INVESTIGATION REPORT:

JANUARY 14, 2008 SHORING COLLAPSE AT 9 DOMINICK ST. (AKA 246 SPRING ST.) MANHATTAN

EXECUTIVE SUMMARY

On January 14, 2008 at approximately 2PM, concrete formwork and the two-story high shoring system which supported it collapsed at a 42 story high rise residential hotel under construction at 246 Spring Street (Trump Soho) in Manhattan. The accident occurred while concrete was being poured to create the northeast corner of the 42nd floor. One worker fell to his death, and two others were injured. The collapsed corner had required a two-story high support for the formwork because the building was designed with a two-story high recess between the 40th and 42nd floors at this location.

New York City Department of Buildings (DOB) forensic engineers started an investigation the same day to establish the physical causes of the accident and to verify compliance with the New York City Building Code and proper engineering practice.

The investigation revealed that employees of the concrete contractor, DiFama Concrete, Inc. (DiFama), had installed the two story shoring system without following plans prepared by the licensed engineer. Those plans were required by Building Code 27-1035 (c) for formwork set at heights over 14 feet.

Following a systematic examination of the physical evidence, the DOB investigators were able to render the configuration of the shoring system substantially as it was immediately prior to the accident. In essence, the installed (and subsequently failed) two story system consisted of a one-story tall shoring system that supported (via aluminum stringers and wood joists) a plywood floor, and on top of this plywood floor, another one story aluminum shoring system that supported the formwork. Because some of the legs of the top shores were not positioned directly above the wood joists, when concrete was poured into the actual formwork set atop this second tier, the plywood on which the upper tier was resting was susceptible to punching. The engineering calculations clearly show that the loads (i.e., the weight of the concrete) supported by a shore leg were significantly higher than the capacity of the plywood to resist punching. The punching capacity of the plywood was obtained by tests performed by a specialized testing lab, Wood Advisory Services. The examination of debris revealed several cases of punching.

Tests at Lehigh University proved that the Patent Construction Systems aluminum shores used on site were capable of carrying the loads for which they had been rated. However, the shore towers were intended to be used on a strong base, rather than the weak plywood base actually present. Our engineering calculations show that when exposed to significant deflection at the base, the shore towers start to fail.

The shoring manufacturer had provided specific instructions on how to avoid setting the shores upon a weak base, but they were ignored as wood sills required by the manufacturer were not found in the debris. Calculations show that excessive deflection caused by the use of low quality wood and improperly placed shore legs could have led to the failure of a tower, even in the absence of actual punching.

As installed, the two tier system transferred the load of the top tier shores to the bottom tier via aluminum stringers. The investigation determined that numerous aluminum stringers were placed improperly, contrary to drawings and manufacturer's instructions. Tests at Lehigh University showed that such improper placement significantly reduced the factor of safety of the tower system.

When the concrete was poured, some legs punched the plywood. Parts of the top tier system then lost stability, and the weight of the concrete was redistributed to the legs that remained stable. This redistribution increased the load on these legs. The increased leg load was then transferred to the shore system below (at the 40th floor) by way of eccentrically placed aluminum stringers. The eccentric transfer of the increased load led to the collapse of the lower tier shores.

If the shoring system in question had been properly installed, it would have had sufficient vertical shore towers to carry the weight of the material above. However, the assemblage was not provided with sufficient positive (dedicated) connections to resist or transfer lateral forces or movements. Contrary to Building Code requirements, the shoring system lacked installations necessary to provide resistance and capacity to transfer lateral forces. This deficiency was a further weakness of the shoring installation.

The investigators found the following further significant defects in the installation of the shores that might have contributed to the collapse:

1. The stringers were in many cases not fastened to the top plate;
2. The extension of the head leg exceeded the 12" indicated as the maximum on the manufacturer's drawings;
3. The nailing of legs to plywood and joists to aluminum girders was poor, and there were only few tie backs installed to stabilize the aluminum towers and transfer lateral loads to floors;
4. Tests by Wood Advisory Services found the wood to be of a quality inferior to what was requested on the shoring drawings.

In conclusion, based on evidence provided by calculations, testing and findings in the debris field, the investigation found that improper installation, without the benefit of engineering consideration and in disregard of both Building Code requirements and proper construction practice, caused the failure of the two story shoring system. DiFama, the concrete contractor, failed to follow the shoring manufacturer's instructions and the drawings found at the site for construction of the support of formwork system.

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1 Accident

In the afternoon on January 14, 2008, formwork collapsed at the northeast corner of the 42nd floor of the new building being erected at 9 Dominick Street, Manhattan. The project is also known as 246 Spring Street or the Trump Soho Hotel. The collapse resulted in the death of one worker, Yuri Vanchytskyy, and injuries to three others.

1.1 Construction Activities at the Time of the Accident

In the afternoon on January 14, 2008 concrete had been poured over most of the entire north end of the 42nd story. At the time of the collapse, only a small area of the northeast corner was yet to be poured. The collapse occurred exactly in that area. The collapse zone was an area about two bays north and two bays east (40 ft by 40 ft). The collapsed corner had required a two-story high support for the formwork, as the building architecture required a two-story high recessed space (see Photos 4,5,6). As a result of the collapse, the recently placed concrete, still wet, flowed onto the floors below and onto the street. The concrete was 5,000 psi, with super-plasticizer. It was furnished by NYCON, of Long Island City. Concrete Controlled Inspections were being performed by Macia Inspection and Testing Laboratories (Macia).

The contractor's intent that day was to pour 237 cubic yards of concrete in slabs, beams and columns at the 42nd floor (including columns 101, 102 and 103 in the area that would collapse that afternoon). The concrete was being lifted at the elevation in a bucket, dropped in place on the formwork and spread. The amount of concrete scheduled to be poured, and the pouring methodology had been used for the other floors.

1.2 Construction Site Organization

The official address of this site is 9-19 Dominick Street. The owner is listed in the New York City Department of Buildings (DOB) applications as Bayrock/Zar Realty, LLC, 423 West 55th Street, New York, New York 10019-4460. The Construction Manager/General Contractor was Bovis Lend Lease (Bovis).

The concrete construction was being performed by DiFama Concrete, Inc. (DiFama). The shoring system was furnished by Patent Construction Systems (Patent), which also provided drawings for support of formwork. Patent had been engaged by DiFama. DeSimone Consulting Engineers (DeSimone) was the structural engineering company of record, meaning DeSimone designed the concrete structure. DeSimone had been the applicant of record for the concrete work and also performed controlled inspections for all concrete work. Testing of concrete was performed by Macia, an approved testing laboratory. The task of ensuring site safety had been delegated to several Site Safety Managers from Bovis. Signing as Site Safety Manager was Kareem Muhammad of Bovis. Martin Bonsignore of Bovis signed as Superintendent of Construction in the Work Permit application form (PW-2) of 6/1/06 for the new building.

1.3 Immediate Stabilization

Immediately following the collapse, additional post shores were installed to improve the stability of the debris pile and remaining concrete. These posts were distinguished by blue paint marks. In addition, a wide net was installed over the debris, and Howard Shapiro and Associates of Lynbrook designed a cantilevered platform with a protecting screen that allowed access to the debris field and served to catch any element that might have escaped from the debris pile.

The removal of the debris proceeded from top to bottom under the supervision of the investigating team. The rebars were burn-cut into manageable segments. Burning occurred also in the last stages, when elements had to be disengaged from the concrete. The concrete that had inadvertently flowed onto the 40th floor was removed by jack-hammering.

Following an investigation of the condition of the remaining concrete around the accident area by De Simone, the structural engineering company of record, a larger portion of the 42nd floor slab was demolished. The removal was requested by the engineer of record, who deemed the concrete poured in that area compromised. The investigation by DeSimone was focused on the condition of the remaining concrete, as it had been disturbed by the collapse and by the interruption of the concrete pour due to the accident. The DeSimone investigation was not related to the present report and findings. The concrete removal was allowed only after the debris removal in the collapse area had been completed.

McLaren Engineering Group, an engineering company commissioned by DOB, performed tests and analyzed the concrete poured at other floors and did not find any problem with the concrete strength.

2 Investigation

2.1 Organization

A technical investigation was performed by DOB Chief Structural Engineer Dan Eschenasy, PE. with assistance from DOB Forensic Engineer Naweed Chaudhri who participated during data collection. GuoZhan Wu, PE, was specially detailed to the forensic unit to prepare the engineering calculations.

The testing and inspection of wood elements was performed by Matthew Anderson and Al DeBonis of Wood Advisory Services. See Appendix B. Testing of Patent Aluminum Shoring was performed at the Structural Test Laboratories Lehigh University, ATLSS Multidirectional Laboratory - Fritz Engineering Laboratory (Lehigh). See Appendix C. Both series of tests, the Wood Advisory Services and Lehigh took place in the presence of representatives of DOB, Thornton Tomasetti and various parties.

2.2 Material Evidence Collection at the site.

The physical evidence at the site was collected by DOB in conjunction with Thornton Tomasetti, an engineering forensic firm representing Bovis. The protocol (agreed to by DOB and Thornton Tomasetti) included tagging, storage and determination of elements that could be discarded.

The tags indicated the type of element, location, zone and elevation, from where the element was collected or recovered. A running number was maintained. Each tag was signed by both Thornton Tomasetti and DOB. The New York City Department of Investigations stored the material. See Figure 1 for tagging areas. The material elements that were considered of potential interest were stored in closed containers. Investigators for the other parties were allowed to photograph and measure evidence on their own. See Figure 1 for zones indicated on tags. The on site investigation was carried out from January 14, 2008 to the end of March 2008.

2.3 Testing

A basic protocol for testing of wood was proposed by DOB and accepted by Thornton Tomassetti. The purpose of the testing was to establish the engineering properties of the wood found onsite, including the capacity of the plywood to sustain concentrated forces. The report by Wood Advisory Services (WAS) is attached in Appendix B. Note that Wood Advisory Services had also been commissioned to observe the wood collected at the site and report on its condition. The various wood tests were performed by Matt Anderson and Al DeBonis, Ph.D of Wood Advisory Services at their Millbrook, NY lab. A protocol for testing of aluminum shores was prepared by DOB and accepted by Thornton Tomasetti. Frank Stokes, Manager of the Fritz Engineering Lab, at the ATLSS Engineering Research Center of the Lehigh University performed and oversaw the tests. The results are attached in Appendix C. Both the Wood Advisory Services and Lehigh tests were conducted in the presence of the various parties who wished to attend.

2.4 Debris Field and Preliminary Observations

The collapsed corner had required a two-story high support for the formwork, as the building architecture required a two-story high recess (see Photos 5, 6 and 7- Partial Plan Concrete at 40th, 41st and 42nd floors and Figures 3, 4 and 5 on shoring reconstruction). The collapse field covered the entire two-story high bays area. Notably, the collapse also did not extend in a meaningful way beyond these bays. All other shores that were supported on the concrete slab at the 41st floor were still standing, although several of these, immediately bordering the bays, exhibited some effects of the collapse (leaning, damage or displacement). As a result of the collapse, the recently placed concrete, still wet, flowed on to the floors below and on to the street.

The slab being installed in that north east corner included significant transfer beams that were heavily reinforced. These beam bundled rebars, together with the slab-reinforcing mesh, prevented the collapsing debris from falling off the building. As a lucky consequence, only one or two heavier formwork shoring elements fell to the street, and no significant pedestrian injury was registered at street level.

As a result of the accident, the rebars and the formwork collapsed, creating a steep surface. The considerable weight of the rebars crushed all the debris, complicating the task of the investigators. Differentiating between the damage and fracture that initiated the collapse and the subsequent damage was extremely difficult. Numerous pictures of the debris field were taken, including a three-dimensional scan of the area.

The debris pile included numerous failed elements including:

- broken or sheared wood joists and plywood;
- bent and fractured extension leg heads;
- bent or fractured aluminum shores;
- failed welds at various members of the shores;
- twisted stringers.

The types of material failure found in the debris pile are described and characterized separately for each element in 3.4.

Photo 1 Collapse Area - Looking West

Investigation around the boundaries of the debris field allowed a better understanding of the general construction layout. Observations in that area revealed instances of improper installation that are described in the report. The investigators strongly believe that such improprieties also existed in the area that completely collapsed. One of the main observations based on the layout of the debris, was that whatever the initiating cause, the extent of the total failure was limited to the two-level of shores installation. Clearly, the failure did not progress in any manner past the edge of the already poured 41st floor (see Photo 1).

Additionally, the layout of the debris suggested a failure of the vertical support systems, most likely tower buckling. If instead the towers had overturned, it is likely one or more would have fallen onto the street.

Photo 2 Debris Field 41st fl.

Photo 3 42nd Fl. Debris - Looking North East

The formwork is assumed to have followed the concrete drawings for shape and elevation. The formwork system was inspected prior the start of the pour by an inspector representing the

Controlled Inspector, DeSimone, for general layout of beams and reinforcing. During the initial interviews that took place at the beginning of the investigation, it was learned that the contractor had installed the formwork shoring without referring to any drawings.

The support of the formwork, i.e., the shoring system, that existed immediately prior to the collapse was reconstructed based on the data collected (formwork and shoring debris found, pictures and measurements taken during the investigation). The reconstruction assumes that the shoring met basic dimensional conditions required by the geometry of the formwork. The errors flagged in this report were identified during the inspection and debris collections.

Also, an accounting of the number of shores existing in the collapse area was established. The table contains all shores or shore fragments that were found in the collapse zone and it establishes with credible accuracy the number of shores and their type. The corresponding pictures identify the condition of the elements post-collapse. A catalog with keyed pictures of all these shores is provided as well.

