crane engineer's submission (by Stroh Engineering), ref [1].
6) Prototype applications for M440-D Tower Crane, ref [2].
7) Operating, Maintenance and Parts Manual, ref [3].
8) Drawings for prototype application for M440-D tower crane, ref [4].

Note: The as-built crane differs in some dimensions from the prototype defined in the supplied documentation. This difference has been addressed by using on-site measurements

7.3 Description of Model

7.3.1 Idealization of Structure

The tower crane, steel collars, tie-beams and 9th floor concrete slab have all been idealized in order to define a finite element (FE) representation of the tower crane system, as shown in Figure 7.1 and Figure 7.2. This section of the report describes the assumptions made when defining each component and how they interact with each other.

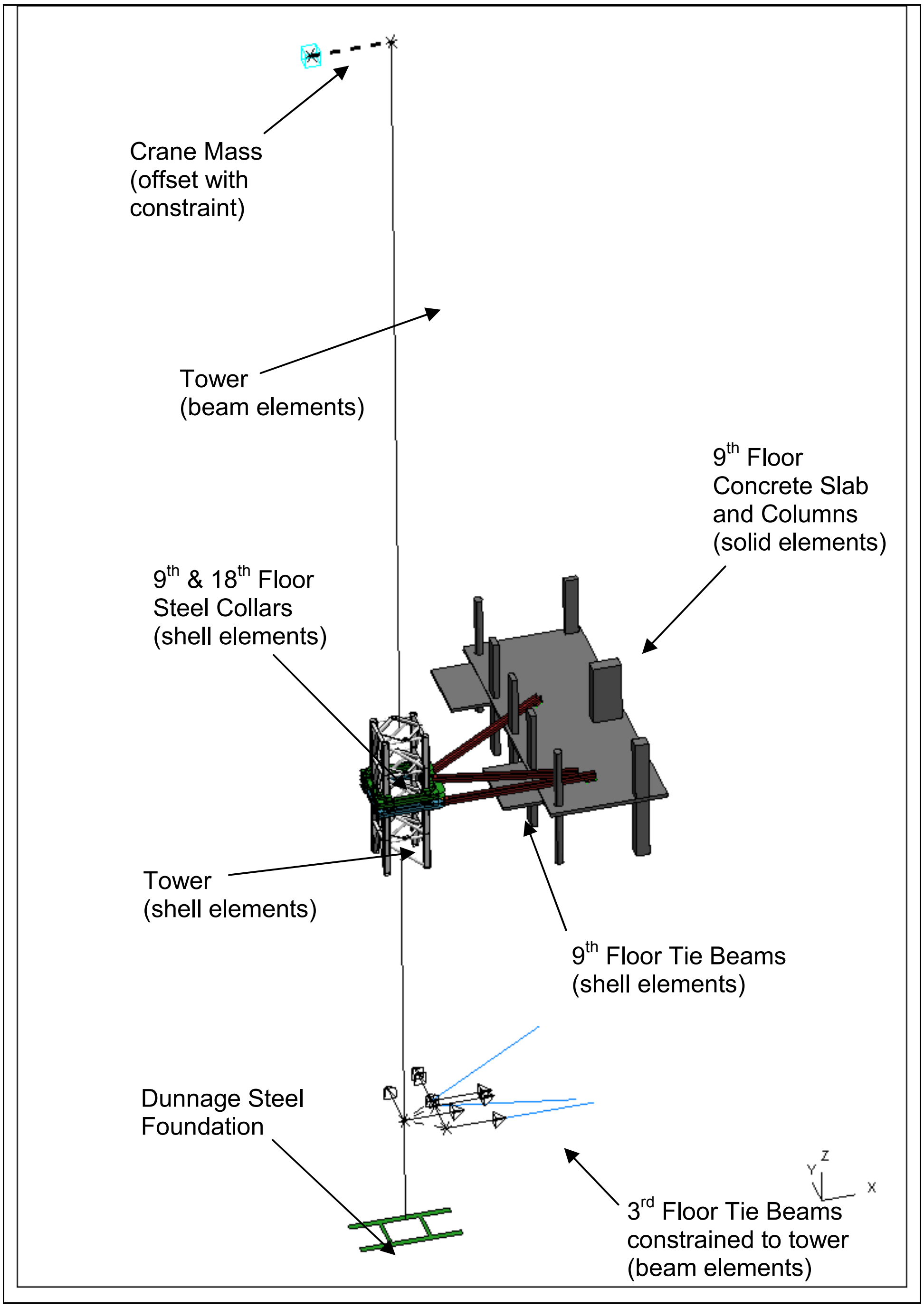


Figure 7.1 - Global view of FE Model. Prepared by Arup.

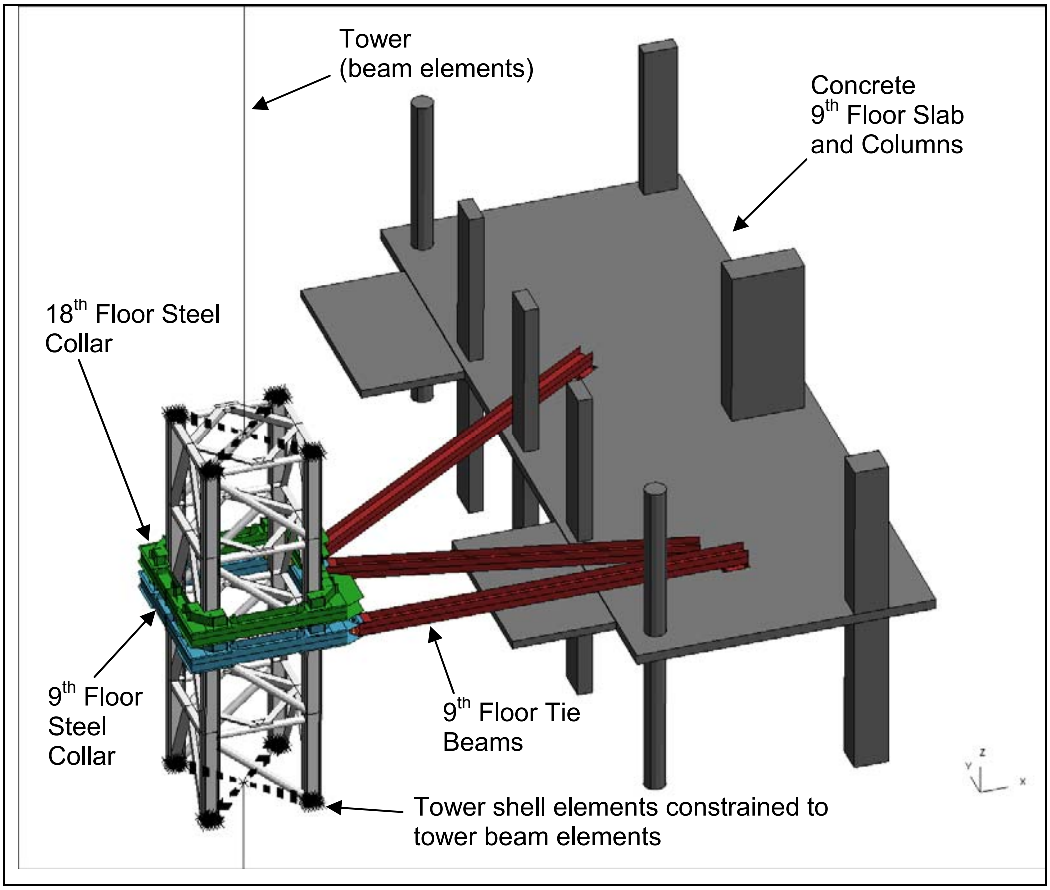


Figure 7.2 - Close-up view of Detailed Area of FE Model. Prepared by Arup.

7.3.1.1 Crane Structure

The crane above the top of the tower has been modeled as a lumped mass of 202×10^3lb (91.6 metric tonnes), located 15ft (4.56m) from the tower centre-line horizontally away from the building and 15.7ft (4.78m) vertically above the top of the tower. The centroidal mass moment of inertia was also included, defined as 1.48×10^8lb.ft^2 (6,250,108kg.m^2). This simulates the rotational inertia of the crane about its centre of mass.

The direction of the counterweight overhang has been defined as away from and perpendicular to the building, consistent with the available photographic and operational information as well as witness statements. It is our professional opinion that the effect on the analysis results of any minor deviation from this assumption will be negligible and will have no effect on the overall conclusions.

7.3.1.2 Crane Tower

The main area of interest with regard to collar impact is within the region of the 9th floor collar at 85.3ft (26m). This area has been modeled in fine detail using shell elements, as can be seen in Figure 7.3. This consists of two full tower segments each of 13.1ft (4m) height, located with one segment above the 9th floor and one below. The tower truss segments include a combination of I-section, L-section and tubular members, all of which were modeled explicitly in shell elements. The truss was orientated such that the tubular members were perpendicular to the edge of the building. The material properties for the shell element representation of the tower can be seen in Table 7.1 and 7.2.

The segments of the tower away from the area of interest, i.e. from the dunnage steel to a height of 72.2ft (22m) and from 98.4ft (30m) to 236.2ft (72m), have been modeled more abstractly using beam elements. The beam element tower was constrained to the shell main I-sections of the tower using rigid body constraints. This transfers all axial force, shear force and bending moments through the connection, as illustrated by the black dashed lines in Figure 7.2.

Linear elastic material properties were assumed for the beam elements (see Table 7.1 and 7.2). The area and bending properties were based on the four main corner I-sections of the truss. The torsional and shear stiffness were based on FE shell model tests of the tower. The section properties used for the beam representation of the tower were as follows:

Area = 95in^2 (0.0613m^2)

I_{yy}/I_{zz} = 162,260in^4 (0.0675m^4)

J = 54,808in^4 (0.0228m^4)

Shear Area = 9.9in^2 (0.00636m^2)

The densities for both the shell and beam elements were factored to match the actual labeled 13.1ft (4m) segment mass of 10,692lb (4.85 metric tonnes). The total tower mass was therefore 192,574lb (87.35 metric tonnes) for 236.2ft (72m) of tower. Factoring the mass takes into account the non-structural components that have not been included in the model, e.g. ladders and decks.

For contact with the collar chocks (see Section 7.3.1.4), additional elements perpendicular to the ends of the main I-section flanges have been included.

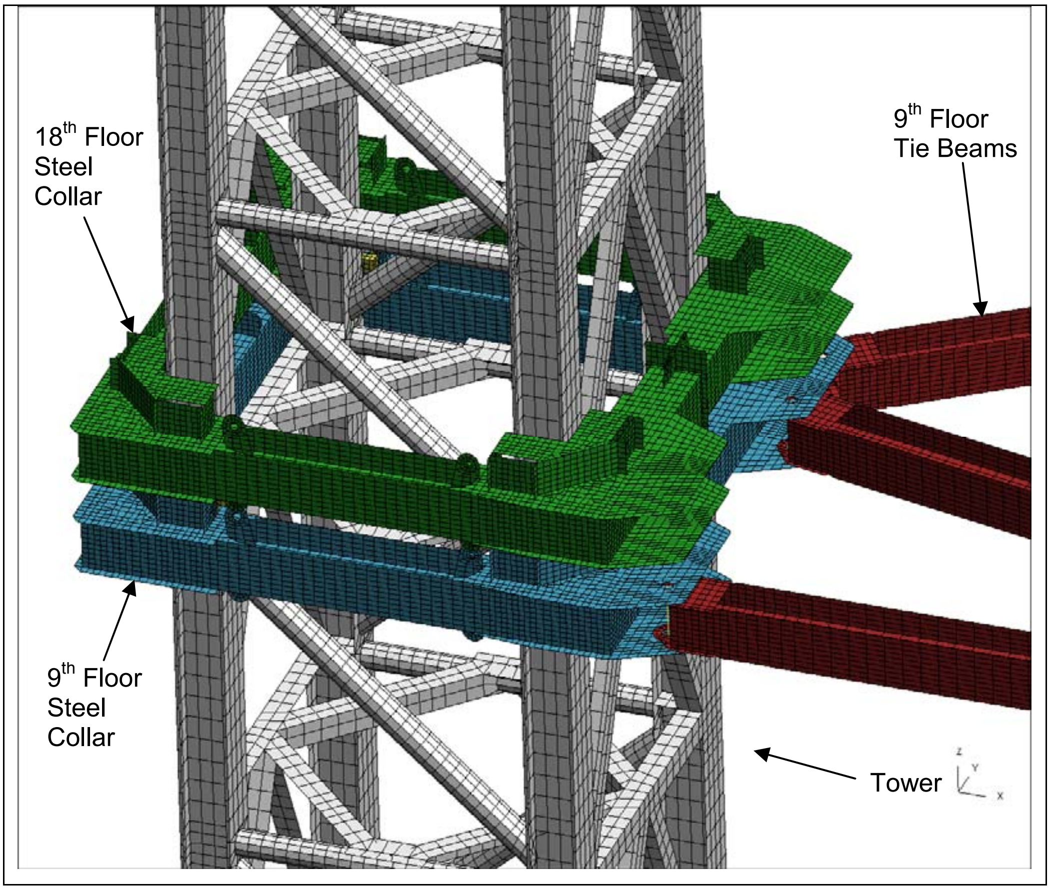


Figure 7.3 - Close-up view of Collars and Tower. Prepared by Arup.

7.3.1.3 9th & 18th Floor Structural Steel Collars

Each collar consists of steel plated I-sections with the flanges aligned vertically with the 2in (0.050m) thick top and bottom plates and chock housings (shown in Figure 7.5) at each corner. The collars are made in two halves that are bolted together around the tower. Both the 9th and 18th floor steel collars were modeled using shell elements, as can be seen in Figure 7.3. The two halves were constrained to each other. The collar density has been factored to match the labeled total collar mass of 11,279lb (5.12 metric tonnes).

Contact has been modeled between the top plates of the chock housings on the 9th floor collar and the bottom plate of the 18th floor collar. The tallest lugs have not been included in the contact element groups.

The material properties used for the collars can be seen in Table 7.1 and 7.2.

The 9th floor tie-beams were connected to the collar with 3in (0.076m) diameter pins (Note: the pins were not connected at the 18th floor collar). The pins were modeled using beam elements. The holes were modeled at the pin diameter with the pin end nodes constrained to the bolt hole nodes of the collar, as can be seen in Figure 7.4. The material properties assumed for the pins can be seen in Table 7.1 and 7.2.

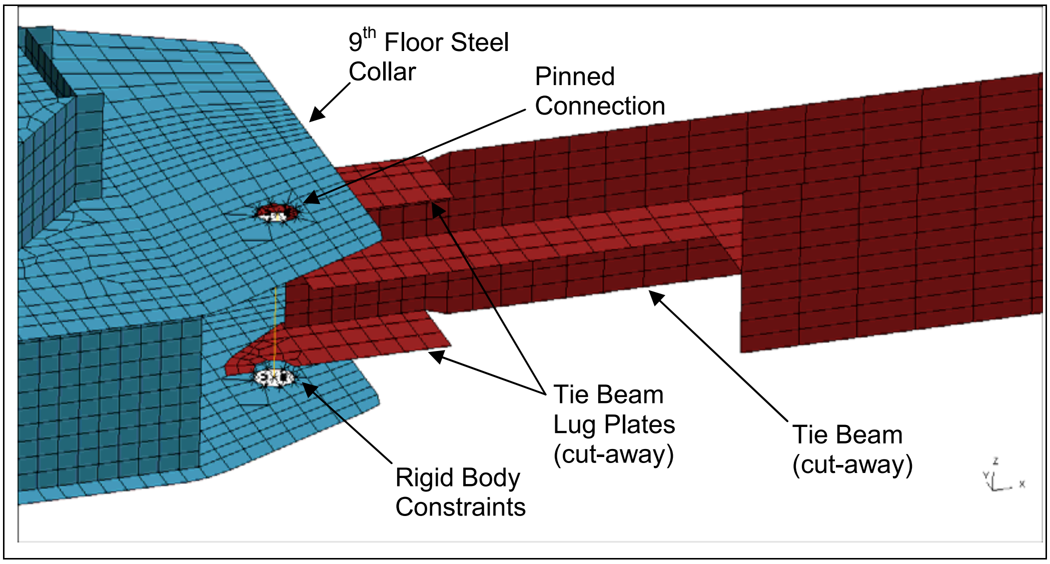


Figure 7.4 - Close-up view of Tie-Beam to Collar Connection (tie-beam cut-away). Prepared by Arup.

7.3.1.4 Chocks

The chocks have been modeled with solid elements in the 9th floor collar only, as the chocks at the 18th floor would not have been deployed at the time of the accident. The model's 9th floor chock elements were constrained to their respective chock housings. No preload has been applied to the chocks, i.e. they have been assumed to be just touching the tower. Although the chocks may have been pre-loaded in reality, the level of pre-load is currently unknown.

The material properties assumed for the chocks can be seen in Table 7.1 and 7.2.

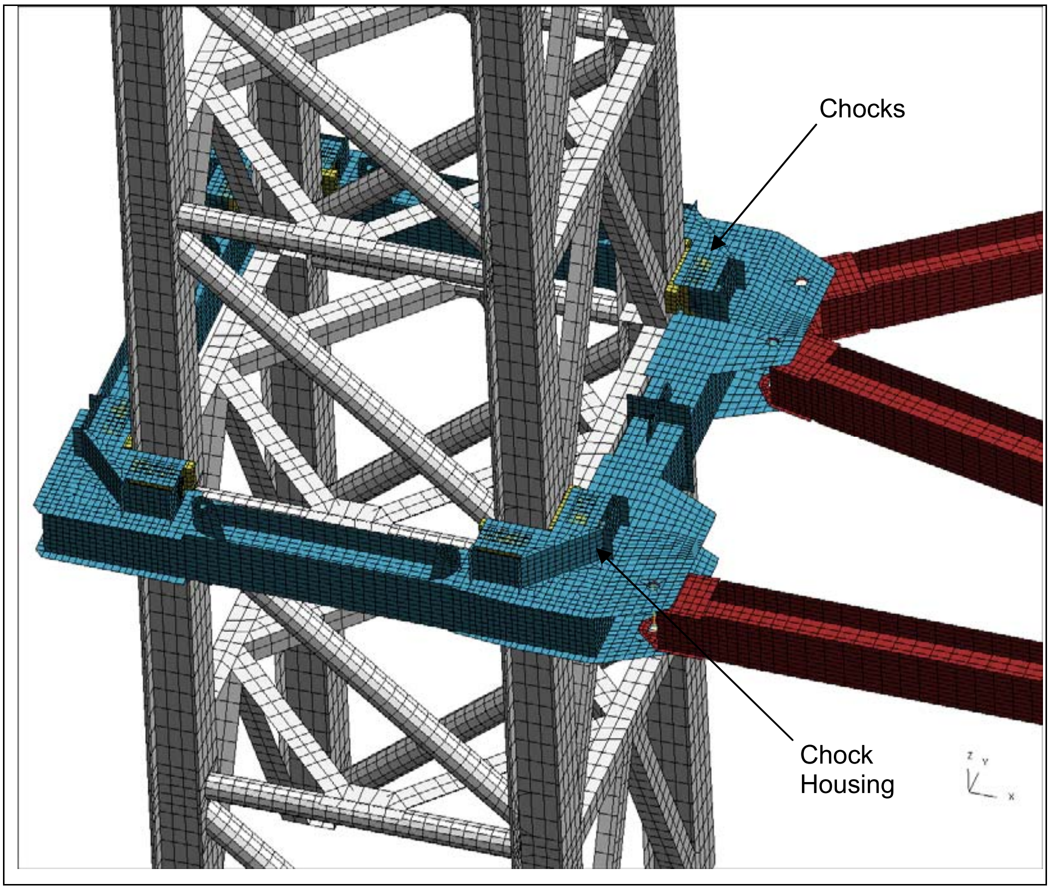


Figure 7.5 - View on 9th Floor Collar and Chocks. Prepared by Arup.

7.3.1.5 3rd Floor Tie-Beams

The 3rd floor tie-beams have been modeled with beam elements connected back to fixing points corresponding to the building and constrained horizontally (x & y) to the tower, i.e. free to slide vertically (see Figure 26). The section properties were based on AISC beam type W12x79, and a linear material has been assumed (see Tables 7.1 and 7.2).

7.3.1.6 9th Floor Tie-Beams

The 9th floor tie-beams and base plates were modeled using shell elements, as shown in Figures 7.5 and 7.6. The section dimensions were based on AISC beam type W12x79. The material properties used for the tie-beams can be seen in Tables 7.1 and 7.2.

The fillet weld connection of the tie-beam to the 9th floor base plate was modeled using shell elements with the thickness of the weld throat and the height of the weld leg, 0.22in (0.0056m) and 5/16in (0.0079m) respectively, as specified in the shop drawing (Figure F1.3, Appendix F). The weld runs the full 1.83ft (0.56m) length of the base plate along one side of each tie-beam flange, as specified. The ultimate strain for the weld elements was defined as 6.5% ref [9]. The material properties assumed for the base plate welds can be seen in Tables 7.1 and 7.2.

The floor slab base plate (see Figure 7.6) corner thickener plates have been assumed to be attached to the base plate over their whole area, in reality they are welded around the edges. The bolt holes were modeled at their correct diameter with the nodes around the holes constrained to the top nodes of the beam elements representing the bolts. It is noted that no failure of these components was observed from the recovered debris. There would therefore be no effect on the behavior of the model from this assumption.

The 1.5in (0.038m) anchor bolts were modeled as beam elements inlayed through the floor slab. The material properties used for the bolts can be seen in Tables 7.1 and 7.2.

The holes in the lug plates (see Figure 7.4) at the collar end of the tie-beams were modeled at the correct diameter with the nodes around the edge constrained to the 3in pins (see Section 7.3.1.3) in the horizontal (x & y) directions. This allows the tie-beam lug plates to slide vertically relative to the pin. Contact has been modeled between the tie-beam lugs and the collar, providing vertical constraint between the tie-beams and collar. The supplied photographs of the lugs indicate thickening plates around the pin connections; these have not been included as they do not appear on the drawings supplied.

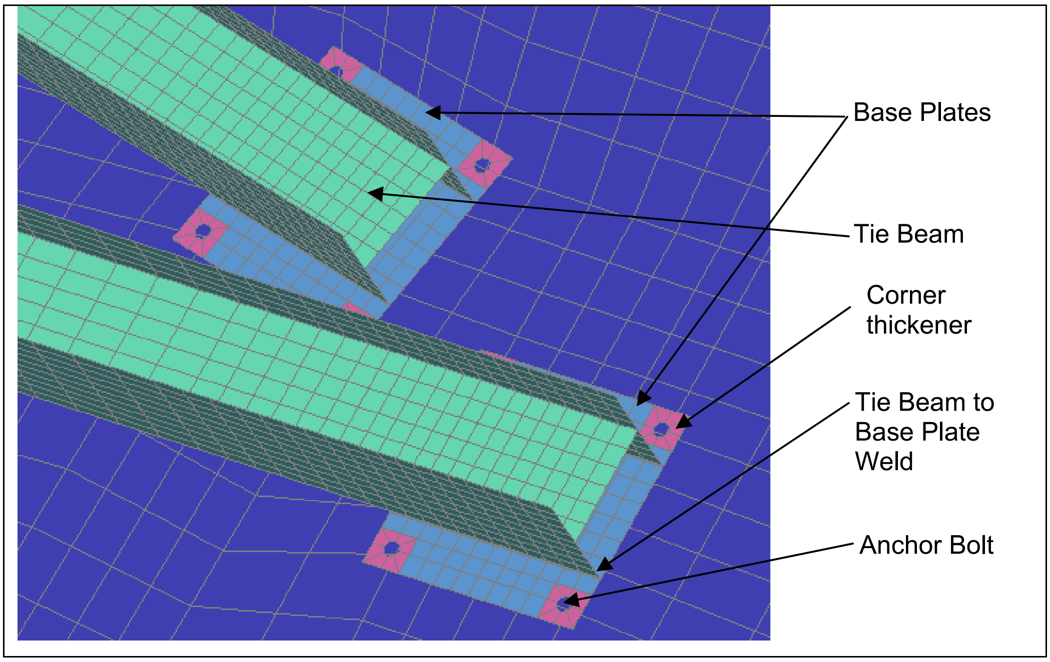


Figure 7.6 - Tie-Beam/Base Plate Connection to Floor Slab. Prepared by Arup.

7.3.1.7 Wire Rope System

Additional support to the 9th floor collar prior to the impact of the 18th floor collar was provided by four wire ropes. The wire rope used was ¼˝ 7x19 aircraft cable, attached between lugs on the collar and the tower, as shown in Figure 7.7. Details regarding their connection to the tower have not been provided. It has been assumed that they were attached at the cross members on the tower above the 9th floor collar at panel points adjacent to the tower legs. We note that the analytical results demonstrated that these cables would fail under the most favorable of circumstances and, as such, the precise nature of the cable attachment was not relevant to the analysis.

The load deflection curve assumed for the wire ropes, see Figure 7.8, was based on the following assumptions. Existing specifications for 7x19 aircraft cable stipulate that the maximum stretch is 1.5% for 60% of the minimum breaking load, which defines the stiffness, ref [8]. Similarly, the maximum breaking load is 7000lb (31.14kN), based on the larger galvanized rope value (rather than 6400lb for corrosion resistant steel), ref [8]. The capacity was then reduced by 20% to 5600lb (24.9kN) taking into account connection efficiency when using compression clips, ref [11]. The post yield ductility was based on load deflection data for a single strand from ref [7]. Although based on a single strand, the post yield extension of approximately 0.6% is representative of the cold drawn carbon steel used for wire rope manufacture.

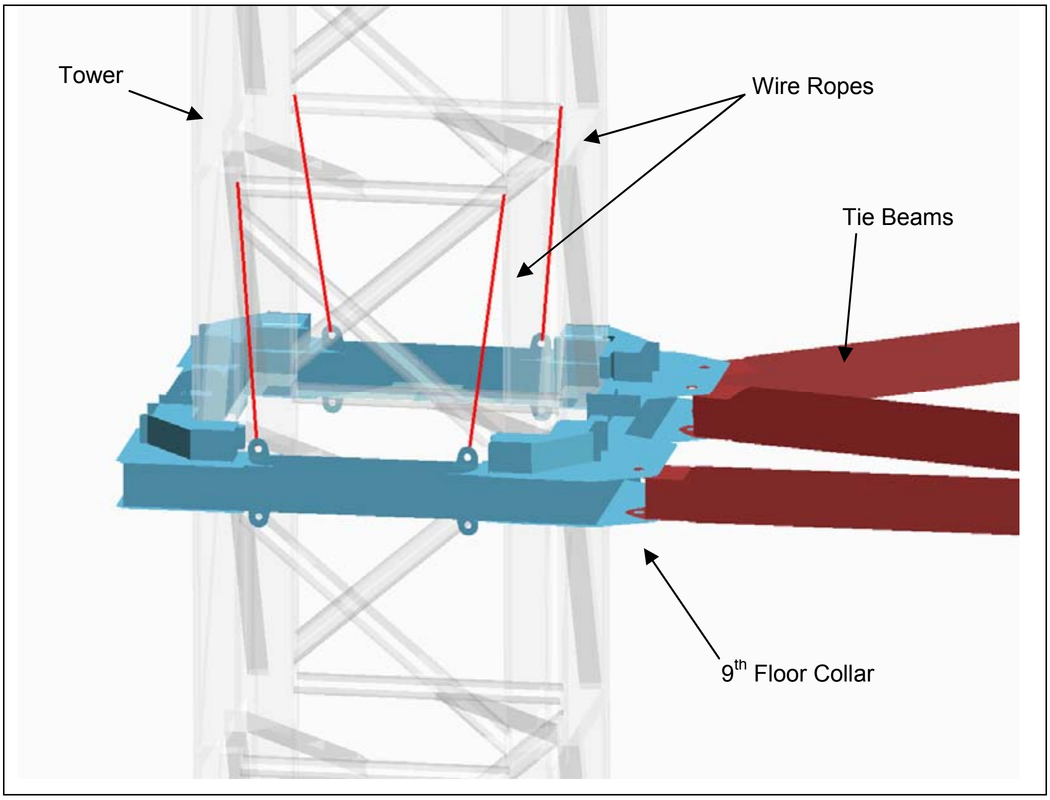


Figure 7.7 - Mesh plot indicating Wire Ropes in red. Prepared by Arup.

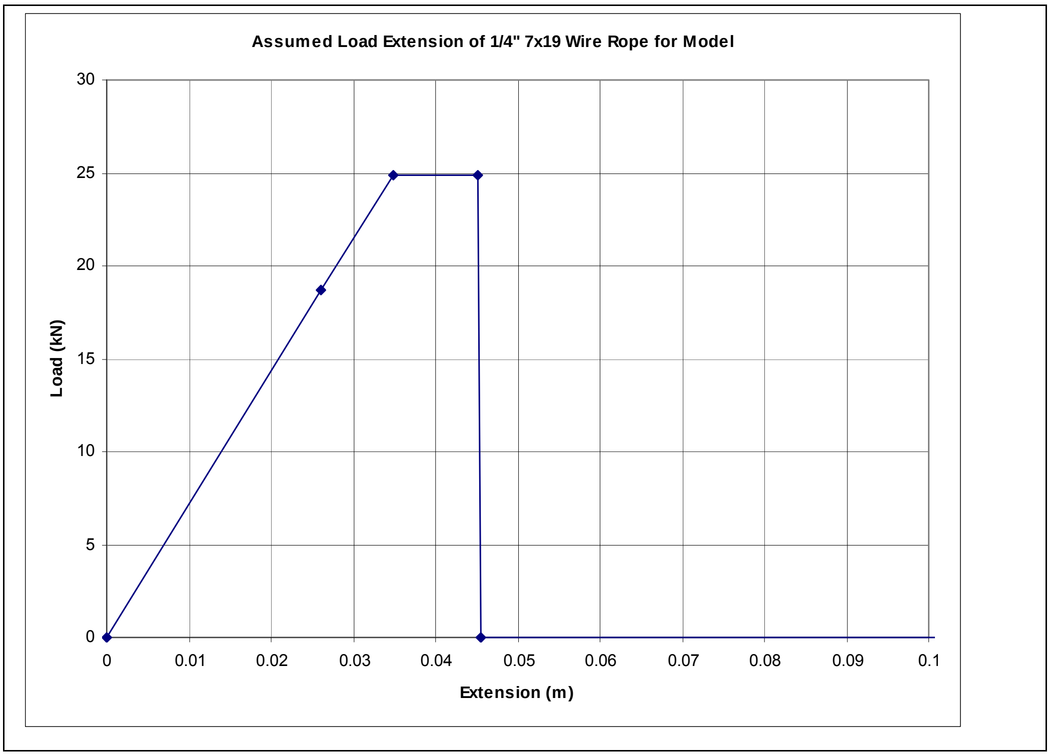


Figure 7.8 - Assumed Load Extension Curve for Wire Ropes. Prepared by Arup.

7.3.1.8 9th Floor Concrete Slab and Columns

The concrete slab and columns have been modeled using solid elements, with the columns fully fixed one floor above and below, as can be seen in Figure 7.2. The material properties assumed for the concrete can be seen in Tables 7.1 and 7.2.

7.3.2 Foundation

The foundation dunnage steel has been assumed to be rigid but has been defined as the correct footprint of the I-sections, as can been seen in Figure 7.1. Contact with friction between the dunnage steel and plywood base (modeled as a rigid plane) was simulated. This allows the dunnage foundation to rock or slide if necessary.

Adjustments to the modeling of the tower to the dunnage steel interface were made for the global stability analysis, as described in Section 7.5 of this Chapter.

7.3.3 Material Properties

The material properties assumed are tabulated in Table 7.1 for imperial units and Table 7.2 for metric units.

The tie-beam material has been defined as ASTM A36 steel, as specified in the drawing (Figure F1.3, Appendix F). The 1.5in (0.038m) diameter bolt material was specified as A193 B7 threaded rod on the same drawing. A193 material has also been assumed for the 3in (0.076m) diameter pin at the tie-beam to collar connection as this was not specified.

The tower has been defined as AS3678-350 plate (equivalent to ASTM A572-50) for the main I-sections and AS3678-250 plate (equivalent to ASTM A36) for the secondary members, ref [2].

The collar and chocks were assumed to be ASTM A36 steel.

All of the non-linear elastic-plastic steel properties have been defined with strain rate dependent properties. The strain rate effects in steel have been modeled using the Cowper-Symonds relationship. The values of the constants used in the relationship for mild steel have been defined as $C=40.4s^{-1}$ and p=5, as recommended in ref [5] for strains of 2-4% which is appropriate for the majority of the tie-beam steelwork.

The concrete grade of 8ksi (55MPa) has been taken from Figure F1.5 (Appendix F). As the floor is loaded in bending it was assumed to behave in an elastic perfectly-plastic manner. The photographic evidence suggests that the damage to the concrete is limited to the balcony section. Therefore, the yield stress was back calculated from the plastic moment of the reinforced concrete balcony section and the plastic section modulus of the balcony geometry. The plastic moment was calculated assuming a standard rebar grade of 60ksi (414MPa) with bar size 6 (Φ19.05mm) at a spacing of 12in (0.305m).

Table 7.1 - Material Properties used in Model (Imperial units). Prepared by Arup.

Components	Young's Modulus (ksi)	Poisson's Ratio	Density (lb/in^3)	Yield Stress (ksi)	Post-Yield Stiffness (ksi)	Ultimate Strain
Tower I-Sections (shells)	30,450	0.3	0.33*	50.8	73.1	18%
Tower Secondary Members (shells)	30,450	0.3	0.33*	36.3	106.0	18%
Tower (linear beams)	30,450	0.3	0.72*	N/A	N/A	N/A
Tie-Beam (shells)	30,450	0.3	0.28	36.3	106.0	18%
Tie-Beam (linear beams)	30,450	0.3	0.28	N/A	N/A	N/A
Base Plate Weld	30,450	0.3	0.28	36.3	106.0	6.5%
Top Collar	30,450	0.3	0.26*	50.8	73.1	18%
Bottom Collar	30,450	0.3	0.26*	50.8	73.1	18%
0.038m Bolts	30,450	0.3	0.28	105.0	127.8	16%
0.076m Pins	30,450	0.3	0.28	105.0	127.8	16%
Concrete Slab and Columns	4,350	0.2	0.087	1.0	0	N/A

*Note: Factored to equal actual mass

Table 7.2 - Material Properties used in Model (metric units). Prepared by Arup.

Components	Young's Modulus (MPa)	Poisson's Ratio	Density (t/m^3)	Yield Stress (MPa)	Post-Yield Stiffness (MPa)	Ultimate Strain
Tower I-Sections (shells)	$210x10^3$	0.3	9.17*	350	504.2	18%
Tower Secondary Members (shells)	$210x10^3$	0.3	9.17*	250	731.2	18%
Tower (linear beams)	$210x10^3$	0.3	19.81*	N/A	N/A	N/A
Tie-Beam (shells)	$210x10^3$	0.3	7.85	250	731.2	18%
Tie-Beam (linear beams)	$210x10^3$	0.3	7.85	N/A	N/A	N/A
Base Plate Weld	$210x10^3$	0.3	7.85	250	731.2	6.5%
Top Collar	$210x10^3$	0.3	7.27*	250	731.2	18%
Bottom Collar	$210x10^3$	0.3	7.27*	250	731.2	18%
0.038m Bolts	$210x10^3$	0.3	7.85	724	881.5	16%
0.076m Pins	$210x10^3$	0.3	7.85	724	881.5	16%
Concrete Slab and Columns	$30x10^3$	0.2	2.4	6.88	0	N/A

*Note: Factored to equal actual mass

7.3.4 Friction Coefficients

Frictional contact was simulated between the components shown in Table 7.3. The assumed upper and lower bound friction coefficients are also defined in Table 7.3.

Table 7.3 - Friction Coefficients used in Model. Prepared by Arup.

	Lower Bound	Upper Bound	Description
Top Collar to Tower	0.15	0.5	Steel onto steel
Bottom Collar to Tower	0.15	0.5	Steel onto steel
Chock Blocks to Tower	0.15	0.5	Steel onto steel
Tie-Beams to Collar	0.15	0.15	Steel onto steel
Tie-Beams to Concrete Slab	0.3	0.3	Steel onto concrete
Collar to Collar	0.15	0.15	Steel onto steel
Dunnage Steel to Plywood	0.35	0.35	Steel onto wood

Notes:

1. A representative friction coefficient of 0.35 was used to represent the interface between the dunnage steel and plywood for the global tower stability analysis. There was only one load case reported for this global stability. The actual loading is just the collar impact.
2. The friction coefficients were based on values listed in Bautabellen für Ingenieure, Klaus-Jürgen Schneider, 13^{th} Edition 1998- Reference 6.

7.3.5 Impact Velocity

The analyses began with the 18^{th} floor collar situated immediately above the 9^{th} floor collar and moving with a velocity associated with its fall to that point. Upper and lower bound values for the impact velocity were used in the analyses. The upper bound impact velocity, 79.5ft/s ($24.2ms^{-1}$), of the collar assumes that it has not had any retardation during its approximate 100ft (30m) fall from the 18^{th} floor. A lower bound was also considered to allow for wind resistance on the falling collar as well as friction with the tower. We note however that there was little damage to the paint on the tower above level 9; suggesting little effect from friction. The lower bound was assumed to be 39.8ft/s ($12.1ms^{-1}$). This magnitude, while considered as extreme as a minimum based upon considerations of possible frictional resistance only, is thought to be a reasonable value for the lower bound based on half the velocity, and hence, a quarter of the energy of the full impact velocity when also considering possible obstructions to free-fall of the collar. It is a very conservative and underestimated value used in the analysis.

7.3.6 Software and Analysis Procedure

The analyses were performed using the commercially available software LS-DYNA written by Livermore Software Technology Corporation. This is an explicit finite element code specifically written to simulate non-linear dynamic events such as impact.

Each analysis was performed in two stages. The initial stage involved the application of self-weight of the structure, defining the pre-collapse condition. The pre-collapse condition has the correct initial stress conditions for all of the components of the tower crane assembly including the friction at the foundation.

For the second analysis stage, the initial velocity is applied to the 18^{th} floor collar prior to the impact.

7.4 Results for 9^{th} Floor Collar Impact Analyses

The assumed parameters for each analysis are shown in Table 7.4.

Table 7.4 - List of Analyses with Varied Parameters. Prepared by Arup.

Run	Chock Friction Coefficient	Drop Velocity
A	0.15	79.5ft/s (24.2m/s)
B	0.15	39.8ft/s (12.12m/s)
C	0.5	79.5ft/s (24.2m/s)
D	0.5	39.8ft/s (12.12m/s)

7.4.1 Description of Behavior

7.4.1.1 Run A – v=79.5ft/s, 0.15 Friction

As the 18th floor collar falls onto the 9th floor collar, the force transferred causes an almost immediate failure in the base plate welds. Although the tie-beams detach from the 9th floor slab, the collars slow down until vertical equilibrium is reached. The collars eventually do fall; however this portion of the analysis was stopped after the welds failed, which occurred first, this is discussed further in Section 7.5. The collars do not immediately fall through to the 3rd floor. Subsequent analysis for the tower stability shows that they do fall through. It happens later in the analysis that the first run captures.

In any case, this aspect of the response is not strictly intended and does not capture reality; specifically the pin connections at the collar are not modelled in sufficient details to capture the rupture of the lug. What the analysis does demonstrate is that even with an overestimate with the lug strength, there is sufficient energy in the falling collar to cause the system to fail.

The collars are held through a combination of friction with the chocks on the tower and support from the tie-beams contacting the floor slab. Extreme plasticity is observed (up to 18.0% strain) in the tie-beam lug plates adjacent to the collars. As a result of the weld failure to the base plates, the three tie-beams remain relatively undeformed, and therefore do not absorb significant amounts of energy.

Without the tie-beam attaching the tower back to the building the tower moves away from it. This is illustrated by the negative x direction displacements of the top of the tower shown in Figure 7.12. The tower stability post impact is studied further in Section 7.5.

7.4.1.2 Run B – v=39.8ft/s, 0.15 Friction

The base plate welds fail within the first 0.25 seconds of impact but the collars do not fall through. The vertical displacement of the collar is about half as much as run A, although the end result, i.e. the tower detaching from the building, is the same.

7.4.1.3 Run C – v=79.5ft/s, 0.5 Friction

A similar model configuration to Run A except the friction between the chocks and the tower was increased to 0.5. The base plate welds still fail almost instantaneously but the peak plasticity in the tie-beam lug plates adjacent to the collars reduces to 9.0%, approximately half the magnitude found in Run A. Approximately 25% more energy is absorbed by friction due to the increased friction coefficient compared to Run A, see Figure 7.14.

7.4.1.4 Run D – v=39.8ft/s, 0.5 Friction

A similar model configuration to Run B except the friction between the chocks and the tower was increased to 0.5. Again, all the base plate welds fail, although not as quickly as in the other analyses, and the collars do not fall through.

7.4.1.5 Model Limitations

The analysis does not predict the failure of the tie-beam to collar connection as was observed for all three beams at Level 9. As described below, the model is not refined enough in these regions to capture that failure mode. Although a more refined analysis could be carried out, we have refrained from that for two reasons. First, even though our analysis overestimates the tie-beam connection strength at the collar (and captures it accurately at the base plate), it predicts a failure of the overall assembly of the connection to the building; a sufficient condition to demonstrate overturning of the tower as described in the next section. Second, the behavior is highly dependent on fine details that are unknown, such as the exact shape of the pin holes, the exact configuration of the weld, and any flaws in the base material. This sensitivity is evident from the fact that all three connections failed in similar, but different modes.

The two most important features that would require refinement to capture failure at the collar are the tie-beam flange to lug plates weld and the tie-beam to collar pinned connection as described below.

The results indicate that the pinned connection to the tie-beam lugs requires more detail in order to fully simulate the behavior. The constraints that connect the lugs to the pins hold the hole in a circular shape and therefore prevent the hole from stretching and ovalizing, i.e. with tension and compression zones. The lug can still deform adjacent to the hole but the stress distribution is not quite correct. As mentioned above, the analysis does demonstrate that even with an overestimate with the lug strength, there is sufficient energy in the falling collar to cause the system to fail.

The welds between the tie-beam lug plates and the tie-beam flanges would require a more detailed mesh representation in order to more accurately reflect their non-linear behavior. A similar approach to that used for the base plate weld was tried, however, the initial impact of the 18th floor collar caused a high compressive force to be transferred through the weld resulting in an unrealistic failure. Rather than failing the lug plate weld in compression, the load was transferred in bearing through the tie-beam flange; whereas, in tension or shear, the load is transferred through the weld. The current model configuration defines full connectivity between the lug plate and the tie-beam flange, i.e. the weld has not been considered. This configuration over estimates the strength of the connection between the lug plates and tie-beam flanges when a tensile or shear load is transferred.

In view of the previous comment, it is felt that the model is not sufficiently refined in the region of the pinned tie-beam to collar and the tie-beam lug plate to flange weld connections to accurately predict failure here. The high levels of plastic strain suggest a potential area of concern and further refinement of modeling could lead to failure being predicted in this region. Although the model does not predict failure in the tie-beam to collar connection it does predict failure between the tie-beams and the base plates. Any weakening of the tie-beam to collar connection would increase the predicted rotation of the tie-beams and correspondingly increase the demand on the tie-beam to base plate connection, therefore making this area even more likely to fail. Therefore, regardless of potential demonstrations of failure at the pin connection that such a refinement might provide, the overall conclusions from these analyses regarding the behavior of the tower crane would remain unchanged.

The shackle lugs that were observed to be completely squashed have not been included in the model, these would absorb some of the impact energy and therefore reduce the predicted damage. However, it is our professional opinion that such effects will not have any influence on the overall behavior of the tower crane and, therefore, our conclusions.

Also, the instructions for the installation of the chocks states that they should 'tighten chocks uniformly onto the tower legs using a rattle gun or similar'. If pre-load were applied the amount of energy absorbed through friction would increase. Sufficient preload could potentially change the behavior of the overall system. However, in reality, the collars did impact each other and proceed to the 3rd floor slab level, it is therefore our conclusion that any pre-load which may not be represented in the model would have an insignificant effect on the model behavior and no effect on our overall conclusions.

7.4.2 Deformed Shape Plots

The deformed shape of the complete tower under self weight is shown in Figure 7.9.

The configuration of the model analyzed has the crane at the top of the tower with its mass offset away from the building. This is illustrated by the bow in the tower causing a maximum displacement of 0.89in (22.6mm).

The undeformed shape has been drawn in a blue dashed line style and the deformation exaggerated by x20 in order to aid interpretation.

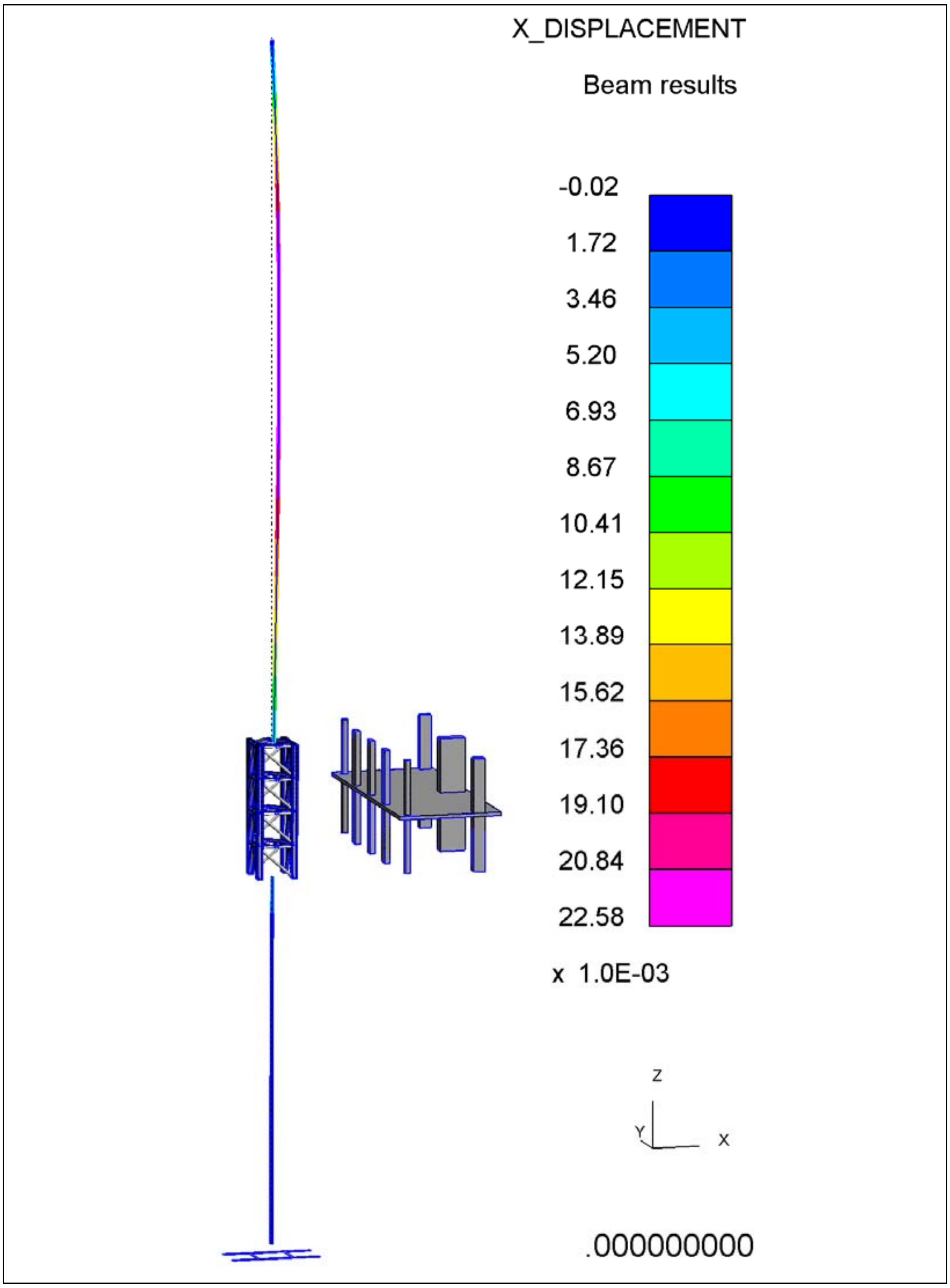


Figure 7.9 - Contours of X Displacement (m) in Tower after Self-Weight (x20 exaggeration). Prepared by Arup.