Based on pictures and measurements taken during evidence collection, the plan and elevation of the shoring was established (see 3.1.1 for Plan Shores at 40th floor and 3.1.2 for Plan Shores at 41st floor). Given the crushing of the debris, the element positions in the reconstruction plans have a degree of approximation of several inches. Wood joists and plywood that did not exhibit special defects were discarded based on a common agreement between DOB and Thornton Tomasetti.

2.5 Governing Design Documents and Material Properties

The issue of design documents is discussed at 5.1.4 and 6.2. A set of drawings were found on site identified by a drawing number, 4607K070, under the title “20KA Shoring Layout Project: Soho Hotel; Location 246 Spring Street; Customer: DiFama Concrete.” The investigation considers the design and installation instructions existing in the *General Notes and Instructions* on sheet 1 of drawing 4607K070 to be relevant to the installed materials (type and properties), and they should have governed the work. In the following examination of debris, the material properties observed are compared with those indicated on sheet 1 of drawing 4607K070.

Note that formwork shoring sketches for the floors lower than 39 were furnished by Vincar Construction Services of Roslyn Heights, and these were in fact sketches prepared by Howard Shapiro and Associates for another site.

Photo 4 Typical Tag and Failure at Connection
Note Name, Zone and Signature

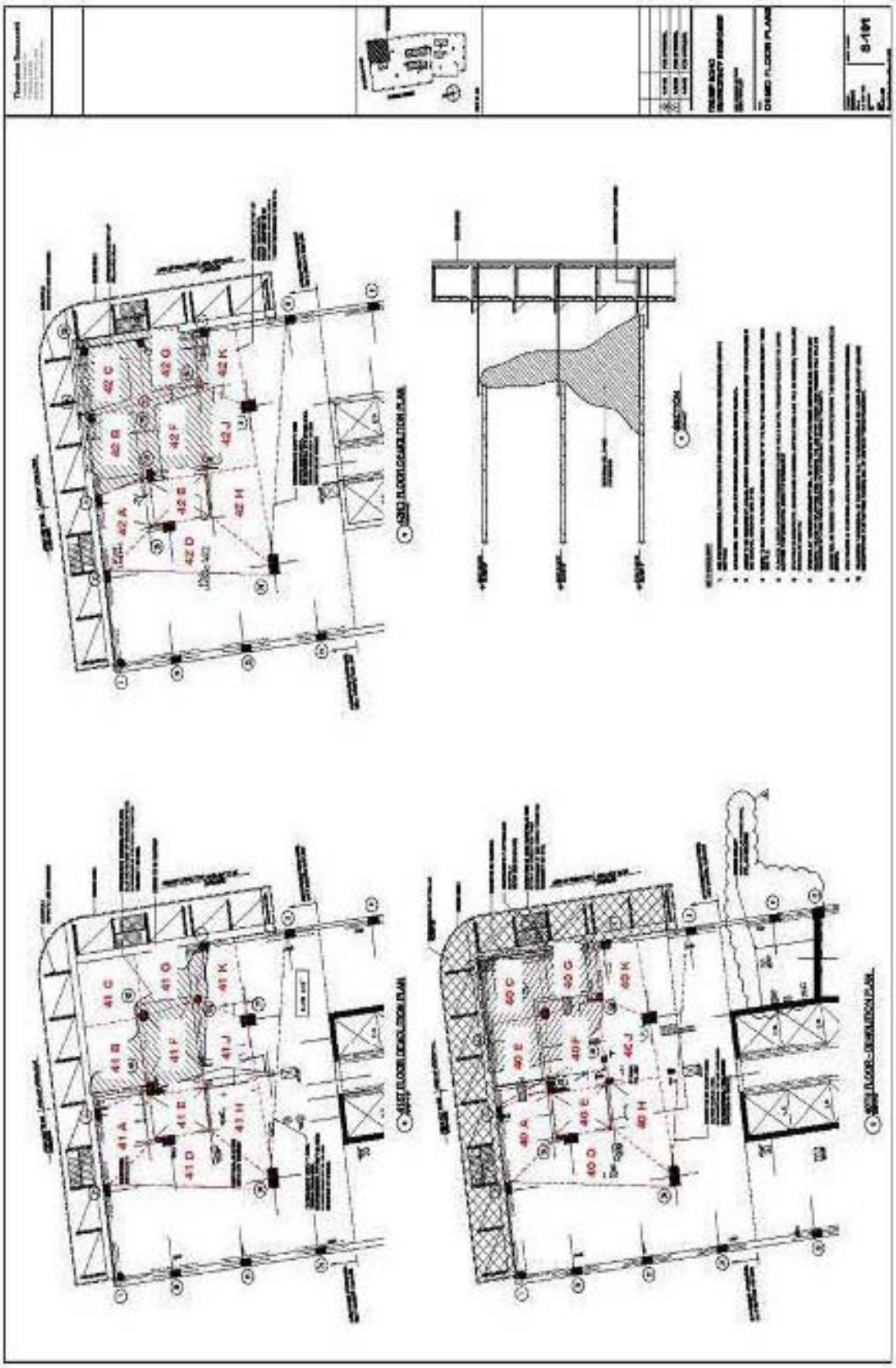


Figure 1 Evidence Collection Zones

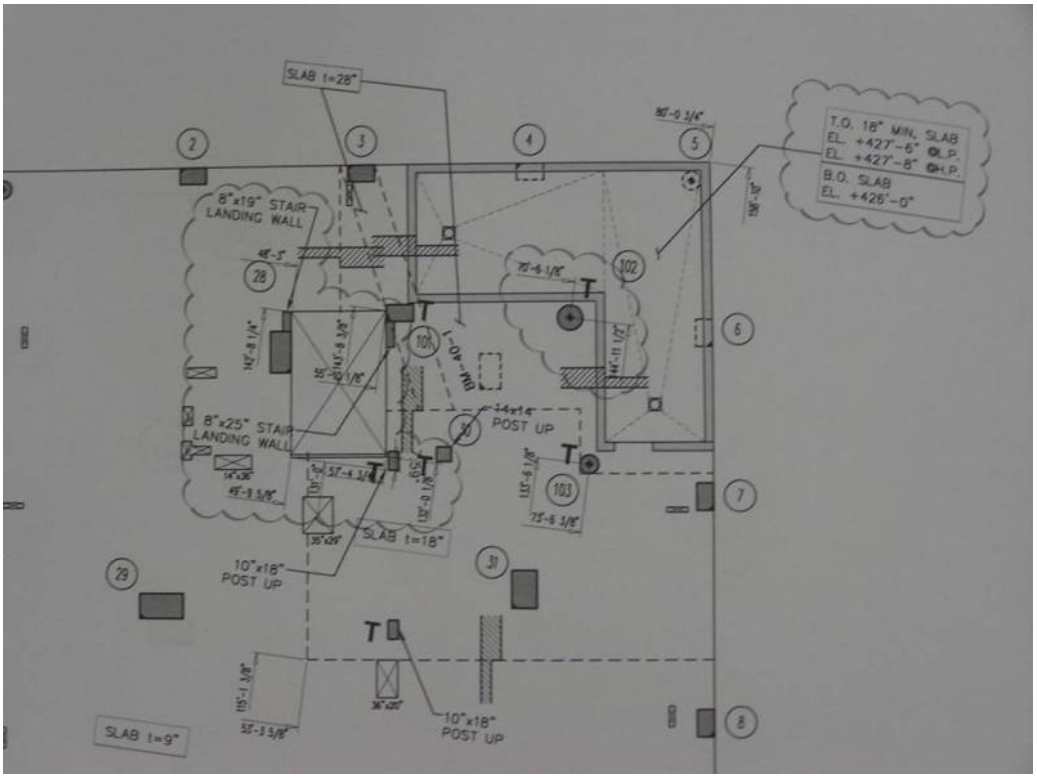


Photo 5 Partial Plan Concrete at 40th Floor by DeSimone Consulting Engineers

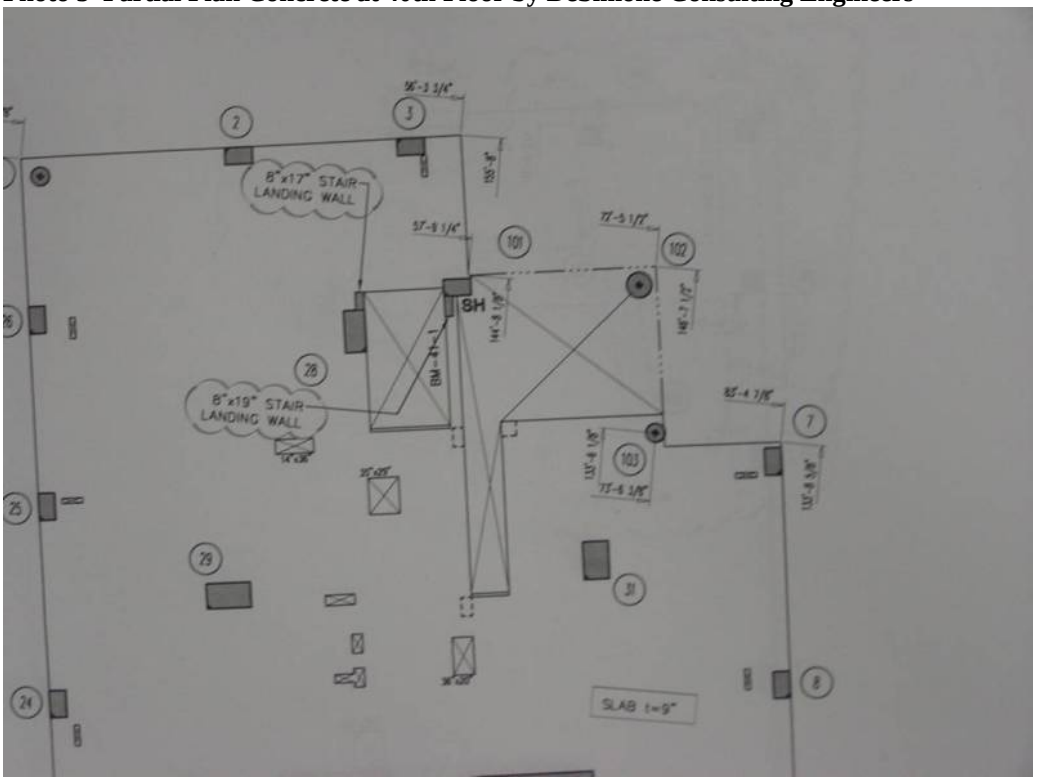


Photo 6 Partial Plan Concrete Floor at 41st Floor by DeSimone Consulting Engineers -

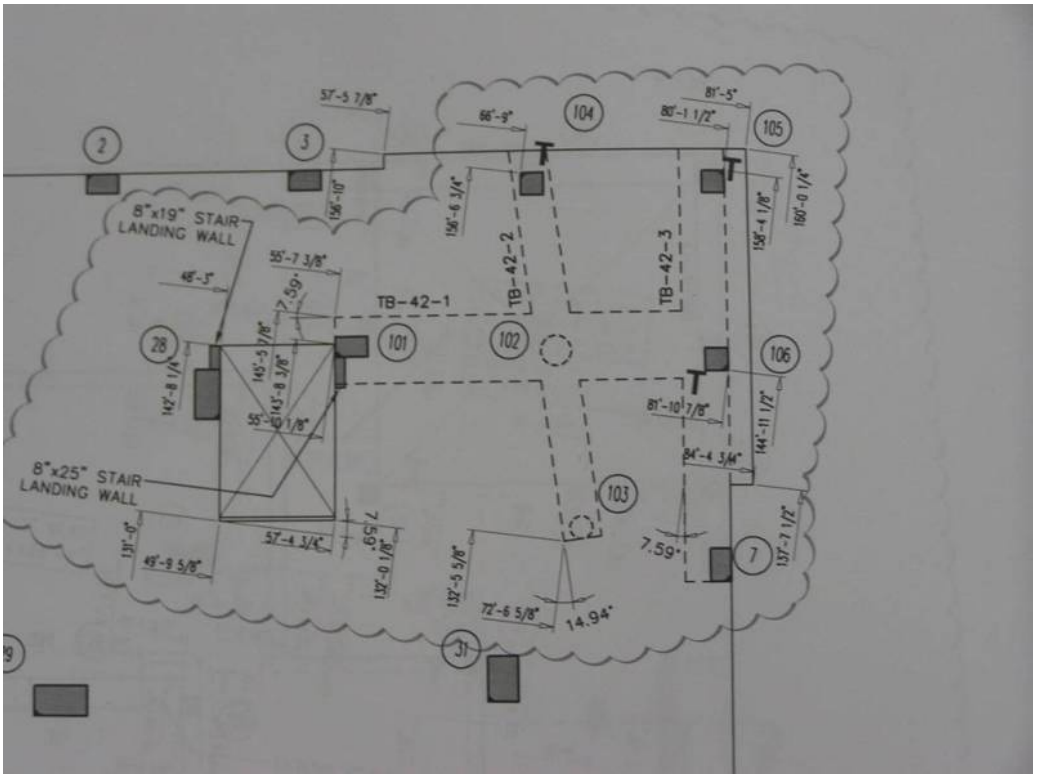


Photo 7 Partial Plan Concrete at 42nd Floor by DeSimone Consulting Engineers.