The following plots, Figures 7.10 to 7.11, illustrate the extreme equilibrium positions of the collar assemblies at the end of the analyses.

The 9th floor collar stops at a maximum vertical displacement of -5.6ft (-1.71m) in run A, and a minimum of -1.8ft (-0.55m) in run D.

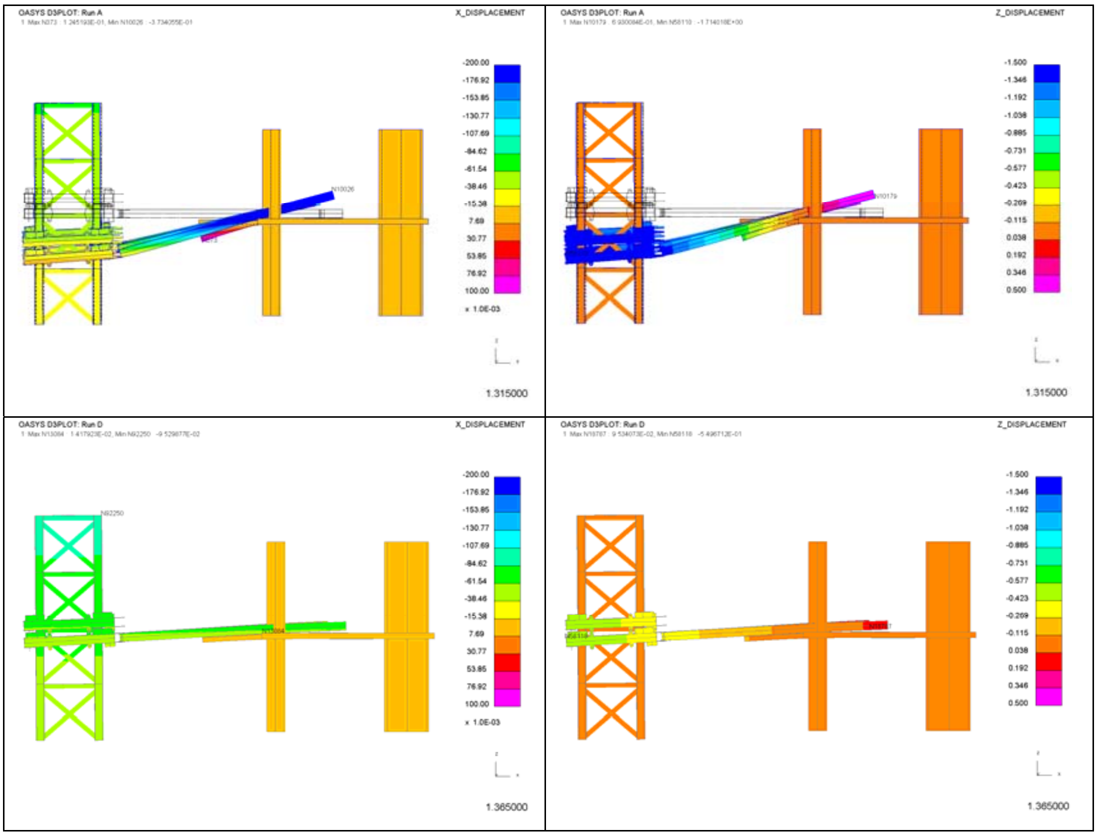


Figure 7.10 - Contours of X & Z Displacement (m) at end of analyses (x1 exaggeration). Prepared by Arup.

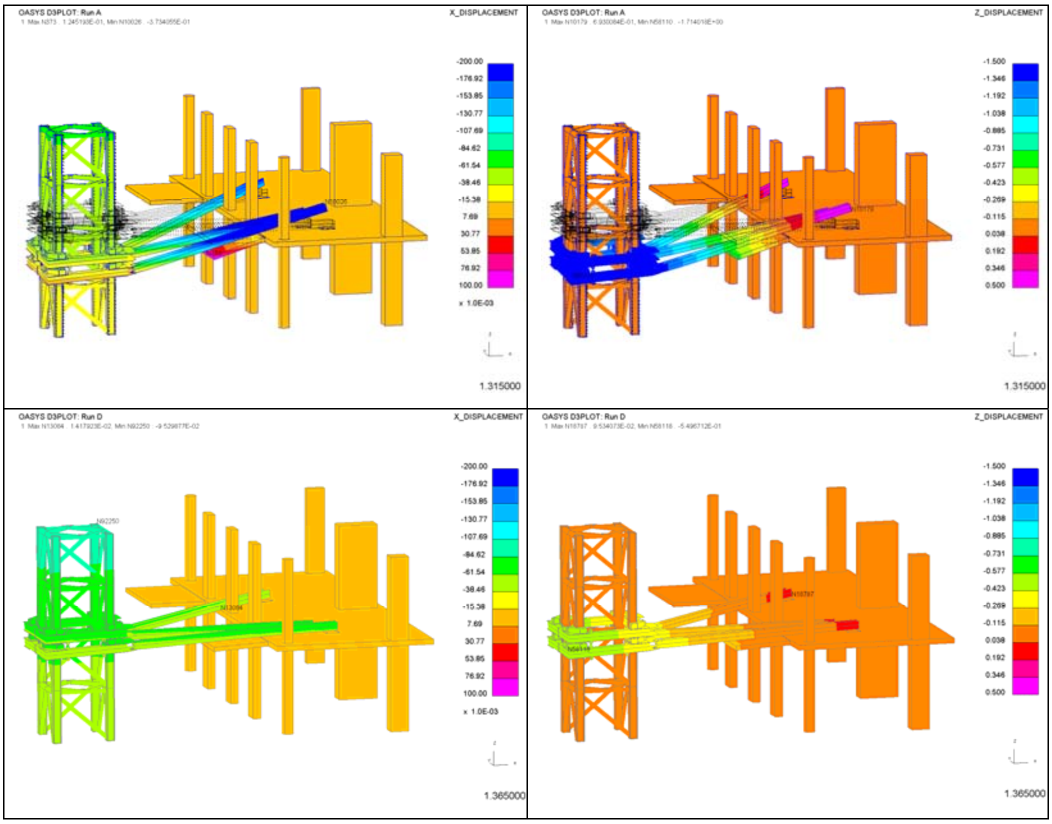


Figure 7.11 - Contours of X & Z Displacement (m) at end of analyses (x1 exaggeration). Prepared by Arup.

The horizontal displacement of the top of the tower is shown in Figure 7.12. For all four runs the crane is still moving at the end of the analysis, this is investigated further in the Global Tower Stability, see Section 5.

Figure 7.12 - Displacement History at Top of Tower (-ve DX away from building). Prepared by Arup.

7.4.3 Energy Histories

The energy histories for the system illustrate the time scale of the impact and the rate at which the energies are transformed from one form to another. The energy history for each analysis for the whole model from the settled self-weight condition onwards are shown in Figure 7.13. In the plots, K.E. represents Kinetic Energy, I.E. represents Internal Energy & T.E. represents Total Energy. It should be noted that the total energy is kinetic plus internal and does not include frictional energy. The kinetic energy for the higher impact speed analyses, i.e. runs A and C, start at 1506kJ (mgh=5.12x9.81x30), but almost halves in the first 0.004s of the impact. The energy is transformed into internal energy of plastic deformation of the two collars and friction energy, mainly at the chock interfaces as can be seen in Figure 7.14.

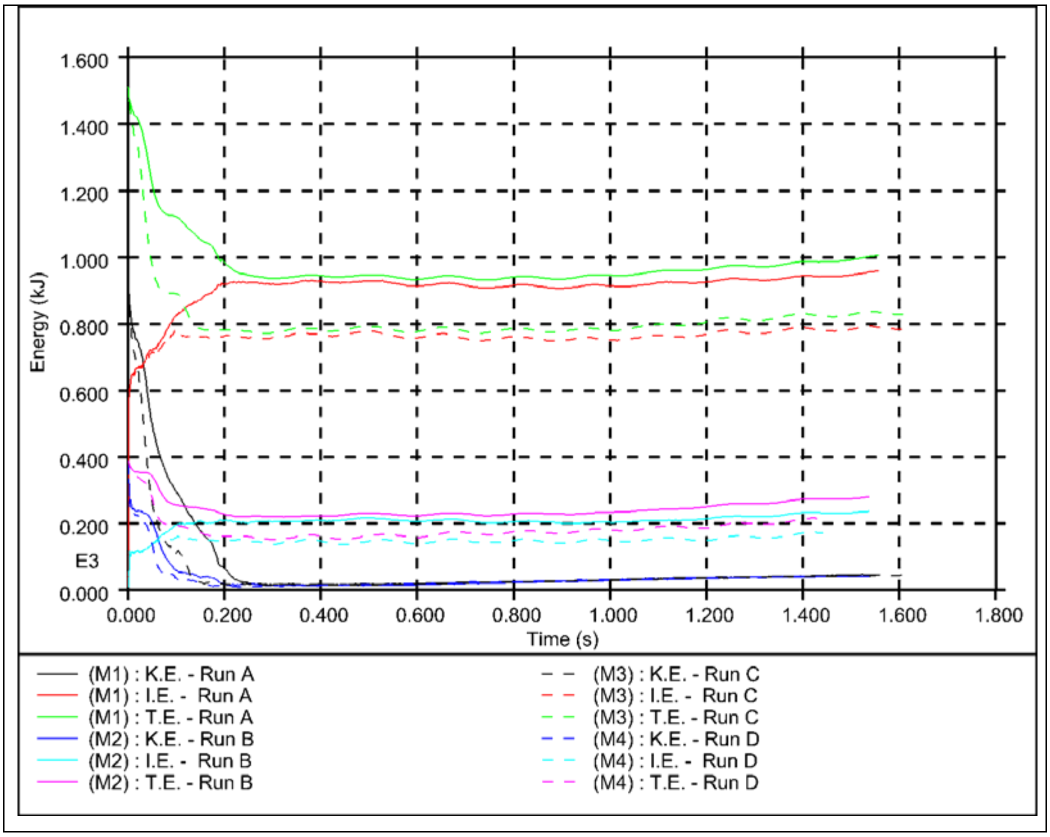


Figure 7.13 - Energy Histories for Whole Model. Prepared by Arup.

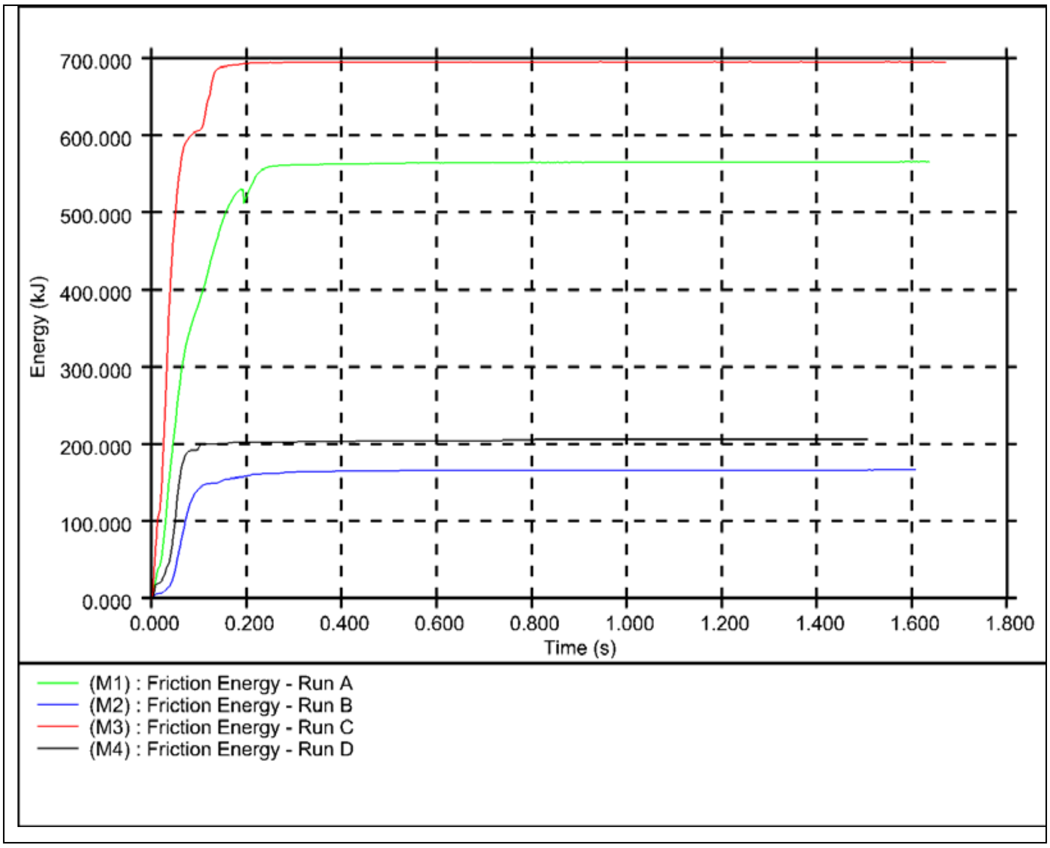


Figure 7.14 - Friction Energy Histories for Chock to Tower Contact. Prepared by Arup.

7.4.4 Contours of von-Mises Stress and Plastic Strain

The von-Mises stress in the tie-beam lugs local to the pinned connection are above the yield of 36.25ksi (250MPa), as can be seen in Figures 7.15 and 7.16.

Plastic strains of 18.0% are predicted around the lug holes for run A, see Figures 7.17 and 7.18. The collar lugs have been drawn translucent so as not to obscure the tie-beam lugs. This region is highly stressed but the modeling issues discussed in Section 7.4.1.5 suggest that the idealization of the pinned connection and welded beam connection are over strong.

All stress and strain plots represent residual values and as such have been extracted from the collars at rest position at the end of the analyses.

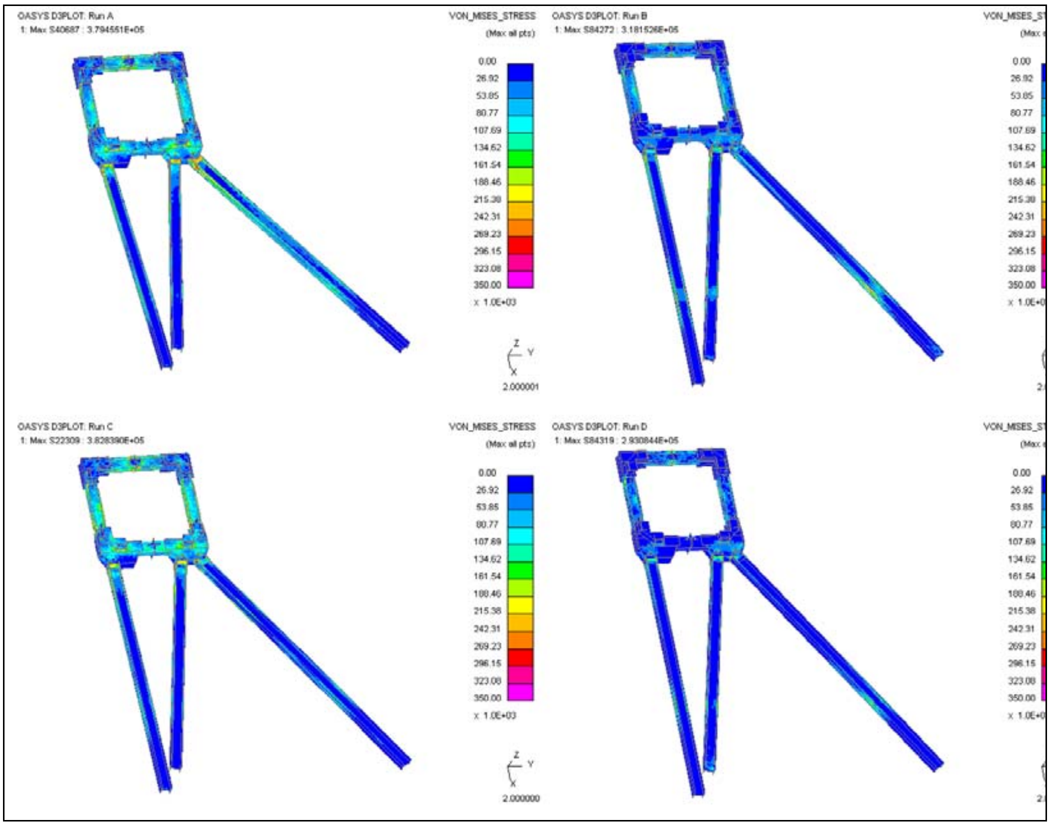


Figure 7.15 - Contours of von-Mises Stress (kPa) in Bottom Collar and Tie-Beams. Prepared by Arup.

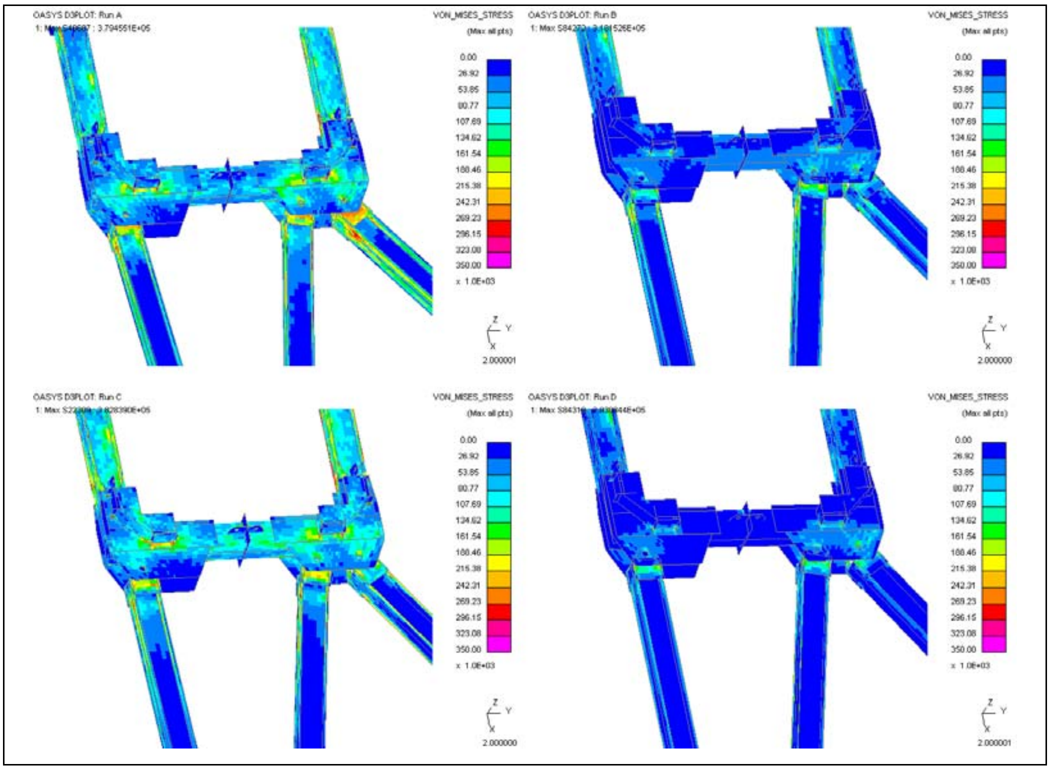


Figure 7.16 - Contours of von-Mises Stress (kPa) in Bottom Collar and Tie-Beams. Prepared by Arup.

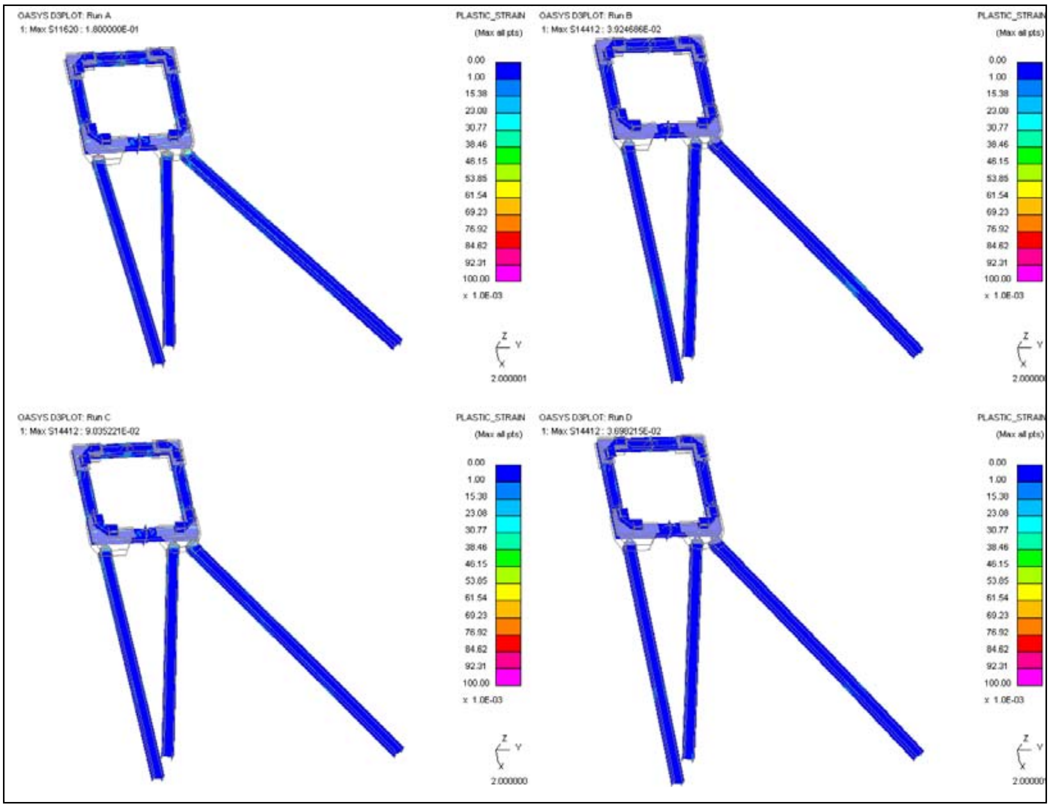


Figure 7.17 - Contours of Plastic Strain in Bottom Collar and Tie-Beam Lugs. Prepared by Arup.

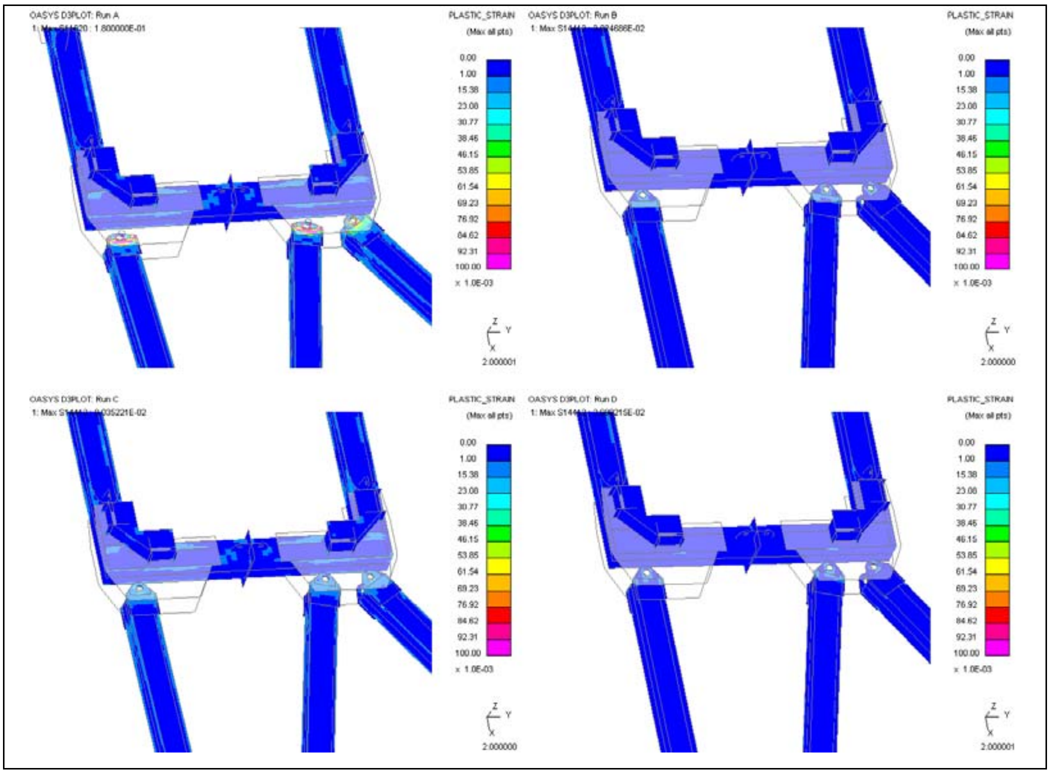


Figure 7.18 - Contours of Plastic Strain in Bottom Collar and Tie-Beam Lugs Close-up. Prepared by Arup.

7.5 Global Tower Stability

A static assessment of the tower crane's global stability, conservatively disregarding the dynamic and large displacement behavior of the tower crane arising from the impact of the falling collar, suggested that the tower would remain standing if only restrained at the 3rd floor, i.e. without restraint at the 9th floor. As this was clearly not the actual observed behavior, it was necessary to investigate the influence of the global dynamics, caused by the collar impact, on the tower stability. For this, the existing model used for the 9th floor collar impact analysis, reported in previous Sections of this report, was enhanced as follows in order to capture the global behavior post collar impact.

The foundation was modified to include contact between the base of the tower and the dunnage steel, see Figure 7.19. Four sets of shells defining the footprint of the tower were constrained to the bottom node of the tower beam elements. Contact was then defined between the 'footprint' shells and the top of the dunnage steel elements. As the actual tower sits in pockets formed by steel sections welded to the dunnage steel a friction coefficient of 1.0 was used which prevented horizontal movement. Vertical lift-off was permitted. The dunnage steel was also modified from being rigid to being a linear elastic material having the stiffness and density properties of steel.

A representative friction coefficient of 0.35 was used to represent the interface between the dunnage steel and plywood.

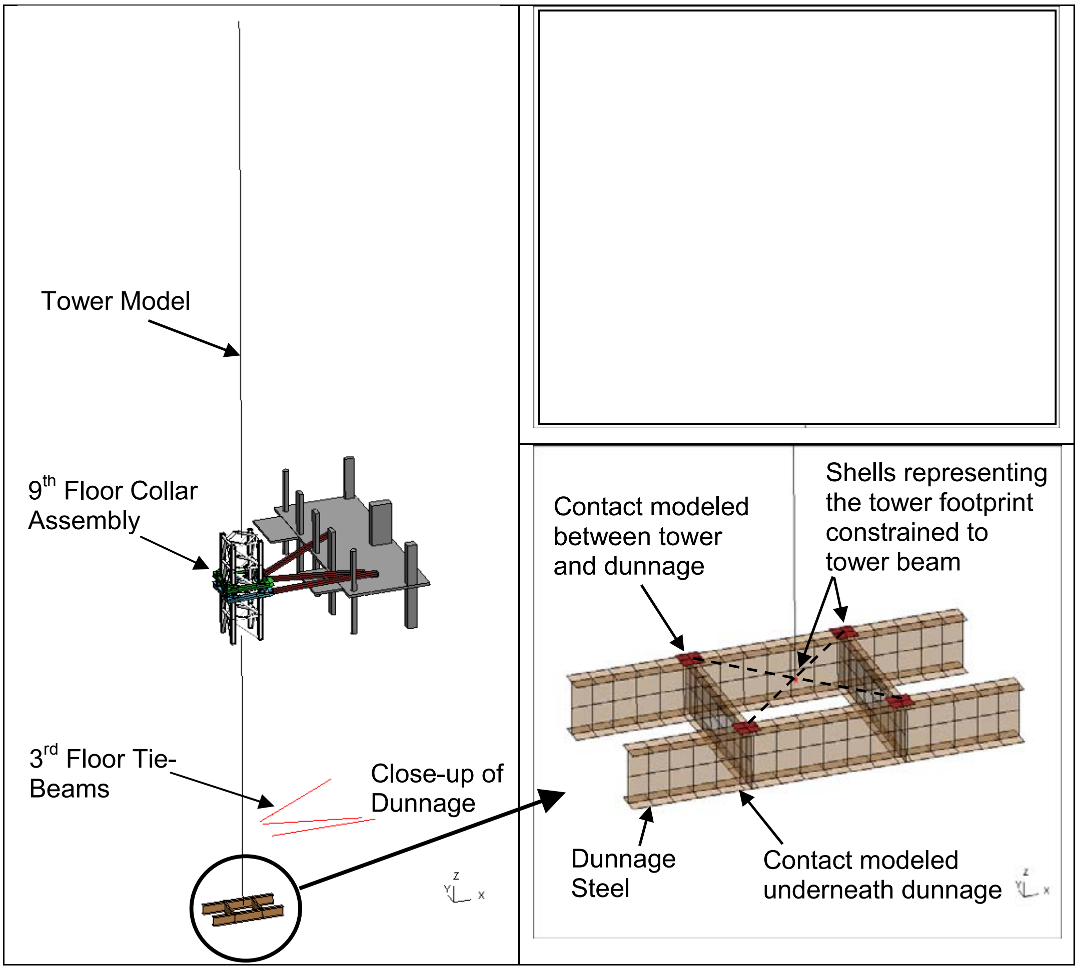


Figure 7.19 - Model Enhancements for the Global Stability Analyses. Prepared by Arup.

After the 9th floor connection is compromised, the stability of the tower is dependent on whether or not the dunnage steel displaces. Specifically, the only available mechanism to prevent the upper portion of the crane from moving away from the building as the tower rotates about the 3rd floor collar is that of friction between the dunnage steel and the concrete base below.

Analysis demonstrated that the horizontal force below the dunnage was great enough to overcome the frictional resistance due to the self-weight of the tower, hence the tower was predicted to fall, as can be seen in Figure 7.20. The dunnage movement can be seen in Figure 7.21. The tower tilts on top of the dunnage as there is no connection limiting this lift off.

Above the dunnage level, the steel brackets that form the ‘pockets’ that the tower base stands in were not explicitly modeled. Instead, a friction coefficient of 1.0 was used so that no slippage would take place. This does not represent the actual failure capacity of the pockets in terms of the resistance to horizontal load. Hence, the assumption is that the resistance of the ‘steel pockets’ to horizontal load is above the magnitude of the applied horizontal load.

The actual observation deformation of one of these pockets can be seen in Photograph 7.1. This indicates that failure of the pockets did not arise due to horizontal movement of the tower legs; rather, it appears that the front steel plates forming the pockets may have squashed as the tower legs lifted and moved. This is consistent with the model's assumption that the pockets had adequate lateral capacity.

Photograph 7.1 - Dunnage foundation. The inward deformation visible in the closeup of the deformed pocket indicates that the steel plate at the end of the pocket was squashed due to a downward force from the tower legs. Had failure occurred due to a horizontal sliding load the plate would have been deformed in the opposing direction. Photo by Arup.

Our findings are based on a coefficient of friction between the bottom of the dunnage steel and the plywood of 0.35; an intermediate figure in the reported range of 0.20 and 0.60 for this condition. With a higher coefficient of friction such as 0.60, the horizontal friction, coupled with the horizontal restraint at Level 3 could be sufficient to restrain the tower. However, the model is under-represents the full effect on the tower of the impact on the Level 3 collar in that it does not capture the fact that the Level 3 collar lowered substantially. The model tends to overestimate the tower restraint. This reduces the leverage between the collar and the base friction and increases the friction demand substantially. Our approximate calculations demonstrate that even with a friction of 0.60, sliding would occur as the collar lowered. This is consistent with the observed behavior of the dunnage foundation (see Photograph 1.5).

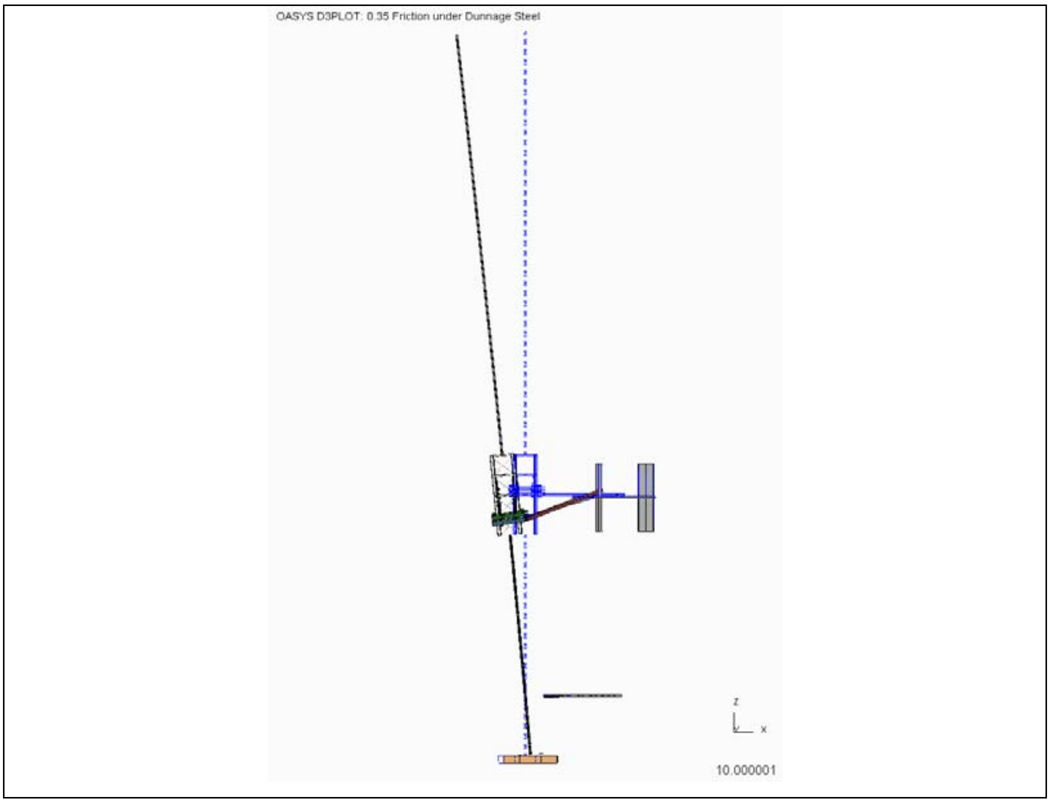


Figure 7.20 - Deformed shape plots. Prepared by Arup.

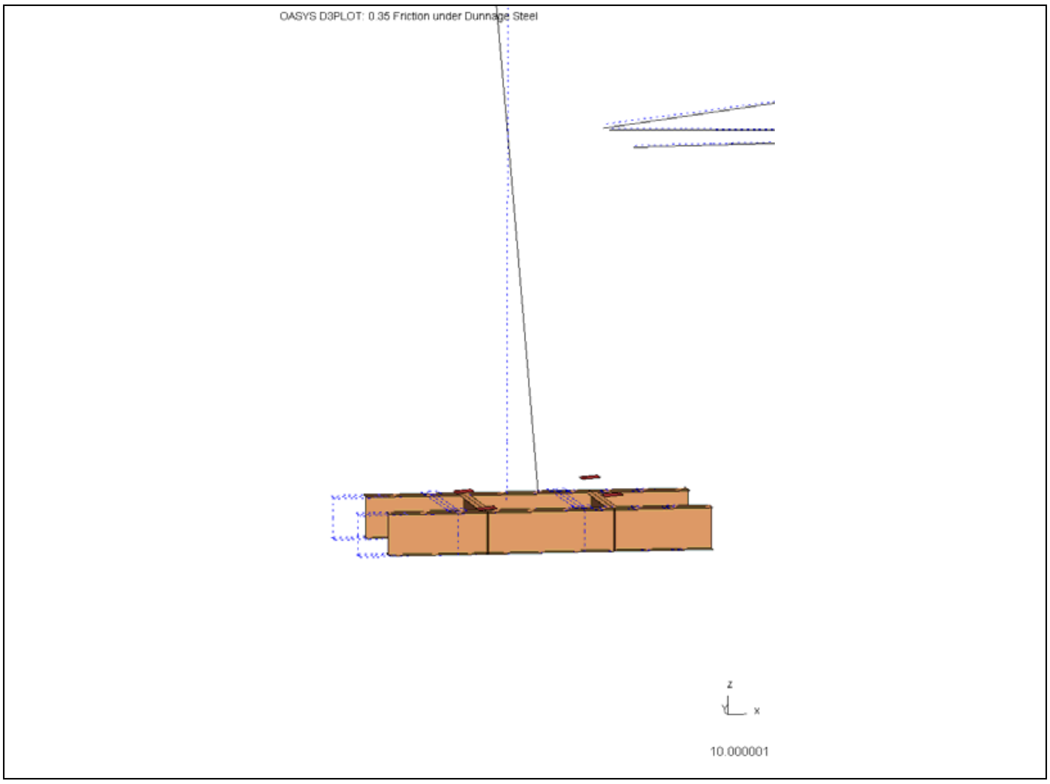


Figure 7.21 - Dunnage Steel close-up at the end the analyses. Prepared by Arup.

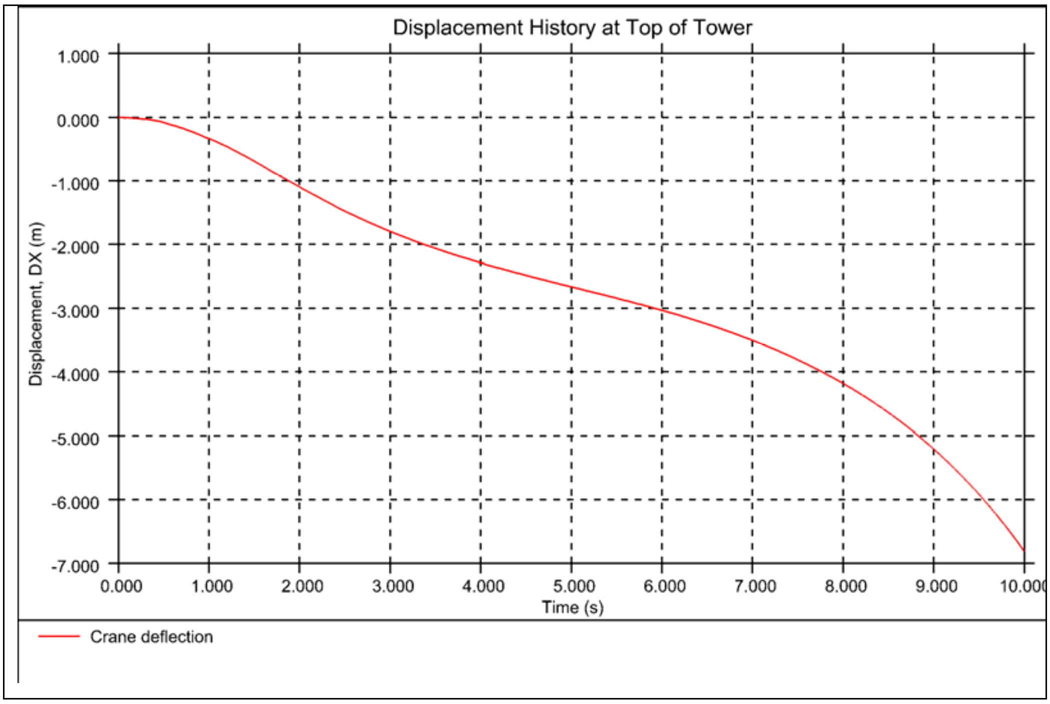


Figure 7.22 - Displacement History at Top of Tower (-ve DX away from building). Prepared by Arup.

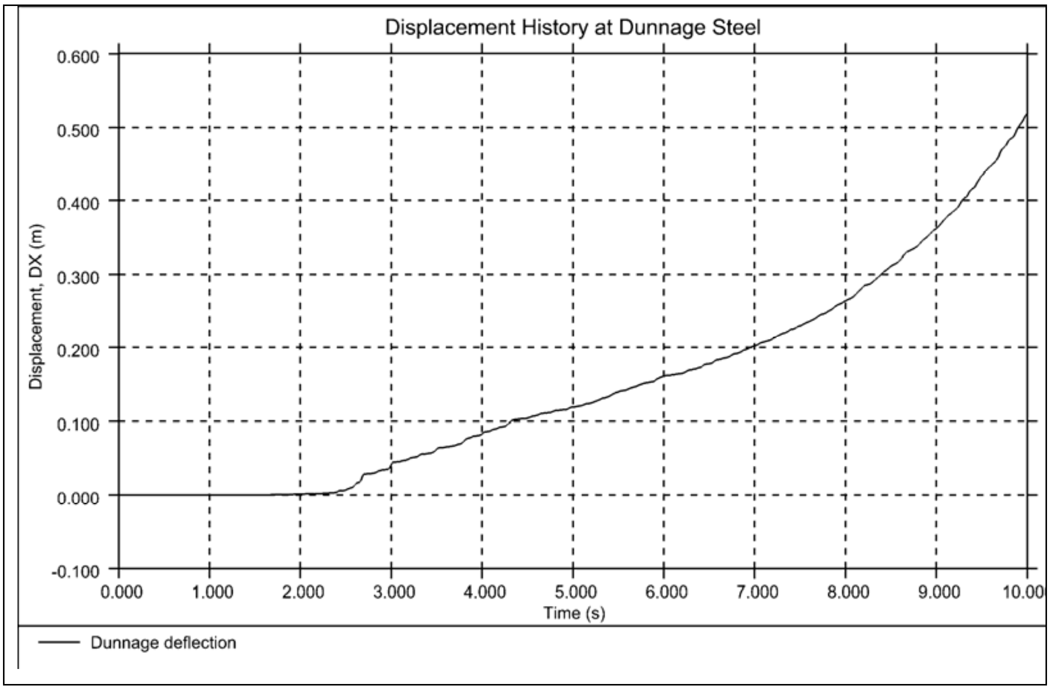


Figure 7.23 - Displacement History of Dunnage Steel (+ve DX towards building). Prepared by Arup.

7.6 Summary of Results

Several non-linear dynamic finite element analyses have been completed to simulate the behavior of the 9th floor collar assembly when it was impacted by the 18th floor collar. Although the model demonstrates that the impact causes an almost immediate failure in the base plate welds and the tie-beams detach from the 9th floor slab, the collars slow down until vertical equilibrium is reached.

The global tower stability analysis used an enhanced model of the 9th floor collar impact analysis to capture the global behavior post collar impact. The foundation was modified to include contact between the base of the tower and the dunnage steel.

After the 9th floor connection is compromised, the stability of the tower is found to be dependent on whether or not the dunnage steel displaces. Analysis demonstrated that the horizontal force below the dunnage was great enough to overcome the frictional resistance due to the self-weight of the tower, hence the tower was predicted to fall. This is consistent with the observed behavior of the dunnage foundation.

7.7 References

[1] Tower crane engineer's submission (by Stroh Engineering) for Certificate of On-Site Inspection approved by NYCDOB on January 17, 2008.

[2] Prototype application for Favello Favco Model M440-D tower crane approved by NYCDOB on March 10, 2000.

[3] Favelle Favco tower cranes operating, maintenance and parts manual submitted as part of prototype application for Favelle Favco M440-D tower crane approved by NYCDOB on March 10, 2000.

[4] Favelle Favco crane manufacturer's drawings from prototype application for Favelle Favco M440-D tower crane approved by NYCDOB on March 10, 2000.

[5] Design Guide for Steel at Elevated Temperatures and High Strain Rates, Fire and Blast Information Group, SCI.

[6] Bautabellen für Ingenieure, Klaus-Jürgen Schneider, 13th Edition 1998.

[7] Construction Products Brochure, Bridon.

[8] Detail Specification, Wire Rope, Flexible, for Aircraft Control, MIL-DTL-83420M, 1 April 2005

[9] American Institute of Steel Construction LRFD Manual, 13th Edition

[10] Mechanical Engineers Handbook, 6th Edition 1958. McGraw Hill

[11] Wire Rope User's Manual, Fourth Edition, 2005, Wire Rope Technical Board.

8 Polyester Slings

8.1 Introduction

As part of the collapse investigation, material analysis of the slings, dynamic structural analysis, review of OSHA test results and review of pertinent codes and standards related to the use of the polyester slings have been conducted. With the exception of the codes and standards review presented elsewhere, these activities are discussed in this chapter.

Polyester web slings were used both for lifting the collars into place as well as for securing them temporarily in a vertical position while the two collar halves were bolted together and positioned to their final vertical position using the chain falls. The slings used for securing the collars in a vertical position on the tower were manufactured from two-ply, two-inch wide polyester webbing. Three of the slings used on the tower were manufactured by Liftall. The fourth sling was manufactured by Liftex but supplied by Metro Wire Rope. As part of the manufacture of the slings from the polyester webbing, a load bearing splice is formed at the base of the eye, at which location there may be a 3-ply construction in an otherwise 2-ply sling.

As discussed in Chapter 7, Dynamic Analysis for the Collar Integrity and Tower Stability, there was a failure to follow manufacturer's explicit instructions with regard to the selection of lifting and chain block points. As a result, the collar was suspended from the tower by choking the slings around the legs of the crane mast and, additionally, the requisite number of support points was not used. Photograph 8.1 below, taken one hour before the tower crane collapse, shows the polyester slings choke hitched around the tower legs as well as improper selection of the lifting points. The bearing of the sling within one of the "V" grooves is also evident.

Photograph 8.1 - View of tower crane with collar half being lifted into place via polyester web slings attached to improper lifting lugs resulting in a non-horizontal position of the collar half during lift (right of photo) and chain falls incorporating polyester web slings choke hitched around the tower legs. (Source: photograph by Gary Halby).

As noted earlier in Chapter 3, Section 3.1.5, the collar was rigged improperly in that the slings used to suspend the collar were choked around the vertical legs of the crane mast without padding. This resulted in exposure to unprotected/unpadded edges under load. As the polyester web sling was choke hitched around the tower crane leg in the V-groove formed by the junction of the tower diagonal and leg, the sling, under load, would have necessarily been constricted, bunched and pinched.

Observations of the sling parts retained by OSHA revealed that the polyester web slings were exposed to frictional contact at their bearing points arising from the unprotected/unpadded edges of the tower legs around which they were choke hitched. Specifically, the melt of the polyester web slings fibers at the failure lines is strongly indicative of an abrasion melting of the fibres. This is corroborated by the test results carried out by ATLSS under OSHA's directions.

There is no indication that the loads being applied to the polyester web or wire rope slings were known with any degree of certainty. As indicated below, the high variability in stretch would render the ability to accurately assess the tension under load without a load indicating device impossible.

Also as discussed in Chapter 3, Section 3.1.5, there is evidence that at least one sling, the "Metro" sling, was previously exposed to long periods of ultraviolet radiation sufficient to bleach both the outer fibers of the sling and the manufacturer's label. This sling also showed evidence of prior usage, including damage to the sling. This observation was corroborated by OSHA.

8.2 OSHA Tests

8.2.1 Introduction

As part of OSHA's investigation of the tower crane collapse, as stated in their report,

> "OSHA contracted with Advanced Technology for Large Structural Systems Research Center (ATLSS) of Lehigh University, Bethlehem, PA to determine the load carrying capacities of the slings under conditions replicating the actual manner they were used. The tests were performed at Fritz Engineering Laboratory (Fritz) of Lehigh University, PA. OSHA rented a section of the crane mast similar to the one used at the site and transported to the Fritz. 12 slings (9 manufactured by LiftAll, and 3 manufactured by Metro) were tested choked around the column flange and trapped in the V shape, as discussed above…"

Arup assisted with the tests with regard to general review of proposed geometries and potential issues regarding test protocols.

As part of the data collection, load-displacement curves were generated for each of the tests. These were provided to Arup for the sling analysis and are presented in Figures 8.4 through 8.8 below; however, a detailed test report from ATLSS has not been received for review.

A description of the test, as contained in the OSHA report, follows.

> "OSHA rented a section of the crane mast similar to the one used at the site and transported to the Fritz. 12 slings (9 manufactured by LiftAll, and 3 manufactured by Metro) were tested choked around the column flange and trapped in the V shape, as discussed above…
>
> The entire crane mast section was supported on a steel dunnage on the Fritz's concrete floor. Slings were tested one at a time. A tension load was created on the slings. The tensile test fixture included a cylindrical base, a pull bar and a forcing member. The pull bar included a limiting member, a specimen-fixing member and a shaft member. Air was used in the cylinder to produce the tensile force.
>
> The sling was choked around the wide flange of the crane mast leg passing through a V- shaped notch between the angle of vertical knee bracing and the tip of the flange of the mast leg. The sling at the other end was connected to the hook. A chain was passing through the eye bar of the hook and was connected to the cylinder shaft. Air pressure was introduced in the cylinder to induce tension on the slings and the magnitude of tension was increased in segments until the sling failed. The elongation of the slings under load was also measured until the failure of the sling. The graph was drawn for the elongation of the sling against the induced tension load.
>
> The sling was choked clockwise as well as counter-clockwise around the column during the testing. Similar testing was carried out by chocking the slings at other end of the column where round diagonal bracing was located. It was discovered that the sling was invariably trapped in the V-shaped notch created by the leg flange and steel angle brace regardless of whether there was a diagonal pipe brace or not. However, testing was also conducted by forcing the sling to touch the round pipe brace."

Testing was carried out such that displacements were held constant, rather than the applied load. This is reflected in the load charts where sudden drops in the applied load are recorded under constant displacements.

Variations in the test procedures included: (1) positioning of the sling splices such that either the 2-ply or 3-ply portion would be within the “V” shaped groove, (2) use of protective padding against sharp edges in some tests (for comparative purposes with the unpadded results), (3) use of Liftex or Liftall slings, and, (4) “quick” vs. “slow” loading.

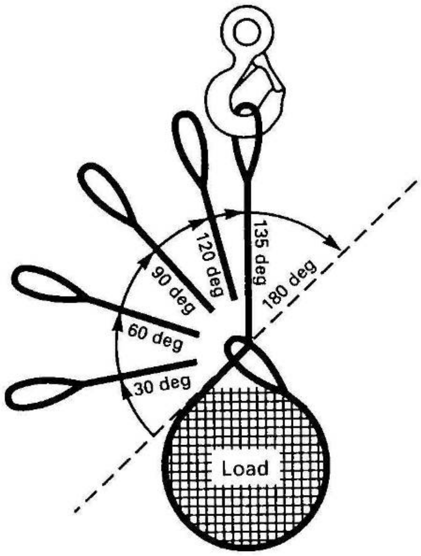
8.2.2 OSHA Test Results

All slings failed significantly below their anticipated actual capacity. Per the WSTDA and ASME standards cited in Section 3, the rated capacity of a polyester web sling, regardless of other considerations including angle of choke and angle of loading, should incorporate a factor of safety of 5. When in a choke hitch there is an additional minimum 20% reduction in rated capacity. Therefore, as the Metro/Liftex slings tested had rated capacities of 5,100 lb and the Liftall slings had rated capacities of 5,000 lb in a choker configuration, actual load capacities should have been approximately 25,500 and 25,000 lb, respectively, in the choker configuration.

Two angles are considered in applying additional reduction factors to the sling per the ASME and WSTDA standards discussed in Chapter 3. The first of these is the angle of choke, which is the angle of the pull to the sling in the plane of the choke hitch. This is shown in Figure 8.1 below, taken from ASME B30.9-2006.

ASME B30.9

Fig. 19 Angle of Choke



Angle of Choke, deg	Rated Capacity, % [Note (1)]
Over 120	100
90–120	87
60–89	74
30–59	62
0–29	49

NOTE:
(1) Percent of sling rated capacity in a choker hitch.

Figure 8.1 - Definition of Angle of Choke, taken from ASME B30.9-2006 Slings

The second angle is that formed perpendicular to the plane of the choke angle, shown in Figure 8.2 below, also taken from ASME B30.9-2006 Slings. Note that the angle of choke is a separate consideration from the angle of loading. If present, the choke hitch in Figure 8.2 would most likely present at the hook but could equally be present at other attachment points.

ASME B30.9-2006

Fig. 18 Angle of Loading

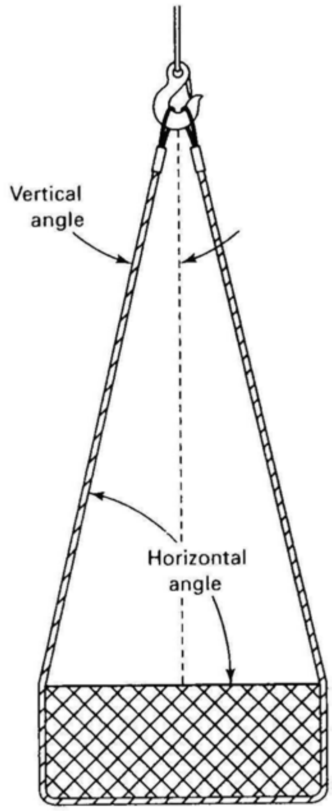


Figure 8.2 - Definition of Angle of Loading, taken from ASME 30.9-2006 Slings

These rated loads are predicated upon a straight pull; i.e., the angle of the sling to the applied load, and maximum choke angle. As indicated, reduction factors are to be applied where the angle of loading deviates from a straight line. The OSHA test geometry was based upon best information regarding the relative heights and positions of the collar and tower crane. The resulting angle of loading was found to be approximately 90-63 = 27 degrees, see Photograph 8.2. At an angle of 27 degrees, the load would be increased load in the sling by a factor of approximately 2.2; i.e., reduction factor of approximately 0.45. See Figure 8.3 below, showing the geometry of the sling, tower and choke hitch. This would effectively reduce the anticipated applied breaking load to approximately 11,500 and 11,250 respectively. This assumes that the sling is not yet bearing on the tower bracing following slippage.

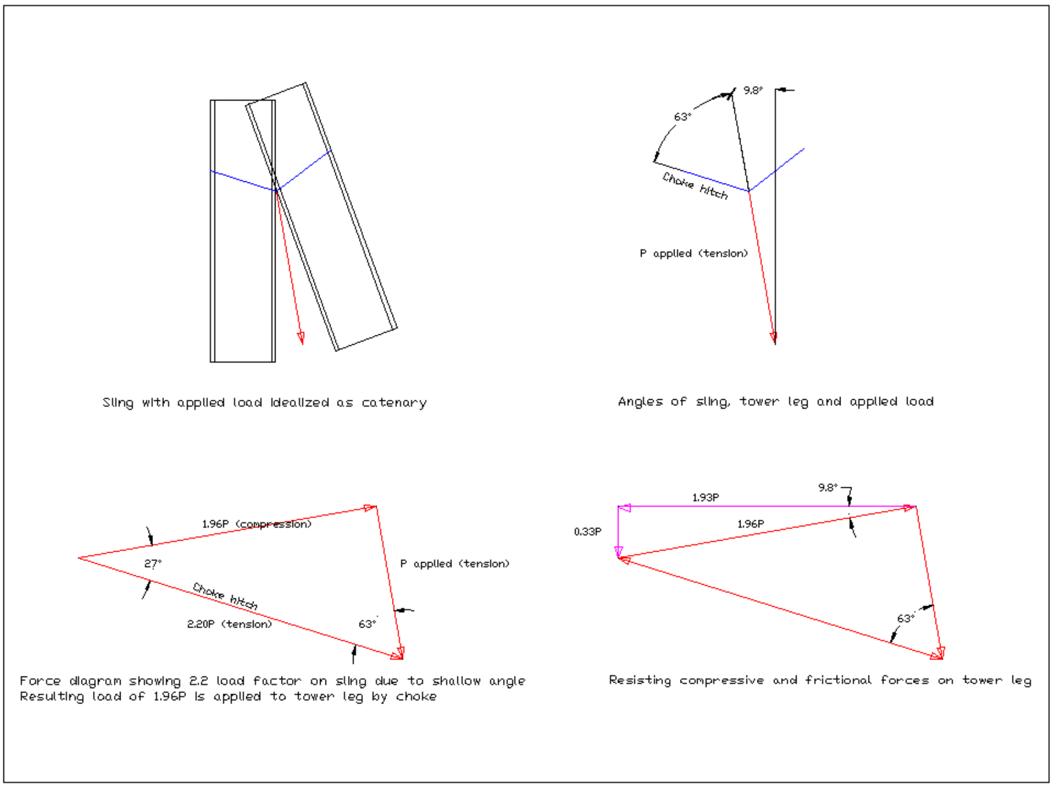
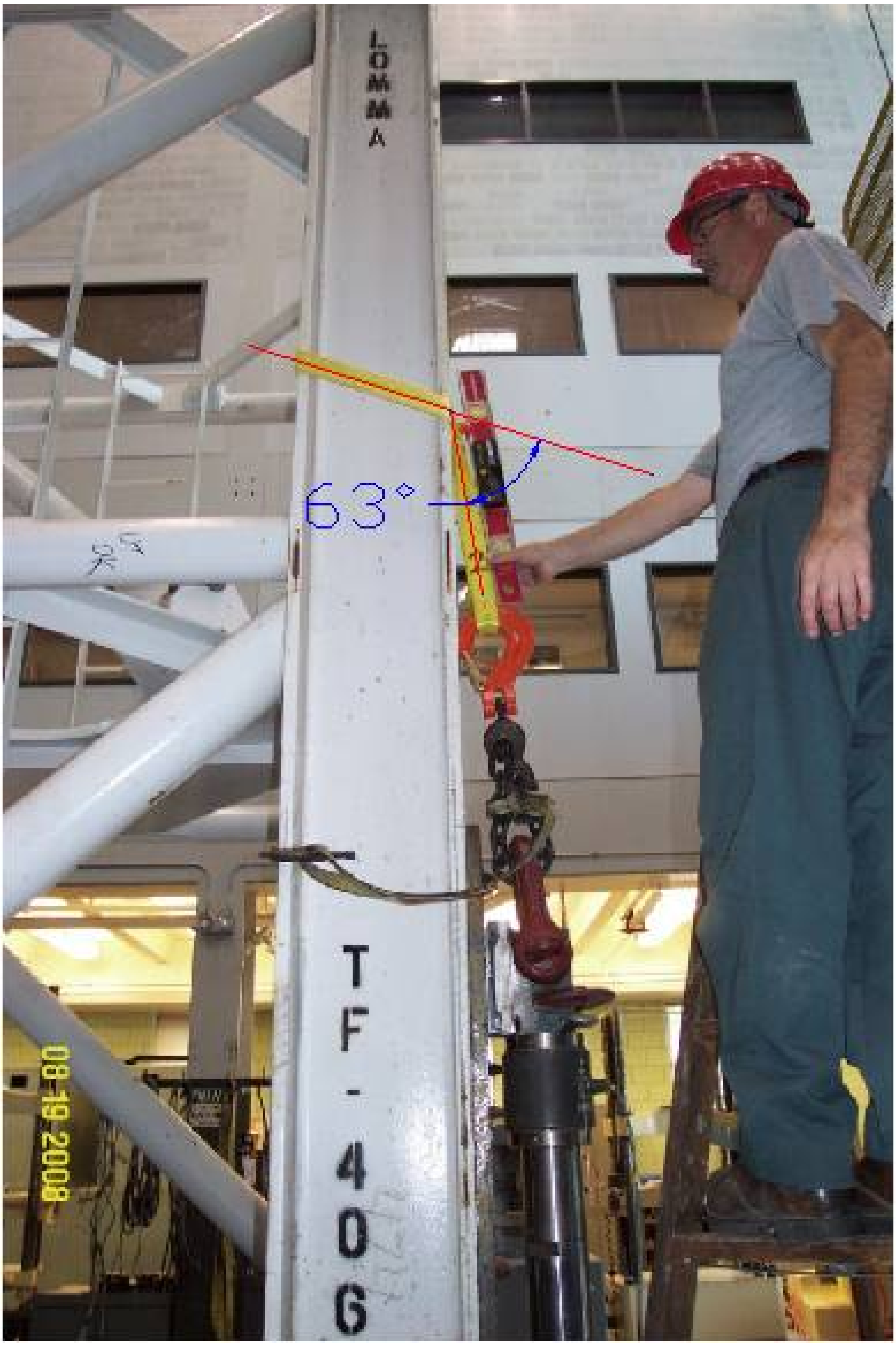


Figure 8.3 - Effective of geometry on sling load prior to slippage. Prepared by Arup.



Photograph 8.2 - OSHA Sling Test - angle of pull. Photograph provided by OSHA.

Note that this reduction factor is linked to the sine of the angle and reflects an increase in the load of the sling due to the angle of the applied load, rather than a reduction in the capacity of the sling. As already indicated, any effect arising from the use of a choke hitch is additional. When the sling is first applied to the tower leg there will be a clearance between the sling and the “V” of the bracing and movement of the sling would be prevented by friction or other resistance. This frictional force is illustrated in the foregoing figure. Hence this increase in load would be realized. Once this resistance is overcome, the sling would displace and jam into the “V” groove. The tension in the polyester sling outside of the tower leg would then be supported vertically by the bracing members and horizontally by the wrapped sling and no reduction in capacity based upon sling angle would be applicable.

This is supported by the test results. Where the slings were neither first pushed down into the “V” groove nor packed, tests #2 - #8 there is a notable drop in support load at relatively low levels. The drop is lower for slings #3 and #8 where only 2 plies, as compared to 3 plies, were located within the “V” groove. Additionally, it is reported in the OSHA report that

> “In all tests, as the slings were being subjected to increasing loads, the slings would abruptly slip and adjust through the choke. These adjusting movements began at about 2500# and continued at various load levels up to breaking strength. Audible tearing of the web filaments began at approximately 5500# and continued until final breaking strength.”

Although there is still reserve capacity in the slings, the presence of “audible tearing”, which is taken to mean failure of groups of individual polyester fibers resulting in audible noise, represents the onset of failure of the slings. It is unknown if the tearing would have continued and resulted in an eventual failure of the polyester slings under a constant load but at much lower levels than the recorded maximum load.

Based upon the photographs of the test setup reviewed, it appears that the angle of choke (see Chapter 3, Section 3.1.5) was within the acceptable limits of the WSTDA and ASME standards without the need for a reduction factor beyond the standard 0.8 factor applicable to a choker hitch.

Test results are presented in Figures 8.4 through 8.8. A summary of the tests and maximum loads achieved in each test is given in Figure 8.9. As can be seen, the maximum load achieved by each sling was significantly below anticipated breaking strengths, although all tests failed above the rated capacity in a choker configuration.

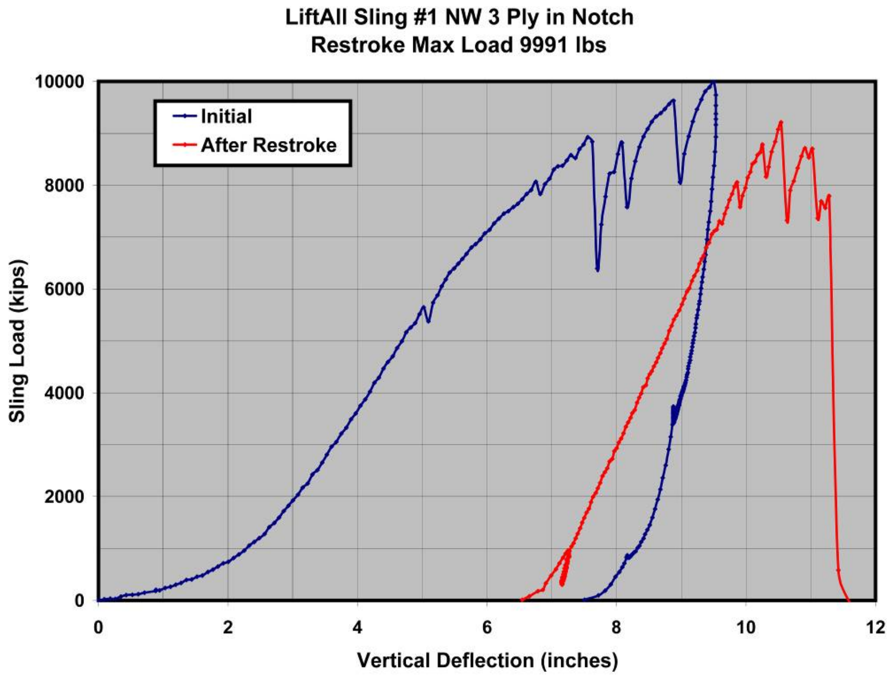


Figure 8.4 - Liftall Sling #1 - OSHA Sling Test Results. Graph provided by OSHA. Graph provided by OSHA.

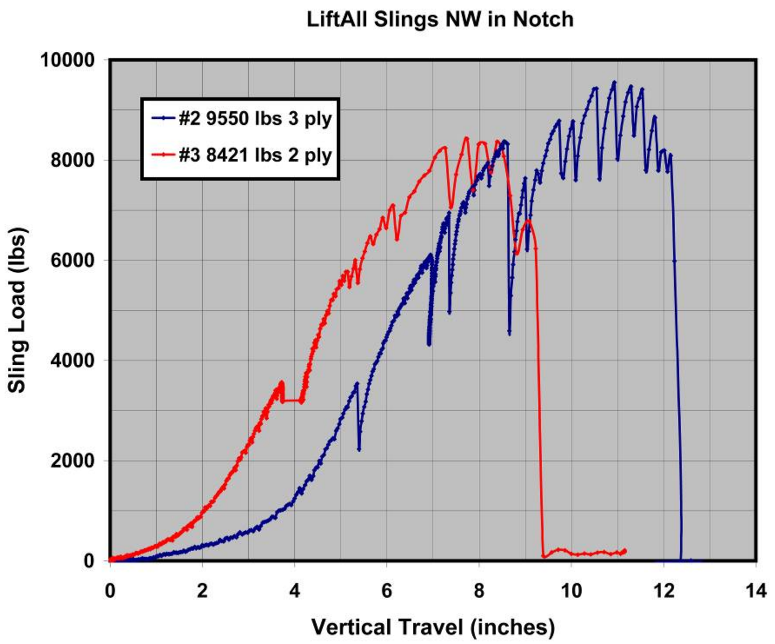


Figure 8.5 - Liftall Sling #2 and 3 - OSHA Sling Test Results. . Graph provided by OSHA.

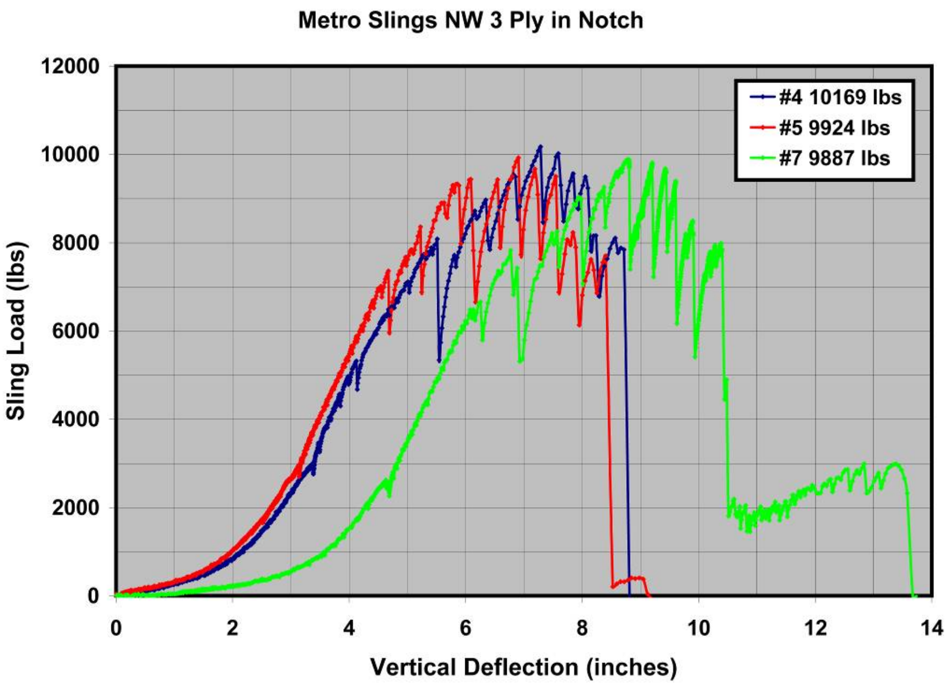


Figure 8.6 - Liftall Sling #4, 5 and 7 - OSHA Sling Test Results. . Graph provided by OSHA.

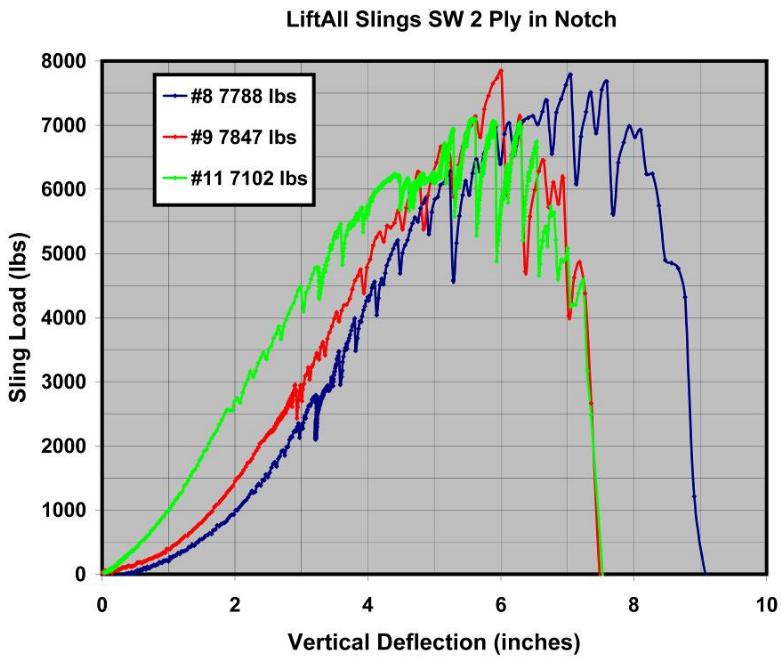


Figure 8.7 - Liftall Sling #8, 9 and 11 - OSHA Sling Test Results. . Graph provided by OSHA.

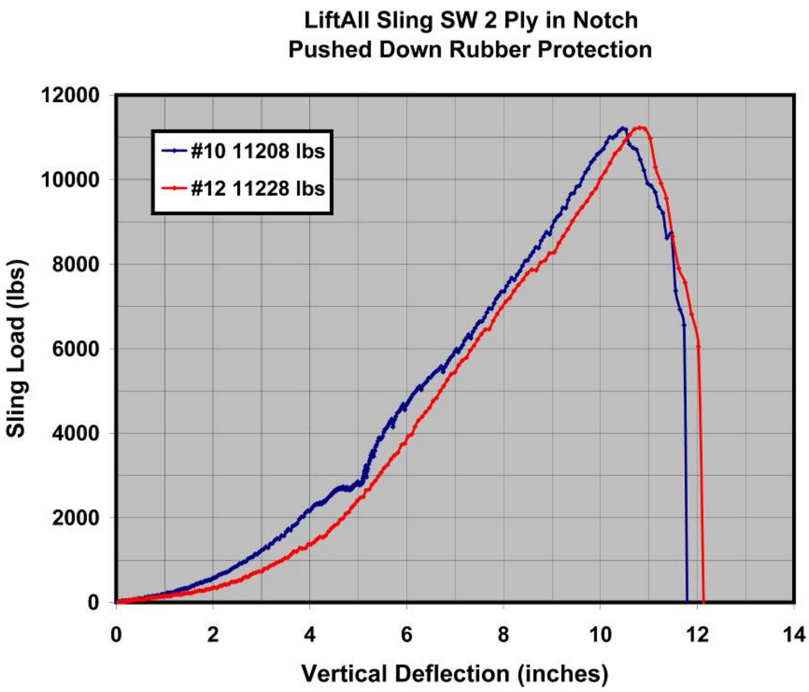


Figure 8.8 - Liftall Sling #10 and 12 - OSHA Sling Test Results. . Graph provided by OSHA.

OSHA 51st Street Crane Collapse Sling Evaluation Summary

Sling	Designation/Test Decription	Angle re: Horizontal (degrees)	Angle re: Vertical (degrees)	Cosine of Angle	Correction Factor	Measured Max Load (lbs)	Actual Sling Max Load (lbs)
1	LiftAll NW, Restroke	N/A	N/A	N/A	N/A	9991	N/A
2	LiftAll NW, 3 ply in notch	84.3	5.7	0.9951	1.0049	9503	9550
3	LiftAll NW, 2 ply in notch	82.5	7.5	0.9914	1.0087	8348	8421
4	Metro NW 3 ply in notch	81.6	8.4	0.9893	1.0108	10061	10169
5	Metro NW, 3 ply in notch	82.4	7.6	0.9912	1.0089	9837	9924
6*	LiftAll NW, 2 ply in notch, fast	N/A	N/A	N/A	1.0087	8300	8372
7	Metro NW, 3 ply in notch, sustained	82.6	7.4	0.9919	1.0082	9807	9887
8	LiftAll SW, 2 ply in notch	84.4	5.6	0.9952	1.0048	7751	7788
9	LiftAll SW, 2 ply in notch, pushed down	84.9	5.1	0.9960	1.0040	7816	7847
10	LiftAll SW, 2 ply in notch, pushed down, rubber	82.1	7.9	0.9905	1.0096	11101	11208
11	LiftAll SW, 2 ply in notch, pushed down, sustained	N/A	N/A	N/A	1.0040	7074	7102
12	LiftAll SW, 2 ply in notch, pushed down, rubber	N/A	N/A	N/A	1.0096	11121	11228

6* Data compromised - not retrievable: 8300 lbs max load based on value onserved on DAS screen during test

Figure 8.9 - Summary of OSHA Sling Test Results. . Graph provided by OSHA.

Also observed during the tests was the wide variability of displacements. There does not appear to be any pattern with regard to manufacturer, presence of padding or initial placement. Elongation measurements at 2800 lb. (1/4 of the collar weight) were as follows:

Sling Test No.	Elongation
2	5 ½ in.
3	4 ½ in.
4	3 3/8 in.
5	3 in.
6	Not recorded
7	4 ¾ in.
8	3 3/8 in.
9	3 in.
10	5 ½ in.
11	2 ¼ in.
12	6 in.

While these measurements represent the combined effects of sliding and stretch, due to the evident inability to predict the extent and variability of displacements, the amount of elongation under load of any particular sling would be unable to be determined. The use of polyester slings to suspend a load would therefore result in an inability to determine the proportion of load shared by each sling in a multi-sling system.

Additionally, the failure mode; i.e., angle of cut, melt of fibres, location of failure on the sling and the appearance of the failed surfaces, from the OSHA tests closely matched those of the recovered sling parts from the failed tower crane.

Based upon their analysis, OSHA reached the following conclusions:

"1. The choice of using polyester slings to suspend the collar at four points was questionable as they are subject to large elongations under tensile loads, thus creating a need to constantly monitor and level the collar.

2. The collar was rigged improperly in that the slings used to suspend the collar were choked around the vertical legs of the crane mast and was seated in the V-shaped groove between the angle bracing and the flange of the crane mast leg. This significantly reduced the load carrying capacity of the slings.

3. The slings were not protected against sharp edges for cuts and abrasions.

4. A deteriorated sling, which should have been discarded if proper inspection of the sling was done prior to its use, was used to suspend the collar.

5. The crane raised the collar from the ground hoisting it at locations different from the crane manufacturer's recommendations. This led the employees to suspend the collar from locations above which there were no horizontal members. This resulted in choking the slings around the legs of the crane mast.

6. Each collar half was suspended at two points instead of at four points as recommended by the crane manufacturer."

8.3 Condition Review of Recovered Polyester Web Slings

Arup independently engaged Dr. Tushar K. Ghosh of North Carolina State University, College of Textiles, Raleigh, North Carolina, to evaluate the polyester slings through a strictly visual inspection. This inspection was supplemented with microphotographs, at an approximate magnification of 200x, taken by Arup and supplied to Dr. Ghosh to assist him in his analysis. Dr. Ghosh's report is contained in Appendix K.

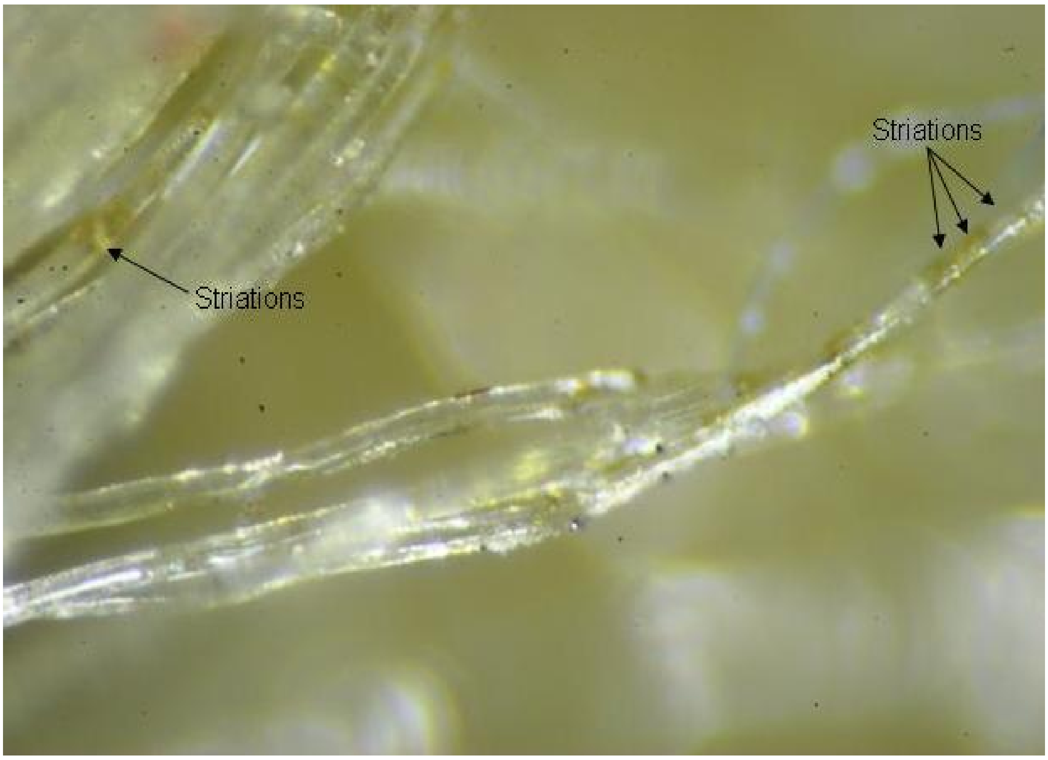
The purpose of Dr. Ghosh's work was to attempt to determine the physical condition of the slings as well as comment on the fracture/failure nature of the slings.

Seven sling parts were recovered from the debris, as observed at OSHA's Manhattan district office. Based upon their respective lengths and physical characteristics, these are considered to be paired as follows, thus creating the original four slings which were used on the collar at floor Level 18.

Part "A" (free end)	Part "B" (chain end)
None recovered	1A
11	2A
12	4A
13	7A

A photo of these pairs is given in Dr. Ghosh's report.

Of particular interest are the observations regarding evidence Item 1A, the Metro (Liftex) sling, understood from OSHA to have been the first sling to fail. Observations are of a slow speed tensile failure. Characteristics of a slow speed tensile failure are presented in the photographs; including both fiber pull-out and thinning of the fibers. See Photograph 8.3 below, showing striation of the fibers, taken from sling 1A. The evidence of abrasion damage in the sling could also be observed in the microphotographs. The fibers in the vicinity of failure show mangled ends and fibrillation. In addition, the striations along the cross direction of the fibers, also visible in the microphotographs could be signs of initiation of cracks during this incident or due to prior use.



Photograph 8.3 - Microphotograph of Metro/Liftex sling failure surface showing striations. Photo by Arup.

As stated in Dr. Ghosh's report

> "Sling Piece 1A: According to the label, the sling piece is made of polyester and manufactured by Metro Wire Rope Corporation. The piece, 57" in length, shows a number of high compressive stress and/or abrasion lines, approximately along the cross direction, see Figure 3a. In some instances the fibers are broken (or cut) along the stress lines. Figure 3b shows a close up of the fractured end of the sling-piece 1A. The failure surface of the sling is along a diagonal line to the long direction of the sling. Visual examination of the fractured end clearly shows two distinct areas. Almost ¾ of the sling failure surface seems to present a high-stress compression and/or abrasion failure from a sharp edge while the rest looks almost like slow-speed tensile failure. The slow-speed tensile failure is characterized by fiber pull-out and thinning of fibers as seen in Figure 3b. The evidence of abrasion damage in sling-piece 1A is clearly evident in a few micrographs presented in Figure 4. The fibers in the vicinity of failure show mangled ends and fibrillation. In addition, the striations along the cross direction of the fibers, visible in Figure 4b and in others could be signs of initiation of cracks during this incident or due to prior use."

Also of note are the observations of Sling Piece 2A which, although it appears to be relatively new, shows at least one line of very high compressive stress, with a fracture surface mostly at a diagonal to the axis of the sling. More than half of the failure surface seems to present a well-defined linear cut due to contact with an unprotected/unpadded edge.

The failure surface of evidence Item 4A shows clear evidence of melting of broken fiber ends resulting from heating of the fibers, either due to high frequency and/or stress abrasion or high speed tensile loading.

The failure surfaces to evidence Items 7A, 11 and 12 are similar to that reported for Item 4A. Item 12 additionally exhibits possible cutting due to contact with an unprotected/unpadded edge and one high stress line along the cross direction. The fracture surface for Item 13 was observed to be very similar to Item 12.

Almost every piece of sling presents signs of moderate to severe abrasion and/or lateral compression damage. There is some evidence of cracking of the fibers along the cross direction, indicating possible prior loading or use. There is considerable evidence of fiber failure at different speeds and abrasive weakening of the slings leading to failure. The rates of strain (or speed), involved in the failure of slings are indicative of “time to failure” and in turn the sequence of failure of the slings.

8.4 Installation of the Polyester Web Slings

It is evident from the A1-1100.123 erection sequence drawing, Figure 1.13 (also see enlarged details in Figures 1.9, 1.10, 3.4 and 3.5) that eight chain blocks were to be used for the installation/erection sequence during the stage of the work in progress at the time the tower crane collapsed. Only four slings were actually used.

As previously noted in Section 1.2, the wrong lugs were used for the crane to lift the collar and for the slings to suspend the collar from the tower. The collar was actually lifted by the crane from lugs intended for the slings. With their point of attachment already in use, the slings were then attached to the lugs intended for lifting by the crane. This was quite significant as the points actually used for the slings aligned with the columns, which prevented a proper connection to the horizontal beams. Figure 8.10 illustrates the correct arrangement, including the use of the intended crane lifting points for the collar and the use of the intended sling attachments. This situation would have resulted in direct attachment of the slings to the horizontal steel as shown. Figure 8.11 shows actual points used to lift the collar and attach the slings.

Due to the use of the incorrect lugs, the positioning of the slings as installed was inconsistent with the manufacturer’s instructions. The slings were choke-hitched around the crane tower columns and drawn into the V-shaped grooves between the diagonal trussed members and the columns. None of the slings were protected against the edges by padding or other means which was required by the sling manufacturers as stated on labels affixed to the slings. See Photograph 8.4 below.

> *“WARNING Failure to comply with warning may result in personal injury or death*
> *Inspect sling before each use*
> *Always protect web slings from being cut or damaged by edges*
> *Do not exceed rated capacities*
> *Do not expose to temp above 200 deg F”*

It is noted that wood packers are also included in the tower crane manufacturer’s erection procedures. The bunching and lack of padding are both in violation of industry standards.

Photograph 8.4 - Warning labels on slings recovered from the collapsed tower crane clearly require inspection before use and use of protection at all edges. Photo by Arup.

Photograph 8.5 was taken about an hour before the accident and shows one half of the collar being lifted from the wrong points and the slings supporting the other half being choked around the columns without any protection. The collar half on the upper right is seen being lifted by the wrong lugs while the slings supporting the other half can be seen in the lower left of the photograph.

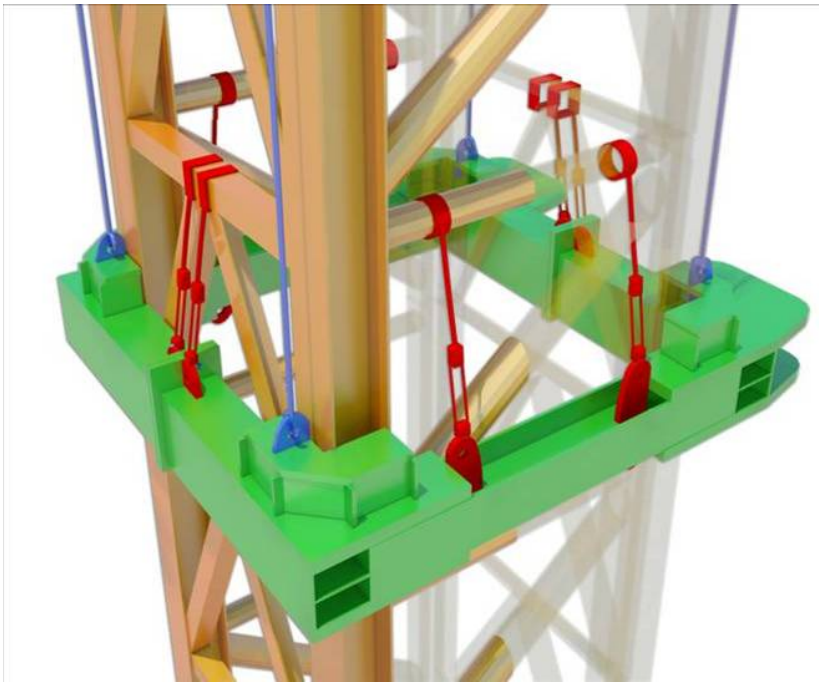


Figure 8.10 - Elevation of the Collar Tie showing the specified/ideal crane lifting points (in blue) and the proposed sling attachment points (marked in red). Prepared by Arup.

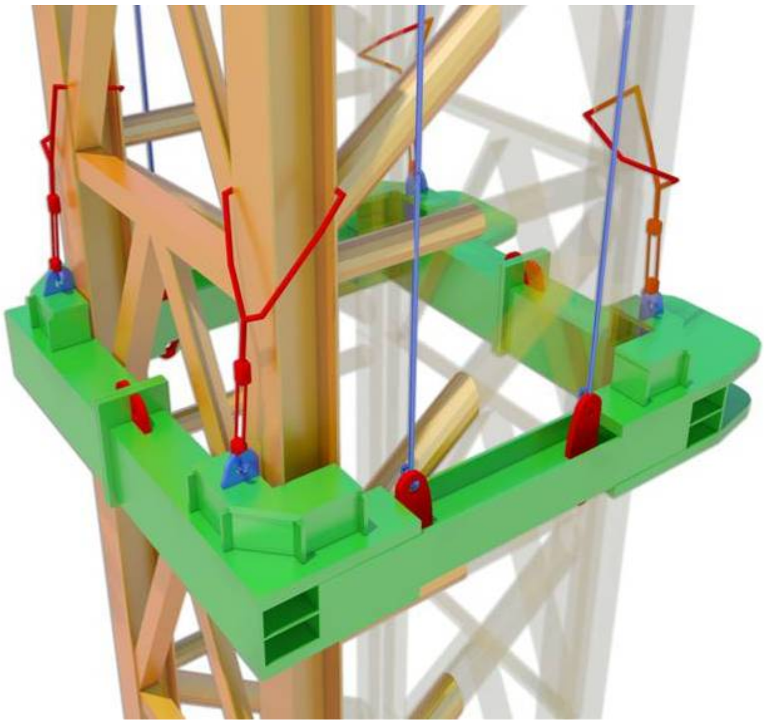
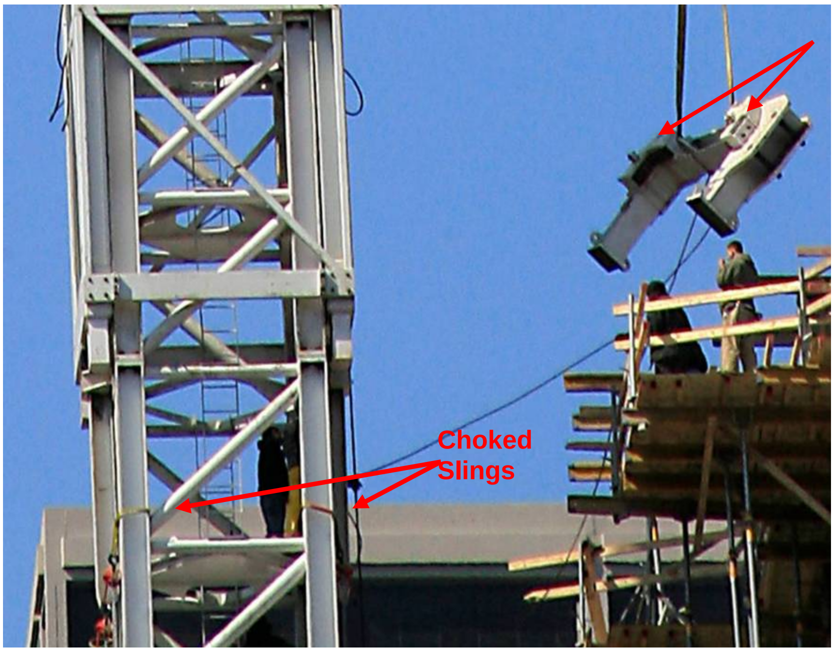


Figure 8.11 - Elevation of the Collar Tie showing the mix up in the use of the lifting points and the sling attachment- Result: choking the slings against unprotected edges. Prepared by Arup.



Photograph 8.5 - Installation of the 2nd section of the collar at the 18th floor. The slings are choked around the tower crane legs without protection against the flange edges at the 1st section of the collar (red arrow). (Source: photograph by Gary Halby).

One of the slings that was used- referred as the "Metro" or "Liftex" sling in the report- was already frayed and deteriorated before it was used to support the collar. Use of damaged or faded slings is in violation of accepted industry standards. See Photograph 8.6 below. The faded appearance of the sling and the illegible warning label are easily observed. As noted in the OSHA report

> "*Post-incident examination of the Metro sling revealed that the sling was already frayed and deteriorated even before it was used to support the collar… The situation worsened when the sling was choked around the column, landing it in the V-shaped groove. The degradation and damage to the slings were so extensive that the Metro sling should have been discarded and not used.*"

Photograph 8.6 - Metro/Liftex sling recovered from tower crane. Not the faded label and faded appearance of the sling exterior. Photo by Arup.

The weight of the collar is given by Favelle Favco as 11,280 lb. The distribution of this weight among four slings with variable stretch would be unknown; it is conceivable that ½ of the weight, 5,640 lb., could be carried by one sling. Without packing, the anticipated breaking capacity of a new sling, based upon the OSHA test results would be as low as 7100 lb. WSTDA tests on slings exposed to ultraviolet light (see Section 3.1.5, with reference to WSTDA-UV-Sling-200) show a possible reduction in capacity of 10-20% with moderate exposure, thus reducing the sling breaking capacity further to as low as approximately 5,680 lb. to 6,390 lb.

8.5 Dynamic Analysis of Polyester Web Slings

In addition to the dynamic analyses described in Chapter 7 of this report, a nonlinear study of the polyester sling collar support system was also performed. The non-linear analyses examine the dynamic amplification effects of one sling failing and the load being redistributed into the remaining three slings. Both even and uneven load distribution scenarios were studied.