3 Examination of Debris

3.1 Examination of Elements of Formwork and Shoring Debris

The lack of drawings (see discussion at 2.5) made the reconstruction extremely difficult. In the following paragraphs the material type and size make-up for the main formwork and shoring elements are identified. The characteristics and the failures are described. The comments on proper installation or material adequacy are derived from comparison with the instructions in the 4607K070 Sheet 1 for the typical slab formwork. The findings discussed in Chapter 4 refer to the installation and failure of constituent materials. The discussion of the formwork shoring as a system is presented in Chapter 5.

3.1.1 Patent Shores

The shoring supporting the plywood used Patent Shoring Systems modules 20KA. This signifies that a shore frame had a base-rated capacity of 20,000 lbs (20 kips) and that the material was aluminum. A four-leg shore tower was rated to 40,000 lbs (40 kips). The system is assembled using frames that are manufactured in several heights. The typical frame used had a 4 ft width. The system also includes connectors, top and bottom plates and adjustment legs, all furnished by Patent.

3.1.1.1 Instructions/Specifications

The instructions for the installation of the typical tower are contained in the manual provided by the manufacturer. The drawings for the typical formwork also contained instructions that essentially reproduced those in the manufacturer's manual – there were no significant contradictions. In any event, the more restrictive requirement should have always controlled.

One drawing note in particular merits mention. On Sheet 1, a note indicates a 12 in. maximum leg extension for a capacity of 10,000 lbs (see Photo 19). It is not clear whether the extension is limited to this dimension for all leg extensions, or only when such capacity is required. It is the opinion of the investigators that the engineer who prepared the drawings meant that 12 in. was the extension for a certain capacity (specifically, 10,000 lbs.), not for all capacities. The investigators so conclude because the design engineer had the obligation to establish the parameters under which the tower was to work (see also 6.1.2). The aluminum frame here (two legs) was rated at 20,000lbs. The installer had no other guidance for installation and had no way to determine on his own what the rating would be for a larger extension.

3.1.1.2 Failure Modes by Shore Components

<u>Tube Frame</u>.
The investigation found several modes of failure. The most common were:

- Failure of the tube next to the weld connecting the horizontal or diagonal elements to the vertical legs. While a special failure analysis of the weld was not performed, it appears that in most cases the tube walls sheared at the weld. In one or two cases there might

have been separation of the weld from the tube material (see Photo 4 Typical Tag and Photo 10 Fracture Near Weld).

- Failure of the tubes (diagonal or horizontal). This might have occurred at connections of horizontals with diagonals or in the aluminum tube at the weld line to the vertical leg (see Photo 8 –Failure Modes).

Failure of the vertical leg was less common, but several cases were noted where the leg sheared at the level of the connecting pin. Buckling of the vertical leg was not noted.

Braces

A large number of diagonals were bent out of shape, and in a few cases the diagonal fractured in the area of the connection hole.

Photo 8 Failure Modes

Adjustment Screw Extension Legs

The investigation noted several cases where the extension legs fractured at the line of insertion to the vertical aluminum leg (see Photo 12 Fractured Adjustment Screw Failed extension leg. Note lack of attachment of wood joists to stringer).

Top Plates

Several top plates were found bent (see Photo 9 Bent Top Plate). Also, two or three locking cams were found broken.

3.1.1.3 Actual Installation

The investigation did not identify any major issues with regard to the assemblage of the tube frame into a tower. The extension of the leg adjusting fillet tube reached in some cases 20 in. when measured from the top of the aluminum tube to the top of the plate.

Photo 9 Bent Top Plate

Photo 10 Fracture near Weld

Photo 11 Adjustment Screw with Extension Over 12"

Photo 12 Fractured Adjustment Screw

3.1.2 Aluminum Stringers

3.1.2.1 Instructions/Specifications

The stringers (or joists) were aluminum type, manufactured by Patent. Combinations of various lengths of stringers were used (10, 12 and 16 ft).

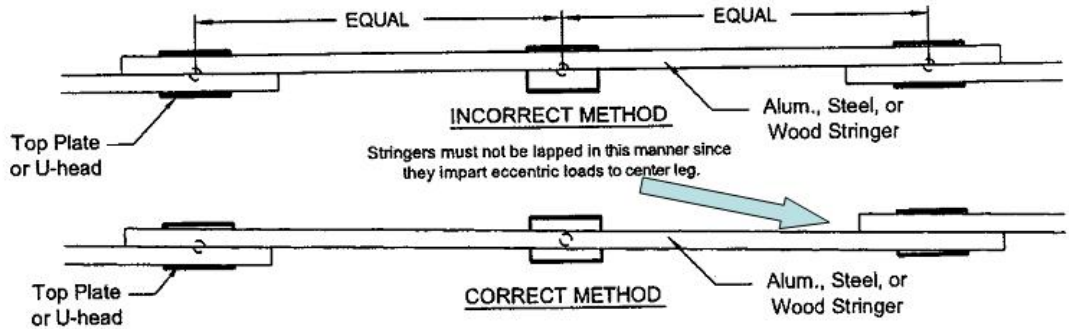


Figure 2 Stringer Setting - Patent Instructions

Photo 13 Stringer Set Eccentric on Head Does Not Follow Patent Instructions

3.1.2.2 Failure Modes

Only a few fractured aluminum stringers were observed, but several bent or torsioned stringers were found in the pile.

3.1.2.3 Actual Installation

In some cases it was observed that the stringers had not been fastened with clamps at the plate support. The layout and splicing of stringers on the shoring plate is discussed at 4.2 and verified in calculations at Appendix A 3.2.2 and 3.3.2. See also Photo 13. The stringers on the east side of the area had been attached with perforated metal bands to the 4x4 wood underneath.

Photo 14 Stringer Supporting Stringer

Photo 15 Improper Stringer Setting

3.1.3 Wood Joists

3.1.3.1 Instructions/Specifications

Drawing 4607K070 Sheet 1 has the following notes for Lumber Design values:

Suggested lumber details shown are based on the use of lumber with allowable unit stresses increased per ANSI/AF&PA NDS 1997 for short term loading to the limit values below:
Extreme fiber stress in bending…1640 psi
Horizontal shear …180 psi
Modulus of Elasticity 1,600,000 psi

3.1.3.2 Failure Modes

The investigators observed that the most common failure of the 3x4 lumber was at the edge of the underlying aluminum stringer. The Wood Advisory Services investigation report (Appendix B) noted that the 3x4 dimension lumber had *a high percentage of brashness which is associated*

with wood decay and/or low specific gravity values. The B (brash) failure mode was associated with 20% of the 3x4 lumber, and the BT (combination brash and tension failure) was observed in 42% of the samples.

3.1.3.3 Actual Installation

The Wood Advisory Services investigation report (Appendix B) concluded that the lumber had *a high percentage of low grade material*. Several pieces (7%) were classified as Economy, that is, with no established structural properties. About one quarter of the lumber (23%) was visually graded NO 3.

The report classifies the lumber as Spruce-Pine-Fir (S_P_F) mill run from Canada. The published allowable stresses for the lumber as graded by Wood Advisory Services are significantly below the specifications.

3.1.4 Plywood

3.1.4.1 Instructions/Specifications

Drawing 4607K070 Sheet 1 requires:

Face grain of plywood must run at right angles to its support. Plywood suggested in the layout assumed to be APA plyform Class I, B-B exterior type PS i-95 or equal. Costumer[sic] must make allowances for lower grades or condition of plywood used.

3.1.4.2 Failure Modes

The typical mode of failure of the plywood was through bending at locations corresponding with the failure of the supporting dimension lumber underneath (see Photo 11). In several locations, punching of the plywood was observed. Local failure at edges was observed also.

3.1.4.3 Actual Installation

Per Wood Advisory Services, the installed plywood was a 5-ply with melamine on both sides marked “Feldman Lumber” or “Mid-South Lumber Company”. The Mid South Lumber ply met the specification, while the observations made on the Feldman Lumber product were inconclusive.

3.1.4.4 Round Column Formwork

The round column formwork at column 102 was Poli New Form, as manufactured by Newark Products (see Photo 16). The rest of the column forms on the site were Sonotube formwork. The choice of different types of round formwork was probably determined by the fact that Sonotube does not manufacture forms taller than 20 feet. Both formwork manufacturers represent their products as calculated to resist the pressures produced by freshly poured concrete. Although, based on the manufacturer literature, the formwork does not appear to need any stiffening, as the stresses are equalized in loops, the usual practice is to stabilize the formwork against possible lean or separation from the horizontal forms. Here, the system was reinforced by vertical pieces of wood tied together with wire. The investigation found the bottom nine feet of the round paper form not torn. The proper practice required that concrete had to be poured in the column prior to the slab pour and vibrated as well.

Photo 16 Round Formwork

3.1.4.5 Beam Formwork

The beam formwork was composed of plywood reinforced with 3x4 wood ribs and kept together with Meadow-Burke ties set at 30”. Some of the forms (the 3 sides) were found in the debris. The investigation was not able to recover intact formwork in significant amounts (see Photo 17). The investigation also could not reconstitute the means of support (if any) of the top of the beam side formwork (where it meets the horizontal forms). The snap tie hardware is from Meadow Burke, with the ties having a diameter of .22”. The ties are attached via wedges to pairs of 3x4 joists (see Photo 18 –Beam Formwork –Ties and Ribs).

.

Photo 17 Beam Form (Upside-down)

Photo 18 Beam Formwork -Ties and Ribs

4 Reconstruction of Formwork and Shoring system

Although the formwork shoring system was installed without following any drawings (3.1), based on the examination of the physical evidence, the investigators are confident that their reconstruction of the formwork and shoring (described in this chapter) is very close to what existed prior to the accident.

4.1 Absence of Design Documents

The Building Code 27-1035(c) (in effect at the time of this accident) requires that formwork related drawings and design be prepared by a licensed engineer, but it does not require that such drawings and design be submitted to DOB. A set of drawings for formwork was found on site. The set was identified by a drawing number, 4607K070, under the title "20KA Shoring Layout Project: Soho Hotel; Location 246 Spring Street; Customer: DiFama Concrete." The set contains four drawings, only the first three of which are signed and sealed (Professional Engineer - Michael Salvatore D'Alessio). The drawing title block differs only by the sheet numbers. The first sheet contains general notes; Sheets 2 and 3 contain plans and sections for shoring at the 40th and 41st floors. The unsigned drawing (Sheet 4) contains plans and sections for the 42nd floor.

The investigators established that the shoring system that collapsed had an intermediate plywood "mud floor" at the 41st floor level of the building and thus bore no resemblance to the plan on the unsigned Sheet 4, nor to the General Notes and Instructions on the signed and stamped Sheet 1 (specifically, to the material type and properties specified).

4.2 Horizontal Layout of Shoring

The shore towers supporting the formwork were assumed to have maintained the spacing and alignment that was found in place in the non-collapsed areas. For the shores under the 42nd floor, the alignment had to be maintained, since the stringers discovered with one end supported in the non-collapsed area had to have been supported along the same centerlines (otherwise their ends in the collapsed area would not have been supported at all).

Because shore legs were found embedded in the concrete that had flowed down during the collapse, the actual location of the shores supporting the 41st floor sheeting was precisely established for many towers.

As mentioned above, the aluminum stringers in the area had different lengths (10, 12 and 16 ft). While the stringer centerline plan position (alignment) was established with good reliability, the identification of each stringer length location is less definitive (see Figure 3 Plan Shoring 41st Fl. and Figure 4 Plan Shoring 42nd).