8.5.1 Supplied Data

The input data included the following items:

1. Sling properties as determined by tests carried out by ATLSS under direction of OSHA
2. The following drawings as provided earlier in the report and in Appendix F:
   a. Tower crane elevations and jump schedule (Figure F1.1)

b. Collar and tie-beam part plan and details (Figure F1.2)
c. Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 (Figure 1.13; Chapter 1)
d. Favelle Favco Drg. A2-7161.151 Building Ties – Details. (Figure F1.7) Received from NY Crane Bates No. NYcrane000000444.
e. Tie beam A, B & C Detail (Figure F1.3)
f. Crane truss (prototype) (Figure F1.6)

3. Photographs of the failed assembly pieces and of the connections to the building.
4. Sketches of the failed collar assembly pieces with notable measurements including overall dimensions and thicknesses.
5. Prototype applications for M440-D Tower Crane, ref [2], Chapter 7.
6. Operating, Maintenance and Parts Manual, ref [3], Chapter 7.
7. Drawings for prototype application for M440-D tower crane, ref [4], Chapter 7.

Note: Where the as-built crane differs in some dimensions from the prototype defined in the supplied documentation the on-site measurements were used.

8.5.2 Sling Analysis

Using the results of the physical tests performed by OSHA for each sling, a nonlinear dynamic analysis was performed to assess the behavior of an assembly of such slings. More precisely, these analyses assumed the sudden failure of the first of the four slings and assessed the response of the remaining three.

8.5.2.1 Model Description

The finite element model consists of three main components: the tower, the collar and the four slings, as can be seen in Figure 8.12.

The tower and collar were modeled using shell elements. As the tower and collar stiffnesses were not of interest they were both defined as rigid, with the tower fixed in space and the collar free to displace. Contact surfaces were defined between the collar and the tower to constrain the movement of the collar.

The collar was attached back to the tower with a sling at each corner. The slings were modeled using discrete spring and beam elements, as can be seen drawn in red in Figure 8.13. The slings were attached to the collar at the crane lifting lugs and to the tower at the intersection of the vertical leg and the cross member above the collar, see Figure 8.14.

The analysis was performed in two stages. The first stage considers the application of self-weight and uses an artificial critical damping to 'settle' the collar in equilibrium. In this type of analysis, this stage is necessary to create an initial condition of equilibrium. The second stage begins with the assumption of sudden failure of one sling and then models the response of the system. That stage has no artificial damping.

Two configurations of the model were analyzed. The first, with an even load distribution between all four slings and the second, with an uneven load distribution such that the entire load was carried by two slings: the sling assumed to fail first and the sling diagonally opposite. The witness statements for the actual sling failure suggest that it was the south west (SW) sling that broke first, which was replicated for both configurations in this study.

The three slings that do not initially fail, i.e. the north east (NE), north west (NW) and south east (SE), were defined for both model configurations with non-linear properties, as described in Section 8.5.2.2.

For the first model configuration, the SW sling was defined with linear properties having the same stiffness as the initial stiffness of the non-linear springs. It was attached in series to a nominally stiff beam element onto the collar. The stiff beam element was then deleted after self-weight equilibrium was achieved, thus replicating the failure of the sling. Deleting the beam effectively models the sling failure as instantaneous. The dynamic response of the collar and subsequent load redistribution were then simulated.

For the second model configuration, the SW sling was defined using a beam element with thermal expansion coefficient of $1.0\text{x}10^{-4}$ C^{-1}. After self-weight equilibrium was achieved, the temperature of the beam element was then reduced, thus shrinking the element and approximately doubling the load in the sling. This replicated a potential uneven load distribution in the slings due to jacking at one corner. The opposing sling (NE) also increases its load in order to maintain equilibrium. This case is a more onerous condition as there is more load to redistribute to the remaining slings once the SW sling fails. Again, the beam element was deleted replicating an instantaneous sling failure.

It is to be noted that the actual load distribution between the four slings just before failure is unknown. Due to the method of erection and variable time-dependent elongation of the slings, as noted in the OSHA report, the load in any particular sling could have ranged anywhere from zero to half of the collar weight.

Note: The centre of mass of the collar is located towards the building due to the tie-beam flanges, therefore there is initially more axial force in the combined northerly slings.

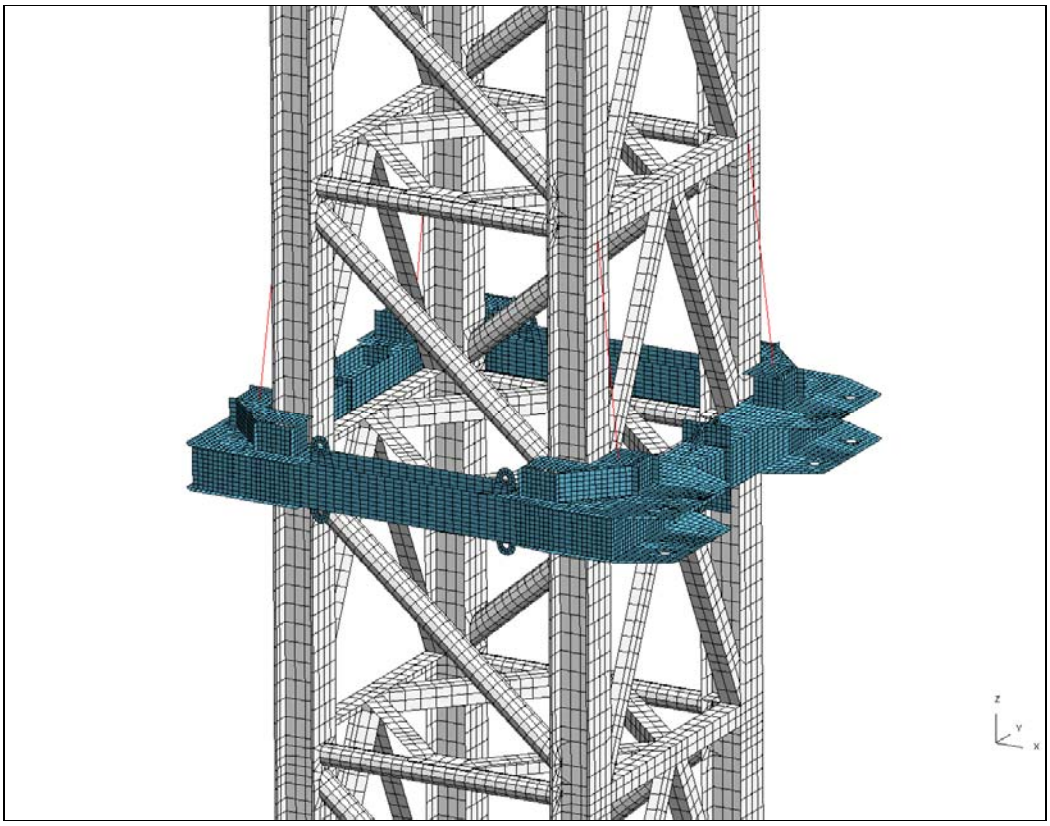


Figure 8.12 - Mesh Plot of Tower, Collar and Slings. Prepared by Arup.

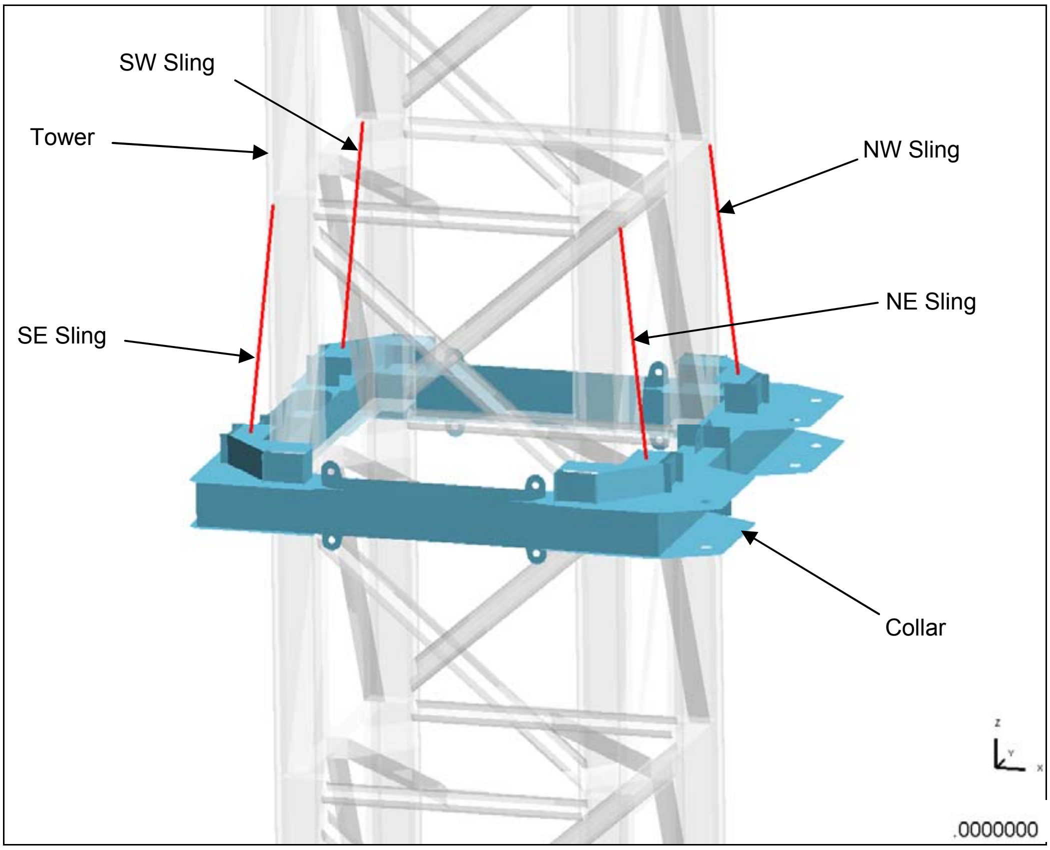


Figure 8.13 - Mesh Plot of Tower (transparent), Collar and Slings. Prepared by Arup.

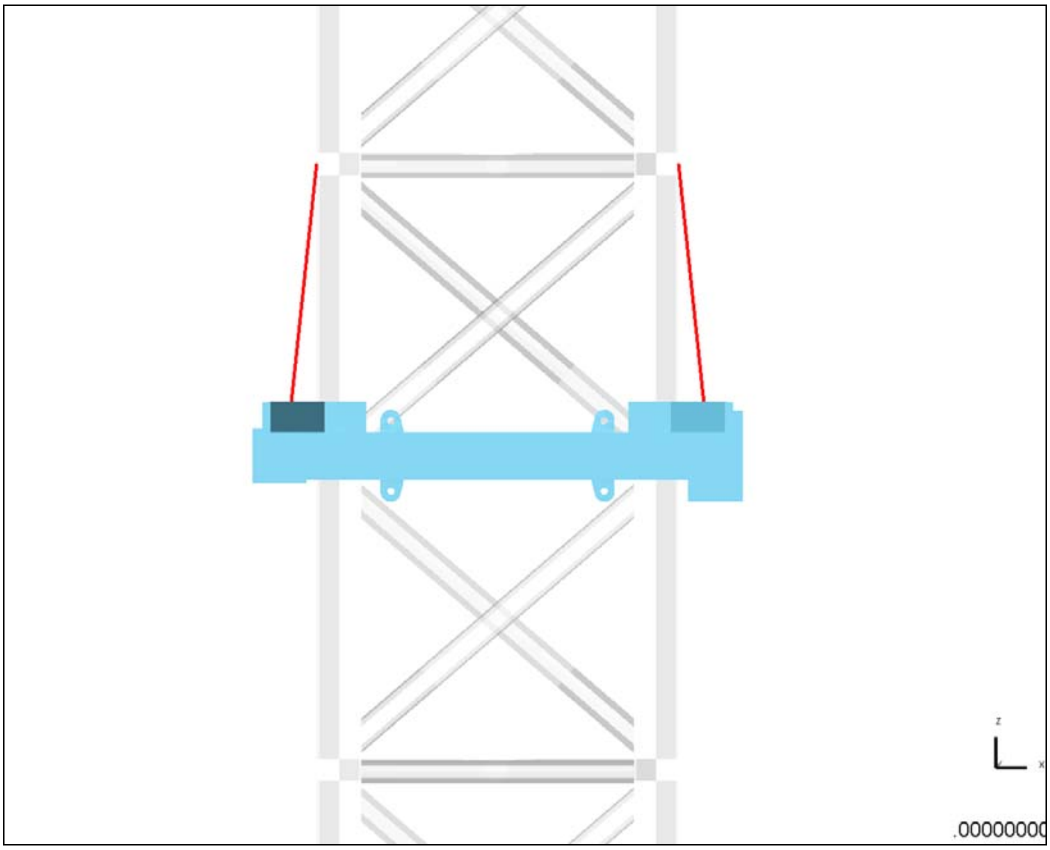


Figure 8.14 - Mesh Plot of Tower (transparent), Collar and Slings (red) – Side View. Prepared by Arup.

8.5.2.2 Test Data Interpretation

Separate FE analyses have been performed for three of the load curves generated from the OSHA testing described above. This provides a range of results for potential variation in sling capacity and ductility. The curves for 2-ply slings manufactured by LiftAll were used as they represent the three slings which did not fail initially. Note that this data was provided courtesy of OSHA.

Curves #3, #9 and #11 were selected and converted into load deflection curves suitable for analysis, as can be seen in Figures 8.15 & 8.16 respectively. Basically, the load test results were reduced into a series of linear segments, as shown in Figures 8.17 & 8.18 in imperial and metric units respectively. It should be noted that the initial low stiffness in the test results is due to slack being taken out of the system. This is representative of the test conditions but not the field conditions and has not been included in the FE curve data. Also, the 'dips' in the test response, most obvious in curve #2 Figure 8.15, have also been ignored as they do not affect the overall capacity.

Curve #3 (Figure 8.15) has the greatest capacity, 8.4kips (37.5kN), and is the most ductile. Curve #9 has a medium capacity, 7.8kips (34.9kN), but the least ductility. Curve #11 (Figure 8.16)has the least capacity, 7.1kips (31.6kN), and a medium level of ductility. All the curves were defined with effectively zero stiffness in the compressive direction.

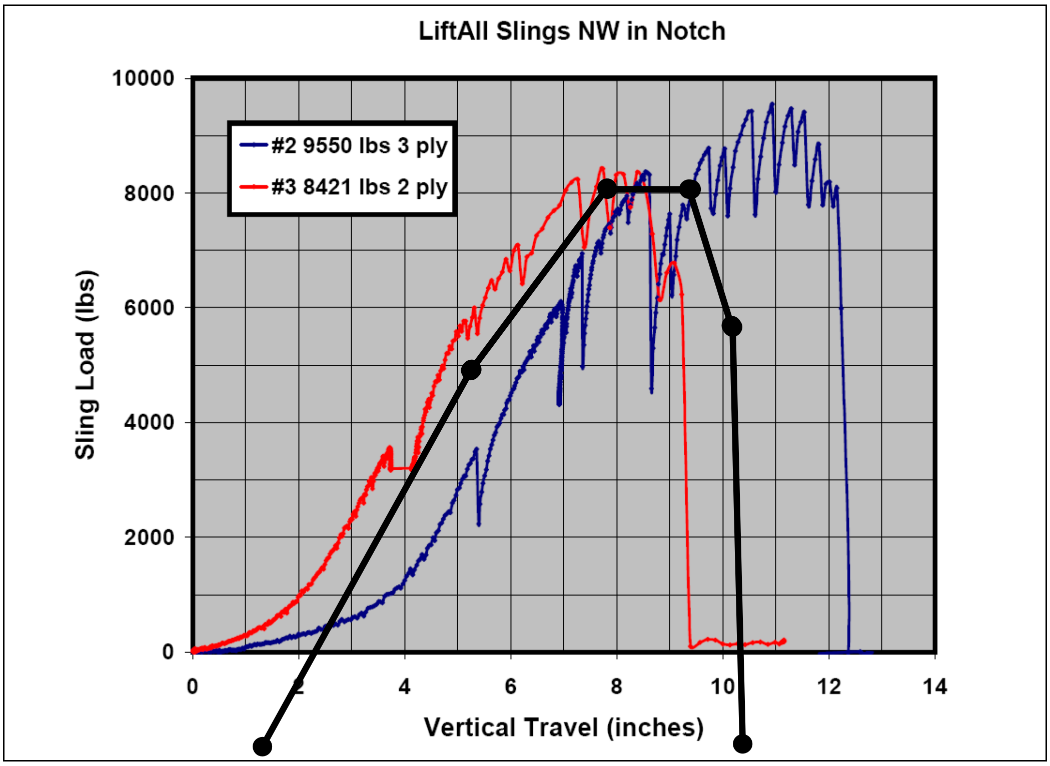


Figure 8.15 - Test Data for Tests #2 & #3 (simplification of #3 drawn in black). Prepared by Arup.

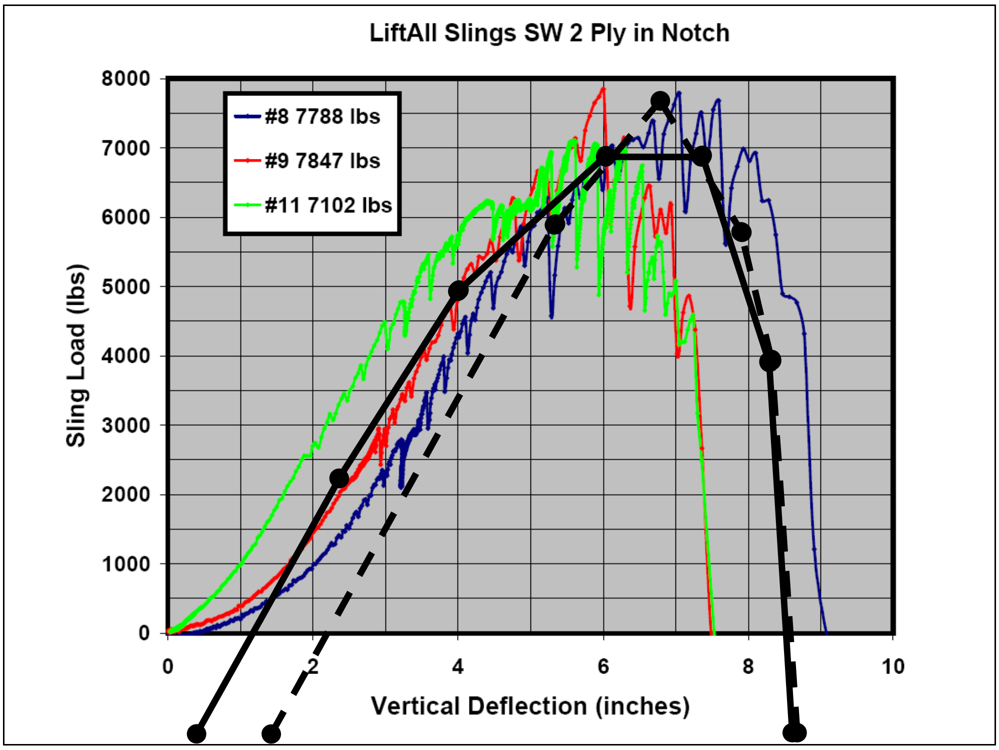


Figure 8.16 - Test Data for Tests #8, #9 & #11 (simplification of #9 & #11 drawn in black). Prepared by Arup.

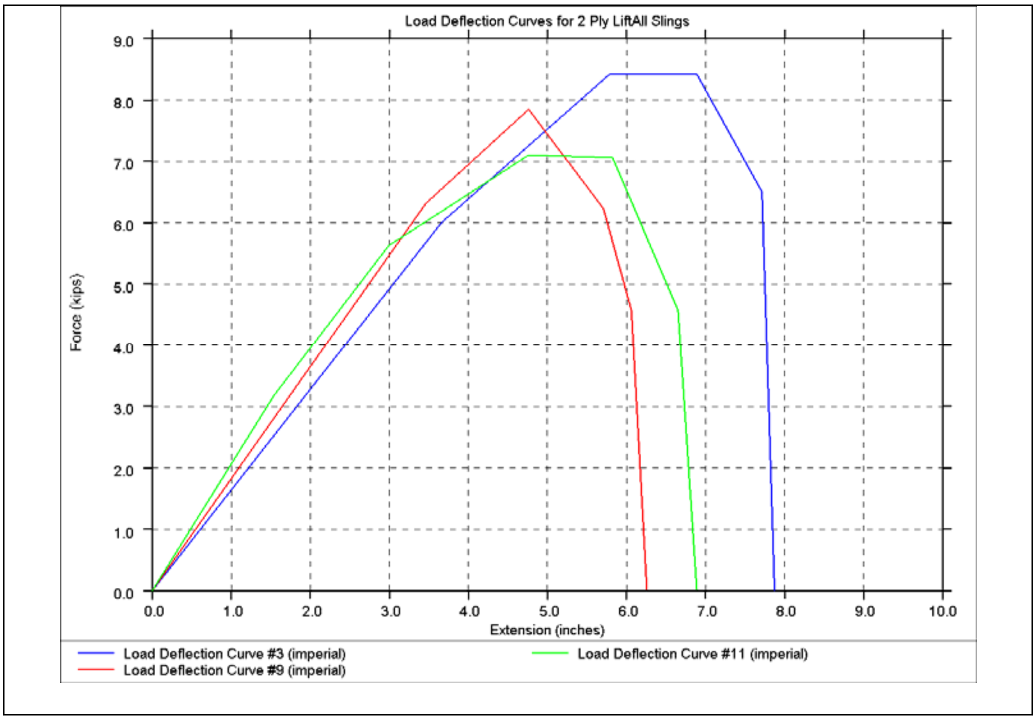


Figure 8.17 - Simplified Load Deflection Curves used in Model (imperial). Prepared by Arup.

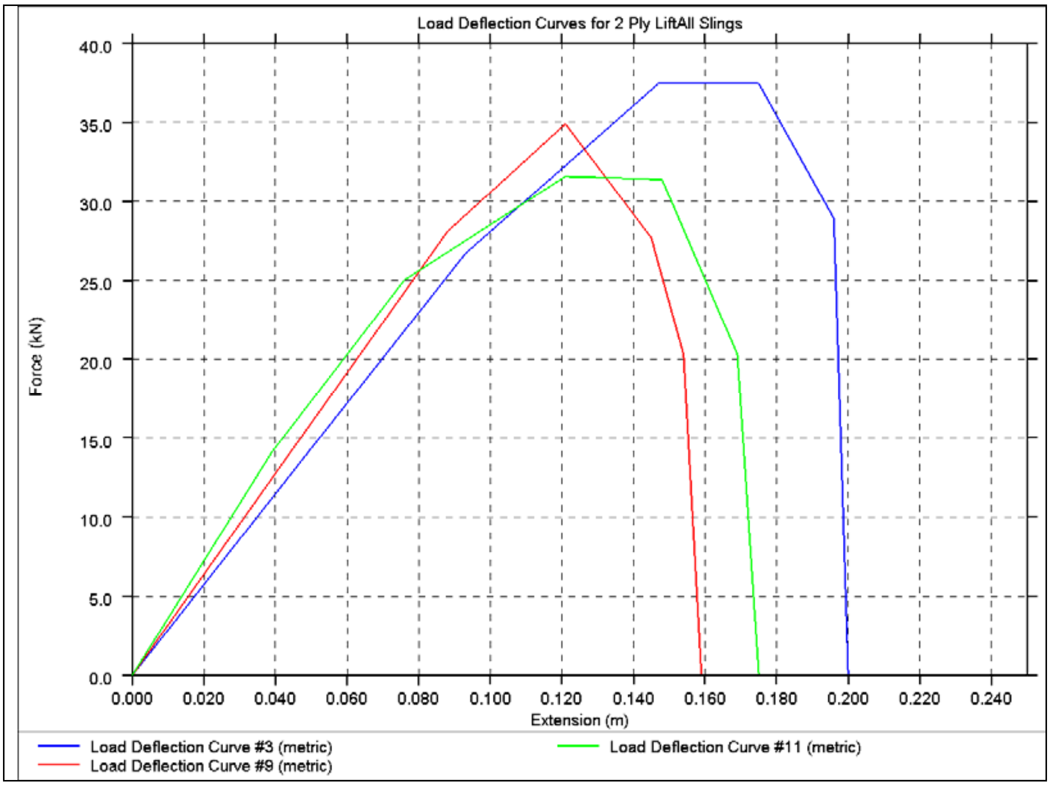


Figure 8.18 - Simplified Load Deflection Curves used in Model (metric). Prepared by Arup.

8.5.2.3 Results

A series of six analyses were performed for the SW sling suddenly failing first:

1. Uniform Initial Sling Loads with Curve #3 Properties
2. Non Uniform Initial Sling Loads with Curve #3 Properties
3. Uniform Initial Sling Loads with Curve #9 Properties
4. Non Uniform Initial Sling Loads with Curve #9 Properties
5. Uniform Initial Sling Loads with Curve #11 Properties
6. Non Uniform Initial Sling Loads with Curve #11 Properties

Deformed shape plots and sling axial force histories (Figure 8.19 to 8.27) are presented for each analysis respectively. The first 2 seconds of the axial force time histories represent the self-weight settlement of the model. It can be seen that the northerly slings take a greater load, 3.4kips (15kN) each and the southerly slings take 2.2kips (10kN) each. This combines to 11.2kips (50kN), which is the weight of the collar. The load can be seen to increase in the SW and NE sling between 2 and 4 seconds for the non uniform initial sling load analyses, with a corresponding unloading of the SE and NW slings.

Neither of the analyses in which the sling properties from curve #3 are used result in failure of the remaining slings, see Figure 8.19. Most of the load is initially transferred onto the NW sling for both analyses. The uniform initial sling load analysis is less severe, in which the NW sling reaches approximately 90% of its capacity; see the orange response in Figure 8.20. When the SW sling fails the load is lifted off the NE sling which is then intermittently loaded and unloaded.

The non uniform initial sling load analysis causes some yielding in the NW sling, as can be seen by the plateau in the response at 4.4s, see Figure 8.21. The capacity of the sling, however, is enough to support the load it experiences.

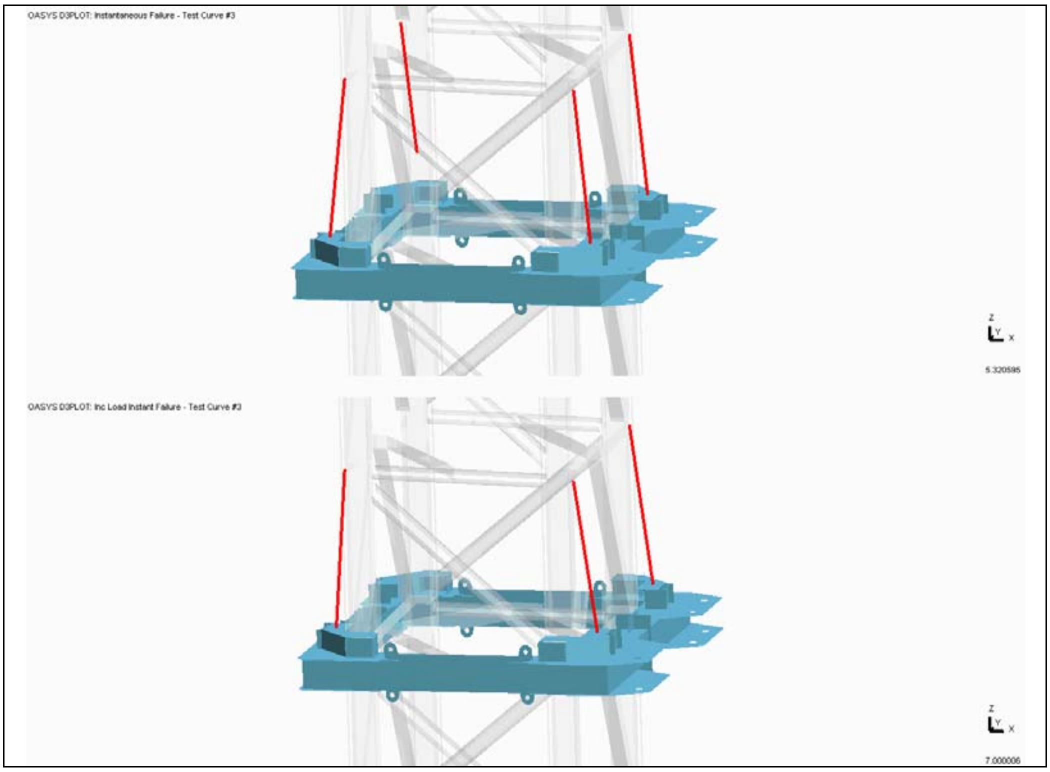


Figure 8.19 - Deformed Mesh at end of Curve #3 Analyses (uniform initial sling load top – non uniform initial sling loads bottom). Prepared by Arup.

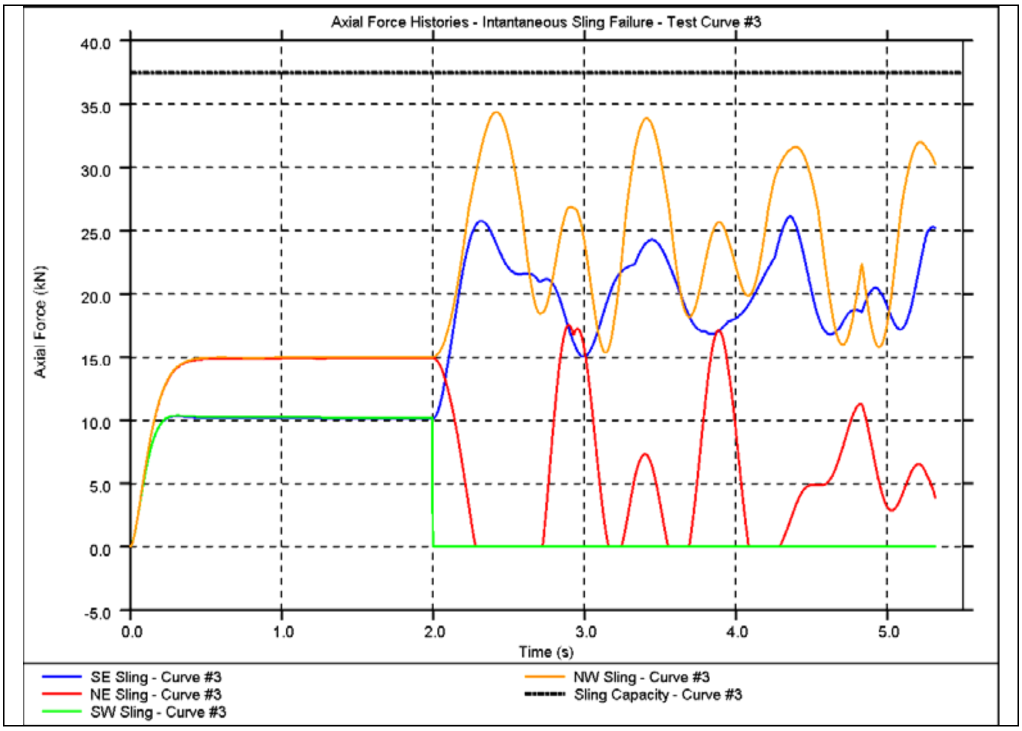


Figure 8.20 - Sling Force Time History –Uniform Initial Sling Loads – Test Curve #3. Prepared by Arup.

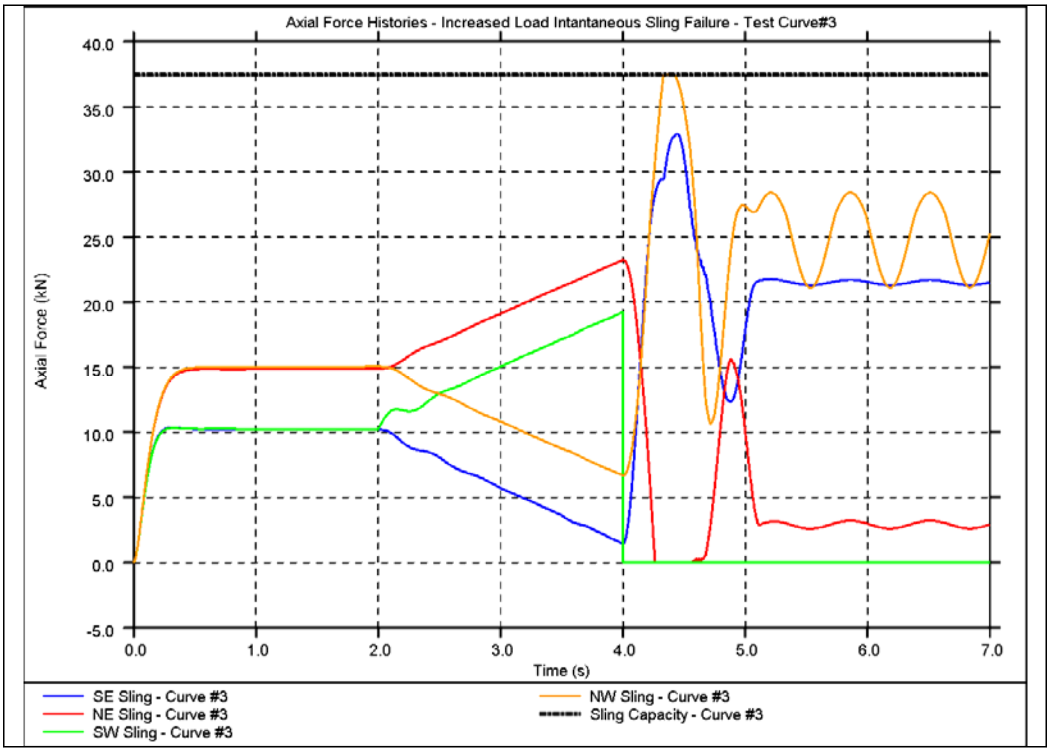


Figure 8.21 - Sling Force Time History –Non Uniform Initial Sling Loads – Test Curve #3. Prepared by Arup.

The uniform initial sling load analysis with curve #9 supports the collar after the initial sling failure, see Figure 8.22. Again, most of the load is initially transferred onto the NW sling, which reaches approximately 97% of its capacity; see the orange response in Figure 8.23. The remaining slings have sufficient capacity to support the collar.

The non uniform initial sling load analysis with curve #9 predicts that support for the collar will fail, as shown by the axial force in all slings reducing to zero in Figure 8.24. The subsequent failure of the NW sling causes the entire load to be transferred to the remaining NE and SE slings. Even though they still have sufficient static capacity, the dynamic amplification of this load transfer is large enough to cause further failure. The disappearing collar in Figure 8.22 indicates that the support system has failed. It should be noted that the NW, SE and NE slings have failed even though they are still drawn in the figure.

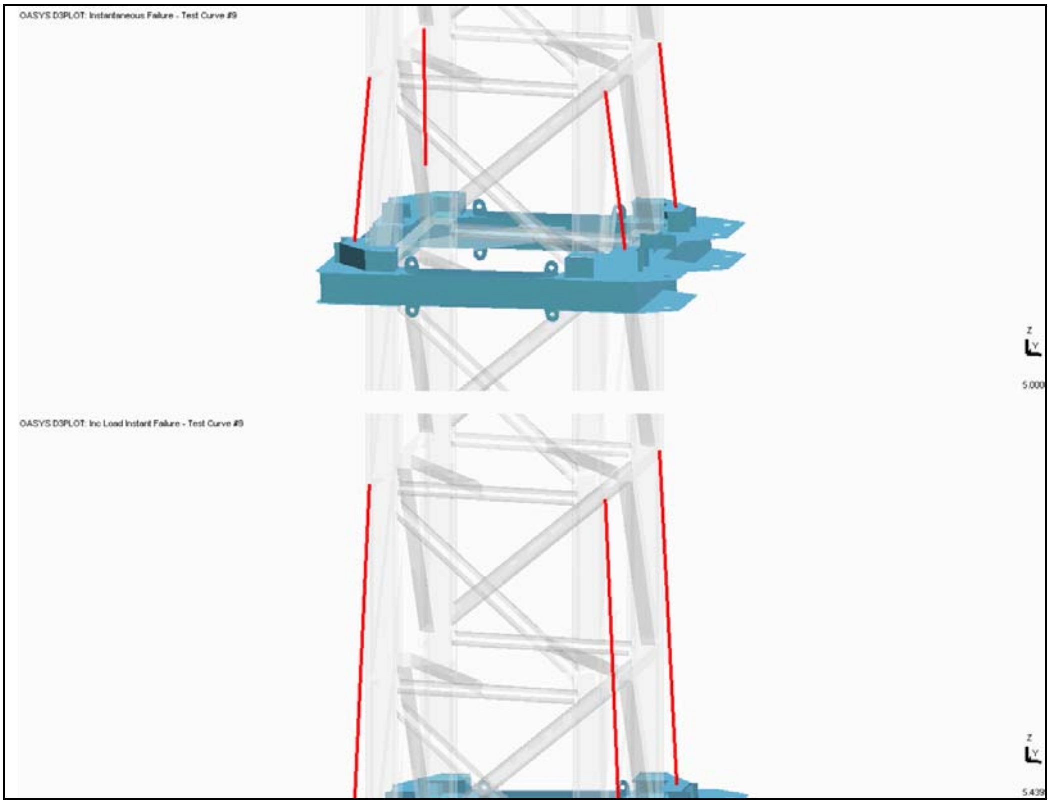


Figure 8.22 - Deformed Mesh at end of Curve #9 Analyses (uniform initial sling load top –non uniform initial sling load bottom). Prepared by Arup.

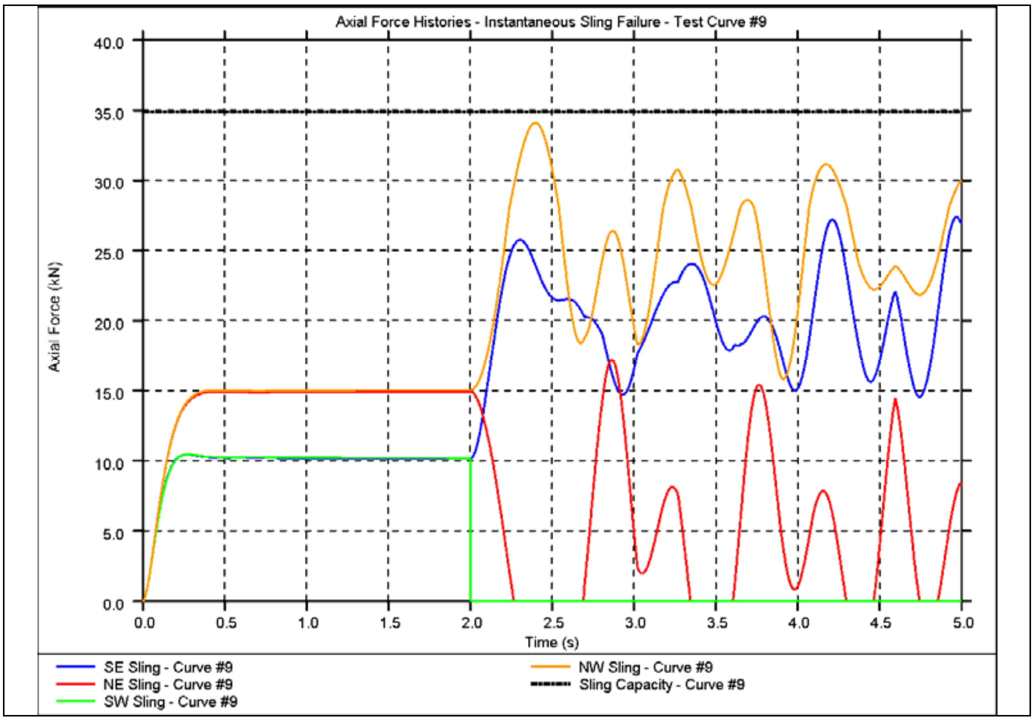


Figure 8.23 - Sling Force Time History – Uniform Initial Sling Loads– Test Curve #9. Prepared by Arup.

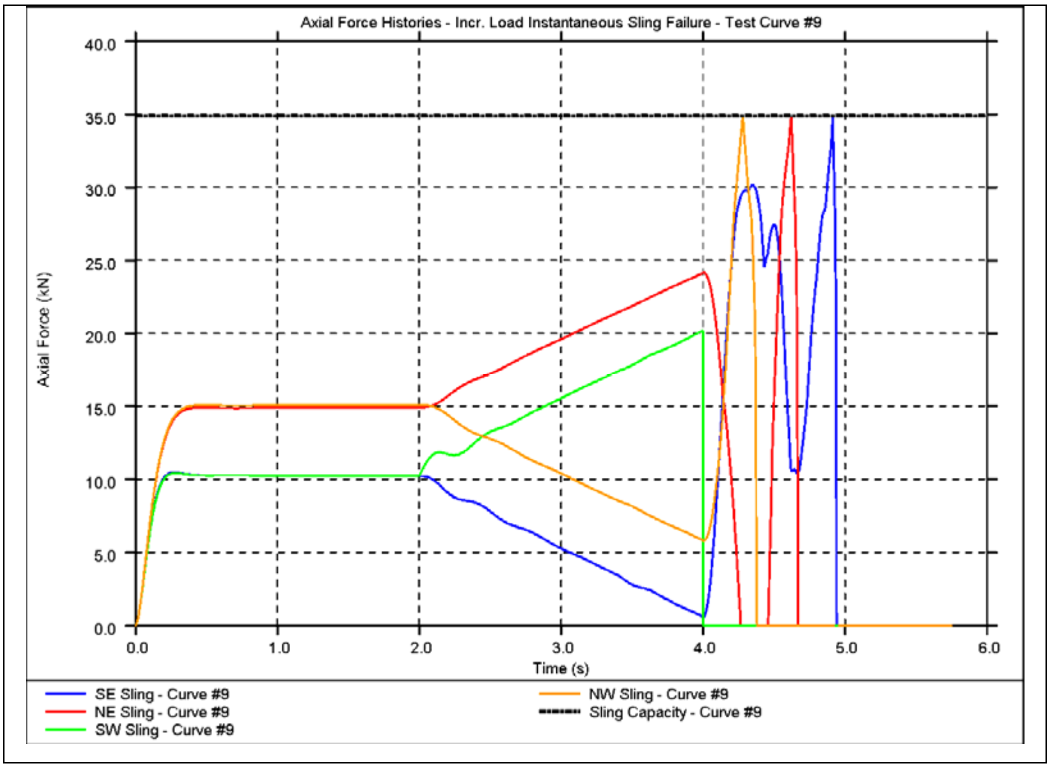


Figure 8.24 - Sling Force Time History – Non Uniform Initial Sling Loads– Test Curve #9. Prepared by Arup.

The uniform initial sling load analysis with curve #11 supports the collar after the initial sling failure, see Figure 8.25. Again, most of the load is initially transferred onto the NW sling, which causes some yielding, as can be seen by the plateau in the orange response at 2.4s (Figure 8.26). The sling yields again in subsequent load oscillations, but not enough to cause failure. The remaining slings have sufficient capacity to support the collar.

The non uniform initial sling load analysis with curve #11 predicts that support for the collar will fail, as shown by the axial force in all slings reducing to zero in Figure 8.27. The subsequent failure of the NW sling causes the entire load to be transferred to the remaining NE and SE slings, thus following the same failure mechanism described above for the curve #9 analysis. The disappearing collar in Figure 8.25 indicates that the support system has failed. It should be noted that the NW, SE and NE slings have failed even though they are still drawn in the figure.

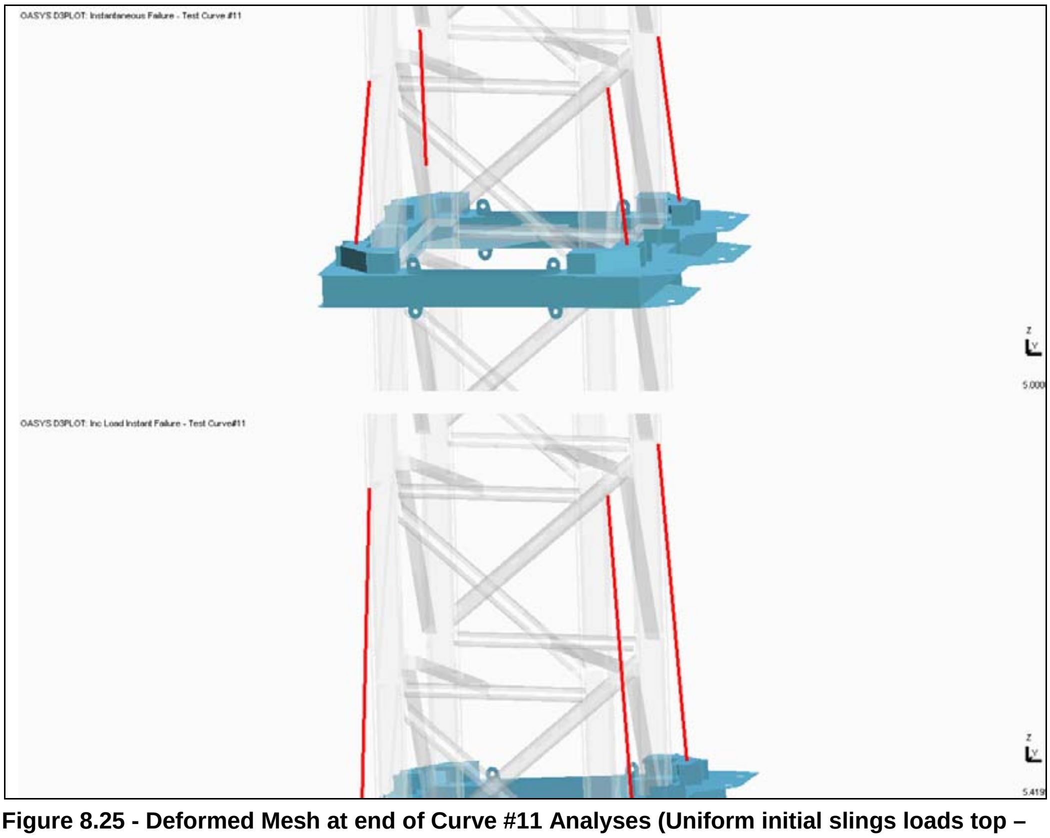


Figure 8.25 - Deformed Mesh at end of Curve #11 Analyses (Uniform initial slings loads top – non-uniform initial sling loads bottom). Prepared by Arup.