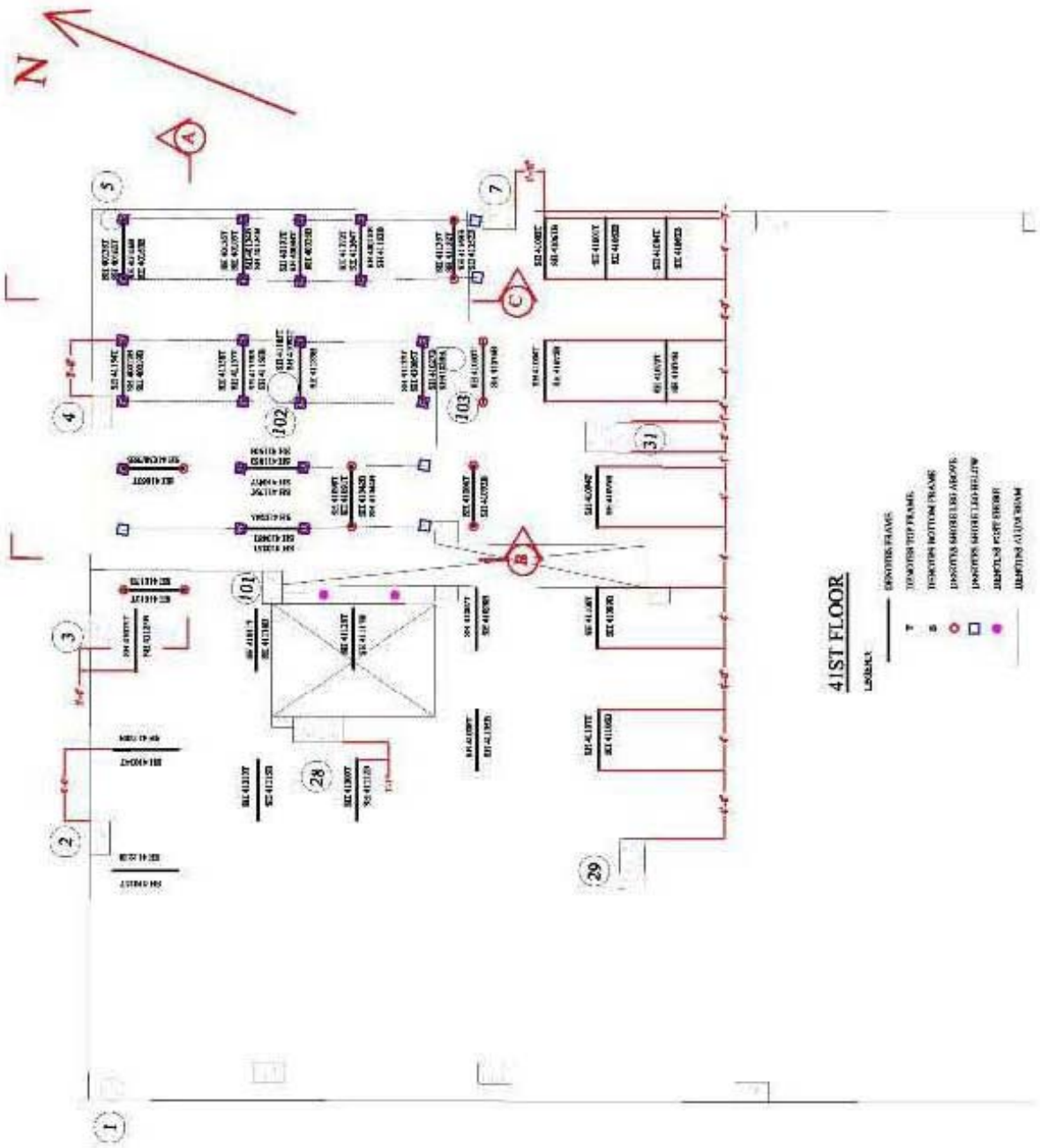


Figure 3 Plan Shoring at 41st Fl. Reconstruction

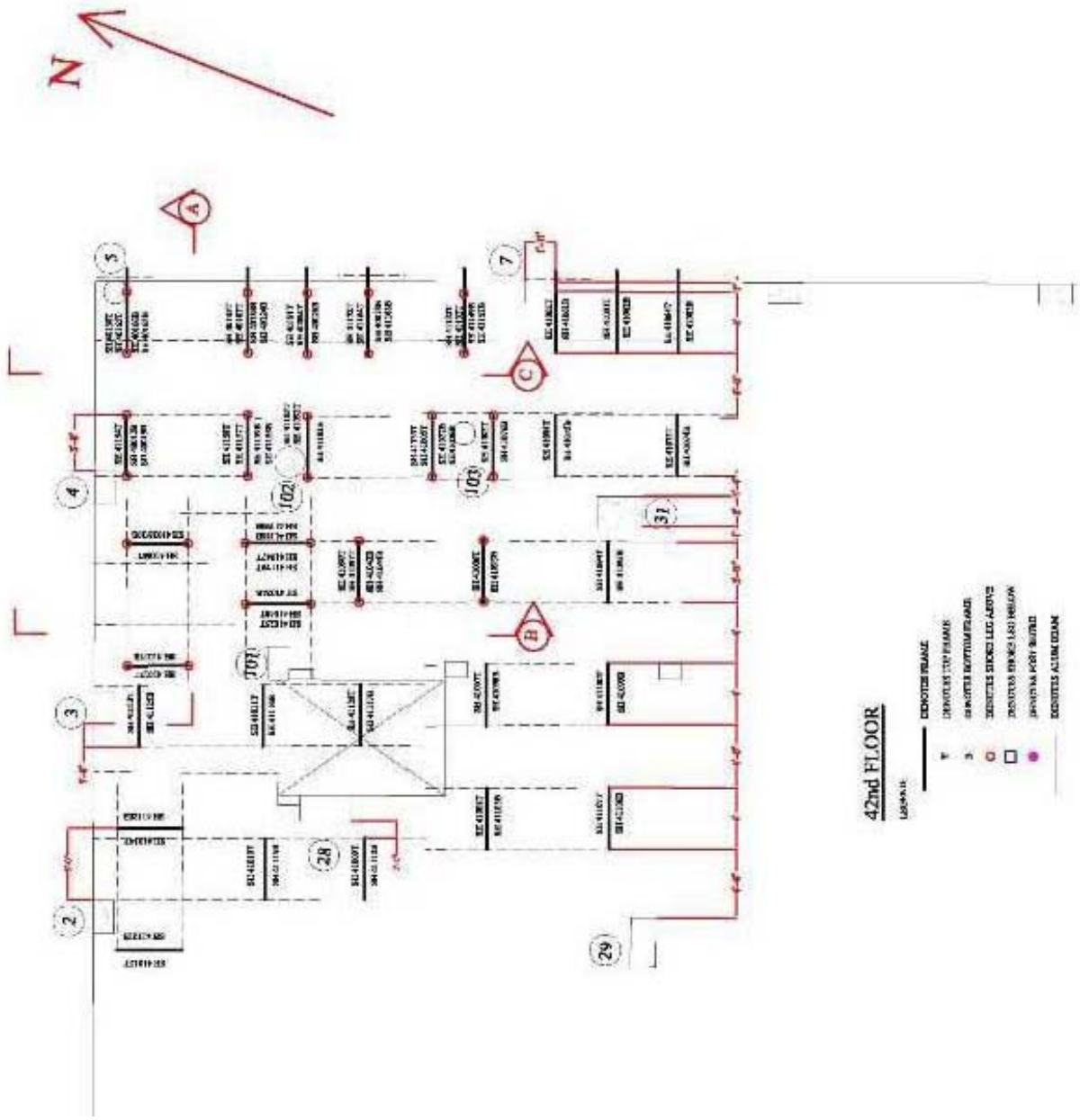


Figure 4 Plan Shoring at 42nd Fl. Reconstruction

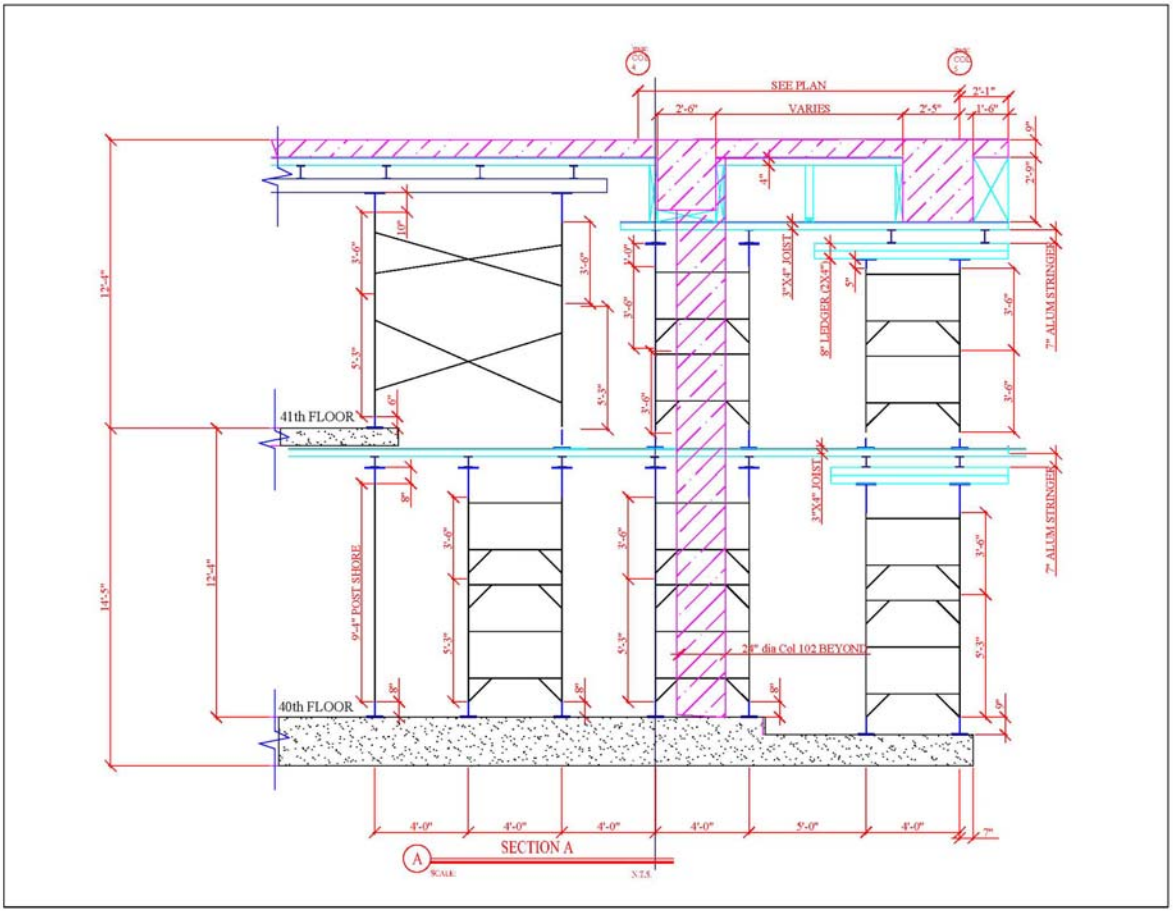


Figure 5 Section A Reconstruction Shoring (Looking North)

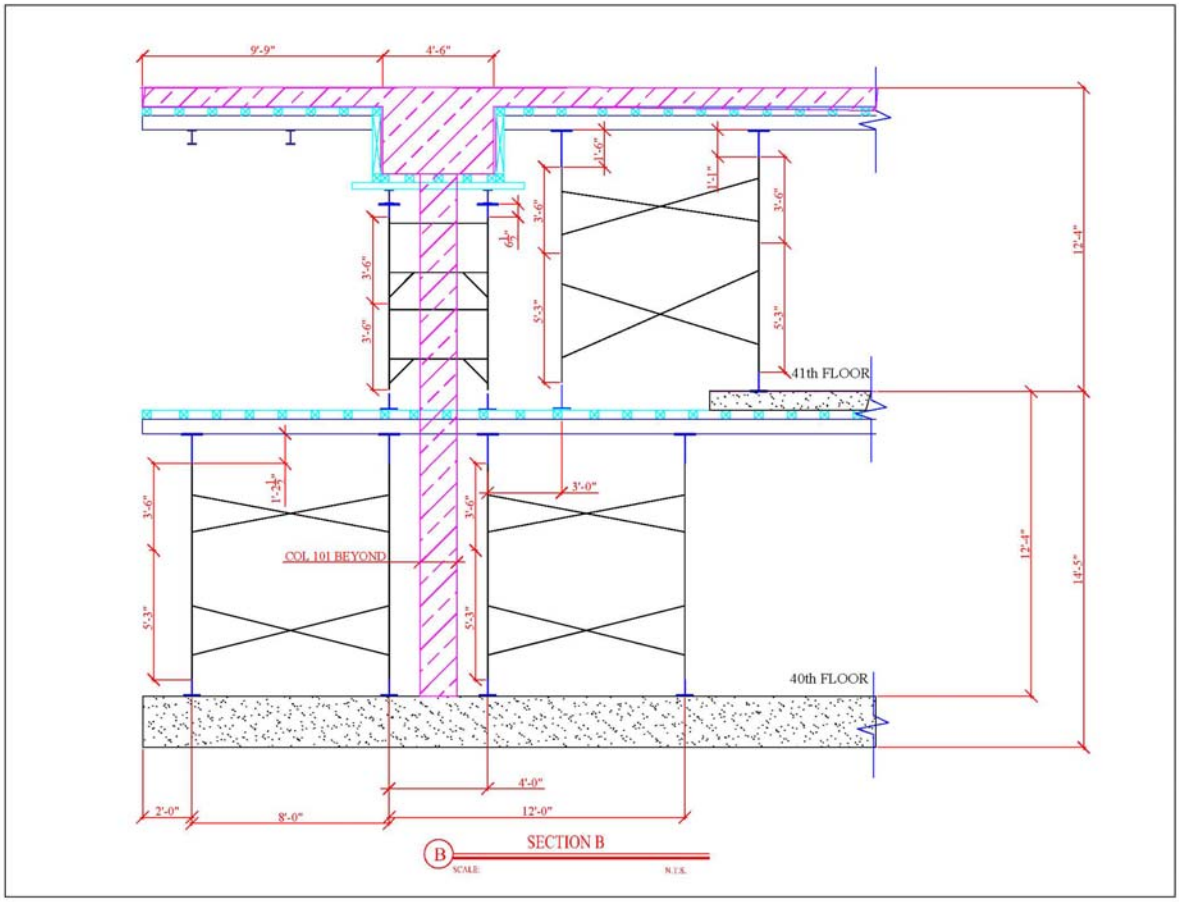


Figure 6 Section B. Reconstruction Shoring (Looking East)

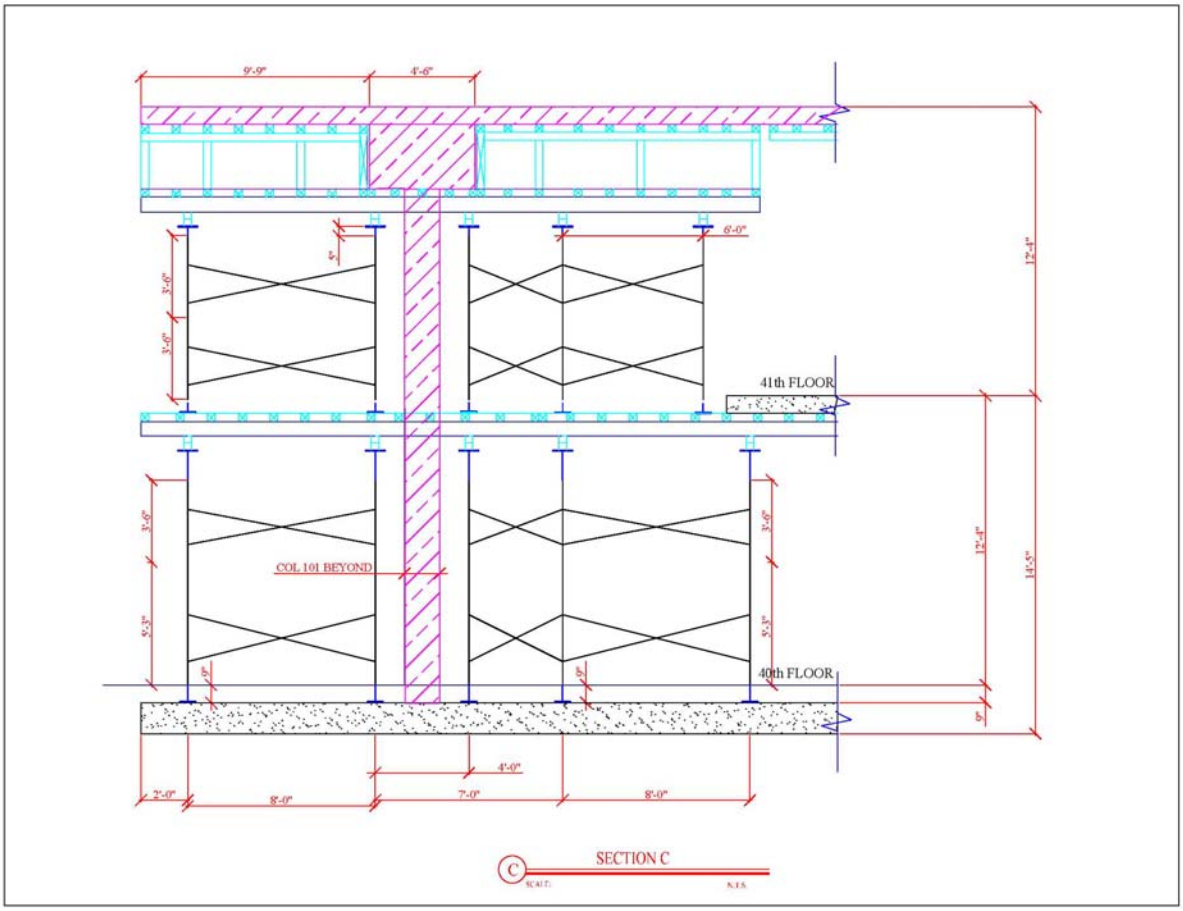


Figure 7 Section C. Reconstruction Shoring (Looking East)

Every drawing in the 4607K070 set contains instructions and a sketch indicating that the stringers shall be set at an angle when supported by more than two posts. This is clearly intended to ensure a concentric application of the load on the middle post. Such an arrangement, following the instructions, was not found at any location on this entire site.

4.3 Vertical Layout of Shoring

The elevation reconstruction was based on geometrical considerations and took into account the given location of the concrete beams. The only element that we inferred based on limited evidence was the support of the slab formwork between the concrete beams. The number of shores and their height resulted from the investigators' accounting/reconstruction work. The shores' heights reached only to the bottom of the beams. Although the evidence is not overwhelming, we indicate that the support was obtained by short wood stubs, which is a common shoring method.

The leg extension shown in our drawings is not based on actual field measurements, but rather on the elevation difference. Field measurements were recorded for each leg and usually vary from 8 to 14 inches (in some cases exceeding).

4.4 Formwork Shoring System

The formwork shoring system—as revealed by the investigation—is shown in Figures 3 -7. In essence, DiFama's personnel supervising the formwork support system installation had the workers create a supported plywood platform at the 41st floor, on top of which a supporting system was erected for the 42nd floor formwork. The supporting 41st floor platform was similar in construction to a flat slab formwork. In the area of interest, the top of the 40th floor concrete slab was slightly sloped due to a rain drainage system.

The formwork for the 42nd floor was more elaborate because of the presence of the heavy transfer concrete beams. On the east side, the shoring system cantilevered about 2 feet via two timbers strapped with bands to the aluminum towers. Aluminum stringers were set on top of the stringers without any clamps. In addition, wood joists were rarely nailed to the stringers. As a result, the stability for this system was dependent in a large proportion on friction.

5 Adherence to Regulation Covering Formwork[1]

There are several sets of requirements regulating the concrete formwork. The contractor and the design professional were required to conform to, among other things, the standards set forth in the New York City Building Code of 1968. The contractor was also subject to, among other things, the rules and regulations enforced by the Occupational Safety and Health Administration ("OSHA"). The regulations quoted below are from Subchapter 19 of the Building Code of 1968. The contractor was also required to follow the instructions provided by Patent, the shoring manufacturer. The Patent shores should not have been expected to function properly if they were used in a manner that they were not designed for.

5.1 New York City Building Code

5.1.1 General Requirements for Concrete Formwork

The New York City Building Code of 1968 has specific and relatively detailed instructions for concrete formwork in § 27-1035, "Concrete formwork". These instructions are similar with those of the American Concrete Institute (ACI) and cover the construction, inspection and design of the formwork and supporting elements. Unless otherwise noted, the code paragraphs cited in Ch. 5.1 are cited from §27-1035.

(a) General requirements.-
(1) Formwork, including all related braces, shoring, framing, and auxiliary construction shall be proportioned, erected, supported, braced, and maintained so that it will safely support all vertical and lateral loads that might be applied until such loads can be supported by the permanent construction.

DiFama failed to comply with the section (a) above for the pour taking place on January 14, 2008 since the shoring collapsed. That is, it did not "safely support all vertical and lateral loads that might be applied." In fact, our investigation found that the shoring system was not tied together or braced, and the system did not conform to any existing design drawings as required in §27-1035 (a)(3) (*Forms shall be properly braced or tied together so as to maintain position and shape, and shall conform to the sizes and shapes of members as shown on the design drawings*).

5.1.2 Inspection Non-Compliance

The mandated inspection of formwork provided for in the Building code section 27-1035 requires verification that the actual field installation conforms to a preexisting, engineer-designed drawings or instructions**.** The relevant sections provide:

(b) Inspection.-
(1)…. *In addition, such forms shall be inspected for conformance with the form design drawings, when such drawings are required by the provisions of subdivision (c) of this section; and/or conformance with the provisions of this section. Such inspections may be made by the person superintending the work.*

[1] All code citations in this report refer to codes in effect at the time of this accident.

...
(b)*(3) A record of all such inspections shall be kept at the site available to the commissioner, and the names of the persons doing the inspecting and the name of the foreman in charge of formwork shall be posted in the field office.*
...
(d)(5*) Any unsafe condition or necessary adjustment revealed by inspection shall be remedied immediately. If, during construction, any weakness develops and the falsework shows any undue settlement or distortion, the work shall be stopped, the affected construction removed if permanently damaged, and the falsework strengthened.*

In this case, the floor to floor distance was over 24 ft. and shoring drawings and calculations were absolutely necessary. Moreover, we do not have records of an inspection of this particular two story stack shoring system, nor is DOB aware of evidence that any inspection was performed at all. In any case, if an inspection had been performed, the inspection required by the Building Code §27-1035 would have had no basis of verification because the only available (non sealed) design was not consulted. In addition, DiFama did not follow the manufacturer's instructions for installation.

5.1.3 Construction Non-Compliance

The code is specific in requiring that the shoring be braced. Each of the towers was braced internally for stability, but the bracing of the entire system would have required a positive attachment of the horizontal wood system at the 41st floor. We did not find any evidence of such attachment. Section 27-1035(a) (2) of the Building Code provides:

(a)*(2) Vertical shores for multi floor forms shall be set plumb and in alignment with lower tiers so that loads from upper tiers are transferred directly to the lower tiers, or adequate transfer members shall be provided. Provision shall be made to transfer the lateral loads to the ground or to completed construction of adequate strength.*

Further, the installation of wood headers was contrary to section 27-1035(d)(3):

(d)(3) Vertical shores shall be so erected that they cannot tilt, and shall have firm bearing.

If DiFama had intended to align the shores above the 41st floor platform with the shores under this platform, the lack of direct visual reference points would have made it complicated to execute. Even if this alignment had happened, there was no direct transfer of forces in some cases or use of *firm bearing*. The legs on the top floor shoring should have been set on top of wood blocks. The 12 to 16 inch spacing between the wood joists allowed the possibility of the top leg falling in between the joists. As Wood Advisory Services reports (based on field findings at several locations) the plywood was punched by the leg.

5.1.4 Design of Concrete Formwork - Non-Compliance

The investigation found that the formwork was installed without a design, although the code clearly requires one:

27-1035 (c) Design of concrete formwork.-
Wherever the shore height exceeds fourteen feet or the total load on the forms exceeds one hundred fifty psf, or wherever power buggies or two-stage shores are used, the forms, including shoring foundation, shall be designed as provided in section 27-1015 of article one of this subchapter, and shall be constructed in conformance with such design. Formwork drawings shall be prepared. The allowable stresses for design shall meet the requirements of subchapter ten of this chapter. A copy of the design drawings and any construction drawings and specifications shall be kept on the job available to the commissioner.
(1) VERTICAL LOADS.-Vertical loads shall include the total dead and live loads. Dead load shall include the weight of formwork plus the weight of the reinforcement and fresh concrete. Live load shall allow for the weight of workers and equipment, with allowance for impact, but in no case shall less than twenty psf be allowed.

Our calculations verified that the number of shoring towers were sufficient in number to carry the vertical load (see Appendix A 2.1 and A3.1). The noncompliance with the Building Code §27-1035 (c) (3) instructions (listed below) is discussed in 7.3.2 and the engineering calculations 3.2.1.

a. Braces and shores shall be designed to resist all external lateral loads such as wind, cable tensions, and inclined supports, dumping of concrete, and starting and stopping of equipment.
b. In no case shall the assumed value of lateral load due to wind, dumping of concrete, and equipment acting in any direction at each floor line be less than one hundred plf edge or two percent of total dead load of the floor, whichever is greater.
(3) EXTERNAL LATERAL LOADS.- a. Braces and shores shall be designed to resist all external lateral loads such as wind, cable tensions, inclined supports, dumping of concrete, and starting and stopping of equipment.
b. In no case shall the assumed value of lateral load due to wind, dumping of concrete, and equipment acting in any direction at each floor line be less than one hundred plf edge or two percent of total dead load of the floor, whichever is greater.
c. Except for foundation walls that are poured against a rigid backing, wall forms shall be designed for a minimum lateral load of ten psf, and bracing for wall forms shall be designed for a lateral load of at least one hundred plf of wall, applied at the top. The lateral load acting on walls greater than fourteen feet high shall be determined by analysis of conditions applicable to the site and building.
(4) SPECIAL LOADS.-The formwork shall be designed for any special conditions of construction likely to occur, such as unsymmetrical placement of concrete, impact of machine-delivered concrete, uplift, and concentrated loads.
(5) SHORING AND BRACING.- a. When patented or commercial devices that are not susceptible to design are used for shoring, bracing, or splicing, they shall be approved.
b. Splices shall develop the full strength of the spliced members.
c. Where shore height exceeds ten feet, or when necessary to provide structural stability, diagonal bracing shall be provided. Struts, anchored into masonry or to panel joints of adjacent braced bays, may be used to prevent buckling of individual members not supported by the diagonal bracing; but, bracing an entire tier of shores with struts without diagonal bracing will not be permitted unless the system can be demonstrated to be braced by other rigid construction.
d. The unbraced length of shores shall not exceed the maximum length determined in accordance with the applicable reference standard in subchapter ten of this chapter for the structural material used.
(6) FOUNDATIONS.-Foundations for shores more than ten feet high and supported on the ground shall be designed.

(7) SETTLEMENT.-Falsework shall be so constructed that vertical adjustments can be made to compensate for take-up and settlements. Wedges, jacks, or other positive means shall be provided for this purpose.

5.1.5 Use of Plywood "Mud Floor" at the 41st Floor as a Construction Platform

It is not clear what the operational purpose of the 41st plywood floor was, but if it was intended for worker circulation, one could interpret this platform as being access scaffolding. But in this case the installation did not follow Patent instructions on planking. Sheet 1 has notes requiring *"For access scaffolding defined as a temporary elevated platform and its supported structure…used to support users and materials, or both"* that all sawed scaffold planks be of a "*scaffold plank grade and shall be certified and bear the stamp grade of a grading agency*." These requirements match those from the Building Code RS 19 27-1044 (c).

5.2 Scaffold, Shoring and Forming Institute

The instructions provided by the Scaffold, Shoring and Forming Institute (SSFI) are not mandatory. But they were referenced and quoted by Patent in the general instructions and specific instructions. Accordingly, the investigators studied them. The instructions quoted by Patent from SSFI are basic. Among them: *"A shoring layout shall be available at the job site at all times*" (as quoted by Patent in brochure SS670R1).

5.3 OSHA and other National Engineering codes.

The OSHA requirements are mandatory for any construction site. The regulations for formwork are found mainly in OSHA Construction Standards, Part 1926, "Subpart Q, Concrete, Concrete Forms, and Shoring". Since OSHA performed its own assessment of the accident, this report will not cover the lack of compliance with OSHA's requirements.

Other instructions for design and installation of formwork are set forth in ACI 347-04: Guide to Formwork for Concrete.