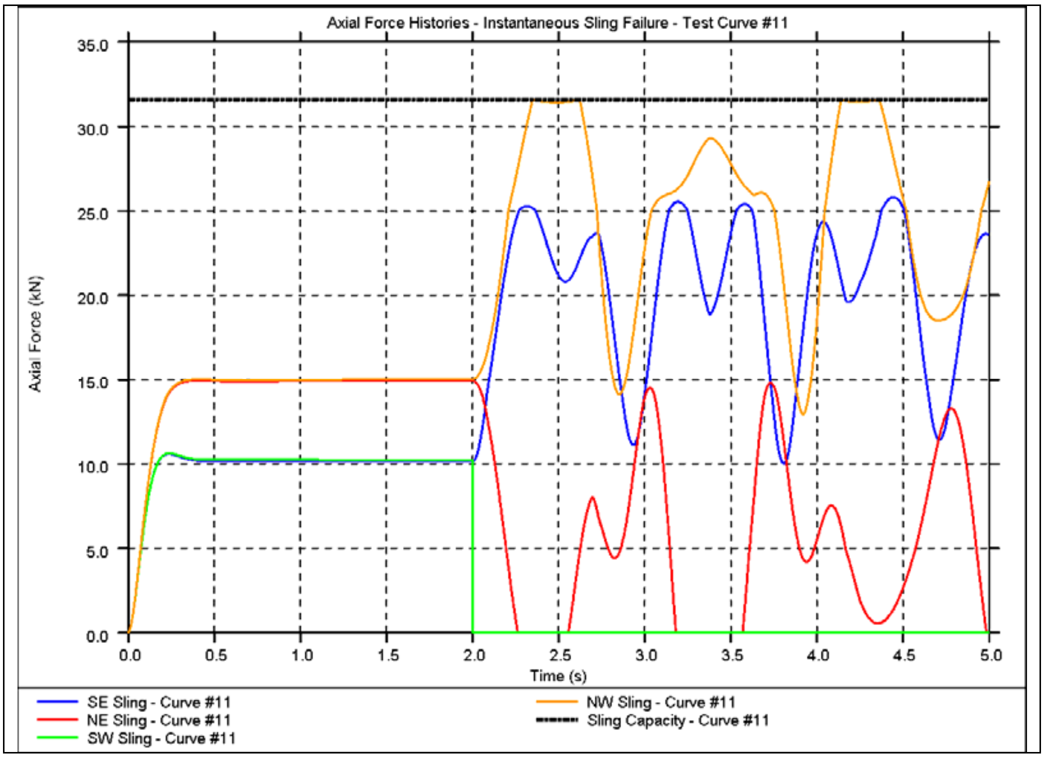


Figure 8.26 - Sling Force Time History – Uniform Initial Sling Loads – Test Curve #11. Prepared by Arup.

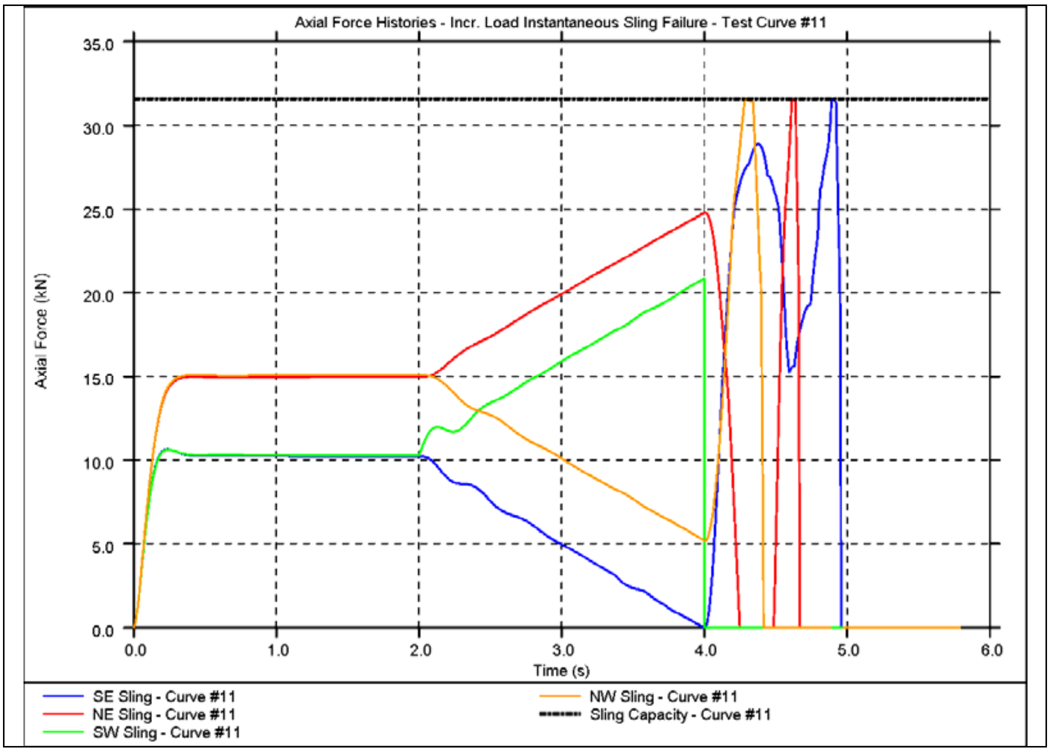


Figure 8.27 - Sling Force Time History –Non Uniform Initial Sling Loads– Test Curve #11. Prepared by Arup.

Of the six analyses performed, four retained support for the collar after the initial sling failure, and two analyses predicted complete failure of the sling support system. Although four of the analyses did not predict failure, they generally either had some yielding or were within 10% of yielding in certain slings, i.e. they were close to failure, as illustrated by the analyses that fully failed.

The analyses completed were based on the following assumptions:

- The test data was based on brand new slings, i.e. no aging or degradation had taken place.
- The slings had no padding protecting them against the tower steel edges; e.g., for the Curve #3 results, padding could have significantly increased their capacity from 8.4 kips (37.4kN) to 11.2kips (49.8kN).
- The slings were mounted so that the 2-ply segments were mounted against the tower steel edges rather than 3-ply; e.g., for the Curve #3 results this could increase the capacity from 8.4 kips (37.4kN) to 9.6kips (42.7kN).
- The slings had no initial damage.
- The initial sling failure was instantaneous.
- A friction coefficient of 0.15 was assumed between the collars and the tower.

Had the SW sling failure been slower, i.e. not instantaneous, the effect of dynamic amplification on load redistribution would have been reduced. For the above assumptions, a slower initial sling failure would therefore be less onerous. However, any damage to the slings, such as from prior use or ultraviolet degradation, would increase the probability of failure.

8.5.3 Summary of Analysis Results

A series of non-linear sling failure simulations were performed. The analyses were used to predict the dynamic load behavior of the four sling collar support system of the 18th floor collar after the failure of a single sling. The analyses were performed for sling failures with both an even starting load distribution and an uneven starting load distribution, and were based on new sling properties i.e. no ageing or degradation had taken place. The properties of each sling used in the analysis were taken from the OSHA test results which were intended to closely simulate field conditions.

Of the analyses performed two of the three analyses based on an uneven load distribution predicted complete failure of the sling support system as the result of the failure of one sling. Variations in results arose from application of the differing OSHA test data in each respective analysis.

Although the specific condition of the slings on site including their strength, ductility and load distribution is not known, the analyses demonstrate that it is indeed possible that the failure of one sling would precipitate the failure of the remaining three. More importantly, the analyses demonstrate the need for careful control of these conditions when using synthetic web slings.

9 Material Testing and Evaluation of Tie Beams Weld

9.1 Introduction

Following the tower crane collapse, concerns regarding the tie-beam design and fabrication were rapidly identified as of particular interest to the investigation. Specifically, failure of the tie-beam welds at the base plate connection to the building was readily evident from the recovered debris. Initial observations suggested that the as-built weld size was below minimum requirements and that other possible deficiencies regarding the quality of the weld may be present.

Due to the importance of the tie-beams to the stability of the tower crane it was considered essential to evaluate these welds further. Accordingly, ATLSS, a National Center for Engineering Research on Advanced Technology for Large Structural Systems at Lehigh University was selected as having the requisite expertise for evaluating the welds and other possible metallurgical items of interest.

Specific tasks respecting metallurgical investigation of the welds included examination of the failed fillet weld connections between horizontal tie-beams B4 and B5 and their respective base plates 9A and 9C from floor level 9. Although the investigations were originally to also include similar tasks for unfailed base plate welds from tie-beams B1, B2 and B3 from floor level 3 and B9 from floor level 9, this was reduced to just the B9 base plate and tie-beam. Thus all three tie-beam/base plate weld connections from floor level 9 were examined. While other design and fabrication issues were of early possible interest, it was determined during the investigation that these were unrelated to the collapse of the tower crane and were therefore not pursued further.

The base plates 9A and 9C for tie-beams B4 and B5 are shown in Photographs 9.1 and 9.2. B9 base plate and tie-beam (still attached) are shown in Photograph 9.3.

The objectives of the metallurgical examination were to determine the quality of the welds, effective weld sizes, fabrication methods used and other features which might arise from a strictly visual non-destructive examination.

The metallurgical investigation was performed by Dr. Eric J. Kaufmann of the ATLSS Engineering Research Center with assistance by Dr. Alan W. Pense of the ATLSS Engineering Research Center. A copy of the ATLSS report is included in Appendix J.

9.2 Methodology

Test protocols for the macroscopic testing; i.e., non-destructive visual examination, were prepared in consultation with ATLSS. The final protocols used for the testing are presented in Appendix H.

9.3 Scope of Testing

As described in the test protocols, the testing was a strictly non-destructive visual examination of the welds and components. No destructive examination, including determination of weld material or strength properties, was undertaken.

9.4 Results

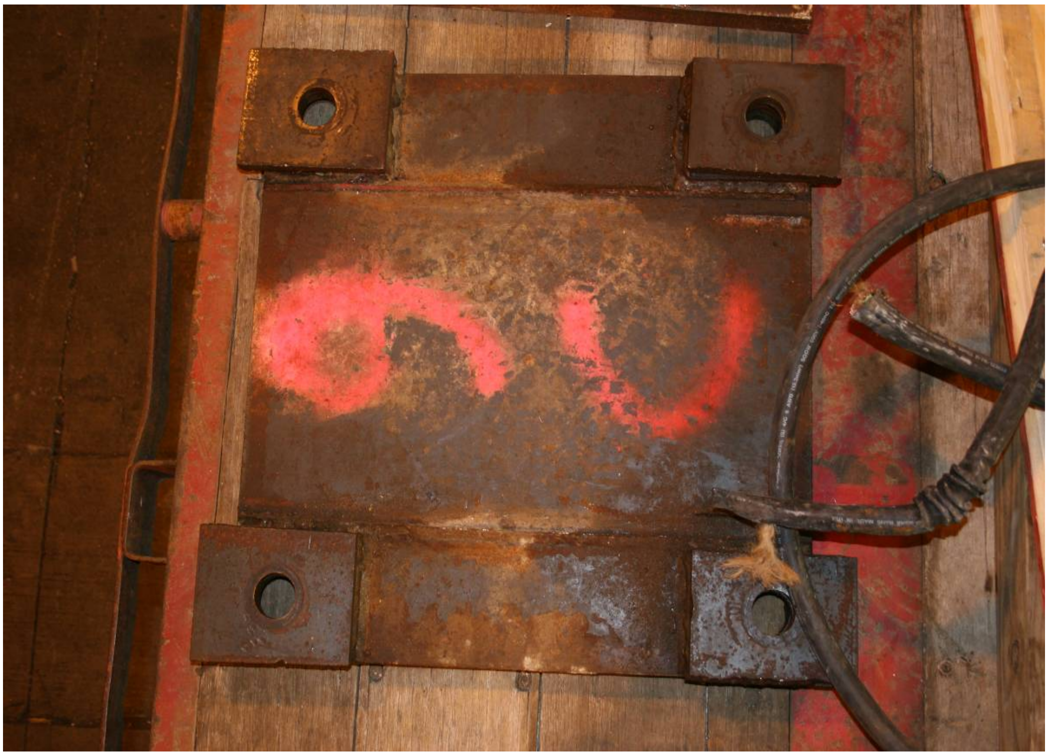
Of the crane components included in the protocol only the three horizontal tie-beams and respective base plates located on the ninth floor of the building were selected for the metallurgical investigation. In accordance with the non-destructive macroscopic examination prescribed in the investigation protocol the components were visually examined in their as-found condition. No surface alteration or preparation was undertaken beyond the specified surface cleaning.

The horizontal tie-beam/base plates consisted of a 1-1/4 in. thick steel plate with a W12x79 rolled shape fillet welded to the plate surface at the beam end. A pair of rectangular washer plates was used at each bolt location, where the base plates had been connected to the building floor slab, and tack welded to the base plate. The base plate components identified as 9A and 9C exhibited fully failed fillet welds between the tie-beam and base plate. The respective tie-beams, also exhibiting weld fracture at one end of the beam, were identified by ATLSS as B-4 and B-5. The third tie-beam/base plate component, identified as 9B, had intact welded connections.

The measured weld lengths varied and ranged from 20 inches to nearly the full length of the base plate. A short length of fillet weld of variable length was also deposited along the interior flange tips at the beam end ranging from 2-1/2 in. to 4-5/16 in. This compares with the fabrication drawings which specified a 22 in. length with no weld along the interior flange tip. See Figure 9.1 below.

Most bolt holes were drilled; a few holes appeared to be torch cut. Missing washer plates on base plates 9A and 9C were apparently not tack welded to the base plate. The section dimensions of the three W12x79 tie-beams were found to be within the permitted tolerances specified in ASTM A6. Note that in the drawing there is no requirement to tack weld the washer plates.

Photograph 9.1 – Base plate 9A from floor level 9. Photo by Arup.



Photograph 9.2 – Base plate 9B from floor level 9. Photo by Arup.

Photograph 9.3 – Base plate 9B from floor level 9. The tie-beam is still attached. Photo by Arup.

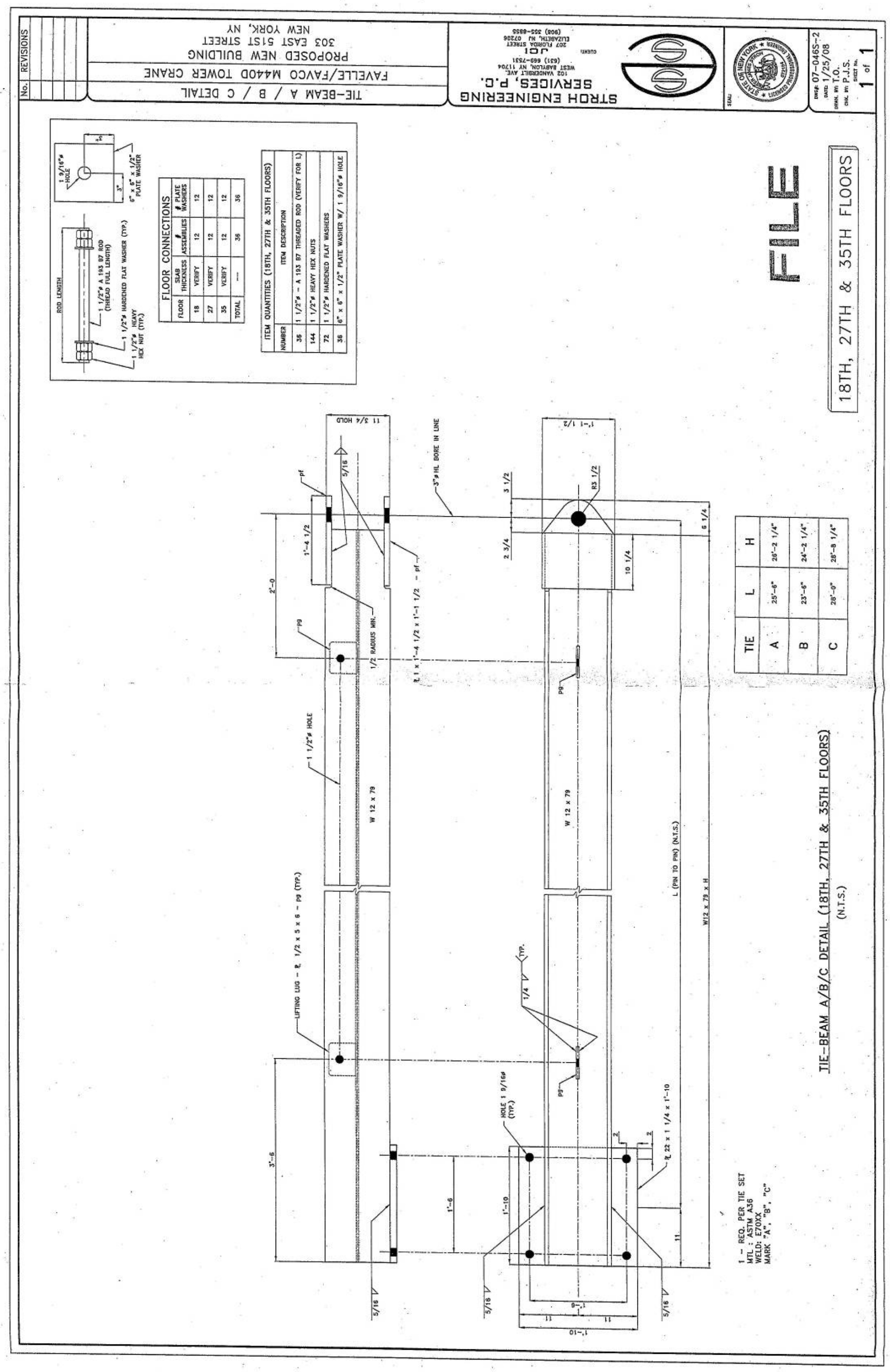


Figure 9.1 - Tie-beam A / B / C Detail, Fabrication Drawing, by the crane engineer Stroh Engineering Services, P.C. Drawing 07-046S-2 Sheet 1 dated 1/25/08.

The appearance of the welds indicated that they were likely deposited using the shielded-metal-arc weld process (SMAW) in a single pass. It could not be determined based upon examination if the welds had been shop or field welded.

Regarding tie-beam B-5, fillet welds generally exhibited unequal leg dimension with the shorter leg existing on the tie-beam. Fillet weld size measurements obtained on all three tie-beam/base plate components indicated a weld leg size ranging from 3/16” to 5/16” with the majority of the deposited weld having a ¼” weld size. This is below the minimum specified by the American Welding Society (see Chapter 3, Section 3.1.6, on American Welding Society Standard D1.1).

Visual examination of the weld fractures on tie-beam B-5 indicated that the welds fractured by ductile shear along the weld leg of the tie-beam. The welds were generally sound and free of weld defects with only isolated weld porosity. No evidence of weld cracking or incomplete fusion was observed.

Tie-Beam B4 weld fractures showed similar fracture features but only a small amount of weld metal deposited on the tie-beam. The small amount of weld fracture observed on the tie-beam indicated an effective weld size much less than the deposited weld size previously measured on the base plate, arising from the tie-beam fit-up to the base plate. However, with the majority of weld having an effective weld size of 3/16”, it is still sufficient to support the design loads (see Section 4 on Peer Review of Tower Crane On-Site Design) without failure, although not with the minimum required factor of safety. As for B-5, this is below the minimum specified by the American Welding Society.

Similar fracture features as observed on Base Plate 9A and Tie-Beam B4 were also observed on Base Plate 9C and Tie-Beam B5. As with Base Plate B4 only a small amount of deposited weld metal was observed on Tie-Beam B5 again due to unequal weld legs and excessive root opening when the connection was welded.

With regard to the still intact assembly, Base Plate/Tie-Beam 9B, the welds appeared to be sound and free of fabrication defects. A weld toe crack on the tie-beam leg was observed in one fillet weld at the weld end, although separation of the components had not yet occurred.

10 Site Visits

As part of the investigation a number of site visits were undertaken to view the collapse site, sling tests and evidence which had been retained by the various parties. These site visits are listed below:

- 2008-03-26 collapse site, 303 51st St., New York
- 2008-04-10 OEM Warehouse, Pier 36, New York
- 2008-04-25 crane yard, New York Crane, New Jersey
- 2008-04-18 OEM Warehouse, Pier 36, New York
- 2008-05-15 OSHA offices, 201 Varick Street, New York
- 2008-05-22 OSHA offices, 201 Varick Street, New York
- 2008-07-10 OEM Warehouse, Pier 36, New York
- 2008-08-01 OSHA offices, 201 Varick Street, New York
- 2008-08-19 ATLSS sling tests, Lehigh University, Bethlehem, PA
- 2208-10-29 OEM Warehouse, coat fracture surfaces, Pier 36, New York

11 Discussion

As described in Chapter 1, following the collapse of an external self-climbing tower crane at 303 East 51st Street, New York, New York, on March 15, 2008, Arup was hired by the NYCDOB to provide engineering and investigative services. This report has described the sequence of events that led to the accident dated March 15, 2008 as well as details of the investigation.

11.1 Crane Collapse – Summary of Events

At the foundation level, the tower was supported by a steel dunnage beam frame atop plywood shims, themselves resting upon concrete walls constructed within the ground. Horizontal restraint of the steel dunnage at the foundation level was provided through friction while vertical restraint was provided through bearing on the foundation. The tower legs at the base of the tower were positioned within pockets atop the steel dunnage beam, thus “fixing” the tower legs horizontally on the steel dunnage beam. Vertical restraint was provided through direct bearing on the steel dunnage beam. Uplift was not restrained but under normal engineering design assumptions would not occur. See Photographs 1.3, 1.4, and 1.5.

Above the tower base prior to the collapse, lateral restraint to the tower was provided at both the 3rd and 9th floors of the building under construction. Each floor-to-tower connection consisted of a structural steel collar assembly surrounding the tower, which was pinned to three tie-beams. The tie-beams were fixed to the building by welded connections to steel base plates, which were in turn anchored to the reinforced concrete slabs of the building. See Figure 1.2.

Immediately prior to the collapse, wire rope slings were being using to provide vertical support to these collars. Based upon photographic records and materials retained by OSHA it does not appear that Favelle Favco recommendations regarding the capacity and location of vertical restraints provided by the wire rope slings were followed, although the number of vertical restraints was met or exceeded. Additionally, the 3rd and 9th floor wire ropes used to provide vertical support to the collars were not installed in accordance with accepted rigging standards. Specifically, the bend radii did not meet minimum criteria; the U-bolt clips used to form the wire rope slings were not installed per industry standards. Finally, it is additionally noted that wire rope slings are not permitted by industry standards for this application (see Sections 3.1.2 and 3.1.4).

On the day of the collapse the weather records indicate no significant wind speeds or other meteorological event occurring.

At the time of the collapse the team had just “jumped” or extended the height of the tower and was proceeding to install a new collar connection to the 18th floor. The collar had to be erected in two halves and connected together so as to surround the tower. One-half of the collar was lifted into place where it was temporarily suspended from the tower by polyester web slings (each extended with a chain fall) using two attachment points not in conformance with any of the tower crane manufacturer specified four attachment points. Following this, the other half of the collar was similarly lifted into place, suspended by polyester web slings and the two halves were bolted together. Thus four attachment points, which were not in conformance with the eight tower crane manufacturer specified attachment points, were being used to temporarily suspend the completed collar at the time of the tower crane collapse

The lifting slings used to lift the collar halves into position should have been attached to the collar at locations designated by the tower manufacturer as lifting points. The lifting points are approximately located long the centroid of the collar mass. See Figure 3.2 showing these respective locations as well as the lugs to be used for later suspending the collar from the tower (“chain blocks”). The sling which had been used to lift the collar halves had been incorrectly attached to the collar using the chain block attachment points. This is evident in Photograph 11.1, which shows that during the lift, as a result of using the wrong lifting points, the collar half was unbalanced and was therefore significantly deviating from a horizontal orientation.

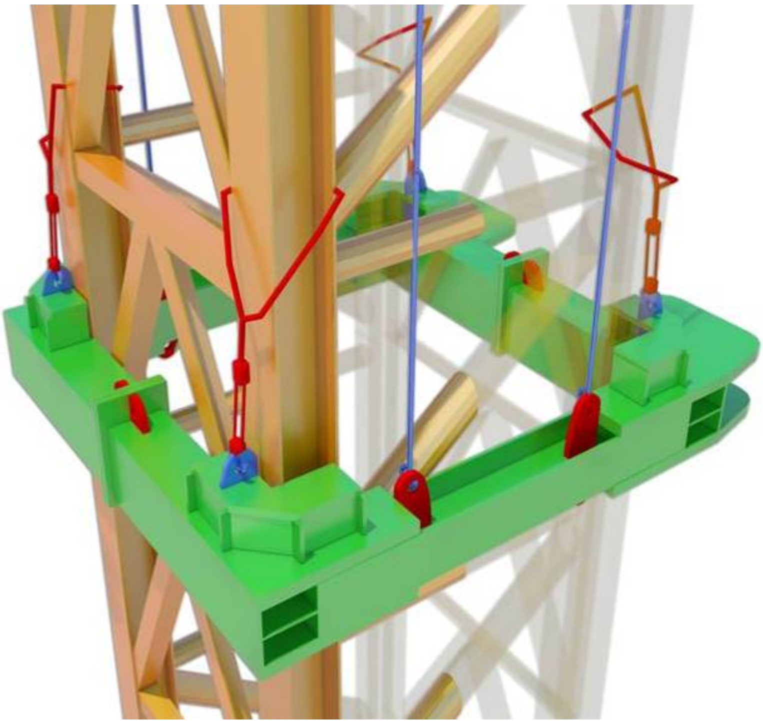


Figure 11.1 - Elevation of the Collar Tie showing the mix up in the use of the lifting points and the sling attachment- Result: choking the slings against unprotected edges. Prepared by Arup.

The two polyester web slings used to suspend the collar from the tower were attached to the lifting point lugs, rather than the designated chain block lugs. As a result, they were in the wrong location with regard to the tower face and had to be choked around the tower columns and diagonal bracing rather than the horizontal bracing. In that configuration, the polyester web slings were drawn into the “V” shaped groove between the bracing and the tower columns.

No padding was used to protect the polyester web slings from the unprotected edges of the tower legs around which they were slung. It would have been necessary for the slings to slide down against the tower leg flanges before they would be fully seated in the “V” groove even if they had been initially pushed into place by the construction crew. This was demonstrated during the testing procedures undertaken by ATLSS under direction from OSHA. Additionally, one of the slings, located at the southwest corner of the tower, was not new and exhibited both fading and surface damage.

As the crane crew began to place the first of its three tie-beams, the collar was suspended from the tower by only four polyester web slings (extended with chain falls). The Manufacturer has specified eight chain blocks to be used for this operation at locations other than those actually used for the polyester web slings. All aforementioned tower crane manufacturer specified locations and numbers of attachment points are per the Favelle Favco External Climbing Collar/Tie Erection Sequence Drawing A1-1100.123 (Figure 1.13). The first tie-beam was suspended from the crane to a position near the collar at one end and bearing on the building slab at the other. The beam was at the collar and being hand positioned between the plates in preparation for insertion of the pin.

Based upon the witness statements, distinct "popping" or "snapping" noises were then heard. The polyester web slings failed, allowing the 18th floor collar to fall along the tower and strike the 9th floor collar below; severing all three tie-beam connections at the collar end and failing two of the weld connection to the base plate connection at the building end. This allowed both collars to fall. Impact of the two collars on the remaining collar at the 3rd floor resulting in significant downward displacement of the 3rd floor collar and bending of the 3rd floor level tie-beams.

After the 9th floor connection failed, only the level 3 connection and the base friction remained to resist overturning of the tower. This system was further compromised by the deflection of the level 3 collar and significant bending of the connected tie-beams from the impact of the collars above. With lateral restraint remaining at Level 3 and the base only, the tower overturned as its base slid towards the building structure to the north and the crane cab at the top fell to the south causing damage to the adjacent buildings and generating seven fatalities and multiple injuries.

The base friction proved inadequate and the tower overturned as its base slid towards the building structure to the north and the crane cab at the top fell to the south. This is consistent with the design criteria used by the tower crane engineer (Stroh Engineering) since the crane base was not designed to make use of the friction that might have developed between the tower and the foundation as per Figure 1.7 in Chapter 1. For the crane configuration that existed at the time of the accident, the crane engineer's calculations and design (by Stroh Engineering) relied on the capacity of the ties at the 3rd and 9th floor levels to provide the lateral resistance of the crane tower and prevent overturning

11.2 Crane Engineer's Design Review

The engineering of the crane for connection to this particular building was undertaken by the crane engineer Peter Stroh (from Stroh Engineering); a registered professional engineer in the State of New York with particular experience in this field. The independent review of the design by a Crane Certified Agent Dale Curtis; a registered professional engineer in the State of California, found that the design was generally in accordance with good practice. While there were some issues noted these were not of concern in this investigation as they had no contribution to the collapse.

One such item concerns the manufacturer's unbraced height limitation. In order to install an upper tie-in collar, the tower sections may sometimes exceed the unbraced height limitation for a short duration when a weather forecast predicts wind velocity less than the upper limit of 20 mph for climbing operations as stated in the ASME/ANSI Standard B

30.3 for construction tower cranes. However, such a free-standing temporary height above a secured tie-in was not analyzed by Mr. Stroh.

Another item involves the failure to incorporate the slewing moment in the tie-beam design. Regardless, the independent calculations by Mr. Curtis indicate that the tie-in beam stresses and welds were found to be within acceptable levels. Similarly, the 4 bolts through the floor were found to possibly be marginal, though acceptable.

Regarding the aforementioned design issues, it is to be noted that, at the time of the collapse, these design loads were not being realized; i.e., as indicated above, the maximum slewing moment, overturning moment and wind speed were not present.

It is noted that the foundation system relied on friction for horizontal restraint. While unusual, this is an acceptable approach. As discussed in the section on design review, the foundation base beams and concrete appear to be well engineered. Mr. Stroh stated in his letter, dated January 14, 2008, that the lateral loads are taken in the 2 ties above and not by the crane steel dunnage frame at the base, which explanation was also accepted by NYCDOB. It should be mentioned that such a base frame is generally preferred to be positively restrained at the concrete below to prevent lateral movement during disassembly of the tower sections when only the lowest tie is attached to the completed building. This is not, however, a requirement.

In summary, the crane engineer's design submission (by Stroh Engineering) is for the most part in accordance with industry "best practice", save for some noteworthy departures as noted above.

11.3 The Use of Polyester Slings

Based on our investigation and analysis, it is our professional opinion to a reasonable degree of engineering certainty that the collapse of the tower was initiated when the polyester web slings supporting a steel collar at the 18th floor failed, allowing the collar to fall. Further, it is our professional opinion that the improper use of the slings resulted in the failure.

As noted in Section 8.4, eight chain blocks were to be used for the installation/erection sequence during the stage of the work in progress, whereas only four slings were used, introducing an immediate reduction of 50% in potential load carrying capacity without any other consideration.

It is noted in the OSHA test results (see Section 8.2) that there was significant variation of the stretch in different polyester web slings during the period during which individual tests were carried out. These could therefore neither be relied on to maintain a vertical position for the collar nor to share the weight of the collar consistently or equally and are therefore inappropriate for consideration as permanent supports. However, at this time it is unknown what, if any, provisions were being provided for replacement of the polyester web slings with a system which would not stretch over time. There is no indication that there were, indeed, any materials at the 18th floor level with which to replace the polyester web slings and, as already noted, there is no provision for replacement of the vertical collar support in the site by Favelle Favco. It is noted, however, that aircraft cable was present on the lower collars (see Section 3.1.4).

As previously noted in Sections 1.2 and 8.4, the wrong lugs were used both by the crane to lift the collar and for the polyester web slings to suspend the collar from the tower. This was quite significant as the points actually used for the slings aligned with the columns, which prevented a proper connection to the horizontal beams.

Due to the use of the incorrect lugs, the positioning of the polyester web slings as installed was inconsistent with the manufacturer's instructions. The polyester web slings were choke-hitched around the crane tower columns and drawn into the V-shaped grooves between the diagonal trussed members and the columns. None of the polyester web slings were protected against the edges by padding or other means which was required by the sling manufacturers as stated on labels affixed to the polyester web slings.

It is again noted that wood packers are also included in the tower crane manufacturer's erection procedures. As discussed in Section 3.1.4, the bunching and lack of padding are both in violation of industry standards.

As stated in Section 8.4, one of the polyester web slings was already frayed and deteriorated before it was used to support the collar. Use of damaged or faded slings is in violation of accepted industry standards.

Test results confirm a significant reduction in the capacity of the polyester web slings as used. Due to the bearing of the polyester web slings against the unprotected/unpadded edges of the leg flanges, the sliding of the slings which occurs under load as the sling "seats" itself in the "V" shaped groove formed by the bracing connection to the leg and the loading configuration by which the polyester web sling is loaded on its edge rather than across its width, the large reduction in capacity is not surprising.

The crane collapse was initiated by the failure of 1 polyester web sling when the workers heard, according to eyewitness account, a popping or snapping sound. Due to the near symmetry of the collar, after the first polyester web sling broke, the polyester web sling diagonally opposite would have carried negligible load. The total load had to be supported by two other diagonal slings. Thus, each of those polyester web slings had to carry half the collar load as well as the impact caused by the dynamic effect of the loading. At this point, those sling loads would have exceeded their capacity causing failure, followed by failure of the remaining sling.

Results of the tests carried out by ATLSS indicated that new polyester web slings rated at 5,000 lb. with a nominal breaking load of 25,000 lb. (based upon an industry standard factor of safety of 5), failed at between approximately 7,000 lbs. and 11,000 lbs. The difference in capacities of the tests resulted from the use of protective packing and the actual location of the plies within the "V" groove. Even under the most favorable test conditions, including provision of edge protection, the capacity of the polyester web slings was less than half of the anticipated capacity, most likely arising from the loading configuration.

As noted in Section 8.4, it is conceivable that ½ of the weight, 5,640 lb., could be carried by one polyester web sling. Without packing, the anticipated breaking capacity of a new polyester web sling, based upon the OSHA test results would be as low as 7100 lb. Such slings exposed to ultraviolet light could show a further reduction in the polyester web sling breaking capacity to as low as approximately 5,680 lb. to 6,390 lb. Additional reductions would be applicable for polyester web slings damaged by other than ultraviolet radiation, such as mechanical damage or prior loading, as well as loading conditions not as favorable as those replicated in the OSHA tests. Based upon these findings and the test results, the capacity of the polyester web sling, as installed, could reasonably be less than the applied loads, based upon static load considerations only. Any dynamic affects arising during installation of the tie-beam, as well as additional load arising from personnel or equipment, would exacerbate this condition making failure of a polyester web sling more likely.

As noted in Section 8.2, “audible tearing” of the tested new polyester web slings commenced at approximately 5,500 lb. indicating the start of sling failure.

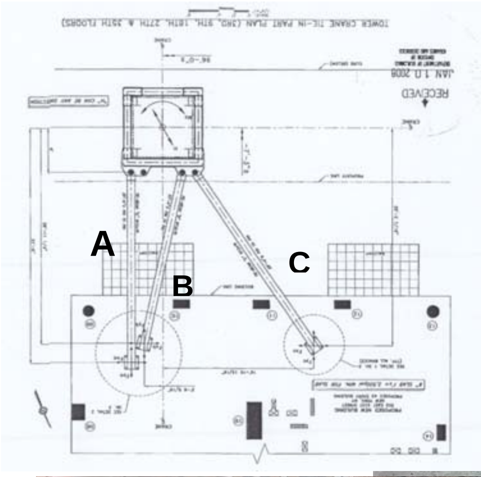
The Metro/Liftex sling was observed to be faded and likely damaged prior to installation. It is therefore probable that the Metro sling, located at the southwest corner of the collar, failed first, as reported by one of the witnesses.

Following failure of one polyester web sling, the combined increased load, dynamic effects and reduction in load carrying capacity of the remaining polyester web slings due to improper rigging practice could result in a general failure of the slings. Dynamic analysis of the collar supported by the four polyester web slings, using the force-deflection data from the OSHA tests, in the configuration as used on the tower crane at the time of collapse, demonstrated the viability of this scenario for the failure of the three slings following the failure of the first. See Section 8.5 for a description of the analyses performed.

11.4 Level 9 Collar Study

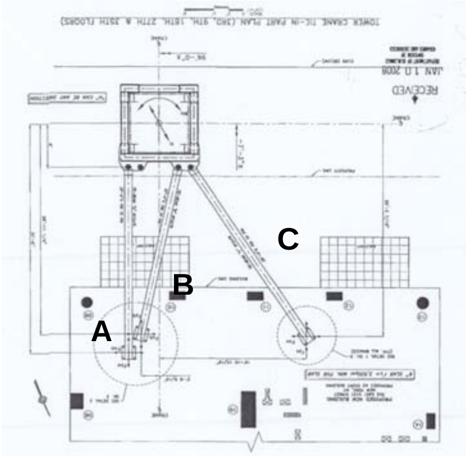
As demonstrated by structural analysis, the unanticipated loads arising from the dynamics of the collar falling from the 18th floor caused the failure of the collar connection at the 9th floor. The precise failure mode proved difficult to replicate analytically. However, even though the analysis over-estimated the strength of some components, it did demonstrate that the falling collar had ample energy to destroy the connection. Photographs 11.1 and 11.2 show the damaged end connections of the tie-beams at the tie-beams at the 9th floor. Photograph 11.1 shows the base plate connections after the incident. The tie-beams, marked 9A and 9C on the photograph, have completely separated from the base plates. Photograph 11.2 shows that all three tie-beams sheared off the collar near pins.

The tie-beam assembly at the 9th floor level was not welded as specified. However, this is not considered to have contributed to the tower collapse. Computer analysis indicates that the welds that connect the tie-in beams to the steel base plates would still have failed if the welds had been properly fabricated per the design. Further, site observations indicated that all three connections of the tie-beams to the collar also failed. Similar observations apply to the pin end fabrication; i.e., the pin end connection are also unable to withstand the impact loads.

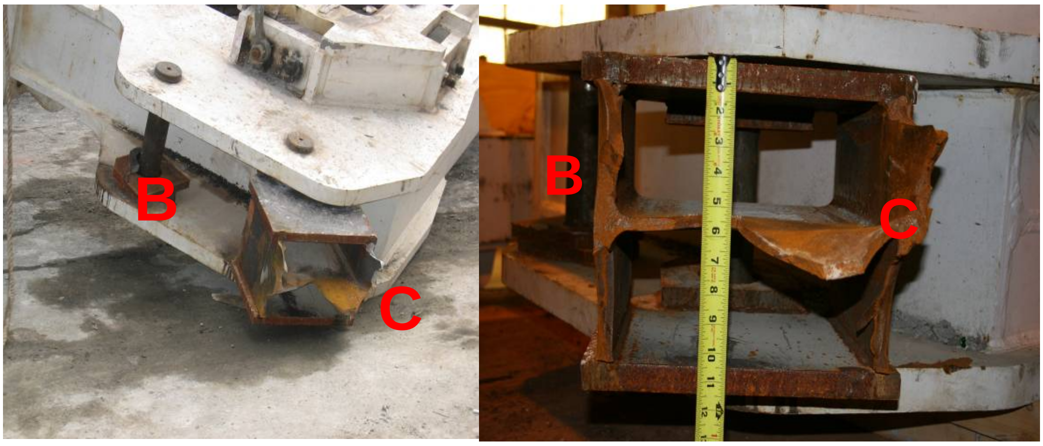


Plan view of typical collar assembly as-designed. Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 Drawing 3 of 4 dated 1/2/08.

Photograph 11.1 - 9th Floor: Failed base plate 9A for Beam B4 (labelled "A" in the photograph) and failed base plate 9C for Beam B5 (labelled "C" in the photograph) prior to its removal from the site (the bent beam in the photograph is Beam 9B). Photos provided by NYCDOB.



Plan view of typical collar assembly as-designed. Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 Drawing 3 of 4 dated 1/2/08.



Photograph 11.2 - 9th Floor: Failed end for Beam B5 (labelled "C" in the photograph) prior to its removal from the site (the pin end of Beam B5 sheared off). Photos by Peter Stroh IMG 0093.jpg and Arup.

11.5 Level 3 Ties Study & Tower Overturning

The loss of the level 9 collar left only the level 3 collar and the base to restrain the crane tower. The base restraint was limited in that resistance to horizontal sliding was provided only by friction and there was no provision to resist uplift.

It is important to note that the base design was in accordance with acceptable engineering practice; however, under the unanticipated and extreme conditions imparted by the falling collars to the tower and foundation support, the ability for the tower legs to lift and the foundation to slide resulted in the mechanism of failure observed. Had the base been fixed and the tower legs restrained against uplift, it is our professional opinion that another mechanism of collapse would have been likely. These have not, however, been studied as part of this investigation.

As demonstrated by structural analysis, the dunnage steel did slip across the foundations at the base of the crane and this resulted in the overturning of the tower. Figure 11.2

shows the slippage of the tower base towards the building and a rotation about the level 3 collar; resulting in the movement of the upper parts of the tower away from the building. As this movement continued, the tower legs at the base of the tower dislocated from the supporting pockets located on the dunnage beams; allowing another type of sliding as the tower legs slid along the dunnage. This is illustrated in Figure 11.3. The dunnage steel eventually slid completely off its foundation wall to the south. Photograph 11.3, taken after the collapse, clearly shows the movement of the dunnage beam and tower base.

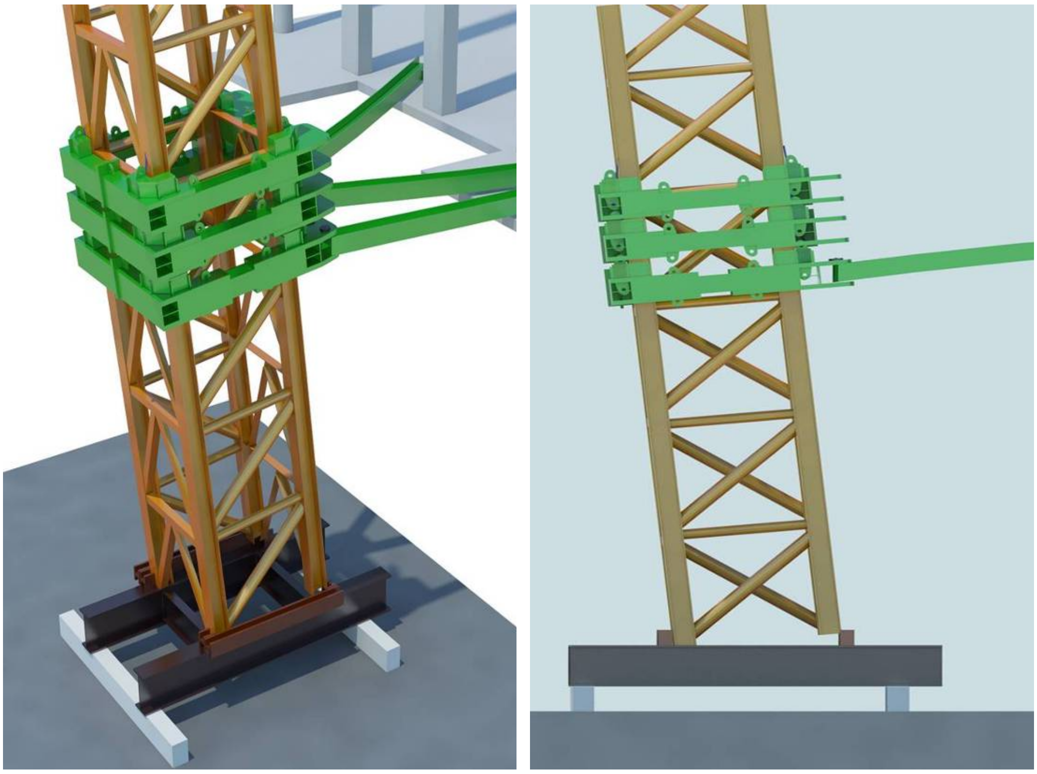


Figure 11.2 - Elevation of the tower at the 3rd floor showing dunnage slippage at the supporting concrete walls and the uplift at the back legs of the tower (initiation of the tower rotation). Prepared by Arup.

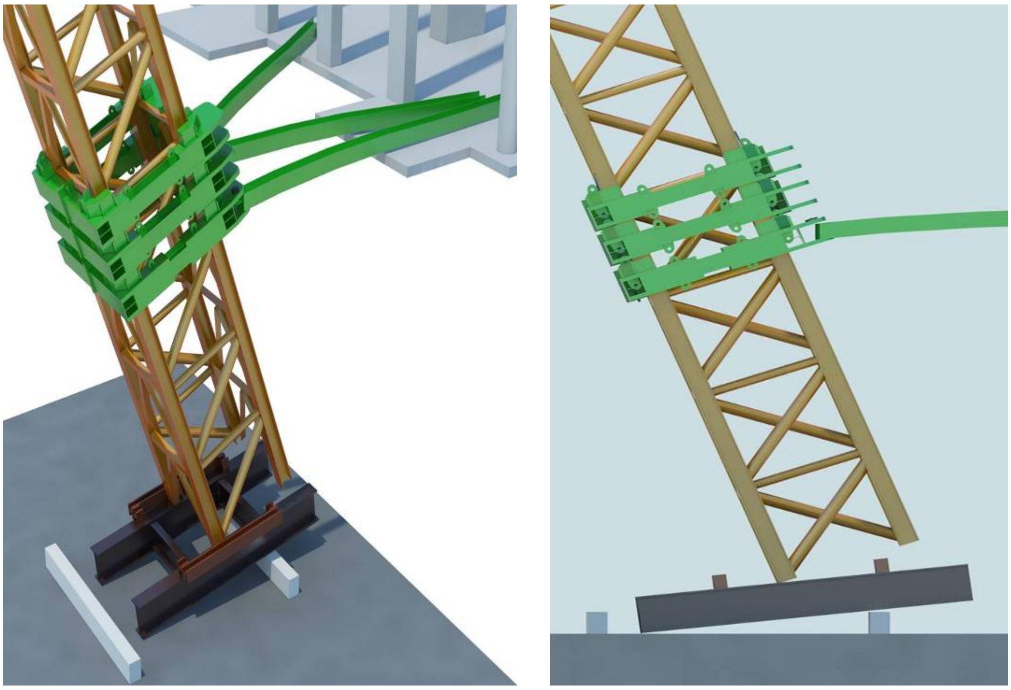


Figure 11.3 - Elevation of the tower at the 3rd floor showing dunnage slippage at the base and the uplift at the back legs of the tower and tower rotation. Prepared by Arup.

Photograph 11.3 - Base of tower crane after collapse. Note the movement of the dunnage beam and displacement of the tower legs from their restraining pockets. Photo from New York County District Attorney IMG_0379_109.jpg.

11.6 Crane Type & Approvals

As a separate task, Arup was requested to review the submittals and applications to the NYCDOB for the crane approvals with regard to the 51st St. tower crane. Our review shows the following:

- The crane is manufactured by Favelle Favco; Model M440E, S/No. 1371 and supplied by "New York Cranes". During construction and at the time of the accident, the crane, installation of the tower collars and associated rigging was under the direction and control of William Rapetti. Rapetti, a tower crane rigger, possessed a valid Tower Crane Rigger's license at the time of the collapse.
- At the time of the accident, the crane had a valid "Certificate of Approval". This represents approval of the prototype number or specific crane model.
- At the time of the accident, the crane did not have a valid "Certificate of Operation". The certificate of operation for the crane, which expired on February 16, 2008, had not yet been renewed. Typically, this certificate should be renewed annually

At the time of the accident, the crane had a valid "Certificate of on-Site Inspection". This type of certificate pertains to the erection of an approved crane at a "specific" job site.