6 Adherence to PATENT Requirements

Patent Construction Systems, a division of Harsco Corporation, is the manufacturer and lessor of the shoring towers and stringers used for this concrete operation. Patent also provided drawings for the installation of the formwork support system. As such, the installation should have followed both Patent's shoring manufacturer and shoring specific design instructions.

6.1 Shoring Manufacturer's General Instructions

Patent's brochure for the products at issue is entitled *Design and General Notes, Specifications and Typical Details for Patent's SS670R1*. The brochure provides general technical and safety instruction for a series of shores, including 20KA Shores (the type used on this site).
The brochure is organized as follows:

- Frame Shoring Safety Rules that reference SSFI. These are described as "common sense" rules and require the existence of a shoring layout. For most cases the rules indicate that the installer should refer to local codes or to an engineer. They also require inspection of the shoring prior to pouring concrete. The implication is that the formwork and supporting system needs to be inspected to meet drawings and instructions.
- Instructions generated directly by Patent. These are divided into General Frame Notes, Typical Stringer Details, and Stability and Lateral Force Consideration on Shoring Towers.
- Specific allowable loads for each type of shore under different usage conditions.

6.1.1 Patent-Specific Instructions - Publication SS670R1.

Below are excerpts from the Patent-specific instructions and the General Frame Shoring notes as well as our observations during the post-collapse investigation. The serious implications of noncompliance with these are discussed elsewhere.

3. The shoring installation must comply with safe practice and with the requirements of governmental regulations, codes and ordinances.
4. Contractor shall design suitable sills to properly distribute the imposed shoring loads.

Only a few sills were found. The lack of sills over the plywood at 41st floor is the discussed at 7.2.4

7. The formwork must be stabilized to poured columns or walls. The layout as shown is designed with the provision that the formwork system is restrained from lateral movement with respect to shoring. The contractor shall provide sufficient lateral support as necessary.

There was no restraining of framework or stabilization to poured columns or walls at 41st floor level. The lateral bracing for towers to control and transmit horizontal loads was not found. In only one instance did the investigators find a long bar that might have been used for that purpose.

12. Imposed shoring loads are computed as applied concentrically to vertical support member, whether frame legs or single post shores. Ledgers must be centered laterally and ledger joints butted or lapped centrally over the vertical support members.

15. Ledgers and stringers must be centered, butted or lapped centrally over their vertical support members.

The investigators found that the manufacturer requires the top stringer to be set at a slight angle to ensure centric loading of the shore legs (see Figure 2 - Stringer Setting –from Patent Instructions and Figure 8 Lapping of stringers – from Patent Instructions). The recommended type of setting stringers was not found at any location, and the violation of this instruction was the subject of detailed engineering analysis

The inspection revealed several cases where the cam was not locking the stringer. This condition was observed both at shoring that was still standing and at failed elements. In the investigators' opinion the cam could not have become loose as a result of the accident.

6.1.2 Allowable loads

Calculations based on Building Code instructions show that the vertical allowable loads indicated on Sheet 1 and in the Publication S670R1 for shoring 20KA were met; likewise with regard to the loads for the stringer. The instructions for 20KA shoring require that specific calculations be performed for ensuring stability against lateral loads. The manufacturer does not indicate any minimum lateral loads, but in other notes it defers to local codes.

The typical drawing S1 and Publication SS670R1 indicate various reductions in tower capacity as a result of increased extension of the tower leg (exposed thread). Note that the manufacturer's instruction allows extensions of the leg, but Patent, as the designer of record for the shoring, indicated on Sheet 1, that "extension shall not exceed 12" for 10,000 lbs." The investigation found this dimension exceeded. In Photo 11 the screw extension is between 15 and 16 inches. The report analyzed in detail the results of this weakening (see Appendix A). Since the shoring installer is not supposed to estimate loads, and would not know if the 10,000 lbs value was reached or not, 12 inches should be in fact considered as the maximum allowed extension for this job.

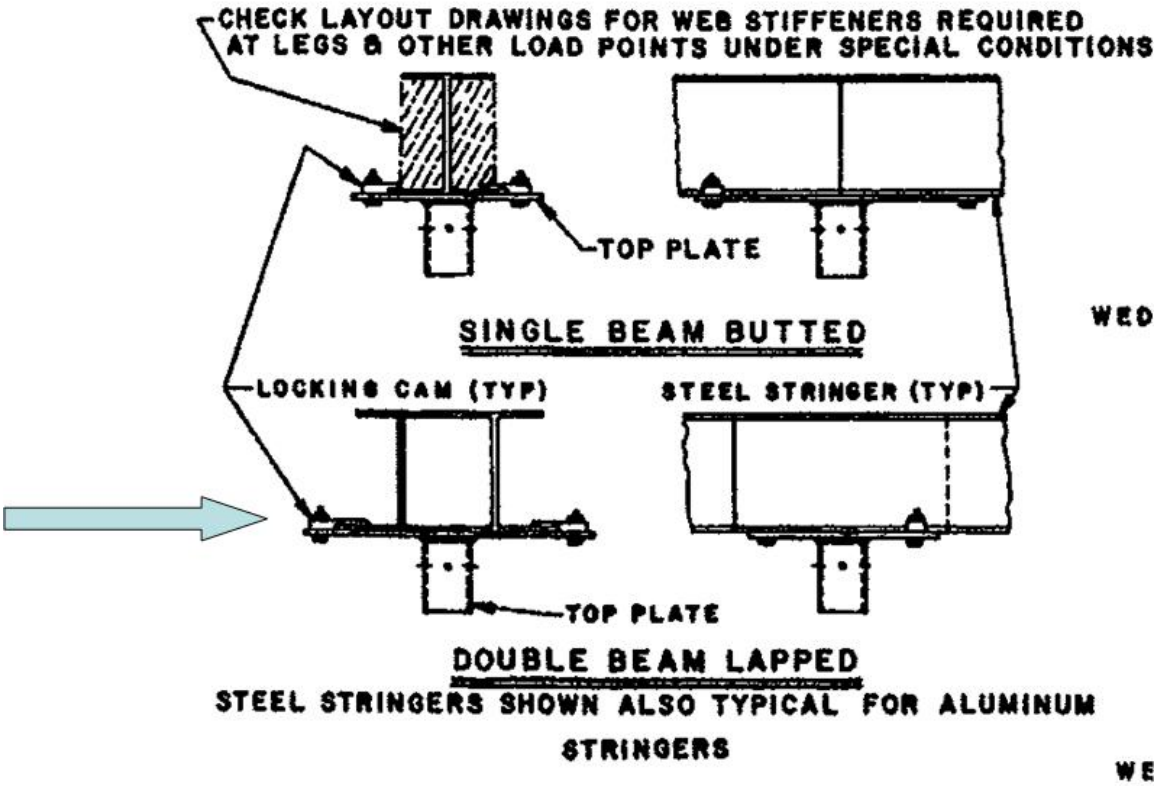


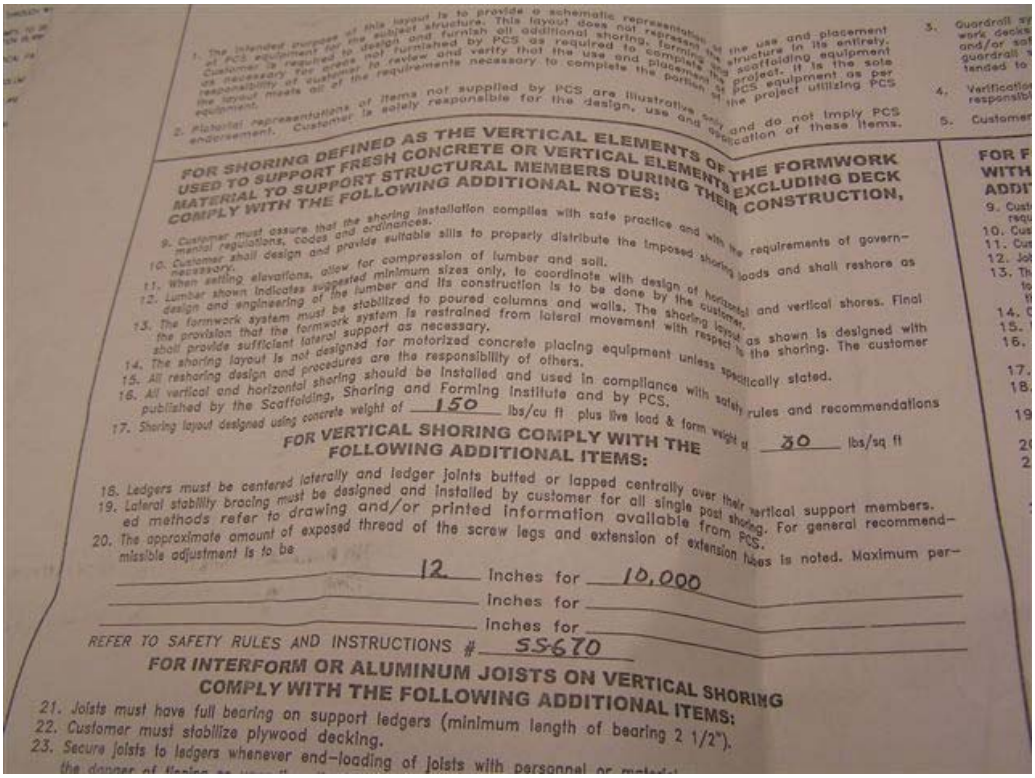
Figure 8 Lapping of Stringers - Patent Instructions

6.2 Patent Drawings for 246 Spring Street

We have analyzed four sheets prepared by Patent Construction Systems. The drawings have the same number, 4607K070, but are differentiated by sheet number. All drawings have the date 9/25/07 in the title block. Sheets 1, 2 and 3 are stamped by Professional Engineer Michael Salvatore D'Alessio, and dated 12/18/07. Sheet 4 was not stamped and does not have a handwritten date. See Figure 9 (Plan of Shoring by Patent).

The general notes on Sheet 1 refer to safety rules and instructions on SS670 and to SSFI instructions mentioned above. All drawings have a "Stringer Lapping Detail" affixed above the title block.

Our calculations (Appendix A 2.2) show that the system proposed was adequate and met code 27-1035 (however, our analysis did not include load combinations including wind, as the drawings indicate sufficient bracing for lateral loads).



1. The intended purpose of this layout is to provide a schematic representation of the use and placement of PCS equipment for the subject structure. This layout does not represent the structure in its entirety. Customer is required to design and furnish all additional shoring, forming and scaffolding equipment as necessary. For areas not furnished by PCS as required to complete the project, it is the sole responsibility of customer to review and verify that the use and placement of PCS equipment as per the layout meets all of the requirements necessary to complete the portion of the project utilizing PCS equipment.
2. Pictorial representations of items not supplied by PCS are illustrative only and do not imply PCS endorsement. Customer is solely responsible for the design, use and application of these items.

FOR SHORING DEFINED AS THE VERTICAL ELEMENTS OF THE FORMWORK USED TO SUPPORT FRESH CONCRETE OR VERTICAL ELEMENTS EXCLUDING DECK MATERIAL TO SUPPORT STRUCTURAL MEMBERS DURING THEIR CONSTRUCTION, COMPLY WITH THE FOLLOWING ADDITIONAL NOTES:

9. Customer must assure that the shoring installation complies with safe practice and with the requirements of governmental regulations, codes and ordinances.
10. Customer shall design and provide suitable sills to properly distribute the imposed shoring loads and shall reshore as necessary.
11. When setting elevations, allow for compression of lumber and soil.
12. Lumber shown indicates suggested minimum sizes only, to coordinate with design of horizontal and vertical shores. Final design and engineering of the lumber and its construction is to be done by the customer.
13. The formwork system must be stabilized to poured columns and walls. The shoring layout as shown is designed with the provision that the formwork system is restrained from lateral movement with respect to the shoring. The customer shall provide sufficient lateral support as necessary.
14. The shoring layout is not designed for motorized concrete placing equipment unless specifically stated.
15. All reshoring design and procedures are the responsibility of others.
16. All vertical and horizontal shoring should be installed and used in compliance with safety rules and recommendations published by the Scaffolding, Shoring and Forming Institute and by PCS.
17. Shoring layout designed using concrete weight of 150 lbs/cu ft plus live load & form weight of 30 lbs/sq ft

FOR VERTICAL SHORING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

18. Ledgers must be centered laterally and ledger joints butted or lapped centrally over their vertical support members.
19. Lateral stability bracing must be designed and installed by customer for all single post shoring. For general recommended methods refer to drawing and/or printed information available from PCS.
20. The approximate amount of exposed thread of the screw legs and extension of extension tubes is noted. Maximum permissible adjustment is to be

12 Inches for 10,000
Inches for
Inches for

REFER TO SAFETY RULES AND INSTRUCTIONS # SS670

FOR INTERFORM OR ALUMINUM JOISTS ON VERTICAL SHORING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

21. Joists must have full bearing on support ledgers (minimum length of bearing 2 1/2").
22. Customer must stabilize plywood decking.
23. Secure joists to ledgers whenever end-loading of joists with personnel or materials