12 Conclusions

Following the collapse of an external self-climbing tower crane at 303 East 51st Street, New York, New York, on March 15, 2008, Ove Arup & Partners, PC was hired by the New York City Department of Buildings to provide engineering and investigative services.

Based upon a review of the tower components, pertinent codes and standards, the OSHA report, computer analyses of the tower structure, photographs and other documentation as described in the foregoing report, the following is our professional opinion to a reasonable degree of engineering certainty.

1) The collapse of the tower was initiated when the four polyester web slings supporting a steel collar at the 18th floor level failed; allowing the collar to fall.
2) Improper usage of the polyester web slings resulted in the failure of the slings.
    a) One of the four polyester web slings was in a deteriorated condition prior to use and should not have been installed. Specifically,
        i) UV degradation was evident on the sling; and,
        ii) physical damage to the sling was evident.
    b) The number of supports provided by the polyester web slings did not meet the tower crane manufacturer's requirements;
        i) Only four support points were used in contrast to the manufacturer recommended specified eight support points.
        ii) With only four slings present, analysis indicates that failure of one sling may have been sufficient to cause failure of the remaining three slings.
    c) The positioning of the polyester web slings as installed was inconsistent with manufacturer's instructions.
    d) The method of attaching the polyester web slings to the tower was not in accordance with accepted industry practice and standards. Specifically,
        i) sharp bearing edges were not padded; and,
        ii) the slings were bunched and edge loaded.
3) As demonstrated by structural analysis, the unanticipated loads arising from the dynamics of the falling 18th floor level collar caused failure of the collar connection at the 9th floor level.
    a) The tie-back system for horizontal restraint had limited vertical capacity
        (1) The system was held up initially by polyester web slings from the collar to the tower and then by cables. In either case, the system had little ductility to resist the effects of falling debris.
        (2) It is noted that this approach is commonly used, and not in violation of any standards to our knowledge.
4) The tie-beam assembly at the 9th floor level was not welded as specified.
    a) Baseplate fillet welds did not meet the design specifications.

(1) Baseplate fillet welds were insufficient to meet the minimum weld size as specified by code.

b) Pin end plate connections did not meet the design specifications.

c) It is noted that computer analysis indicates that the tie-beam welds would still have failed if the welds had been properly fabricated per the design. Further, site observations indicated that all three connections of the tie-beams to the collar also failed.

5) The 3rd and 9th floor wire ropes used to provide vertical support to the collars were not installed in accordance with accepted rigging standards nor in accordance the tower crane manufacturer's requirements.

a) It is noted that structural analysis indicates that the wire rope slings would still have failed if installed per industry standards.

6) As demonstrated by structural analysis, after the loss of the Level 9 collar connection, the remaining supports at level 3 and the base were not sufficient to restrain the tower. The tower rotated about the level 3 collar, with its base sliding towards the building and its top falling away from the building.

7) The dynamics of the 18th and 9th floor level collars impacting the 3rd floor level collar caused slippage of the dunnage steel at the base of the crane resulting in the overturning of the tower.

a) Horizontal sliding of the base was restrained only by friction while uplift of the tower legs was not restrained at all.

i) It is noted that this approach is commonly used, and not in violation of any standards to our knowledge.

8) A review of the crane engineer's submissions (by Stroh Engineering) found that the tower and tower supports were generally well-engineered and designed to industry standards.

9) Permitting procedures were properly followed and completed.

In summary, based upon the information supplied and work completed, it is our professional opinion to a reasonable degree of engineering certainty that the cause of collapse of the tower crane on March 15, 2008 was the failure of the polyester web slings due to improper usage.

These conclusions and recommendations are subject to alteration should additional information be forthcoming, whether as additional documents, document review or site investigation.

Appendix A

HMO & Rigger's License

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Class: B **Status:** A ACTIVE **Expiration:** 08/31/2008 **City** I

Office Address: ,
Business Phone:

Business 1 :

Insurance Type	Required	Company	Expiration [
General Liability	N		
Workers' Compensation	N		
Disability			

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Results by Licensee Name

=== TOWER CRANE RIGGER ====

Licensee	License	Status	Expiration Date	Business 1	Busin
RAPETTI WILLIAM	T000021	EXPIRED	07/31/2008		

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License Details

WILLIAM RAPETTI

Entry Date: 08/12/1999 **Type:** TOWER CRANE RIGGER **License #:** 000021
Class: **Status:** E EXPIRED **Expiration:** 07/31/2008 **City En**
Office Address: ,
Business Phone:
EIN:

Business 1 :

	Insurance Type	Required	Company	Expiration I
General Liability				
Workers' Compensation				
Disability				

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LDINGS

License Application

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Application Must Be Typewritten

Internal Use	
License No.	21
Docket No.	—

1. Application Type

Original ✓ Renewal Change Copy

2. License Number

21

3. License Type

Elevator Inspector	✓ Rigger
Engineer | Stationary | Portable	Sign Hanger
Hoisting Machine Operator Type ______	Site Safety Manager
Master Plumber	Welder
Oil Burning Equipment Installer	Master Fire Suppression Piping Contractor Class ______

4. Application Information

Name William Rapetti [redacted]
Address [redacted]
City [redacted]
State New York [redacted] Zip [redacted] Phone [redacted]
Date of Birth [redacted] SSN [redacted]

5. License Use

✓ Individual
On Behalf of a Corporation
On Behalf of a Partnership/Sole-Proprietorship

6. Business Information

Name

Office	Shop
Address	Address
City	City
State Zip Phone	State Zip Phone

7. Partner or Officer Information

Name	Name
Address	Address
City	City
State Zip Phone	State Zip Phone
License Number	License Number
% Control	% Control
Title	Title
Name	Name
Address	Address
City	City
State Zip Phone	State Zip Phone
License Number	License Number
% Control	% Control
Title	Title

8. Affiliation

This is my only affiliation for this license type:
☑ Yes ☐ No

If No, My Other Affiliation Is With:
Name
Address
City
State Zip Phone

9. Statements and Signatures

As a condition of being granted a License from the Department of Buildings, I hereby comply with all Administrative Code Provisions and Departmental Rules, Regulations and Directives regarding how licensees conduct their specific trade.

Falsification of any statement is a misdemeanor under Section 26-124 of the Administrative Code and is punishable by a fine or imprisonment, or both.

Bribery is a Crime: A person who gives or offers a bribe to any employee of the City of New York or an employee who takes or solicits a bribe, is guilty of a felony punishable by imprisonment or a fine, or both.

Expiration Date ~~07/31/2007~~ 2008

Applicant Name William Rapetti
Signature [signature]
Date 07/01/2007

Internal Use
Clerk's Signature 330
Date 8/8/07
Fee Paid $100
Certificates Issued
Docket Numbers

LIC-2 Revised 5-03

The City of New York
Department of Buildings
Tower/Climber Crane
Rigger License

Issued	9/22/2003
Expires	7/31/2004
License Number	
21	

Commissioner's Signature

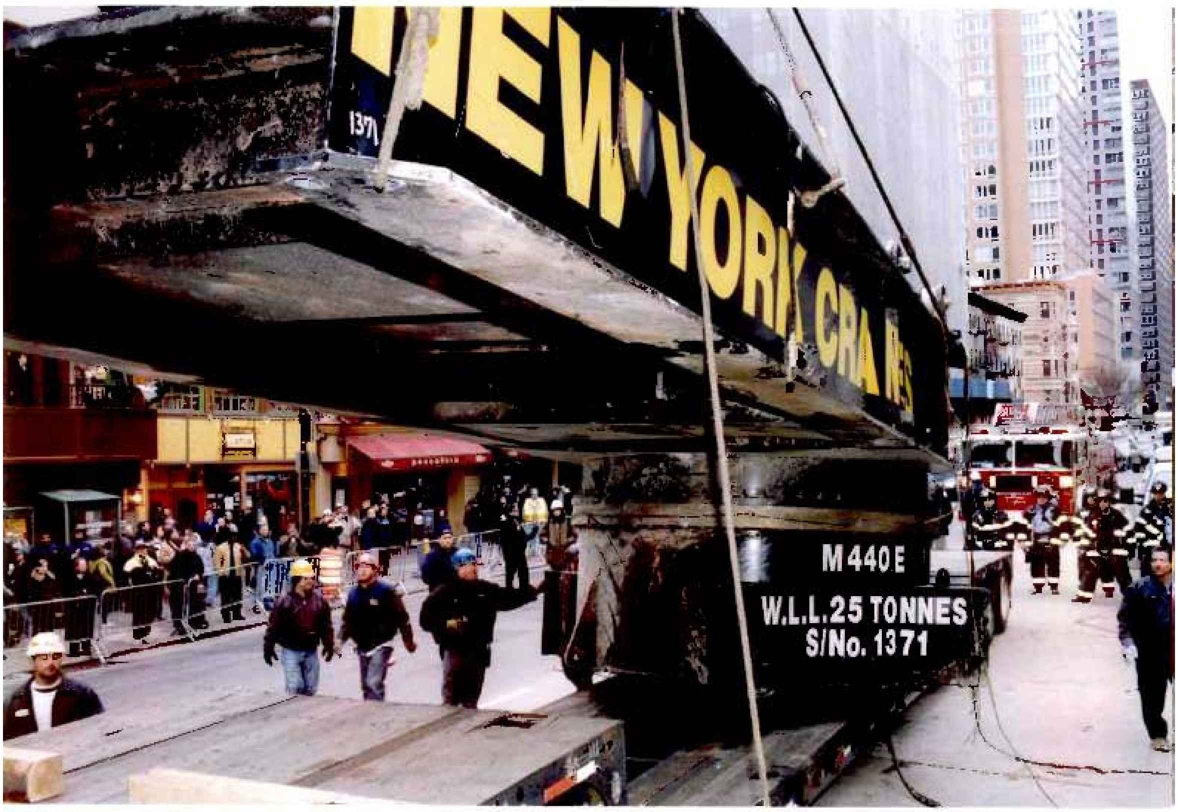
, A.I.A.

Commissioner · LIC 32

Appendix B

Photographs of M440E at Collapse Site- Photographs by the New York City Police Department


M440E
W.L.L.25 TONNES
S/No. 1371

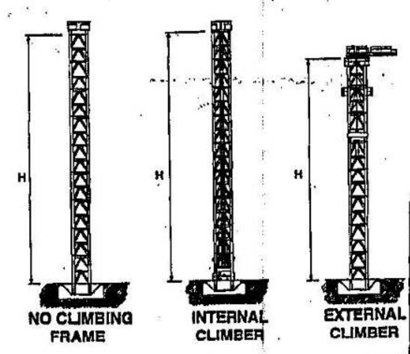

NEW YORK CRANES
1371
M440E
W.L.L.25 TONNES
S/No. 1371

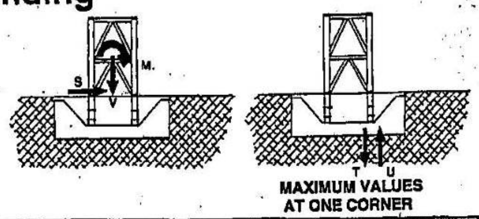
Appendix C

Selected Tower Crane Manufacturer's Data Provided by Favelle Favco for Crane Engineer Peer Review

M440D

Crane on Base – Freestanding



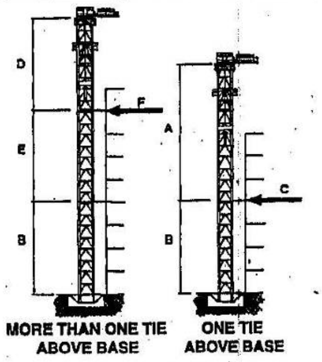


FREESTANDING TOWER HEIGHTS AND FOUNDATION REACTIONS

	NO CLIMBING FRAME			INTERNAL CLIMBING FRAME			EXTERNAL CLIMBING FRAME			UNITS
TOWER	441	442	392	441	442	392	441	442	392	
H	40	48	40	40	48	40	36	44	36	(m)
M	885	1122	877	885	1122	877	930	1191	923	(mT)
V	176	188	178	191	203	193	187	199	189	(T)
S	26	29	25	28	29	25	29	33	29	(T)
T	301	389	341	305	373	345	312	392	351	(T)
U	225	292	261	222	288	257	236	310	273	(T)

In-service wind speeds up to 20 m/s & out-of-service wind speeds up to 42 m/s. Alternatives available upon request. For long (>55m) booms, heights shown are only attainable with extra counterweight.

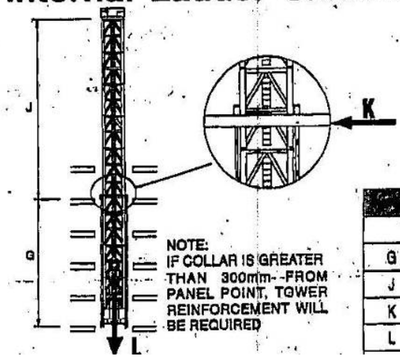
External Climbing

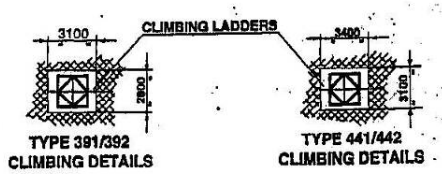


EXTERNAL CLIMBING

	TOWER TYPES:	441	442	392	UNITS
	ONE TIE ABOVE BASE				=105
A	MAX. HEIGHT ABOVE FIRST TIE	32	40	32	(m)
B	MIN. HEIGHT TO FIRST TIE	16	20	20	(m)
C	MAX. HORIZONTAL TIE LOAD	104	111	88	(T)
-	MORE THAN ONE TIE BASE				
D	MAX. HEIGHT ABOVE TOP TIE	32	40	32	(m)
E	MIN. HEIGHT BETWEEN TOP TIES	16	20	16	(m)
F	MAX. HORIZONTAL TIE LOAD	93	99	92	(T)

Internal Ladder Climbing





INTERNAL LADDER CLIMBING

	TOWER TYPES	391	392	441	442	UNITS
G	MIN. DISTANCE BETWEEN COLLARS	10.5	12	12	12	(m)
J	MAX. DISTANCE ABOVE TOP TIE	33.5	28	28	38	(m)
K	MAXIMUM HORIZONTAL LOAD	98.0	71	71	90	(T)
L	TOTAL VERTICAL LOAD	156.0	193	191	203	(T)

Table from the crane manufacturer's catalogue (Favelle Favco)

TOWER CRANE CLIMBING & TIE-IN ANALYSIS

The following initial information shall be obtained from the Specific Tower Crane Operations Manual and from the Owner of the tower crane:

Project name ______________________________
Address ______________________________

Tower Crane Lessee company ______________________________
Contact person ______________________________
FAX # ______________________________
Office Phone # ______________________________
Cell Phone # ______________________________

Tower Crane Manufacturer ______________________ Jib Length ______________
Model # ______________________ S/N (if known) ______________
Tower type ______________________ Height of standard tower ______________

Operations Manual important information:
Foundation Reactions: (for Max. free-standing above base)
Mx (in operation) ______________________
Mx (storm) ______________________
Horiz. at base (in oper.) ______________________
Horiz. at base (storm) ______________________
Slewing Moment (in operation) ______________________
Maximum free-standing above foundation ______________________
Maximum height allowed above a secured top tie-in ____________
Maximum distance allowed between tie-ins ______________________
Height of climbing frame + retracted ram ______________________
Minimum distance allowed between tie-ins ______________________

Date this form completed ______________ By: ______________________
Cell Phone # ______________________

Form by the NYCDOB.

Appendix D

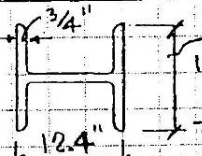
Reviewer’s Independent Calculations for the Tower Crane On-Site Design

CURTIS ENGINEERING CORP.
Engineers Since 1973
SAN DIEGO and NATIONAL CITY
(619) 265-0700
FAX (619) 265-1954

JOB FAVELLE FAVCO TOWER CRANE
SHEET NO. FOUR OF FOUR
CALCULATED BY D.H. CURTIS DATE AUG. 9, 2008
CHECKED BY DALE CURTIS DATE AUG. 9, 2008
SCALE NOT TO SCALE. 2 UPDATE 10-9-08

CHECK TIE-IN STRUTS ALL SAME SIZE

W 12×79
ASTM A36



$\frac{d}{A_f} = 1.39$ $r_T = 3.31$

$\frac{b_f}{2t_f} = 8.2 < 10.8$

for: I $S_x = 107 \text{ in}^3$ $S_y = 35.8 \text{ in}^3$ $r_y = 3.05$

Max $l = 28'$ ± $A = 23.2 \text{ in}^2$ $r_T = 3.31$ $65/\sqrt{F_y} = 10.8$ & 9.19

Max COMP LOAD = 211,973# & FOR ASD LOAD CONDITION D+L+0.7W

$\frac{kl}{r_y} = \frac{1.0 \times 28 \times 12}{3.05} = 110.2$ for k = 1.0 if both Ends pinned

$\frac{kl}{r_y} = \frac{0.8 \times 28 \times 12}{3.05} = 88.1$ for k = 0.8 for 1 end fixed - this project

$F_e' = 19.28$ for $F_y = 36$

For $F_y = 36$ ksi Mat'l, $F_a = 14.32$ for 88.1 & CHECK COMBINED STRESSES:

For Grade 50 Mat'l $F_a = 17.15$ ksi $F_b = 0.75 F_y = 0.75 \times 36 = 27$ ksi

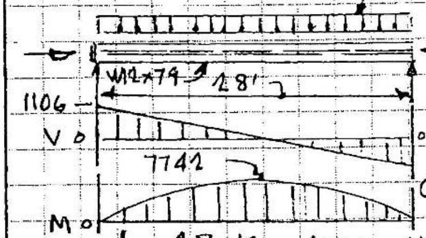
$f_a = \frac{0.7 \times 211{,}973\#}{23.2 \text{ in}^2} = 6396$ psi

check this value.

& $f_a = \frac{211{,}973}{23.2} = 9137$ psi $\frac{f_a}{F_a} = \frac{9137}{14{,}320} = 0.6380 > 0.15$

W = 79 PLF

CHECK SIMPLY SUPPORTED



$M_{max} = \frac{1}{2} \times 14 \times 1106 = 7742'\#$

$f_{b_y} = \frac{M}{S_y} = \frac{7742 \times 12''/'}{35.8} = 2595$ psi

& WIND = $\frac{24 \text{ PSF}}{79} = .3038$

f_{b_x} SIDE WIND $= \frac{0.3038 \times 7742 \times 12''/'}{107} = 264$ psi (SIDE WIND)

$F_{b_x} = 0.6 \times 36 = 21{,}600$ 2

$\frac{l}{r_T} = \frac{28 \times 12}{3.31} = 101.5 < \sqrt{\frac{510 \times 10^3 C_b}{F_y = 36}} = 119$ ok ∴ Check $F_{b_y} = 27$ ksi

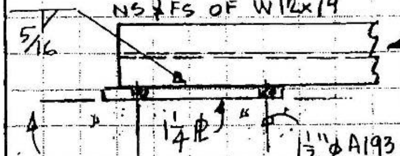
& check $C_m = 1.0$

COMBINED STRESS ANALYSIS

$$\frac{f_a}{F_a} + \frac{C_m f_{b_x}}{\left(1 - \frac{f_a}{F_e'}\right) F_{b_x}} + \frac{C_m f_{b_y}}{\left(1 - \frac{f_a}{F_e'}\right) F_{b_y}} = 0.6380 + \frac{1 \times 264}{\left(1 - \frac{9137}{19280}\right) 21600} + \frac{1 \times 2595}{\left(1 - \frac{9137}{19280}\right) 27{,}000} \le 1.0$$

check $F_y = 36$ for ASTM A36: Combined $= 0.6380 + 0.0232 + 0.1827 = 0.8439 < 1.0$ OK

∴ W 12×79 IS SUFFICIENT ← ALL STRUTS.



MAX COMPRESSION LOAD = 211,973#

FOR WIND 211,973 × .7 = 148,381#

Allow Weld = 928#/1/16 × 5/16 × 21" × 2 SIDES = 194,880#

E7018LH 194,880 > 148,381#

∴ WELDS AT BOTH ENDS OKAY FOR LOADS

$F_y = 81$, Check $F_v = 21$ ksi on Gross Area $21^{\text{ksi}} \times 1.767 \text{ in}^2 = 37{,}107\#$ 2

Allow = 37.1 × 4 bolts = 148,400# close.

000010

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Engineers Since 1973
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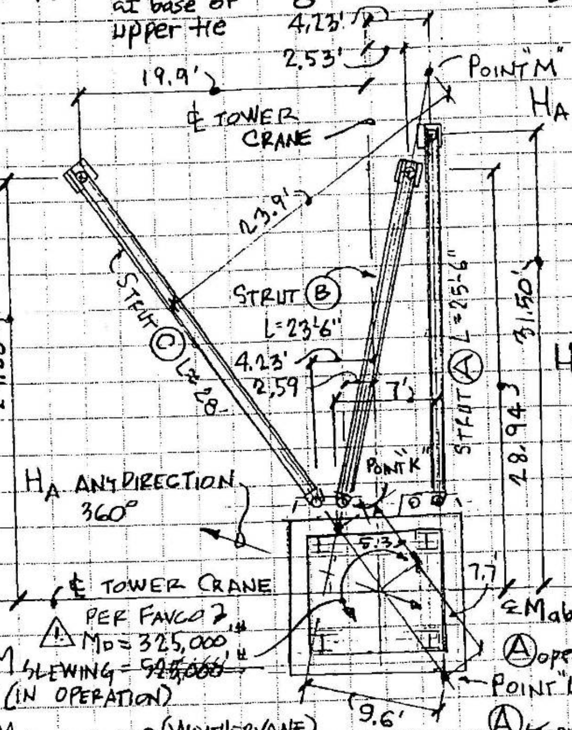
FAX (619) 265-1954

JOB FAVELLE FAVCO TOWER CRANE
SHEET NO. THREE OF FOUR
CALCULATED BY D.H. CURTIS DATE AUG 9, 2008
CHECKED BY DALE CURTIS DATE AUG. 9, 2008
SCALE 1" = 10' REDUCED FOR FAXING 1 REV 9-1-08

ANALYSIS Continued -- FOR FORCES INTO STRUTS (A) (B) & (C):
Initial analysis using Reactions for 45 mph & 95 mph.
Adjust for approved wind velocities of 30 mph & 90 mph:
Format used by most European Tower Crane Manufacturers:

$H_A = H_{orig.} + \frac{3}{8} q\, h(\text{backspan}) + \frac{3M_x}{2h}$ & Calculate for both In-operation wind and Storm Wind:

at base or upper tie



h back-span = 98' (Upper tie-ins)

$H_{A\,in\,oper} = H + \frac{3qh}{8} + \frac{3M_x}{2h}$

$= 28{,}407 + \frac{3 \times 24.4 \times 98}{8} + \frac{3 \times 2{,}639{,}476}{2 \times 98}$

$= 28{,}407^{\#} + 897^{\#} + 40{,}400^{\#}$

$H_{A\,in\,oper} = 69{,}704^{\#}$ ✓

(Compare to FAVCO's 72,000#)

$H_{A\,STORM} = H + \frac{3qh}{8} + \frac{3 \times M_{x\,STORM}}{2h}$

$= 51{,}045 + \frac{3 \times 199.2 \times 98}{8} + \frac{3 \times 5{,}330{,}134}{2 \times 98}$

$= 51{,}045 + 7{,}320 + 81{,}584^{\#}$ ✓

$H_{A\,STORM} = 139{,}949^{\#}$

(Compare to FAVCO's 106,000#)

PER FAVCO 1
1 $M_D = 325{,}000^{\#}$
$M_{SLEWING} =$ ~~525,000~~$^{\#}$
(IN OPERATION)

$M_{SLEWING} = 0$ (WEATHERVANE)
(STORM)

ΣM abt Point "K" FOR STRUT (A) FORCES:

$(A)_{oper} = \frac{325{,}000 + 69{,}704 \times 5.3}{7'} = 99{,}204^{\#}$ 1 ✓

$(A)_{STORM} = \frac{0 + 139{,}949 \times 5.3'}{7'} = 105{,}962^{\#}$ ✓

ΣM abt Pt. "L" FOR STRUT (B) FORCES:

$(B)_{OPER} = \frac{325{,}000^{\#} + 7.7' \times 69{,}704}{9.6'} = 89{,}763^{\#}$ 1 ✓

$(B)_{STORM} = \frac{0 + 139{,}949 \times 7.7}{9.6'} = 112{,}251^{\#}$

ΣM abt PT. "M" FOR STRUT (C) FORCES:

$(C)_{OPER} = \frac{325{,}000 + 36.2 \times 69{,}704}{23.9} = 119{,}175^{\#}$ 1 ✓

$(C)_{STORM} = \frac{0 + 139{,}949 \times 36.2}{23.9} = 211{,}973^{\#}$ ✓

RECAP:

STRUT A_{OPER} = 99,204#	STRUT A_{STORM} = 105,962#	FAVCO A_{OPER} = 51K	FAVCO A_{STORM} = 75K
STRUT B_{OPER} = 89,763#	STRUT B_{STORM} = 112,251#	FAVCO B_{OPER} = 60K	FAVCO B_{STORM} = 87K
STRUT C_{OPER} = 119,175#	STRUT C_{STORM} = 211,973#	FAVCO C_{OPER} = 110K	FAVCO C_{STORM} = 160K
$M_{SLEWING}$ = 325,000# included. 1		FAVCO M_{SLEW} = Not seen in their calcs!	

000011

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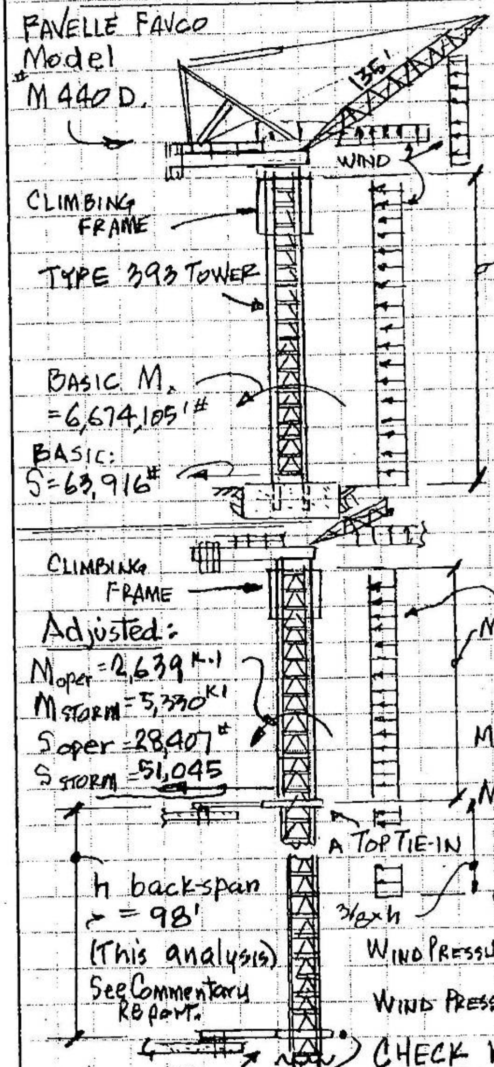
FAX (619) 265-1954

JOB FAVELLE FAVCO TOWER CRANE
SHEET NO. TWO OF FOUR
CALCULATED BY D.H. CURTIS DATE AUG. 8, 2008
CHECKED BY DALE CURTIS DATE AUG. 8, 2008
SCALE NOT TO SCALE

ANALYSIS OF FORCES ON TOWER CRANE – Continued

FAVELLE FAVCO Model M 440 D.



CHECK TO SEE HOW FAVELLE FAVCO OBTAINED THEIR H_A VALUES:

FROM CRANE OPERATIONS MANUAL:
(COPY OF DATA SHEET ATTACHED IN APPENDIX #2) IMPORTANT INFO:
MAX FREE-STANDING = 36 meters = 118'
IN OPERATION WIND = 20 m/sec = 45 MPH
STORM WIND = 42 m/sec. = 95 MPH
HORIZ. S AT BASE = 63,916# (29T)
$Max M_{118'}$ = 923 mT = 6,674,105'#
MAX DISTANCE ABOVE TOP TIE = 32 meter = 105'

THIS INFO IS GIVEN FOR TOWER TYPE 392 WHICH IS BEST AVAILABLE FOR TYPE 393

CITY OF NEW YORK MAX IN SERVICE WIND = 30 MPH
MAX HEIGHT ABOVE TIE-IN = 32 m = 105' FOR WIND = 45 mph & 95 MPH.

ADJUST BASIC M FOR 105' & 30 mph & 90 MPH

Adjusted:
$M_{oper} = 2{,}639^{K \cdot 1}$
$M_{storm} = 5{,}330^{K1}$
$S_{oper} = 28{,}407^{\#}$
$S_{storm} = 51{,}045$

$$M_{OPER} = 6{,}674{,}105 \times \frac{30^2}{45^2} \times \frac{105}{118'} = 2{,}639{,}476 \text{ ft.}^{\#}$$

$$M_{STORM} = 6{,}674{,}105 \times \frac{90^2}{95^2} \times \frac{105'}{118} = 5{,}330{,}134'^{\#}$$

$$S_{OPER} = 63{,}916^{\#} \times \frac{30^2}{45^2} = 28{,}407^{\#} = H_{oper}$$

$$S_{STORM} = 63{,}916 \times \frac{90^2}{95^2} \times \frac{105}{118} = 51{,}045^{\#} = H_{storm}$$

WIND PRESSURE @ 30 MPH: $q = 55^{\#}/_{1} \times \frac{30^2}{45^2} = 24.4^{\#}/_{1}$

WIND PRESSURE @ 90 MPH: $q = 222^{\#}/_{1} \times \frac{90^2}{95^2} = 199.2^{\#}/_{1}$

CHECK HOW FAVCO OBTAINED THEIR VALUES:
ΣM abt. Lower tie-in (FOR HORIZ H_A at Upper Tie-in)

30 MPH: $H_{A\,oper} = \frac{2{,}639{,}476'^{\#}}{98'} + 28{,}407^{\#} = 26{,}933 + 28{,}407 = 55{,}340^{\#} < 72{,}000^{\#}$

90 MPH: $H_{A\,STORM} = \frac{5{,}330{,}134'^{\#}}{98'} + 51{,}045^{\#} = 54{,}389^{\#} + 51{,}045 = 105{,}434^{\#} \approx 106{,}000^{\#}$

FAVELLE FAVCO COMPUTER VALUES

FOR 90 mph Wind:
Adjust Height above Top tie-in: $105' \times \frac{95^2}{90^2} = 117'$ continued for analysis ....

PRODUCT 204-1 (Single Sheets) 705-1 (Padded) NEBS Inc., Groton, Mass. 01471. To Order PHONE TOLL FREE 1-800-225-6380

000012

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Engineers Since 1973
SAN DIEGO and NATIONAL CITY
(619) 265-0700
FAX (619) 265-1954

JOB FAVELLE FAVCO TOWER CRANE
SHEET NO. ONE OF FOUR
CALCULATED BY D.H. CURTIS DATE AUG. 7, 2008
CHECKED BY DALE CURTIS DATE AUG. 7, 2008
SCALE 1" = 10' REDUCED FOR FAXING

ANALYSIS FOR MAX H_A (ANY DIRECTION) AT TIE-IN STRUTS BACK-SPAN = 98'

$M_S = M_D = 525^{k'} = 525{,}000'^{\#}$ IN SERVICE

$H_{A\,op} = H + \frac{3qh}{8} + \frac{3M_x}{2h} = 55{,}100 + \frac{3}{8} \times 24.4 \times 98 + \frac{3 \times 6341.485}{2 \times 98}$

INITIALLY w/o CLIMBER $H_{A\,op} = 153{,}060^{\#}$

$H_{A\,sto} = 63{,}916 + \frac{3}{8} \times 222 \times 98 + \frac{3 \times 6{,}674{,}105}{2 \times 98}$

$H_{A\,storm} = 174{,}229^{\#}$

FAVELLE FAVCO M440 LOADS
REF: DATA SHEET APPEND. #2

$M_{x\,\text{No Climber}} = 877\,mT = 6{,}341{,}485'^{\#}$

$M_{x\,storm} = 923\,mT = 6{,}674{,}105'^{\#}$

$S_{\text{No Climber}} = 25T = 55{,}100^{\#} = H_{op}$

$S_{storm} = 29T = 63{,}916^{\#} = H_{sto}$

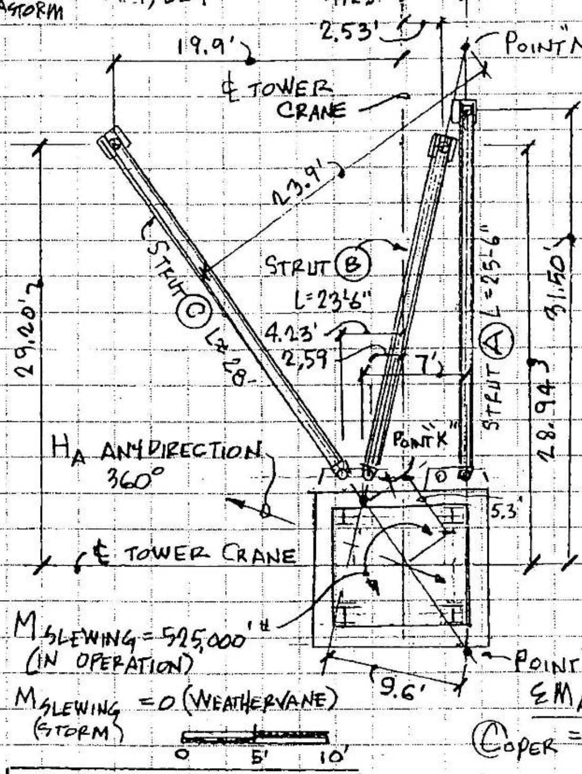
Above for Winds 45 mph + 95 mph.
Per FAVCO Calc. Sheets:

$q_{\text{in serv } 45\,mph} = 55\,^{\#}/_1$

$q_{\text{storm } 95\,mph} = 222\,^{\#}/_1$

For 30 mph in SERV. (PER CHAPTER 1 OF TITLE 27):

$55\,^{\#}/_1 \times \frac{30^2}{45^2} = 24.4\,^{\#}/_1$



$\Sigma M_{\text{ABT POINT "K"}}$ FOR STRUT (A) FORCES

$A_{oper} = \frac{525{,}000 + 153{,}060^{\#} \times 5.3'}{7'} = 190{,}888^{\#}$

$A_{storm} = \frac{0 + 174{,}229 \times 5.3}{7'} = 131{,}915^{\#}$

$\Sigma M_{\text{ABT POINT "L"}}$ FOR STRUT (B) FORCES

$B_{oper} = \frac{525{,}000 + 153{,}060 \times 7.7}{9.6'} = 177{,}454^{\#}$

$B_{storm} = \frac{0 + 174{,}229 \times 7.7}{9.6} = 139{,}746^{\#}$

$\Sigma M_{\text{ABT POINT "M"}}$ FOR STRUT (C) FORCES:

$C_{oper} = \frac{525{,}000 + 153{,}060 \times 36.2}{23.9'} = 253{,}798^{\#}$

$C_{storm} = \frac{0 + 174{,}229 \times 36.2}{23.9} = 263{,}895^{\#}$

$\boxed{\text{INITIAL CALCULATIONS}}$
(See Sht. 3 for final)

Continued......

COMPARE ABOVE TO MAX STRUT FORCES FROM FAVELLE FAVCO COMPUTER:

PRELIM. CURTIS CALCULATIONS	[SHOWN ON STROH DRAWING 3 OF 4] OPER. WIND = 30 MPH, STORM WIND = 95 MPH
$H_{A\,oper} = 153{,}060^{\#}$; $H_{A\,storm} = 174{,}229^{\#}$	$H_{A\,oper} = 72{,}000^{\#}$; $H_{A\,storm} = 106{,}000^{\#}$
STRUT $A_{oper} = 191^K$ STRUT $A_{storm} = 132^K$	STRUT $A_{oper} = 51^K$ STRUT $A_{storm} = 75^K$
STRUT $B_{oper} = 177^K$ STRUT $B_{storm} = 140^K$	STRUT $B_{oper} = 60^K$ STRUT $B_{storm} = 87^K$
STRUT $C_{oper} = 254^K$ STRUT $C_{storm} = 264^K$	STRUT $C_{oper} = 110^K$ STRUT $C_{storm} = 160^K$
$M_{\text{SLEWING IN OPER.}} = 525{,}000'^{\#}$	$M_{\text{SLEWING OPER.}}$ NOT SEEN IN CALCS!

000013

Appendix E

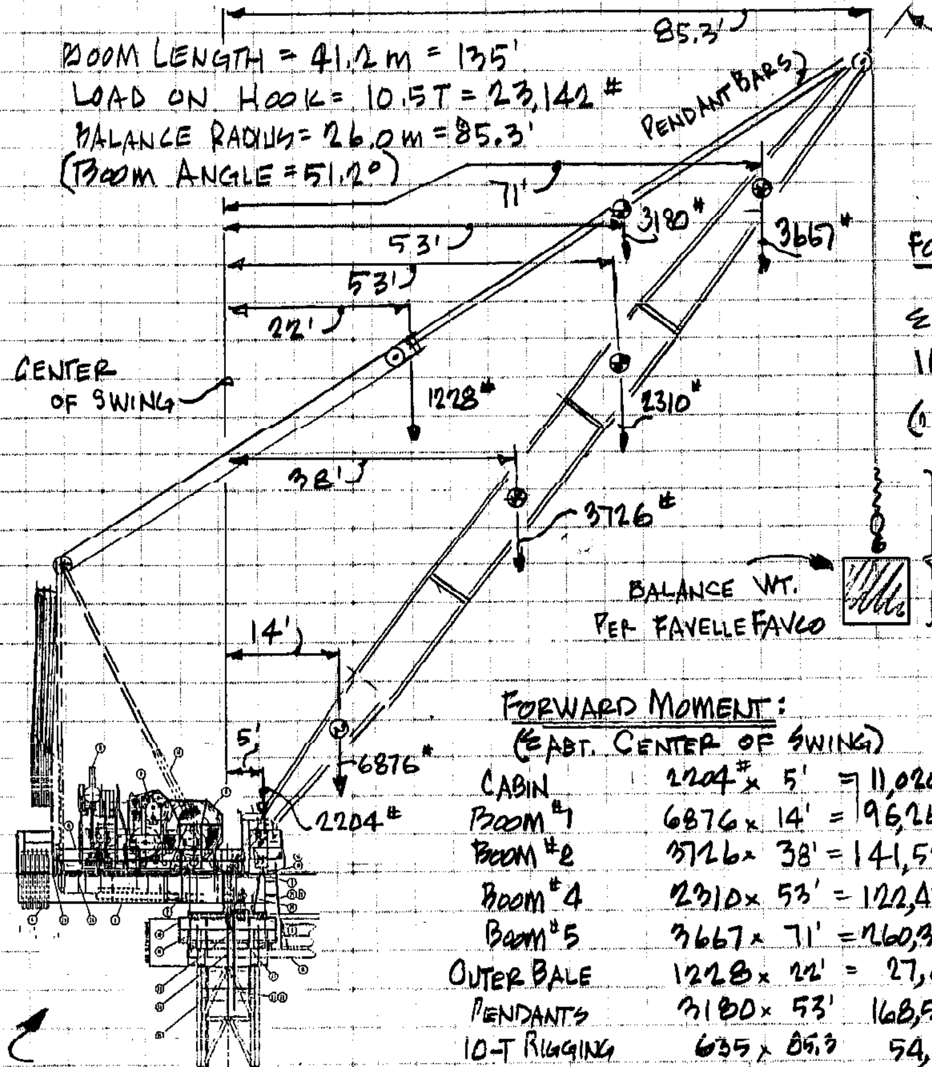
Overturning Moment Calculation for M440E Model Crane

CURTIS ENGINEERING CORP.
Engineers Since 1973
SAN DIEGO and NATIONAL CITY
(619) 265-0700
FAX (619) 265-1954

JOB EAST 51st STREET CRANE COLLAPSE
SHEET NO. 1 OF 3
CALCULATED BY DALE CURTIS DATE SEP. 6, 08
CHECKED BY DALE CURTIS DATE SEP. 9, 08
SCALE 1" = 20'

CALCULATIONS TO CHECK BALANCE POINT
AS SHOWN IN FAVELLE FAVCO OPERATIONS MANUAL
~~FOR MODEL M440E SN 1371 - March 07 Rev A page 7-37/37:~~

BOOM LENGTH = 41.2 m = 135'
LOAD ON HOOK = 10.5 T = 23,142 #
BALANCE RADIUS = 26.0 m = 85.3'
(BOOM ANGLE = 51.2°)



CALCULATE FORWARD MOMENT:

Σ BOOM WT = 16,574 # TO 17,983 #
(10.5-T RIGGING = 1999 #)

10-TON RIGGING 635 #
MISC SLINGS 635 #
10.5 T = 23,142 #
SUM = 23,777 # @ 85.3'

FORWARD MOMENT:
(Σ ABT. CENTER OF SWING) Σ WEIGHTS = 47,603 #

CABIN	2204 # × 5'	= 11,020 '#
BOOM #1	6876 × 14'	= 96,264
BOOM #2	3726 × 38'	= 141,588
BOOM #4	2310 × 53'	= 122,430
BOOM #5	3667 × 71'	= 260,357
OUTER BALE	1228 × 22'	= 27,016
PENDANTS	3180 × 53'	168,540
10-T RIGGING	635 × 85.3	54,166
10.5T BALANCE WT:	23,777 × 85.3	2,027,752
	SUM	= 2,909,559 ft.#

IF FAVCO HAS USED 25-T HOOK RIGGING IN THEIR BALANCE POINT DETERMINATION, THEN 1999 # - 635 = 1364 #:
THE FORWARD MOMENT = 2,909,559 + 1364 × 85.3
= 3,025,911 ft.lb.

SCALE 1" = 20'
(REDUCED FOR FAXING)

CURTIS ENGINEERING CORP.
Engineers Since 1973
SAN DIEGO and NATIONAL CITY
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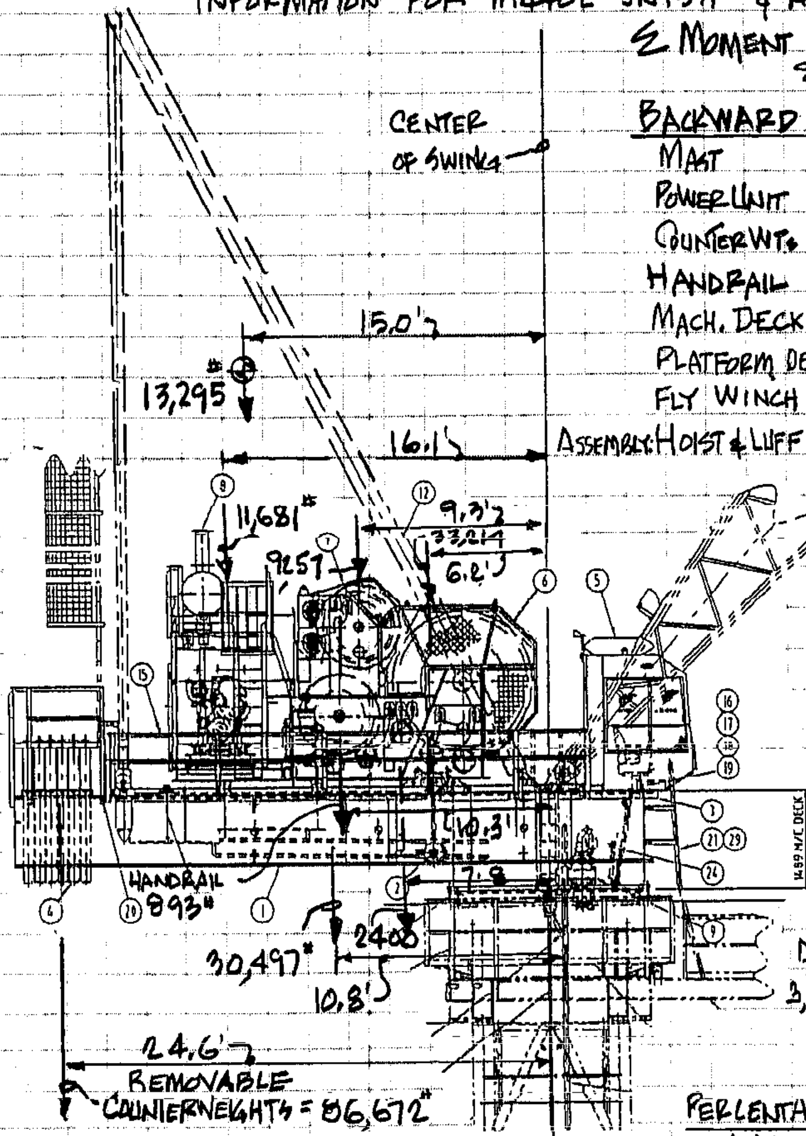
JOB EAST 51ST ST. CRANE COLLAPSE
SHEET NO. 2 OF 3
CALCULATED BY DALE CURTIS DATE SEP. 6, 2008
CHECKED BY DALE CURTIS DATE SEP. 9, 08
SCALE 1/8"=1'-0"

CALCULATE BACKWARDS MOMENT

ALL WEIGHTS & DIMENSIONS OBTAINED FROM BEST AVAILABLE INFORMATION FOR M440E SN1371 & AS SUCH, THEY MAY VARY. (TYP.)

Σ MOMENT ABOUT CENTER OF SWING:

$\Sigma F_{VERT} = 187{,}909$ #



BACKWARD MOMENT:

MAST	13,295# × 15' =	199,425'#
POWER UNIT	11,681 × 16.1 =	188,064
COUNTER WTS	86,672 × 24.6' =	2,132,131
HANDRAIL	893 × 10.3 =	9,198
MACH. DECK	30,497 × 10.8 =	329,368
PLATFORM DECK	2400 × 7.8 =	18,720
FLY WINCH	9257 × 9.3 =	86,090
ASSEMBLY: HOIST & LUFF	33,214 × 6.2 =	205,926'#

SUM = 3,168,922'#

CALCULATED BACKWARDS

& COMPARE "BALANCE" CONDITION PER FAVCO ~~85.3' RADIUS~~;
PER PREVIOUS PAGE:
FORWARD M = 3,025,911'#

DIFFERENCE:
3,168,922 − 3,025,911
= 143,011 Ft-lbs

PERCENTAGE DIFFERENCE:

$\frac{143{,}011'\#}{3{,}168{,}922} = 0.0452 = 4.5\%$

FAVELLE FAVCO
Model M440E
SN 1371
SCALE 1/8" = 1'-0"
(REDUCED FOR FAXING)

4.5% IS FAIRLY CLOSE & ∴ BACKWARD MOMENT OF 3,168,922'# CAN BE USED FOR BOOMED-UP ANALYSIS .....

Continued...

PRODUCT 204-1 (Single Sheets) 205-1 (Padded) NEBS Inc., Groton, Mass. 01471. To Order PHONE TOLL FREE 1-800-225-6380

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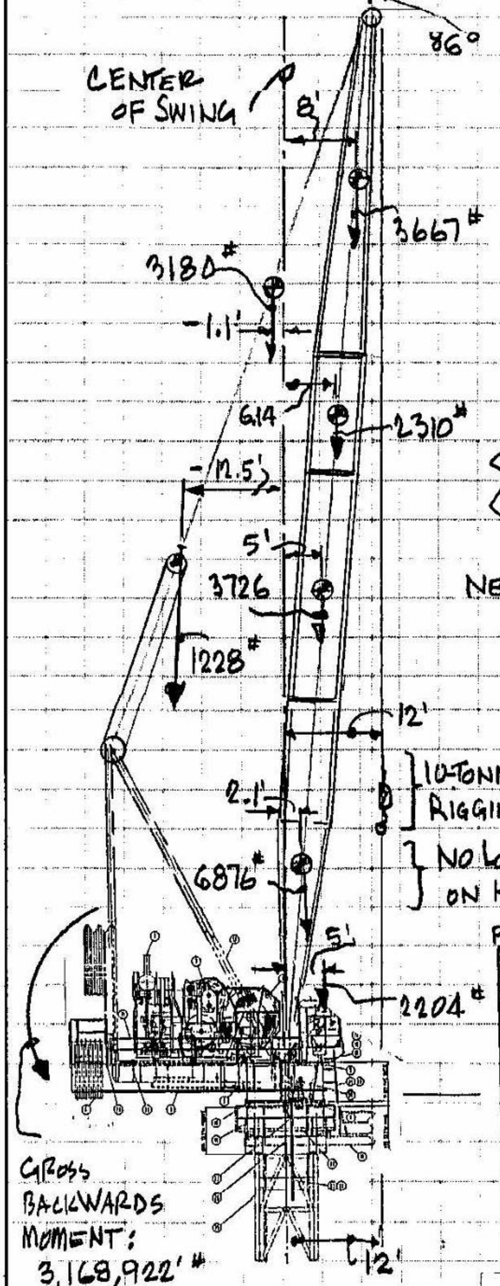
JOB EAST 51st STREET CRANE
SHEET NO. 3 OF 3
CALCULATED BY DALE CURTIS DATE SEP. 6, 2008
CHECKED BY DALE CURTIS DATE SEP. 10, 2008
SCALE 1" = 20' 0 — 20'

CALCULATE NET BACKWARD MOMENT WITH BOOM IN MAXIMUM UP POSITION
AT 12' RADIUS WITH NO LOAD ON HOOK
BOOM L = 135'



MODEL M440E
S/N 1371

FORWARD MOMENT:

ΣM about Center of Swing: ↻+

CABIN	2204 × 5 = 11,020 '#
Boom #1	6876 × 2.1 = 14,440
Boom #2	3726 × 5 = 18,630
Boom #4	2310 × 6.14 = 14,183
Boom #5	3667 × 8 = 29,336
< Less Outer Bale	1228 × 12.5 = −15,350 >
< Less Pendants	3180 × −1.1 = −3,498 >
+ 10-T Rigging	635 × 12 = 7,620

NET ↻ FORWARD MOMENT = 76,381 ft.#

THIS MOMENT IS FOR ALL LOADS ABOVE THE SLEWING RING.

PER THIS ANALYSIS:
NET BACKWARDS MOMENT:
3,168,922'# − 76,381 = 3,092,541'#
ANS

PΔ: A 4' LEAN WILL INCREASE M BY 211,735# × 4' = 846,940'#

FOR COMPARISON ONLY:
30 MPH WIND IN OPERATION:
FAVELLO FAVCO
$M_{x_{OPER}}$ TO TOWER = 2,639,476 ft.#

HOWEVER: TOWER HAS BEEN ENGINEERED BY MANUFACTURER FOR M_{STORM} = 5,330,134 ft.#
(before p-Δ & p-δ effects)

Appendix F

Reference Drawings for Dynamic Analysis

F1 Drawings

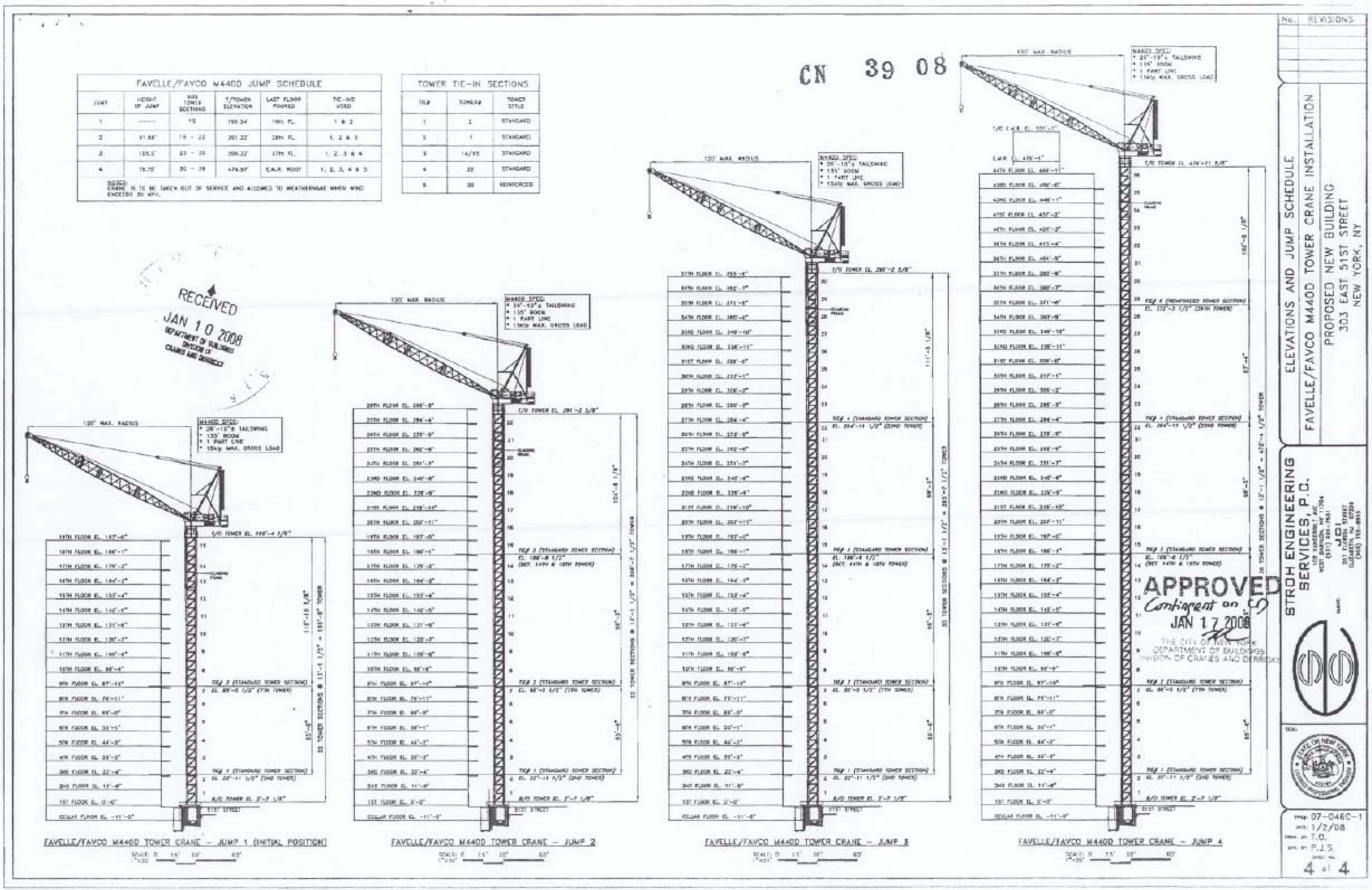


Figure F1.1 - Tower Crane Elevations and Jump Schedule. Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 sheet 4 of 4 dated 1/2/08.

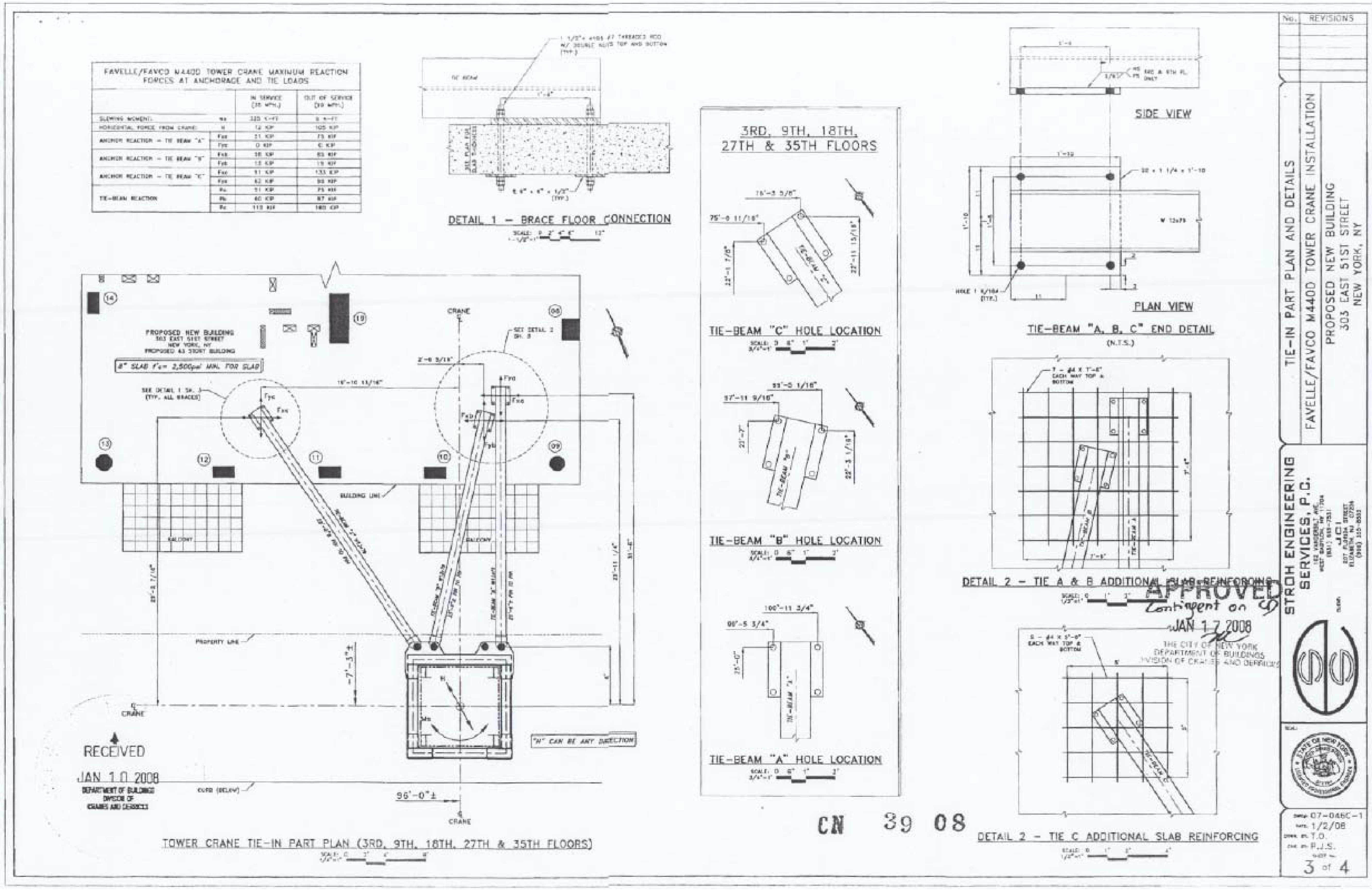


Figure F1.2 - Collar and Tie-Beam Part Plan and Details. Provided by the crane engineer Stroh Engineering, drawing 07-046C-1 sheet 3 of 4 dated 1/2/08.

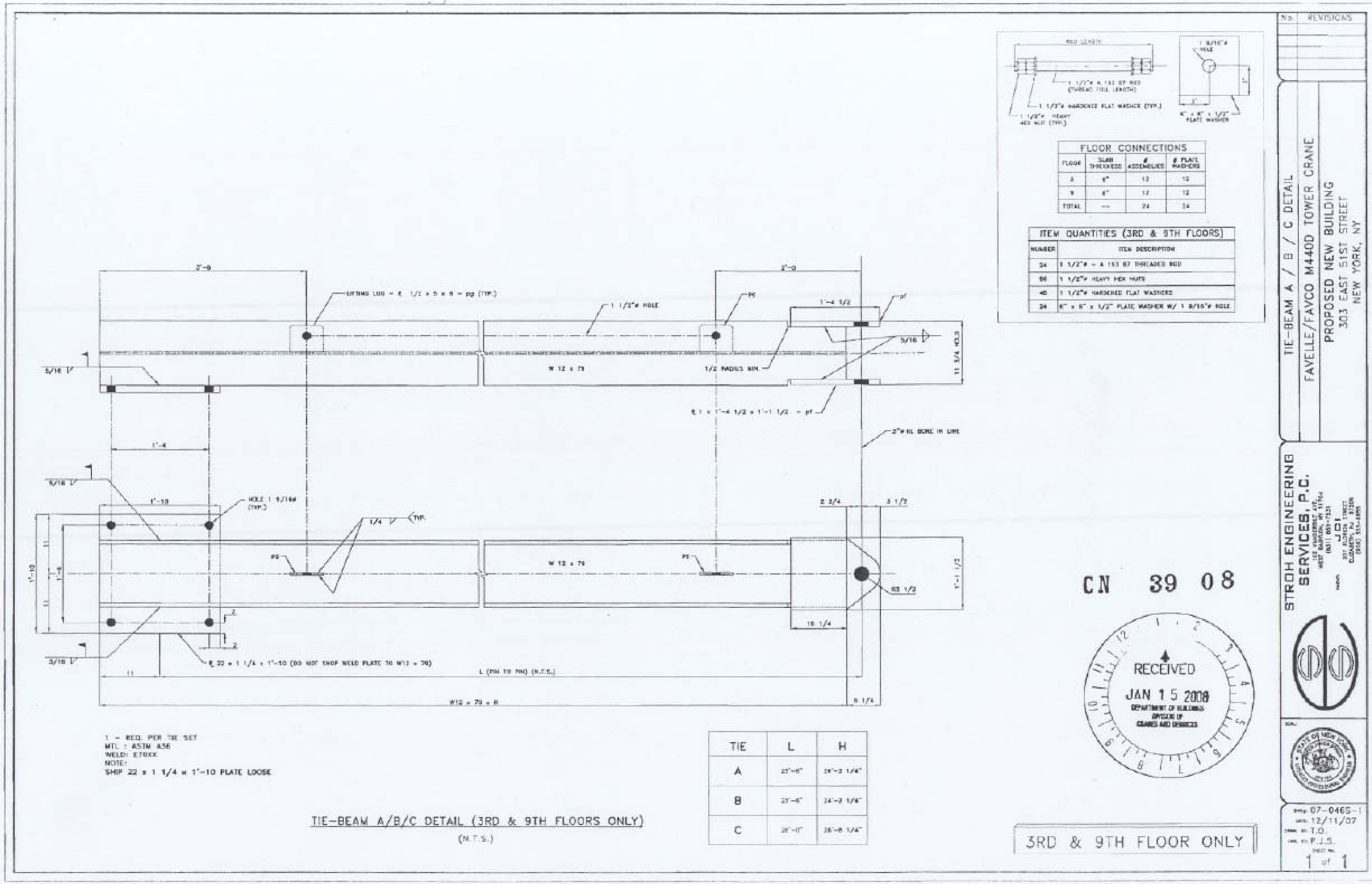


Figure F1.3 - Tie-Beam A, B & C Detail. Provided by the crane engineer Stroh Engineering, drawing 07-046S-1 sheet 1 of 1 dated 12/11/07.

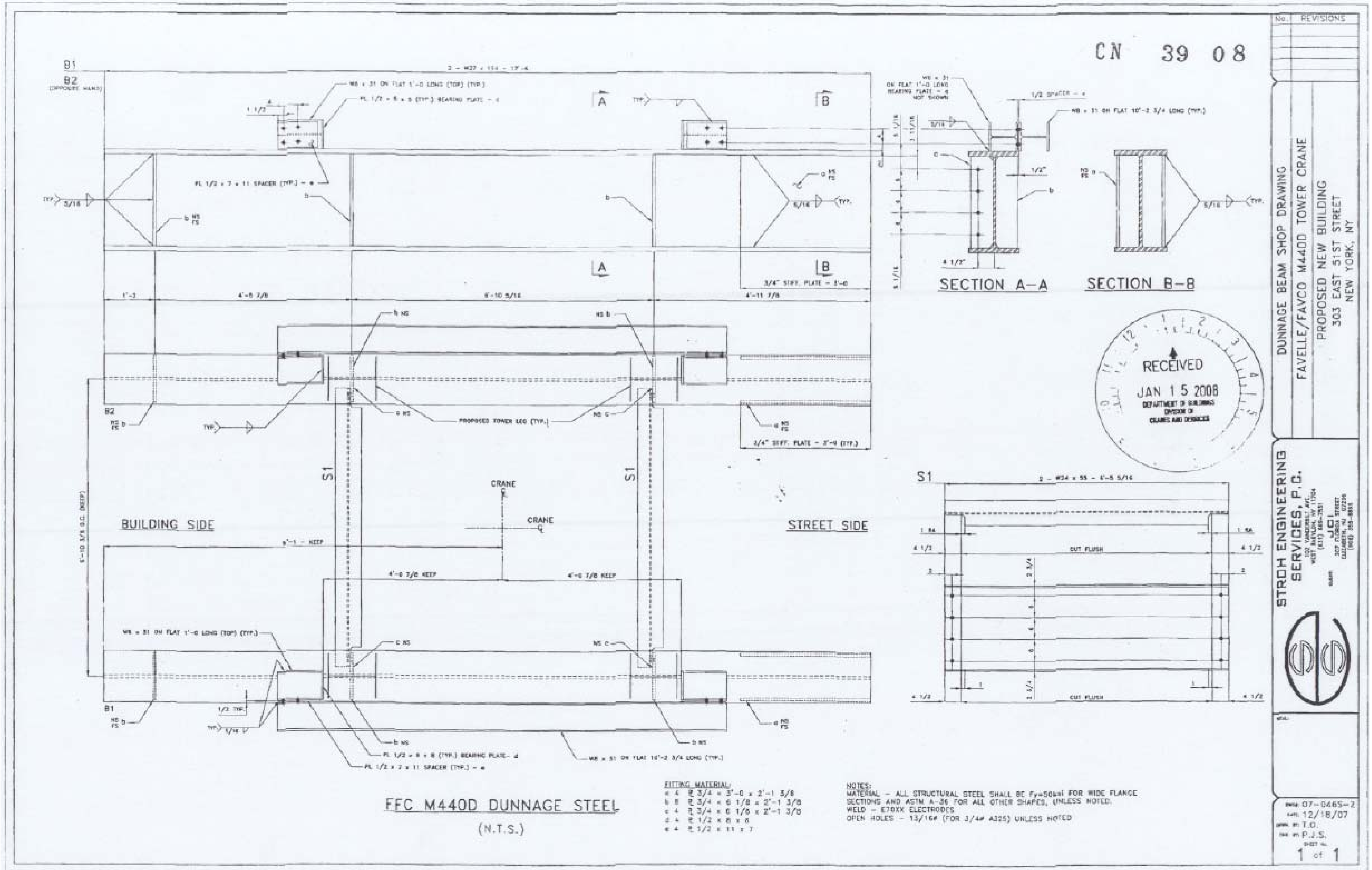


Figure F1.4 - Dunnage Beam Shop Drawing. Provided by the crane engineer Stroh Engineering, drawing 07-046S-2 sheet 1 of 1 dated 12/18/07.

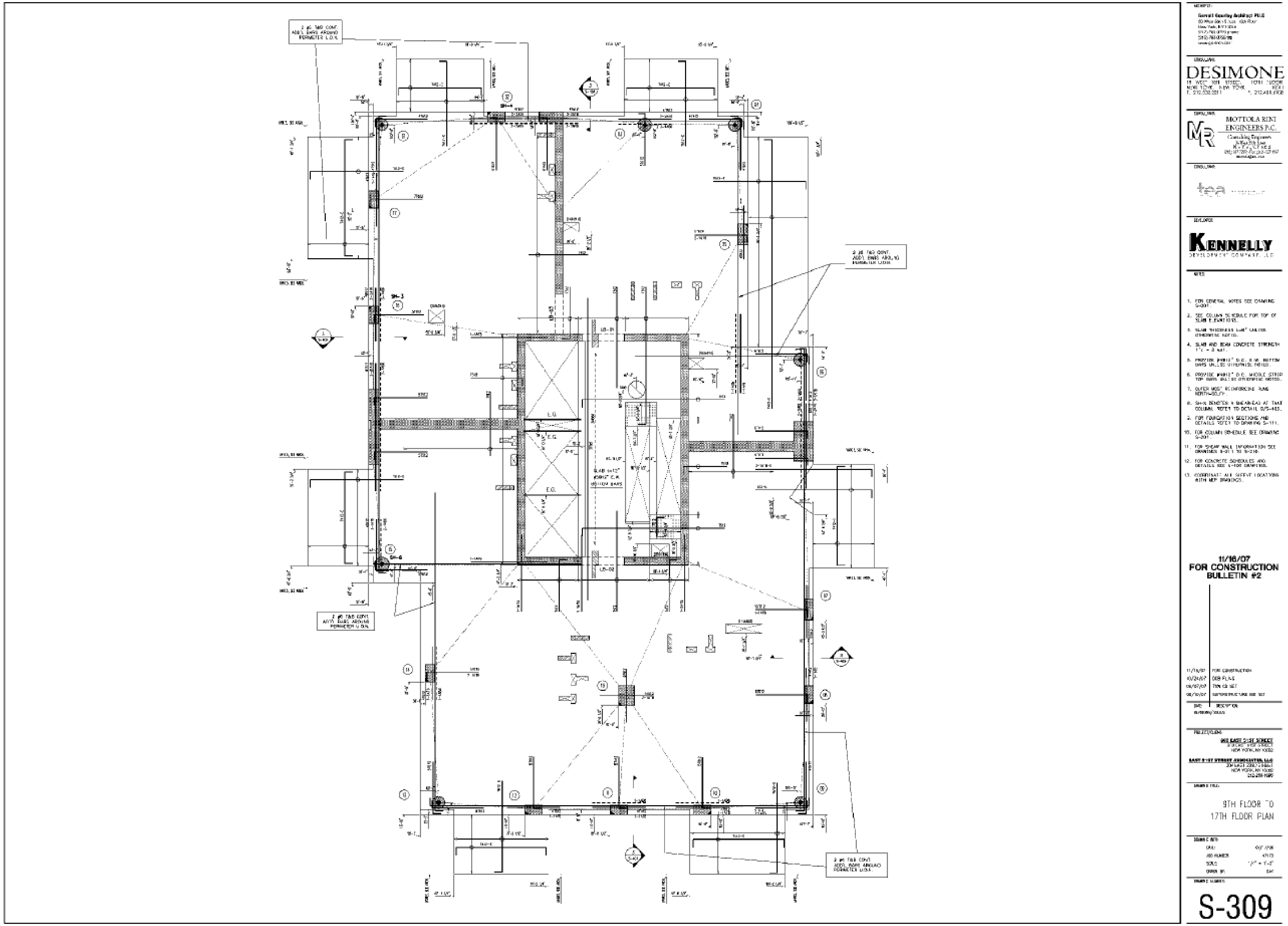


Figure F1.5 - 9th Floor Plan. Garret Gourlay Architect PLC / DeSimone Drawing S-309.

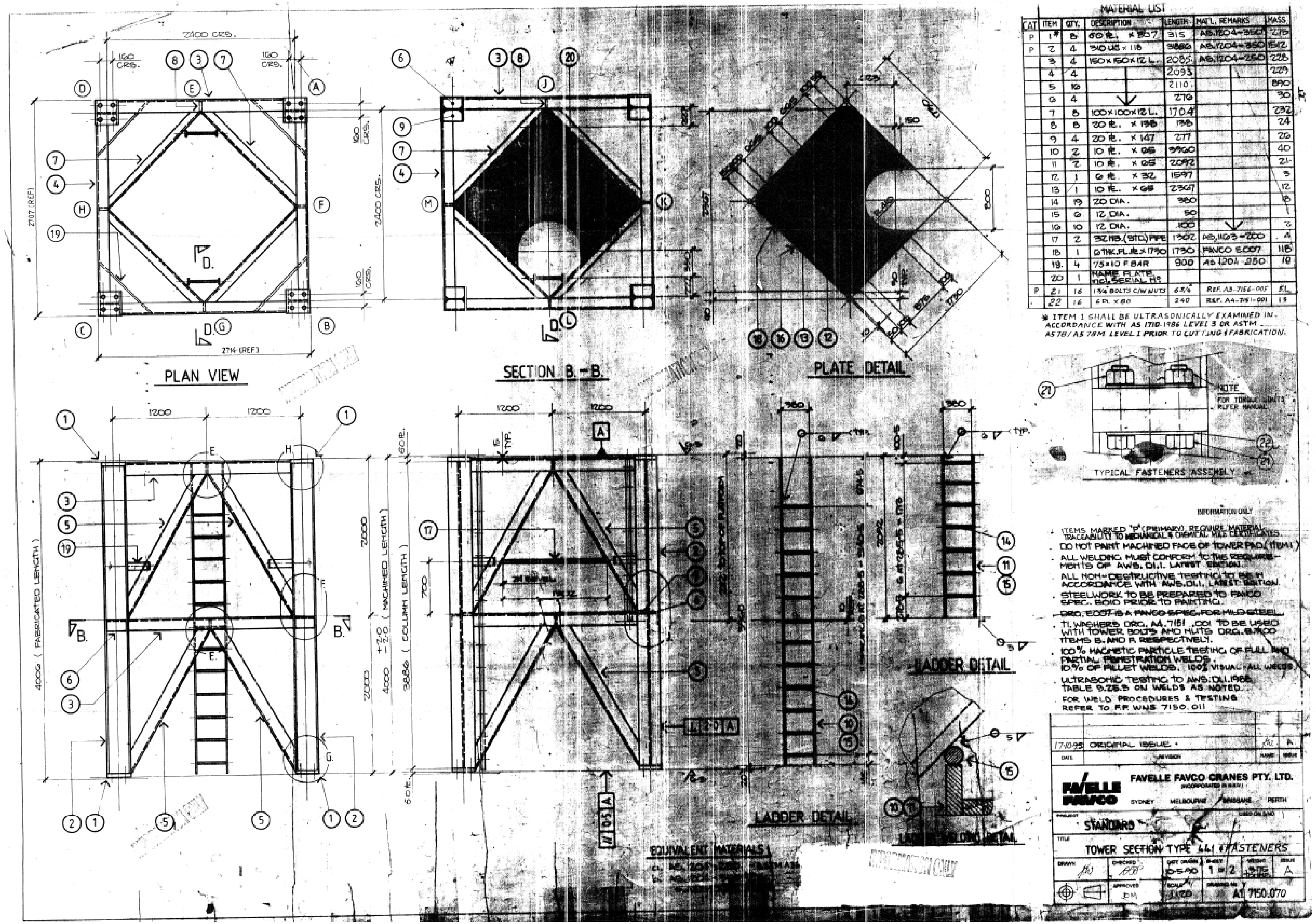


Figure F1.6 - Crane Truss (not matching with actual truss). Favelle Favco drawing A1.7150.070.

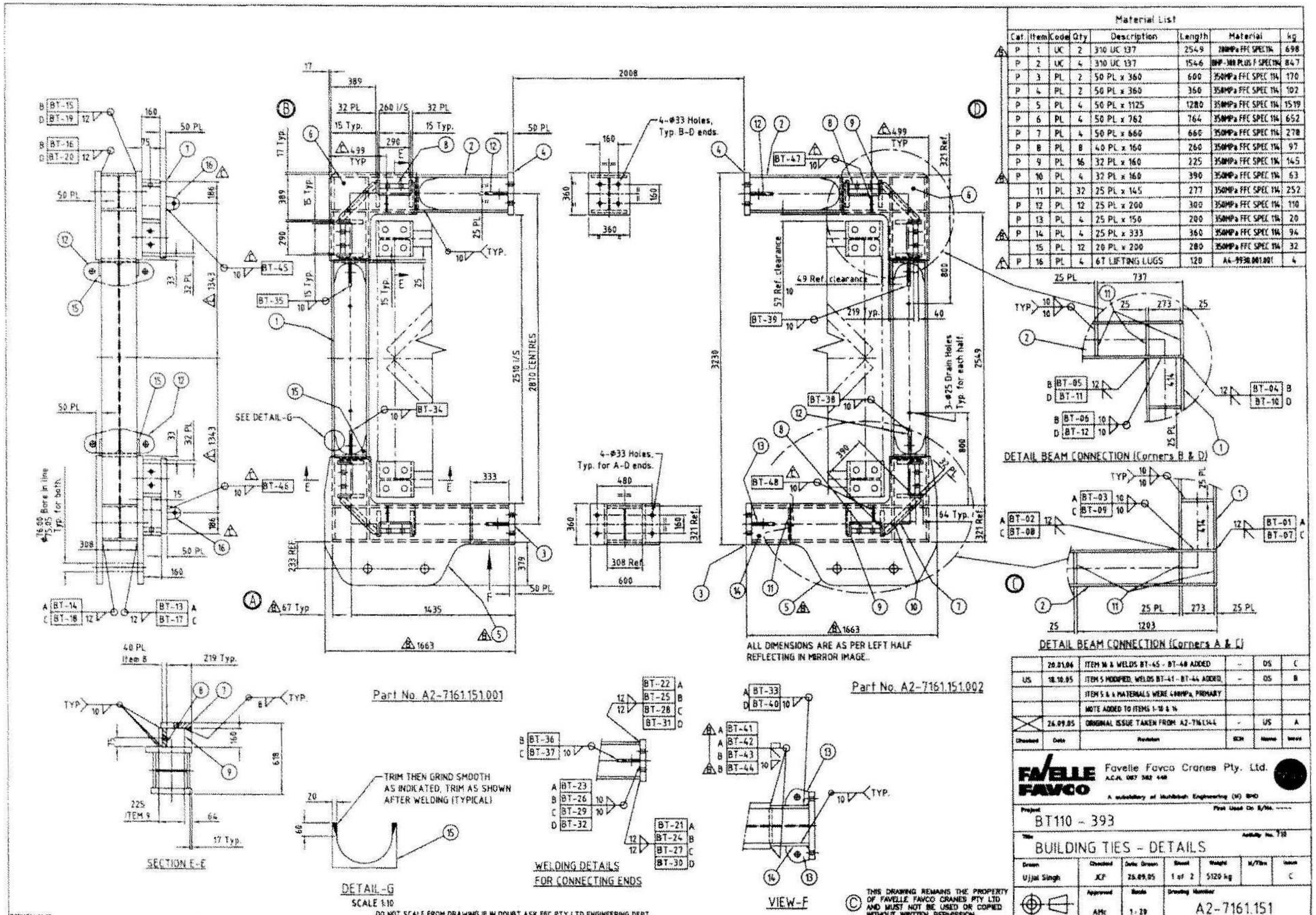


Figure F1.7 – Favelle Favco Cranes Pty. Ltd Dwg. A2-7161.151 – Building Ties-Details. Favelle Favco drawing A2-7161.151.

Appendix G

Weld Ductility Assessment

ARUP

Calculation Sheet

Job title	51st Street Crane Simulation	Job number	Sheet number	Revision
		131951		
Calc title	Weld ductility: lower bound	Member/Location		
		Drg. Ref.		
		Made by DAG	Date 17/09/08	Chd. RG

The section containing minimum material is on a 45° plane going through the root of the weld, see dimension A in Figure 1.

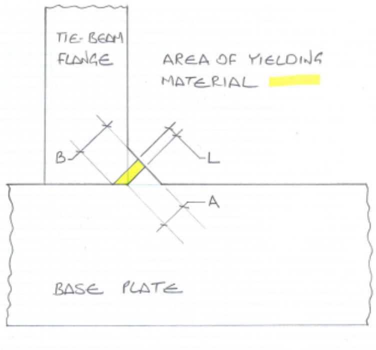


FIGURE 1

If a force is applied which generates yielding on this plane then the width of the yielded section (L) will be limited by the ratio of sigma UTS to sigma yield.

If the maximum width of the yielded section is dimension B (in Figure 1) then assuming equilibrium with the root of the weld being at UTS, and assuming the section is of unit depth then:

Sigma UTS * A = Sigma yield * B

Using trigonometry A, B and L are related:

B = A + L

Combining the above equations and rearranging gives the following relationship:

L = ((Sigma UTS)/(Sigma yield) -1) * A

For the 'as drawn' geometry (5/16 inch fillet weld) and material properties of the weld consumable used (AWA Electrode Number E70xx: Sigma UTS = 482MPa, Sigma yield = 393MPa, Minimum elongation = 0.22) the width of the yielded section is:

L = (482/393–1)*5.61 = 1.27mm

At the point of failure the strain along the yielded width will vary between the ultimate strain at section A to strain at 1st yield at section B. If this variation is assumed linear then the average strain would be:

eAVG = (0.22 + 0.002)/2 = 0.111

Therefore the lower bound deflection at weld failure is L * eAVG = 0.14mm.

In the finite element model used in the dynamic analysis of the crane the weld has been simulated with a shell element having the length of the fillet weld (5/16 inch, 7.94mm) therefore the appropriate lower bound failure strain for these elements = 0.14/7.94 = 0.018 or 1.8%.

Appendix H

ATLSS Testing Report

All photographs, figures and tables in the report are provided by ATLSS

303 East 51st Street Tower Crane Collapse - Macroscopic Examination of Crane Components

E.J. Kaufmann

Final Report
to

Ove Arup & Partners
499 Thornall Street
Edison, NJ 08837

November 26, 2008

ATLSS is a National Center for Engineering Research on Advanced Technology for Large Structural Systems

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303 East 51st Street Tower Crane Investigation

I. Introduction

As part of an investigation of the tower crane collapse at 303 East 51st Street, New York on 15 March 2008 by Ove Arup & Partners for the New York Department of Buildings, ATLSS Engineering Research Center at Lehigh University, Bethlehem, PA was contracted to carry out a metallurgical investigation of selected crane components. The metallurgical investigation was set forth in an investigation protocol prepared by Arup dated 7 October 2008. The protocol identified the crane components selected for investigation and the tests and test methods to which the crane components would be subjected. The protocol specified only non-destructive macroscopic examination of the selected crane components. The following report is the final report on the findings of the non-destructive macroscopic examination of the selected crane components.

The metallurgical investigation was performed by Dr. Eric J. Kaufmann of the ATLSS Engineering Research Center with assistance by Dr. Alan W. Pense of the ATLSS Engineering Research Center. Examination of crane components was carried out at the ATLSS Center on 16 September 2008 and at the Pier 36 storage facility in New York, NY on 6 October and 10 October 2008.

II. Scope of Work

The three horizontal tie-beams identified as B4, B5, and B6, and respective base plates, identified as 9A, 9C, and 9B, located on the ninth floor of the building were selected for the metallurgical investigation. In accordance with the non-destructive macroscopic examination prescribed in the investigation protocol (Section 3) the components were photographically documented and visually examined in their as-found condition. Physical dimensions of the components were recorded including all weld dimensions. Weld regions and weld fractures were subsequently cleaned to remove surface dirt/contamination prior to photographic documentation and detailed visual examination to identify and characterize fracture features, flaws, and defects.

III. Description of Components

The horizontal tie-beam/base plate consisted of a 1-1/4 in. thick steel plate with a W12x79 rolled shape fillet welded to the plate surface at the beam end. The base plates were connected to the building floor slab by four bolts located at the base plate corners. A pair of rectangular washer plates were used at each bolt location and tack welded to the base plate. The base plate components identified as 9A and 9C exhibited fully failed fillet welds between the tie beam and base plate. The respective tie beams, also exhibiting weld fracture at one end of the beam, were identified as B-4 and B-5. As-found views of these components are shown in Figures 1 through 4. The third tie-beam/base plate component, identified as 9B, had intact welded connections (see Figure 5).

Figure 1 Ninth Floor Horizontal Tie Beam Base Plate 9A With Weld Fractures.

Figure 2 Ninth Floor Horizontal Tie Beam Base Plate 9C With Weld Fractures.

Figure 3 Ninth Floor Horizontal Tie Beam B4 End With Weld Fractures. (Welded to Base Plate 9A).

Figure 4 Ninth Floor Horizontal Tie Beam (B5) End With Weld Fractures. (Welded to Base Plate 9C).

Figure 5 Intact Ninth Floor Horizontal Tie Beam and Base Plate 9B.

IV. Component Dimensions

A summary of the measured physical dimensions of the base plates and tie beams are shown schematically in Figures 6 through 9. The three base plates were nominally 22"x 22"x1-1/4" plates with 1.5 in. dia. holes. Most holes were drilled; a few holes appeared to be torch cut. The rectangular washer plates were nominally 4"x6"x1/2". Missing washer plates on base plates 9A and 9C were apparently not tack welded to the base plate. The section dimensions of the three W12x79 tie beams were found to be within the permitted tolerances specified in ASTM A6.

V. Weld Dimensions

Measurements of the weld length and weld size were obtained for the fillet welds joining the W12x79 tie beam to the base plate. For the two failed base plate connections (9A and 9C) measurements were obtained from the weld material remaining on the base plates. Weld sizes were measured using standard fillet weld gages.

The measured weld lengths of the three tie beam/base plates varied and ranged from 20 inches to nearly the full length of the base plate (see Figures 6-8). A short length of fillet weld of variable length was also deposited along the interior flange tips at the beam end that ranged from 2-1/2 in. to 4-5/16 in. At one location on base plate 9A the weld was wrapped around the beam end. The appearance of the welds indicated that they were likely deposited using the shielded-metal-arc weld process (SMAW) in a single pass. This process is used in both shop and field welding applications.

Examination of the weld fractures indicated that fracture occurred along the weld leg joining the tie beam and all or most of the deposited weld remained on the base plates. (Latter reassembly of the connections verified that the fractures occurred along the tie beam weld leg). Fillet welds generally exhibited unequal leg dimension with the shorter leg existing on the tie-beam. Fillet weld size measurements obtained on all three tie-beam/base plate components (see Figures 6-8) indicated a deposited weld size ranging from 3/16" to 5/16" with the majority of the deposited weld having a ¼" weld size (see Figure 10).

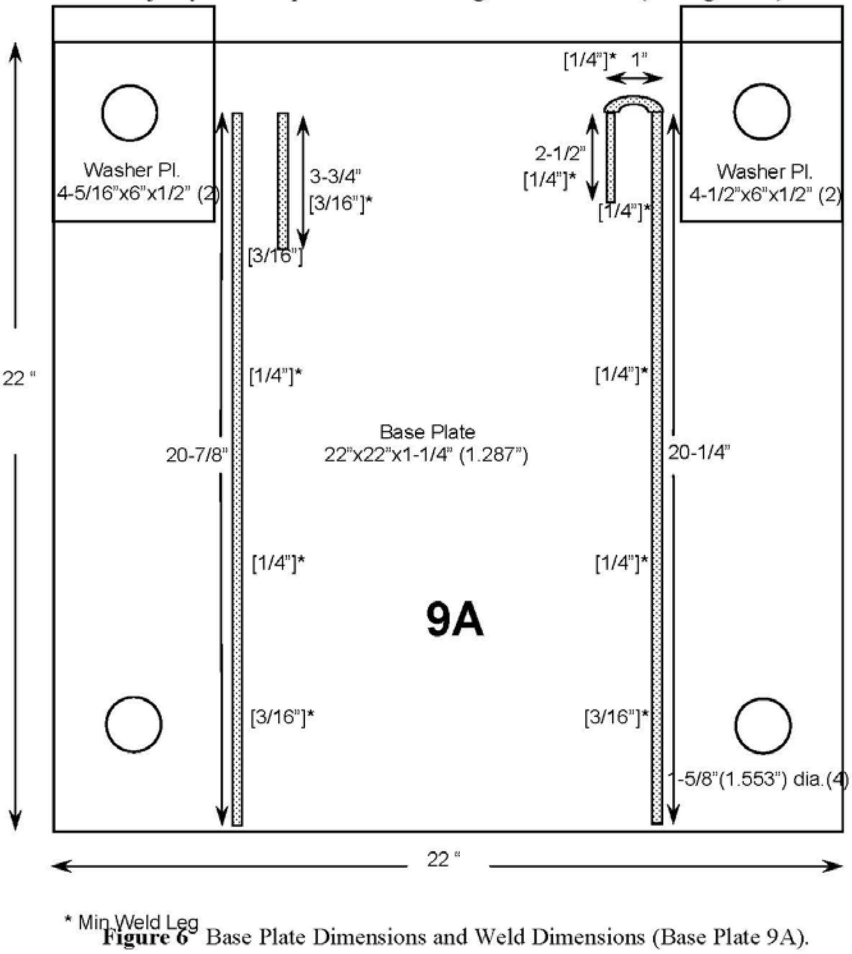

Figure 6 Base Plate Dimensions and Weld Dimensions (Base Plate 9A).

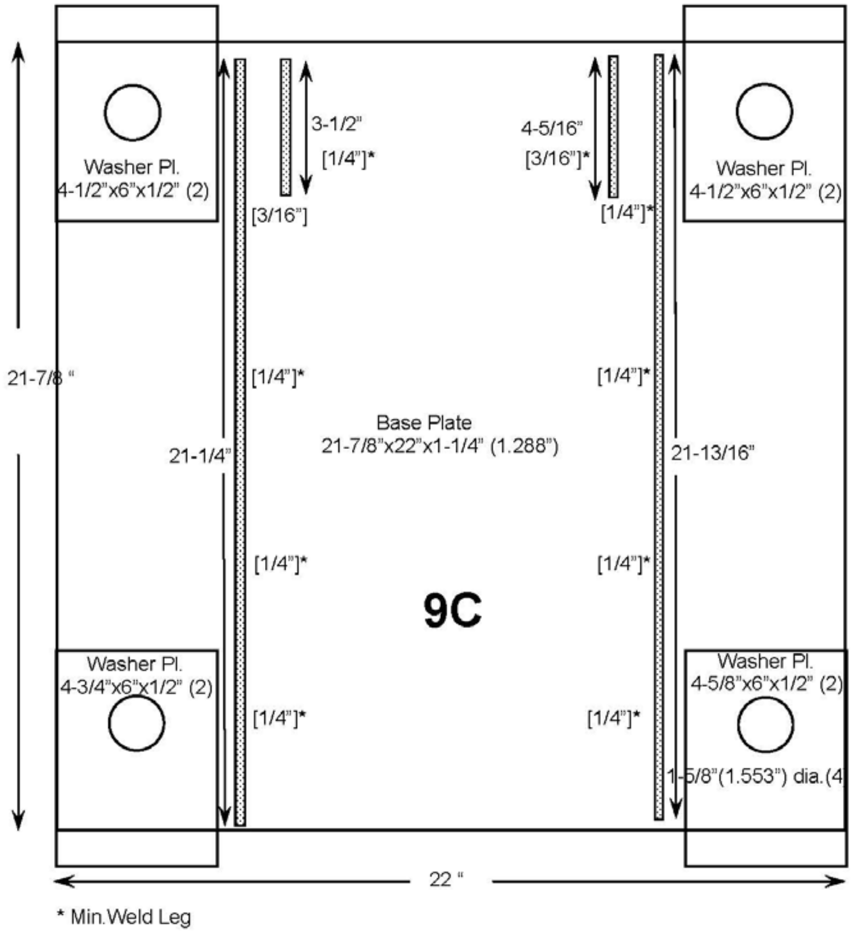

Figure 7 Base Plate Dimensions and Weld Dimensions (Base Plate 9C).

303 East 51st Street Tower Crane Investigation

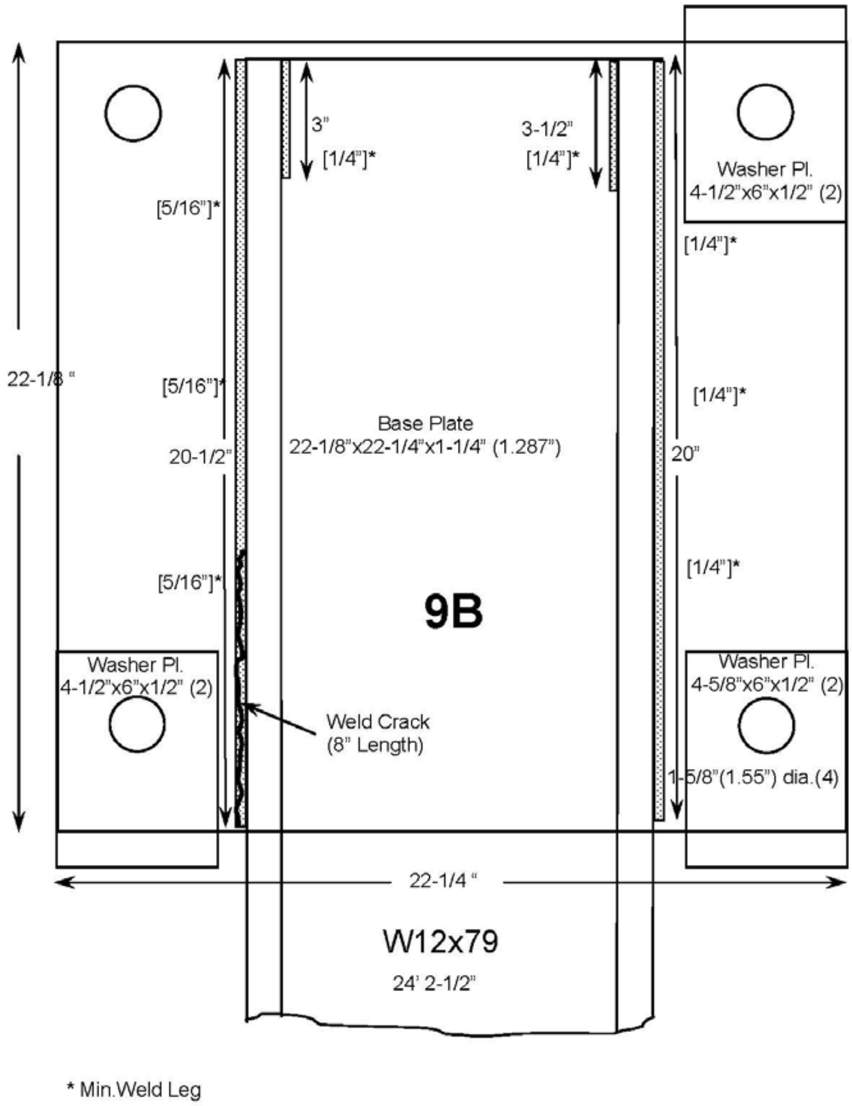

Figure 8 Base Plate Dimensions and Weld Dimensions (Base Plate 9B).