Photo 19 Exposed Thread of Adjustment Screw Leg Instruction by Patent

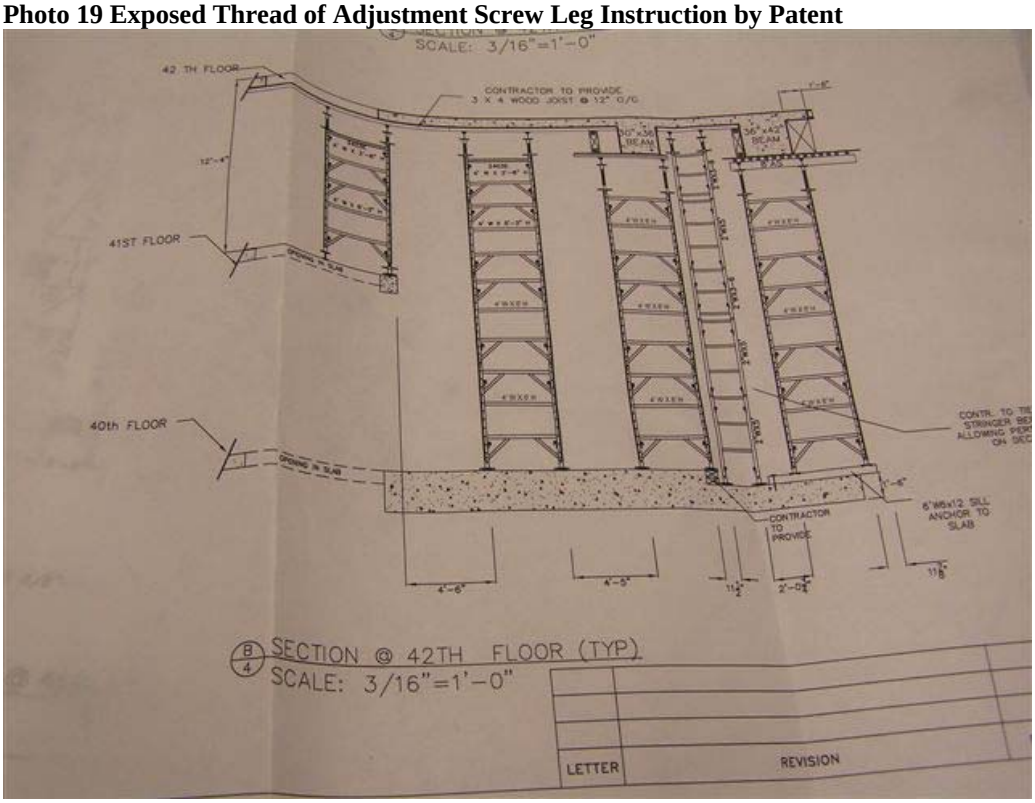


Figure 9 Plan of Shoring by Patent

7 Engineering Analysis

7.1 Adequacy of the Formwork

The Wood Advisory report found the wood joist material used on site to have been inferior to the one specified. However, calculations (Appendix A, 3.4) show that even with the inferior material, the joists had enough capacity to sustain the load of the fresh concrete.

The fragments of beam formwork recovered after the accident did not necessarily indicate a failure of the vertical form under concrete load, but such a possibility could not be totally discounted. The formwork for the beams had snap ties placed on a pattern 30 inches horizontally and 16 inches vertically, with a bottom edge distance of 6 inches. Engineering calculations (Appendix A 3.4 and 3.5) show that the snap ties and the connecting wood joists were adequate, even when a standard tie is considered (rated capacity 2250 lbs with a factor of safety 2). Even more, calculations show that if for any reason a snap tie should fail the joists spanning double distance (5 ft.) would be enough to carry the load to the remaining ties.

The formwork for column 102 was rated to resist pressures resulting from the pour. We also know that the bottom 8-9 ft of formwork did not fail. The bottom would be the area where the largest pressure is exerted. The horizontal cut on the form is clean–almost straight—and does not show any concrete coloring.

7.2 Adequacy of the 20KA Tower Installation

Tests performed at ATLSS, Lehigh University determined that the aluminum shoring towers perform well under concentrated vertical loads (see Appendix C). The towers failed at loads between 152,000 lbs to 159,000 lbs. Consequently, the factor of safety for the towers approaches four (40,000 lbs rated capacity vs. 152,000 lbs failure load). Thus, properly installed, the towers would not have failed under vertical loads. Also the loads imposed by the weight of the concrete were below the rated capacity of the towers. However, the investigation found three significant problems concerning the tower installation as listed below:

- layout of stringers;
- overextension of the leg adjusting fillet;
- placement/support of legs.

7.2.1 Improper Layout of Stringers

Patent's instructions require the stringer to be set in a manner to ensure a centered load on the head plate (Figure 8 Lapping of Stringers – from Patent Instructions). In fact, this layout is shown on each of the drawings, underscoring the importance of the requirement. The extent of the damage in the collapsed zone prevented the investigators from ascertaining whether the stringers were properly installed in the portion that collapsed, but proper installation (as shown in Patent's sketch) was not found anywhere else (i.e., in areas immediately adjoining the collapse or in areas to the south of the building that used the same tower configuration) (see Photo 20 Eccentric Stringer Position). It is thus a reasonable inference that the collapsed portion was similarly improperly installed.

When the stringers are laid using the improper method described above, there will be a 2 inch eccentricity in the application of the stringer reaction to the leg support. The resulting moment will reduce the carrying capacity of the leg, hence the manufacturer interdiction for such a layout.

Photo 20 Eccentric Stringer Position on Head

We observed in the vicinity of the collapse zone stringers supported by other stringers. In fact, the position of the tower based on our layout would not work without some stringers supporting other stringers. Such layout is not necessarily wrong, but the stringers need to be calculated for the loads. Also, at each of the observed stringer support on stringer we could not observe any positive connection. Several of such stacked beams observed in the vicinity of the collapse were twisted.

7.2.2 Overextension of Leg

In one location, leg adjusting fillet extension was found to have reached 19 inches. Extensions of 14 to 15 inches were relatively common. The manufacturer's general specifications (see SS670R1) do not prohibit these dimensions, and the reduction in capacity shown in the SS670R1 booklet tables would have been acceptable. The note on Sheet 1 indicates that the maximum permissible "exposed thread" to be 12 inches for 10,000 lbs. It is not clear if the Sheet 1 instruction prohibits the extension of the filleted area beyond 12" or merely indicates the capacity of the leg for that extension. Notably, from a purely geometrical analysis, given the sizes of the frame and the absence of other additions at some locations, the extensions had to

reach 14 or 15 inches to accommodate the floor to floor distance. Since the installer is not expected to calculate special conditions, and since there is no other indication on the drawing of what capacity reduction to apply, from our point of view the 12" constitutes the limit of the extension (see also discussion at 6.1.2).

7.2.3 Overextension Combined with Improper Layout of Stringers.

The investigation analyzed the very likely case where the improper layout of the stringer coincided with an overextension of the top leg.

Tests at Lehigh (Appendix C) proved that when the load is applied with a 2 inch eccentricity over a leg extended 18 inch and 21 inch, the failure occurs between 52,000 and 61,000 lbs. This indicates a factor of safety of only 1.3 for the tower rating. In the case of the shores at the collapse zone the applied load was of the order of 7,000 lbs for normal conditions (Appendix A 2.2). It is important to note though that the tests measured the combined capacity of the tower system, not the actual individual leg capacity. At the time of the failure, the capacity of each leg was not necessarily equal to the others. In our opinion the capacity of the overextended leg subjected to a 2 inch application of the load is lower than the 13,000 lbs to 15,000 lbs suggested by the total tower carrying capacity measured during the test at the time of failure. Even more, during pour, as one leg deflects, the fluid concrete fills the inclined form resulting in an increase of vertical load. This process might be accompanied by the formation of horizontal loads as well. It is interesting to note that the failures were relatively different during each eccentric test (i.e. test Tower 4 failure occurred by buckling but also with significant bending of an extended adjustment screw that had been loaded in a centered manner and breakage of a horizontal tie. In Test Tower 5 an extension buckled. Test Tower 6 failed due to an excessive bend of the plate). These denote that the eccentricity had the potential to exploit multiple weaknesses once a certain load level was reached.

Our calculations (Appendix A 3.2.2) demonstrate that an extension of 20 inches combined with improper placement of the stringer on top has the capacity to bend the extended leg. The condition fails under a code check analysis per 27-1035 (c), but does not reach ultimate capacity under a normal vertical load. Several such failures were observed in the debris (see 3.1.1.2.).

7.2.4 Placement of Legs

Precise alignment of shores above and below the plywood platform at the 41st floor was difficult to execute. Such alignment is required by the Building Code 27-1035 (d) (2) and manufacturer instructions. The requirements for alignment are directly derived from structural engineering concerns, and they are intended mainly to minimize moments induced by eccentric application of loads as well as shear related problems. Proper alignment simplifies engineering calculations. In the case of the installation on the North East corner, the transfer of the loads imposed by the shore legs on the 41st floor platform should have been specially designed, as alignment of top and bottom shore posts is not entirely sufficient; the transfer of concentrated forces needs to be

performed directly from the top leg to the bottom or via a system capable of sustaining the forces. In our case, the plywood and joists were interposed between the legs (see Figures 6 and 7 and Photo 21). Note in Photo 21 the almost precise contour of the base plate defined by the punch hole and the relative position of the nails. The alignment of nails indicates the position of the joists. Clearly the plate was set between the joists and not on top of them. Several punch-holes like this were found in the debris.

Tests by Wood Advisory (Appendix B) demonstrated that a plywood floor supported by 3x4 joists spanning 4 ft is not necessarily adequate to support and transfer concentrated loads when such loads are applied to the plywood mid-span. When the leg is placed at the center of a 12 inch span the plywood can be punched by a force as low as 3,000 lbs. The calculated forces on the legs of the shore towers vary between 3,000 to 7,000 lbs when properly installed, and the investigation located several cases of punched plywood.

Our calculations (Appendix A 3.3.3) show that a deflection of .8 at the plywood level of 41st floor would have caused the failure of an aluminum tower. Such deflection could be the result of a leg punching the plywood, or even of an excessive displacement of the joist plywood system without any actual breakage, either of which would be the likely consequence of the placement of the legs on plywood midspan.

Photo 21 Plywood Penetrated by Leg Base

7.3 System Structural Adequacy

The northeast corner of the building had required a two-story high support for the formwork because the building architecture required a two-story high recess. In that area, the contractor had installed a supported plywood platform (sometimes known as a “mud floor”) at the 41st floor on top of which a supporting shore tower system was erected for the 42nd floor formwork. As described in 2.4, Debris Field and Preliminary Observations, the collapse did not extend in a significant way beyond these bays (see Photo 1 and Photo 3). One of the main conclusions based

on the debris layout was that whatever the initiating cause, the magnitude of the failure was related to this stacked (two story) installation. The preceding paragraphs 7.1 and 7.2 provide an analysis of the adequacy of individual elements. The following paragraphs present a discussion of the structural adequacy of the two tier shoring system.

7.3.1 Vertical Load Carrying Capacity

Our calculations (Appendix A 3.1) show that the aluminum towers in the collapse area, had they been carefully located, were sufficient in number and strength to sustain the vertical loads imposed by the concrete above, including additional "superimposed" vertical loads as set forth in the Building Code 27-1035 (c) (1).

As a pure gravity carrying structure, the towers might have not collapsed under vertical loads alone had these been transferred properly. This would have required not only exact alignment of shores above and below the plywood platform at 41st floor, but an engineered system to transfer the vertical concentrated loads. As discussed at 7.2.4, a plywood floor supported by 3x4 joists, depending on the placement of the legs, might not necessarily be adequate to support and transfer concentrated loads unless additional engineering details are implemented.

The shore tower rated loads were confirmed by tests, but the rating and the tests assumed firm support at the base. Our calculations (Appendix A3.3.1, 3.3.2, 3.3.3, 3.3.4) show that the aluminum towers are sensitive to deflections of supports and fail when the deflection goes beyond some limits (such as those produced by legs placed on plywood mid-span). The system as installed had various flaws or potential conditions that might have allowed deflection:

- wood joists with weak modulus of elasticity (900,000 psi in lieu of 1,500,000 psi);
- legs set on plywood not always directly on top of joists (Wood Advisory Services tests of plywood punching show that even before failure the plywood can deflect in excess of 1" under a 5,000 lbs load);
- improperly placed stringers (Lehigh tests show that under a load of 3,000 lbs the top plate would deflect vertically ½ inch).

One needs to conclude that while the installation had enough tower elements to sustain the vertical load, the condition of their stacking and their actual installation did not ensure proper vertical capacity for the system.

7.3.2 Lateral Load Carrying Capacity

The Building Code and all relevant ACI publications require that the formwork and supporting scaffold be designed to carry combinations of vertical and horizontal loads. Per our analysis (Appendix A 3.2) the two-floor stack system employed in our case did not have adequate carrying capacity, as it did not allow proper transfer of lateral loads. Nor did it have sufficient capacity to sustain such loads.

It is not clear why the contractor opted for the two-floor stack solution. Had some consideration been given to the transfer of horizontal forces it might have been a structure more capable to resist lateral loads than the one shown in the unsigned and unsealed drawing sheet (Sheet 4 by Patent that was not implemented).

As installed, the two floor stack had no positive attachment to the 41st floor, and as a result, it amounted to an independent two story structure that should have had additional specific provisions for stability. To transfer lateral loads as those indicated in the Building Code 27-1035 (c) (3) one would need to nail or fasten each stacked element to the one below (i.e. plywood to joists, joists to stringers, stringers to towers, tower legs to mud slab and so on). Chapter 3, Examination of Debris, documents a number of cases where no connection existed. The installation lacked a systematic concern for transfer of horizontal loads.