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6

303 East 51[st] Street Tower Crane Investigation

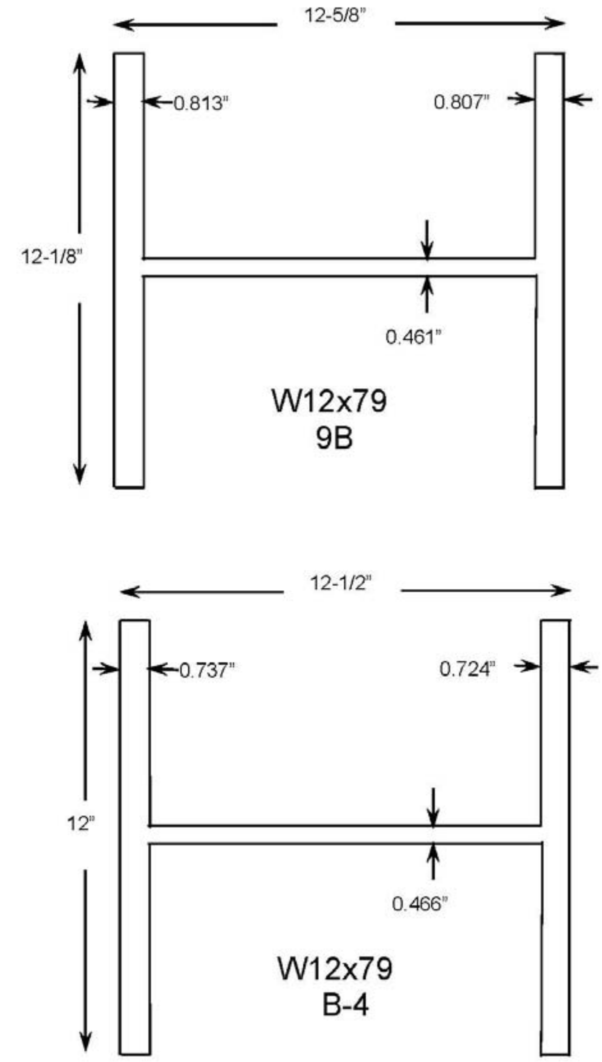

Figure 9 Tie Beam Dimensions.

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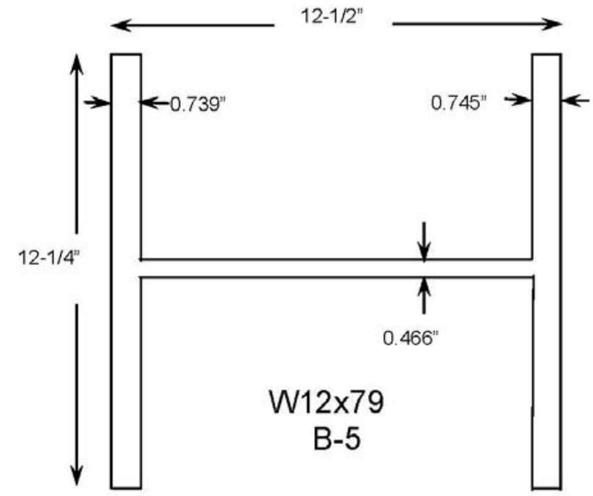


Figure 9 (cont'd) Tie Beam Dimensions.

VI. Macroscopic Examination of Welds

Prior to examination of the welds and weld fractures the surfaces were cleaned with acetone and light brushing with a soft bristle brush. The area was then rinsed with acetone and compressed air.

VI.1 Base Plate 9A/Tie Beam B4

General views of the weld fractures on Base Plate 9A and the respective Tie Beam B4 are shown in Figures 11 and 12. Visual examination of the weld fractures indicated that the welds fractured by ductile shear along the weld leg of the tie beam. The welds were generally sound with acceptable weld quality. Only isolated occurrences of weld porosity was observed. Figure 13 shows the typical appearance of the weld fractures.

The respective Tie Beam B4 weld fractures showed similar fracture features but only a small amount of weld metal deposited on the tie beam (see Figure 14). The small amount of weld fracture observed on the tie beam indicated that the effective weld size of the fillet weld was less than the deposited weld size previously measured on the base plate. Further examination of the base plate weld fractures revealed that the reduced effective weld size was due to the tie beam fit-up to the base plate before welding that resulted in a root opening along the weld that ranged from 0 to 1/16". This can be seen in Figure 13 where weld metal and weld slag flowed into the open root. This reduced the effective weld leg on the tie beam by this amount and resulted in the small amount of deposited weld observed. The reduction in effective weld size of fillet welds caused by root opening is shown schematically in Figure 15. Hence, because of the root opening the effective weld size of the base plate/tie beam connection was reduced to 1/8" to ¼" with the majority of weld having an effective weld size of 3/16". Root openings of the size measured are not prohibited; the deposited weld size must be increased accordingly to result in the required weld size for the connection.

Figure 10 Base Plate/Tie Beam Fillet Weld Size Measurement.

VI.2 Base Plate 9C/Tie Beam B5

General views of the weld fractures on Base Plate 9C and the respective Tie Beam B5 are shown in Figures 16 and 17. Similar fracture features as observed on Base Plate 9A and Tie Beam B4 were also observed on Base Plate 9C and Tie Beam B5. Visual examination of the weld fractures indicated that the welds also fractured by ductile shear along the weld leg of the tie beam. The welds were generally sound and free of weld defects with only isolated weld porosity. No evidence of weld cracking or incomplete fusion was observed. Figure 18 and 19 shows the typical appearance of the weld fractures.

As with Base Plate B4 only a small amount of deposited weld metal was observed on Tie Beam B5 again due to unequal weld legs and weld root opening when the connection was welded. Evidence of root openings ranging from 0 to 1/16" was observed which reduced the effective weld leg size on the tie beam to 1/8" to ¼" with the majority of weld having an effective weld size of 3/16".

Figure 11 General Views of the Weld Fracture on Base Plate 9A.

Figure 12 General Views of Weld Fracture on Tie Beam B4.

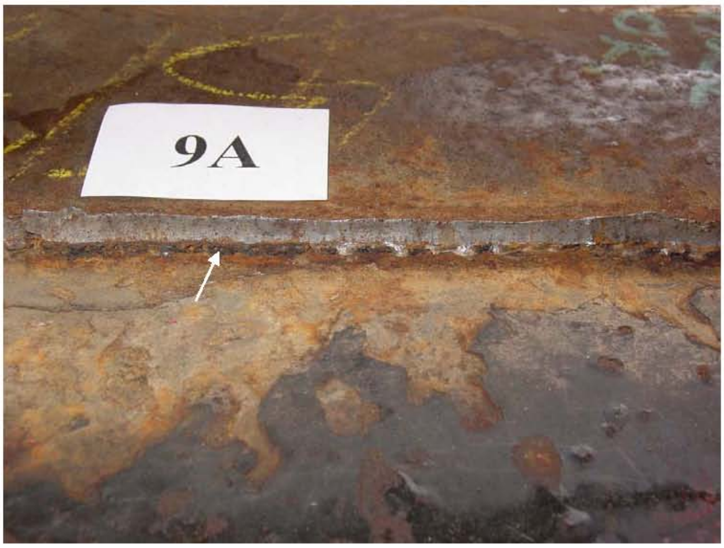


Figure 13 Enlarged Views of Cleaned Weld Fracture on Base Plate 9A Showing Ductile Weld Metal Fracture. Note Weld Metal Flow Into Weld Root Opening (see arrows).

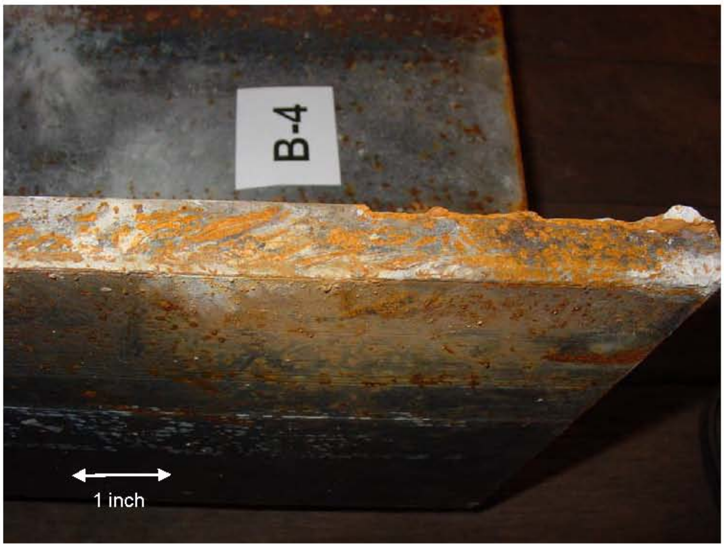
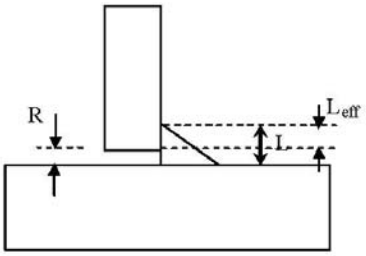


Figure 14 Enlarged Views of Ductile Weld Fracture on Tie Beam B4. Note Small Size of Deposited Weld.



R= Root Opening
L= Weld Size
L_{eff}= Effective Weld Size= L-R

Figure 15 Fillet Weld Dimensions.

VI.3 Base Plate/Tie Beam 9B

A visual examination of the intact base plate/tie beam welds was performed on Base Plate/Tie Beam 9B. General views of the welds are shown in Figure 20. Visual inspection of the welds indicated that they were generally sound and free of unacceptable defects. Deposited weld sizes ranged from ¼" to 5/16". A crack was observed in one weld at the weld end that measured approximately 8 inches in length. The crack was located along the weld toe on the tie beam leg (See Figure 20 and Figure 21).

303 East 51st Street Tower Crane Investigation

Figure 16 General Views of the Weld Fracture on Base Plate 9C.

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15

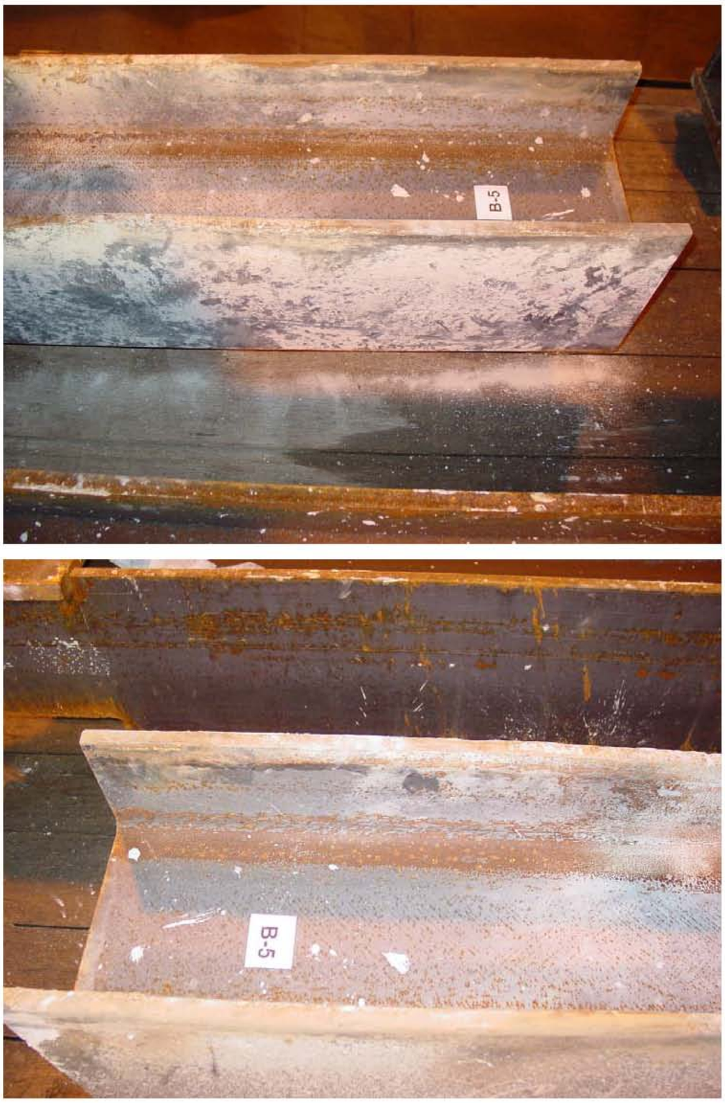


Figure 17 General Views of Weld Fracture on Tie Beam B5.

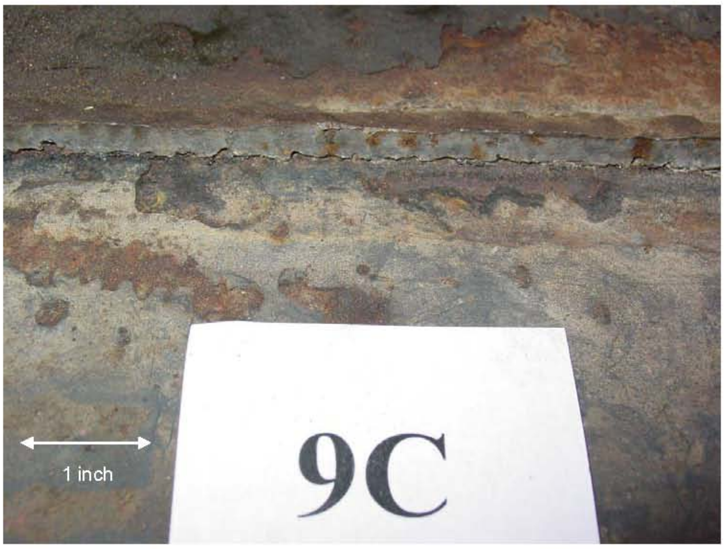


Figure 18 Enlarged Views of Cleaned Weld Fracture on Base Plate 9C Showing Ductile Weld Metal Fracture. Note Weld Root Opening.

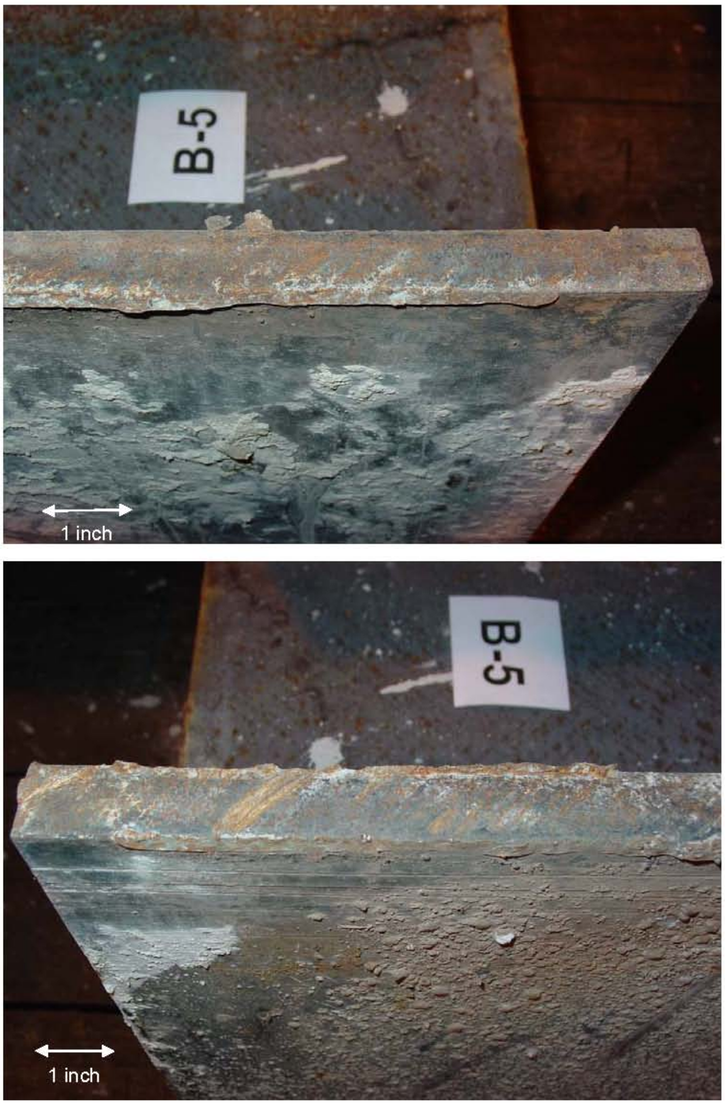


Figure 19 Enlarged Views of Ductile Weld Fracture on Tie Beam B5. Note Small Size of Deposited Weld.

Figure 20 General Views of the Base Plate/Tie Beam Welds From Base Plate 9B.(Arrow Shows Location of Weld Toe Crack)

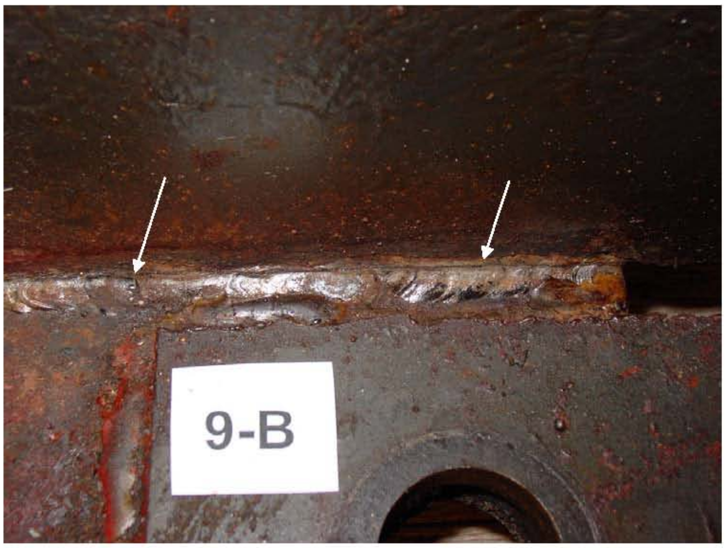


Figure 21 Enlarged View of Weld Toe Crack Observed in Base Plate 9B Weld. (see Arrows)

VII. Summary

Visual examination of the three base plate/tie beam components located on the ninth floor of the building indicated the following:

1. The three components were nearly identical dimensionally with nearly identical weld fabrication.

2. The two failed base plate/tie beam components (9A and 9C) failed by ductile shear fracture of the fillet welds joining the base plate to the tie beam. No significant fabrication weld defects were observed in the welds. A partial weld crack was observed in the intact Base Plate/Tie Beam 9B.

3. The deposited fillet weld size of the weld joining the base plate to the tie beam in the three components ranged from 3/16" to 5/16" with the majority of the weld length having a weld size of ¼". Due to unequal weld leg size and weld root opening caused by misfit-up the effective weld size was reduced to 1/8" to ¼ "with the majority of the weld length having an effective weld size of 3/16".

Appendix I

NYCDOB List of Standard Evidence

		LIST IN PROGRESS - DRAFT 3/3/08				
NYC BUILDINGS			303 East 51st Street Crane Collapse Stored Evidence			
Name	Item Number	DOB Tag Number	Description/Dimension	Photo Number	Storage Location	Equipment Identification Number
Boom	1	B-1	Crane Boom Piece - 230" long	1 thru 1Z	Pier 36 - Montgomery St. & East River	
	2	B-2	Crane Boom Piece - 308" long	2 thru 2R	Pier 36 - Montgomery St. & East River	
Tower	3	Tower Bott	Tower Bottom Piece - 150 1/2" long	3 thru 3K	Pier 36 - Montgomery St. & East River	
Boom	4	B-3	Crane Boom Piece - 221" long	4 thru 4G	Pier 36 - Montgomery St. & East River	
	5	B-4	Crane Boom Piece	5 thru 5E	Pier 36 - Montgomery St. & East River	
	6	B-5	Crane Boom Piece - 86" long	6 thru 6C	Pier 36 - Montgomery St. & East River	
	7	B-6	Crane Boom Piece - 97" long	7 thru 7F	Pier 36 - Montgomery St. & East River	
	8	B-7	Crane Boom Piece - 104" long	8 thru 8C	Pier 36 - Montgomery St. & East River	
	9	B-8	Crane Boom Piece - 115" long	9 thru 9D	Pier 36 - Montgomery St. & East River	
	10	C1-A	Collar Piece -First Half	0 thru 10G	Pier 36 - Montgomery St. & East River	
	11	C1-B	Collar Piece -Second Half	11, 11A	Pier 36 - Montgomery St. & East River	
	12	C2-A	Collar Piece - First Half	12, 12A	Pier 36 - Montgomery St. & East River	
	13	C2-B	Collar Piece -Second Half	13 thru 13F	Pier 36 - Montgomery St. & East River	
	14	C3-A	Collar Piece -First Half	14 thru 14E	Pier 36 - Montgomery St. & East River	
	15	C3-B	Collar Piece -Second Half	15 thru 15E	Pier 36 - Montgomery St. & East River	
Tower	16	3ase Crane Towe	Crane Tower Base Support - 211" long	16 thru 16Z	Pier 36 - Montgomery St. & East River	
Steel	17	B-1 (Steel)	Steel Tie Beam - 172" long	17 thru 17B	Pier 36 - Montgomery St. & East River	
	18	B-2 (Steel)	Steel Tie Beam - 220" long	18 thru 18B	Pier 36 - Montgomery St. & East River	
	19	B-3 (Steel)	Steel Tie Beam - 200" long	19 thru 19C	Pier 36 - Montgomery St. & East River	
	20	B-4 (Steel)	Steel Tie Beam - 314 1/2" long	20 thru 20C	Pier 36 - Montgomery St. & East River	
	21	B-5 (Steel)	Steel Tie Beam - 334 1/2" long markP	1 thru 21B	Pier 36 - Montgomery St. & East River	
	22	B-6 (Steel)	Steel plate Piece - 22" x 22"	22 thru 22B	Pier 36 - Montgomery St. & East River	
Beam	23	18-E	Tie Beam - 290 1/2" long	23A, 18east	Pier 36 - Montgomery St. & East River	
	24	3-E	Beam - 97" long	24, 24A	Pier 36 - Montgomery St. & East River	
	25	3W	Beam - 112 1/2" long	25 thru 25E	Pier 36 - Montgomery St. & East River	
	26	3M1D	Beam - 103 1/2" long	26 thru 26B	Pier 36 - Montgomery St. & East River	
	27	18W	Tie Beam - 350 3/4" long	thru 27B,1	Pier 36 - Montgomery St. & East River	
	28	CC 3	Yellow Beam - 202 3/4" long	28 thru 28C	Pier 36 - Montgomery St. & East River	
	29	CC 1	End Support? - 32 inch long	29 thru 29B	Pier 36 - Montgomery St. & East River	
	30	CC 4	Bracket - 25" x 26"	30, 30A	Pier 36 - Montgomery St. & East River	
	31	CC 2	Crane Bottom Piece - 18" long	31thru 31B	Pier 36 - Montgomery St. & East River	
	32	Without Tag	Support Beam piece? - 18 inch long	32, 32A	Pier 36 - Montgomery St. & East River	
	33	9	Steel Tie Beam - 290 1/2" long	33A, 9tie, 9	Pier 36 - Montgomery St. & East River	
	34	6T	Weld Cut Piece of Tie Beam - 35" long	34, 34A	Pier 36 - Montgomery St. & East River	
	35	1B	Weld Cut Piece of Tie Beam - 55" long	35, 35A	Pier 36 - Montgomery St. & East River	
	36	3B	Weld Cut Piece of Tie Beam - 62" long	36 thru 36E	Pier 36 - Montgomery St. & East River	
	37	10T	Weld Cut Piece of Tie Beam Nose Piece	37	Pier 36 - Montgomery St. & East River	
	38	4B	Weld Cut Piece of Tie Beam - 68" long	38, 38A	Pier 36 - Montgomery St. & East River	
	39	4T	Weld Cut Piece of Tie Beam - 34 1/2" long	39, 39A	Pier 36 - Montgomery St. & East River	
	40	5T	Weld Cut Piece of Tie Beam - 40" long	40, 40A	Pier 36 - Montgomery St. & East River	
	41	7T	Weld Cut Piece of Tie Beam - 31 1/2" long	41	Pier 36 - Montgomery St. & East River	
	42	2T	Weld Cut Piece of Tie Beam - 46 1/2" long	42	Pier 36 - Montgomery St. & East River	
	43	3T	Weld Cut Piece of Tie Beam - 41 1/2" long	43	Pier 36 - Montgomery St. & East River	
	44	2B	Weld Cut Piece of Tie Beam - 75" long	44, 44A	Pier 36 - Montgomery St. & East River	
	45	9TA	Weld Cut Piece of Tie Beam - 21" long	45	Pier 36 - Montgomery St. & East River	
	46	9T	Weld Cut Piece of Tie Beam - 25 1/2" long	46, 46A	Pier 36 - Montgomery St. & East River	
	47	5B	Weld Cut Piece of Tie Beam - 38 3/4" long	47	Pier 36 - Montgomery St. & East River	
	48	1T	Weld Cut Piece of Tie Beam - 25" long	48	Pier 36 - Montgomery St. & East River	
	49	8T	Weld Cut Piece of Tie Beam - 32 " long	49	Pier 36 - Montgomery St. & East River	
See desrip	50	DA1	Crane control board	DA1	James McElligot - DOI/Buildings	
See desrip	51	DA2	Engine handle	DA2	Pier 36 - Montgomery St. & East River	
See desrip	52	DA3	crane manual	DA3	Pier 36 - Montgomery St. & East River	
See desrip	53	DA4	Sheared lug 1	DA4	Pier 36 - Montgomery St. & East River	
See desrip	54	DA5	crane phone	DA5	Pier 36 - Montgomery St. & East River	
See desrip	55	DA6	Sheared lug 2	DA6	Pier 36 - Montgomery St. & East River	
See desrip	56	DA7	rope	DA7	Pier 36 - Montgomery St. & East River	
See desrip	57	DA8	rope	DA8	Pier 36 - Montgomery St. & East River	
See desrip	58	DA9	pin(from roof)	DA9	Pier 36 - Montgomery St. & East River	
See desrip	59	DA10	pin	DA10	Pier 36 - Montgomery St. & East River	
See desrip	60	DA11	pin	DA11	Pier 36 - Montgomery St. & East River	
See desrip	61	9A	plate	9a2,9a1	Pier 36 - Montgomery St. & East River	

LIST IN PROGRESS - DRAFT 3/3/08

BUILDINGS

303 East 51st Street Crane Collapse Stored Evidence

Name	Item Number	DOB Tag Number	Description/Dimension	Photo Number	Storage Location	Equipment Identification Number
See desrip	62	9C	plate	9c1	Pier 36 - Montgomery St. & East River	
See desrip	63	D1	Cable	D1f,d1	Pier 36 - Montgomery St. & East River	
See desrip	64	D2	small pin	D2f,d2	Pier 36 - Montgomery St. & East River	
See desrip	65	D3	rope	d3	Pier 36 - Montgomery St. & East River	
See desrip	66	D4	small bolt	D4f,d4	Pier 36 - Montgomery St. & East River	
See desrip	67	D5	plate connect pin	D5	Pier 36 - Montgomery St. & East River	
See desrip	68	D6	hook	D6f,d6	Pier 36 - Montgomery St. & East River	
See desrip	69	D8	wire cable	D8f,d8	Pier 36 - Montgomery St. & East River	
See desrip	70	D9	spool wire	d9	Pier 36 - Montgomery St. & East River	
See desrip	71	D10	crane phone line	D10,d10f	Pier 36 - Montgomery St. & East River	
See desrip	72	no tag	pins	nspd,pinsp	Pier 36 - Montgomery St. & East River	
See desrip	73	no tag	strap personal protection?	strapers	Pier 36 - Montgomery St. & East River	
ELEMENTS OF MINOR INTEREST						
	1	PDA1	Pipe element	PDA1		
	2	PDA2	Pipe element	PDA2		
	3	PDA3	Pipe element	PDA3		
	4	PDA4	Pipe element	PDA4		
	5	PDA5	tube	PDA5		
	6	PDA6	sheet steel	PDA6		
	7	PDA7	Pipe element	PDA7		
	8	PDA8	Pipe element	PDA8		
	9	PDA9	Pipe element	PDA9		
	10	PDA10	Pipe element	PDA10		
	11	PDA11	Pipe element w joint	PDA11		
	12	PDA12	grating	PDA12		
	13	PDA13	conduit?	PDA13		
	14	PDA14	Pipe element	PDA14		
	15	PDA15	platform steel	PDA15		
	16	PDA16	pin	PDA16		
	17	PDA17	grating platform 15	PDA17		
	18	PDA18	Pipe element	PDA18		
	19	PDA19	Pipe element	PDA19		
	20	PDA20	Pipe element	PDA20		
	21	PDA21	Pipe element	PDA21		
	22	PDA23	Pipe element	PDA23		
	23	PDA24	Pipe element	PDA24		
	24	PDA25	Pipe element	PDA25		
	25	PDA27	Pipe element	PDA27		
	26	PDA28	Pipe element	PDA28		
	27	PDA29	Pipe element	PDA29		

Appendix J

Macroscopic Examination of Crane components

All photographs, figures and tables in the report are provided by Arup

J1 Introduction

On 15 March 2008 a 205 ft-tall tower crane collapsed on 303 East 51st Street, New York. Ove Arup & Partners PC (OAP) has been retained by the New York Department of Buildings (NYCDOB) to investigate the cause of the collapse and to report there-on. The Advanced Technology for Large Structural Systems ("ATLSS") Engineering Research Center at Lehigh University, Bethlehem, Pennsylvania, has been nominated as the test facility for carrying out these tests.

This document sets out the protocol for the metallurgical investigations, resulting from recommendations arising from non-destructive testing complete thus far.

- Movement of base plates 9A and 9C (e.g., item numbers 9A, shown in Photograph 1 and 9C, shown in Photograph 3) from ATLSS back to NYCDOB.
- Examination of failed and unfailed welded connections between horizontal tie-beams and their respective base plates for beams B4 and B5. The investigations shall equally include similar items for items B1, B2, B3, and B5 as well as the B9 plate and beam.

J2 Investigation protocol

J2.1 Introduction

The following text sets out a sequence of steps that shall be followed in the metallurgical investigation.

J3 Non-destructive tests

J3.1 Movement of samples

A chain of custody form prepared by the New York City Department of Investigation (NYCDOI) shall be maintained. Baseplates 9A and 9C shall be moved from ATLSS to a location to be designated by NYCDOB. All movement or relocation of these samples from their current location shall be documented on the CCF until final disposal. Prior to shipment from ATLSS, ATLSS shall take record photographs of each in their current condition providing coverage of all surfaces.

J3.2 Sample preparation for non-destructive tests

Prior to the examination of the listed components ATLSS shall take record photographs of each in their current condition providing 100% coverage of all surfaces to be tested. Labels shall be included in the photograph identifying the view and component.

Following photography, samples shall be carefully lifted, rotated or otherwise repositioned to enable the non-destructive examination to take place. Such lifting shall not involve any direct contact with fracture surfaces, failed welds or other items to be examined. Lifting and movement of the samples shall be undertaken by NYCDOI

A record of component dimensions shall be taken and recorded on sketches (to scale) for comparison with as-built drawings. This shall include all weld sizes and bolt dimensions.

Pertinent features of the components shall be cleaned to remove surface dirt/contamination prior to the examination. For cleaning of entire sample for macroscopic examination:

1. Sample will be brushed with a soft bristle brush and compressed air to remove loose debris.
2. Fracture surfaces will be degreased with acetone and if required brushed with an aqueous solution of Alconox detergent to remove light corrosion product followed by rinsing with water and then ethanol to dry surface.

J3.3 Macroscopic examination

Each component, inclusive of welds, shall be macroscopically examined for the presence of any such features:

1. Shear lips
2. Beach marks
3. Chevron markings
4. Gross plasticity
5. Flaws or defects
6. Secondary cracks
7. Direction of propagation
8. Other surface defects or blemishes
9. The initiation point, if possible, of fracture.

All such features shall be documented and recorded photographically.

J3.4 Reporting

Upon completion of the macroscopic examination the test-house shall produce an interim report detailing the findings of their initial investigations. The report shall include, at a minimum, scope of work, component description(s), test procedures followed, identification of investigator(s) and technician(s), test results and photographs. Following review of the report samples will be nominated for destructive examination.

J4 Destructive examination

Not used

J5 Reporting

Not used

J6 References

As appropriate the test-shall comply with the following ASTM standards:

General

E2028 Standard Practice for Receiving, Testing and Reporting Results of Investigation of Metal, Ore, or Metal Related Samples that Are or May Be Involved in Litigation

Mechanical testing

E3 Standard Guide for Preparation of Metallographic Specimens

A370 Standard Test Methods and Definitions for Mechanical Testing of Steel

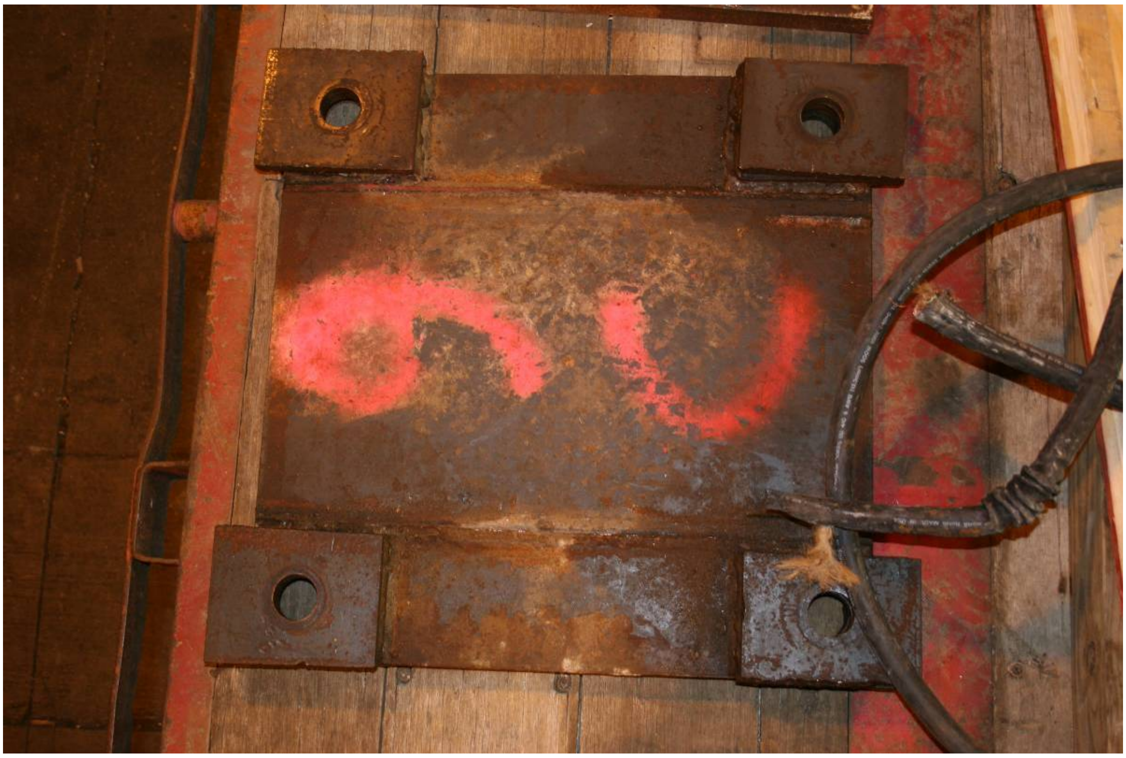
Products

Chemical & spectrographic analysis

A751	Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products
E135	Standard Terminology Relating to Analytical Chemistry for Metals, Ores, and Related Materials
E350	Standard Test Methods for Chemical Analysis of Carbon Steel, Low-Alloy Steel, Silicon Electrical Steel, Ingot Iron, and Wrought Iron
E406	Standard Practice for Using Controlled Atmospheres in Spectrochemical Analysis
E415	Standard Test Method for Optical Emission Vacuum Spectrometric Analysis of Carbon and Low-Alloy Steel
E882	Standard Guide for Accountability and Quality Control in the Chemical Analysis Laboratory
E1009	Standard Practice for Evaluating an Optical Emission Vacuum Spectrometer to Analyze Carbon and Low-Alloy Steel
E1763	Standard Guide for Interpretation and Use of Results from Interlaboratory Testing of Chemical Analysis Methods
E1806	Standard Practice for Sampling Steel and Iron for Determination of Chemical Composition
E1950	Standard Practice for Reporting Results from Methods of Chemical Analysis

Photograph 1

Photograph 2



Photograph 3

Appendix K

Dr. Tushar Ghosh Report

All photographs, figures and tables in the report are provided by Dr. Tushar except Figure 1.
Figure 1: Taken from Favelle Favco External Climbing Collar Tie Erection Sequence Drawing A1-1100.123

Investigation of Crane Collapse at 303 E 51st Street

A Report on the Condition and Possible Failure Mode of Slings

Tushar K. Ghosh, Professor
College of Textiles, North Carolina State University, Raleigh, NC

Introduction

In order to glean information to help understand the potential reasons for failure of the slings involved in the subject incident, pieces of the slings were visually (unaided) examined during a visit to the OSHA office in New York. Unaided visual examination can be useful in gathering qualitative information that can be useful in understanding the modes of failure of the sling fibers. The observations reported here are based on my visual examination of the slings as well as the images of the slings provide by Mr. James Cohen, Arup.

The purpose of this report is to convey my observations on the physical condition of the slings as well as comment on the nature of fracture/failure of the slings.

Background

It is understood that, as part of the crane operating procedures, the height of the crane would be increased periodically using pre-approved established jumping (i.e., climbing) procedures supplied by the tower crane manufacturer. At the time of collapse on March 15, 2008, the tower climbing procedure had been completed.

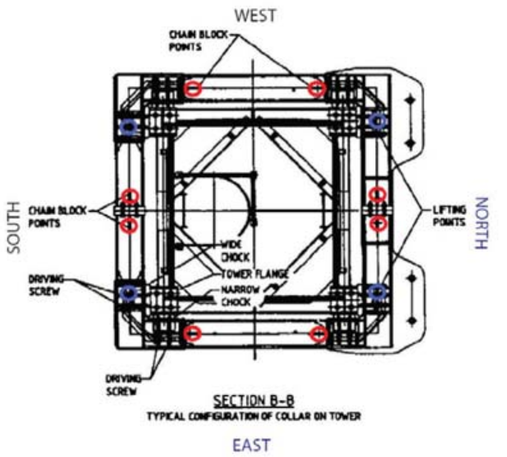


Figure 1. Positions of slings on the collar are shown by blue circles.

It is also understood that, following completion of a jump, an external collar would be applied as two symmetrical halves around the tower and tied back to the concrete building via steel beam tie-backs, themselves anchored to the concrete slab. This would take place at various heights, determined by a professional engineer. The procedure for attaching the collar and tie-backs was provided by the manufacturer.

For this particular tower, manufacturer's information indicates that the collar/tie erection sequence includes five stages of work. The erection of the collar had been completed through Stage 3; i.e., the two collar halves had been bolted together and each half was being held in place vertically by two chain-falls. Each chain-fall was in turn supported by a polyester sling, which was choked around a tower leg. Stage 4, which involves installation of the "tie-bars" (tie-backs), had begun to the point of positioning the first tie-back into its slot in the collar where it awaited the insertion of its pin. This is reportedly the point of time when the sling failure occurred. The arrangement of the chain-falls and the slings on the assembled collar before the crash is shown by blue circles in Figure 1.

OBSERVATIONS

Pieces of the four slings involved in the incident are shown in Figure 2. Each piece is aligned with another, understood to possibly be the matching, missing part, to suggest the complete sling before failure. The numbers used in identifying are those assigned by OSHA. Note that one of the pieces that should be paired with piece 1A is missing and was reportedly not found.

Figure 2. Recovered pieces of slings involved in the crash.

Sling Piece 1A: According to the label, the sling piece is made of polyester and manufactured by Metro Wire Rope Corporation. The piece, 57" in length, shows a number of high compressive stress and/or abrasion lines, approximately along the cross direction, see Figure 3a. In

Figure 3. (a) Sign of stress on sling piece 1A, (b) Fracture surface of sling piece 1A

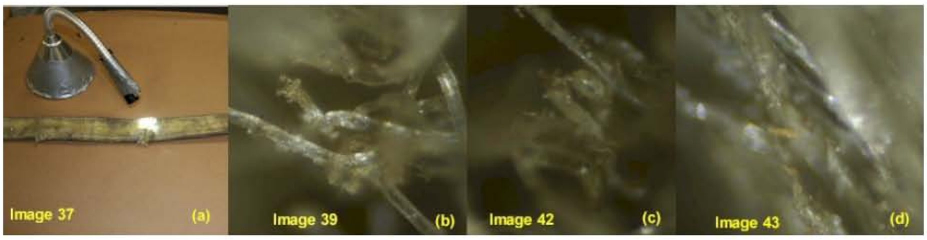


Figure 4. Micrographs (b-d) of fibers in sling 1A showing abrasion damage. Image (a) represents the position of the fibers presented in the micrographs.

some instances the fibers are broken (or cut) along the stress lines. Figure 3b shows a close up of the fractured end of the sling-piece 1A. The failure surface of the sling is along a diagonal line to the long direction of the sling. Visual examination of the fractured end clearly show two distinct areas. Almost ¾ of the sling failure surface seems to present a high-stress compression and/or abrasion failure from a sharp edge while the rest looks almost like slow-speed tensile failure. The slow-speed tensile failure is characterized by fiber pull-out and thinning of fibers as seen in Figure 3b. The evidence of abrasion damage in sling-piece 1A is clearly evident in a few micrographs presented in Figure 4. The fibers in the vicinity of failure show mangled ends and fibrillation. In addition, the striations along the cross direction of the fibers, visible in Figure 4b and in others could be signs of initiation of cracks during this incident or due to prior use.

Sling Piece 2A: The sling piece looks relatively new and according to the label, is made of polyester yarns and manufactured by LiftAll Corporation. This piece is 58" in length and shows at least one line of

Figure 5. (a) Sign of stress on sling piece 2A, (b) Fracture surface of sling piece 2A

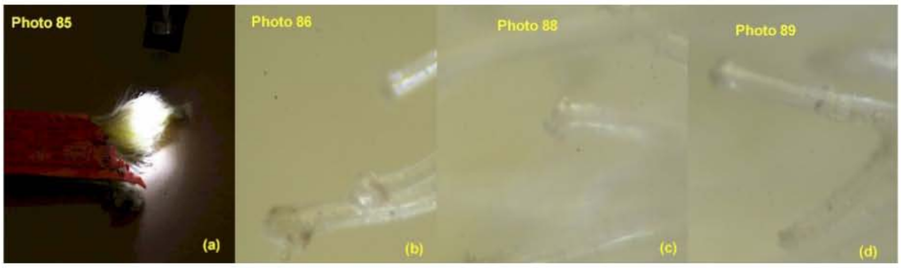


Figure 6. Micrographs (b-d) of fibers in sling 2A showing mushroom-like fracture ends . Image (a) represents the position of the fibers presented in the micrographs.

very high compressive stress, approximately along the cross direction as shown in Figure 5a. Some of the fibers along the stress line are broken.

The fracture surface is mostly at a diagonal to the axis of the sling, see Figure 5b. More than half of the failure surface seems to present a well-defined linear high-stress cut due to contact with a sharp edge. The evidence of abrasion melting is also present. The other part of the fracture surface presents a mix evidence of cutting, relatively slow tensile pull out, and high speed failure characterized by melting due to high-speed drawing. Limited evidence of mushroom-like fiber ends resulting from high speed failure are presented in Figure 6.

Figure 7. Fracture surface of sling piece 4A.

Figure 8. Fracture surface of sling piece 7A.

Sling Piece 4A: The sling piece is 24" long and appears faded in color. It is unclear whether this is due to prior use/storage or because of the exposure after the incident. The interesting feature of this piece is the failure surface shown in Figure 7. It shows clear evidence of melting of broken fiber ends resulting from heating of the fibers, either due to high frequency and/or stress abrasion or high speed tensile loading. It is well known that at slow speeds (rate of extension or loading) most manmade thermoplastic fibers show ductile failure characterized by crack formation and high-yield extension. On the other hand, in high-speed failure (e.g. falling pendulum), where the time to break is very small (fraction of a second) the broken end often shows evidence of melting due to heat of drawing. The process of slow-speed failure is isothermal, as opposed to adiabatic conditions at high-speed.

Sling Piece 7A: The sling piece labeled 7A is about 27" long. Interestingly, its failure surface presents clear evidence of fiber melting and is similar to that reported for sling-piece 4A, see Figure 8.

Figure 9. Fracture surface of sling piece 11.

Figure 10. Fracture surface of sling piece 12.

Sling Piece 11: The sling piece is approximately 13" long and the fracture surface shows signs of melting due to heat of drawing similar to that of 4A, see Figure 9. Once again the fracture or cut is at a diagonal to the long axis of the sling.

Sling Piece 12: The piece looks relatively new and according to the label, is made of polyester yarns and manufactured by LiftAll Corporation. The piece is about 48" long. As shown in Figure 10 the failure surface shows evidence of high speed failure as described for 4A, as well as possible cutting due to contact with a sharp edge. It also shows at least one high stress line along the cross direction.

Sling Piece 13: The sling piece looks relatively new and according to the label, is made of polyester yarns and manufactured by LiftAll Corporation. The piece is about 47 in long. It presents evidence of one high stress line along the cross direction, see Figure 11. The fractured surface is very similar to piece 12.

Figure 11. Evidence of high stress along a line on sling piece 13.

CONCLUDING REMARKS:

The observations presented here are based on limited sources of information. Almost every piece of sling presents signs of moderate to severe abrasion and/or lateral compression damage. There is some evidence of cracking of the fibers along the cross direction, indicating possible prior loading or use. There is considerable evidence of fiber failure at different speeds and abrasive weakening of the slings leading to failure. The rates of strain (or speed), involved in the failure of slings are indicative of "time to failure" and in turn the sequence of failure of the slings. These parameters may be crucial in the reconstruction of the crash.