A proper installation would have transferred the lateral forces developed at each individual shore tower. Nailing, which would have assured such transfer of lateral forces, was not always present at the site, especially at shore legs. Only a few leg bases were nailed into the 41st wood floor. The legs at the 40th floor that could not have been nailed into the concrete should have been placed on and nailed to sills. What resulted was a gravity system that relied on friction to transfer horizontal loads (friction develops in the presence of vertical (gravity) forces and is directly proportional with vertical forces). At some phases during the concrete pour process some bays were not loaded, and, as a result of continuity effects, some portions of girders or posts might even have experienced a tendency to uplift. Consequently, at some locations there might have been no friction to transfer the loads.

Photo 22 Shoring at the East Side.
Note absence of ties

Photo 23 Form for Beam

Photo 24 Tie (Assumed)

At the site's east side the formwork relied on a cantilever system that was loosely set. Only one tie-back was positively identified in the debris when each frame at both levels should have been tied back. Our calculations show that the system as installed failed analysis required by the code (27-1035).

7.3.3 General Stability

As shown by engineering calculations (Appendix A, 3.3.1, 3.3.2, 3.3.3, 3.3.4), the carrying capacity of aluminum shores is sensitive to large deflections. Here, relatively large deflections were made possible by the installation of a wood flooring system at the 41st floor, and aggravated by the use of substandard wood. This diminished the system's ability to carry vertical loads.

One should note that the pattern of failure observed during the Lehigh direct load tests was not fracture or buckling of legs, but failure of braces or horizontal bars that allowed the shore legs to overturn. This corresponded with the failure modes observed in the debris. But testing was performed on isolated towers. In a well laced system, the legs, after the failure of a supporting element, could have been kept stable by other structural elements. The correspondence observed might support the opinion that the system as installed was not sufficiently interconnected.
A system installed without moment connections and that is not laced and cross-braced to transfer horizontal loads is for most cases incapable of safely adjusting and finding new load paths when a vertical element fails. While each individual shore tower was cross braced internally, the two story structural frame lacked a bracing of the entire system. To make matters worse, when a failure occurs during a concrete pour, an additional effect develops: the fluid concrete develops horizontal forces due to the concrete lateral pressure on the inclining formwork. Another set of forces is produced by the friction associated with the flow of the concrete. The system's lack of capability to resist or transfer lateral forces was a contributing factor of the collapse. Several engineering calculations show that the system could not pass a code check for the combination of vertical and horizontal loads.

The system could not take advantage of positive effects resulting from member continuity over several spans. On the contrary it seems that the collapse was propagated by elements that were continuous over several towers such as 4 foot x 8 foot plywood sheets, long stringers, formwork for the beams and the extremely heavy reinforcing bars. Most of these were found atop the pile and in a less damaged condition than the elements below.

8 Discussion and Conclusions

8.1 Initial Failure

The layout of the debris suggested a failure of the vertical support systems, most likely tower frame failure. 90 degree overturning of the entire installation or individual towers did not happen - had some towers overturned, some would have likely fallen out onto the street, which did not occur. As a result, it was concluded that the initial failure was due to a vertical load. The only significant vertical load was the load of the concrete being poured. This load was not extraordinary, and the system was supposed to have been engineered for such load.

8.1.1 Punching of the Plywood

In the investigators' opinion punching of the plywood was most likely the initial failure that started the chain of collapse.

Photo 25 Punched Plywood

Our calculations show that the shoring towers were likely to fail when the punching of the plywood occurred at the 41st floor, which corresponds with the punched plywood observed in the debris (Photo 25 and Appendix B). Engineering calculations (see discussion at 7.2.4) prove that the load on the leg far exceeded the capacity of the plywood to resist punching. The intensity of loads that were capable of punching the plywood was determined by Wood Advisory, which performed tests on plywood found at the site. Photo 21 is a clear example of punched plywood. It also demonstrates that the leg was not atop a joist.

The aluminum shoring was not intended to accommodate and sustain excessive leg deflections, but that is precisely what was imposed upon it. Calculations show that loss of support or excessive deflection at the base of one tower leg will induce failure of the shoring frame (which

lies opposite the settling leg). The shoring towers (above the 41st level) were thus likely to fail when the punching of the plywood occurred at the 41st floor level.

The scenario of initial failure described above is confirmed by:

- physical elements found in the debris field that match the description (e.g. Photo 21);
- results of testing of material collected from the site (e.g., Wood Advisory tests);
- engineering calculations that are simple and do not use any other loads but those clearly existing at the time of the collapse. That is, a lower level of loading than required by code. (27-1035 (c) (1) *Live load shall allow for the weight of workers and equipment, with allowance for impact, but in no case shall less than twenty psf be allowed.* Our calculations do not include this additional 20 psf).

The destruction in the area of consideration was such that other scenarios of failure might be proposed, albeit with a lower level of probability.

Photo 26 Broken Adjustment Screw Extension

8.1.2 Other less probable initial failure causes.

Formwork blowout is probably the most common accident occurring during a concrete pour. Usually, it occurs at pours against vertical forms, where the pressure produced by the concrete head can be substantial. In our case this could have happened at the column or the beams. Still, the formwork for column 102 did not fail in its bottom half (where it would have been more likely). Some witnesses describe the accident as having occurred while pouring the beams. The witnesses place the accident at a time after the bucket had been emptied and concrete was being spread. A high rate of concrete placement could increase pressure on vertical formwork, but this rate could not have been extreme at the very moment of the collapse as the bucket had already moved away. A concrete blowout of the beam formwork remains a possibility, although this would not have been entirely consistent with the debris layout (which suggested tower buckling). A beam formwork breach would have most probably resulted in a V shape plywood collapse. This was not observed. Our calculations, using the maximum pressure (hydrostatic) on the vertical formwork do not predict failure.

Excessive deflection of the plywood system is another possible scenario of initial failure. This is supported by the poor quality of the wood in the mud floor assembly. This scenario is not

substantially different from the punching of the plywood discussed at 8.1.1 in that the collapse would have followed a similar pattern and been rooted in the same failure to understand the same general principles of load transmission.

8.2 Two Floor Stacked System Failure

8.2.1 Stack System Failure.

The punching of the plywood shown in Figure 11, Detail A (or even the excessive deflection of a base under a leg) produced the increase on the load of the remaining tower legs. Several scenarios supported by engineering calculations demonstrate this.
Calculations (see Appendix A 3.3.1, 3.3.2, 3.3.3, 3.3.4) show that when a leg loses support, the diagonally opposite leg would discharge (or even see some uplift) and the two remaining legs would see their loads doubled.
Other calculations (see Appendix A 3.1) show that a 12 ft. stringer, supported on three legs, would not fail under the loads that existed when one support was lost. The deflection of this stringer would be less than the movement allowed by the punched plywood. The loads would just redistribute to the remaining legs.
The result of the redistributions discussed above would bring the load on one leg to 11,000 to 13, 000 lbs. The Lehigh tests showed that under such load levels an adjustment screw extending 18" and with a 2 inch load eccentricity will fail (see Figure 11). This failure would be followed by the collapse of the entire tower.

This scenario (Figures 10 and 11) clearly explains the global failure of the entire stacked system and is supported by calculations, lab testing and numerous pieces of broken adjustment screws found in the debris. The scenario follows naturally from the mode of failure on the floor above (see 8.1.1). Since such failure would have occurred in any tower under the central beam (where the loads are maximum) the resulting general collapse would be consistent with the layout of the debris existing at the site.

8.2.2 Other System Weaknesses

No matter what the initial cause of the formwork failure, it is the fragility of the two floor stack system that caused the catastrophic extent of the accident. The collapse of one shoring tower allowed failure of the formwork above. The collapsing elements together with the concrete flowing on inclined surfaces produced lateral loads that the system was not capable of sustaining. Simple engineering principles tell us that once a post is starting to incline (head moves laterally) horizontal loads are generated in the elements trying to resist this movement. The horizontal forces can reach over 10% of the applied vertical force. Thus the failure of one shore tower was easily capable of bringing down the entire shoring system, especially since it was not braced or laced. Without bracing and/or lacing, stability could not be fully attained, or in the case of lateral loads, resisted by the system. The mode of failure described here is also consistent with the debris layout.

The system was also weakened by overextended legs on top of which the stringers were placed without being centered (see 7.2.2). Tests at Lehigh proved that significant deflection and rotation at the top head occur under an eccentrically applied load. Calculations show a significant reduction of the factor of safety under such conditions. Once the collapse was initiated, the towers and the legs started to fail.

The report puts in evidence the main elements that made the system weak:

a) Substandard wood for formwork
Because substandard wood was used for the formwork, the structure was likely to incur formwork failure or—even more likely—have high deflections. Calculations show that the Patent shoring towers are sensitive to large deflections, and the manufacturer's instructions clearly require firm footing for the legs.

b) Lack of design for wind and for horizontal loads
We did not obtain design calculations for the shoring system as installed. Our simplified verification led us to assume there had not been consideration or calculation for wind and horizontal forces. The lack of design for lateral loads (wind as well as lateral loads indicated in the code to be considered in relation to concrete pouring activities) led to the installation of a structure that had limited or no capability to transfer lateral loads. We found only one round bar that might have been used to tie the shoring tower system to the concrete floor. Proper design for the lateral loads would have required bracing and attachment to the rest of concrete. The attachment to the rest of the structure could have been implemented by explicit engineering calculations or as a result of various code and manufacturer instructions.

c) Lack of standard practice measures such as lateral bracing, including lack of attachment of the system to the 41st floor already-poured concrete
Despite the dearth of engineering discussed above, observance of basic safe construction principles might have prevented or limited the extent of the collapse. The lack of engineering consideration was aggravated by a failure to install devices required by standard practice: braces and ties. The result was an installation that was not attached to the rest of the structure or otherwise properly braced.

d) Lack of sufficient nailing
The installation as erected relied mainly on friction for its stability and transfer of lateral loads. Lack of sufficient nails made the structure reliant on friction, but friction is present only when gravity loads are acting. Deflection of wood formwork, especially in a continuous system, is known to reverse deflections or loads in certain cases during pour. As a result, in some cases, where uplift occurs, there might be no friction present at all.

e) Eccentric application of loads
The eccentric application of the load on extended shore heads created the possibility of large displacements. Such overextended shore heads did not meet code or the manufacturer's specific instructions. Consequently, the factor of safety for the structure was significantly reduced.

All of the above can be traced to the failure to follow engineered drawings and to have subsequent inspection.

The responsibility for erecting the structure was that of the concrete contractor, DiFama. Contrary to fundamental code requirements (see Code 27-1035(b)), the structure was erected without following any drawing. Inspection by the contractor's inspector, if it took place at all, did not flag the gross failure to conform to code and shoring manufacturer instructions. Note that it took several days to erect this system and there was ample time for inspection. It would have been obvious to an inspector that drawings were not followed.

The investigation also uncovered several defects in the installation and use of the formwork and shoring at the site in addition to the contractor's failure to follow engineered drawings. These defects were:

1. The contractor did not follow basic manufacturer general instructions;
2. None of the contractor's inspectors questioned the installation's failure to comply with these instructions;
3. Substandard material was used for formwork.

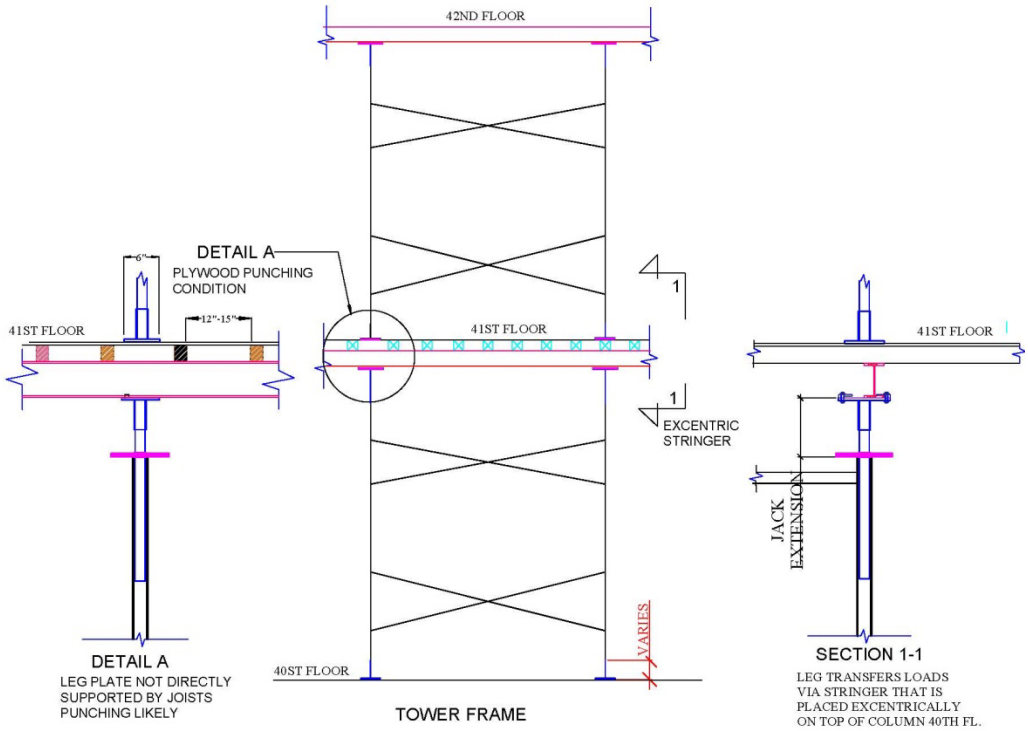


Figure 10 Transfer of Loads at "Mud Floor" – Before Loading and Collapse

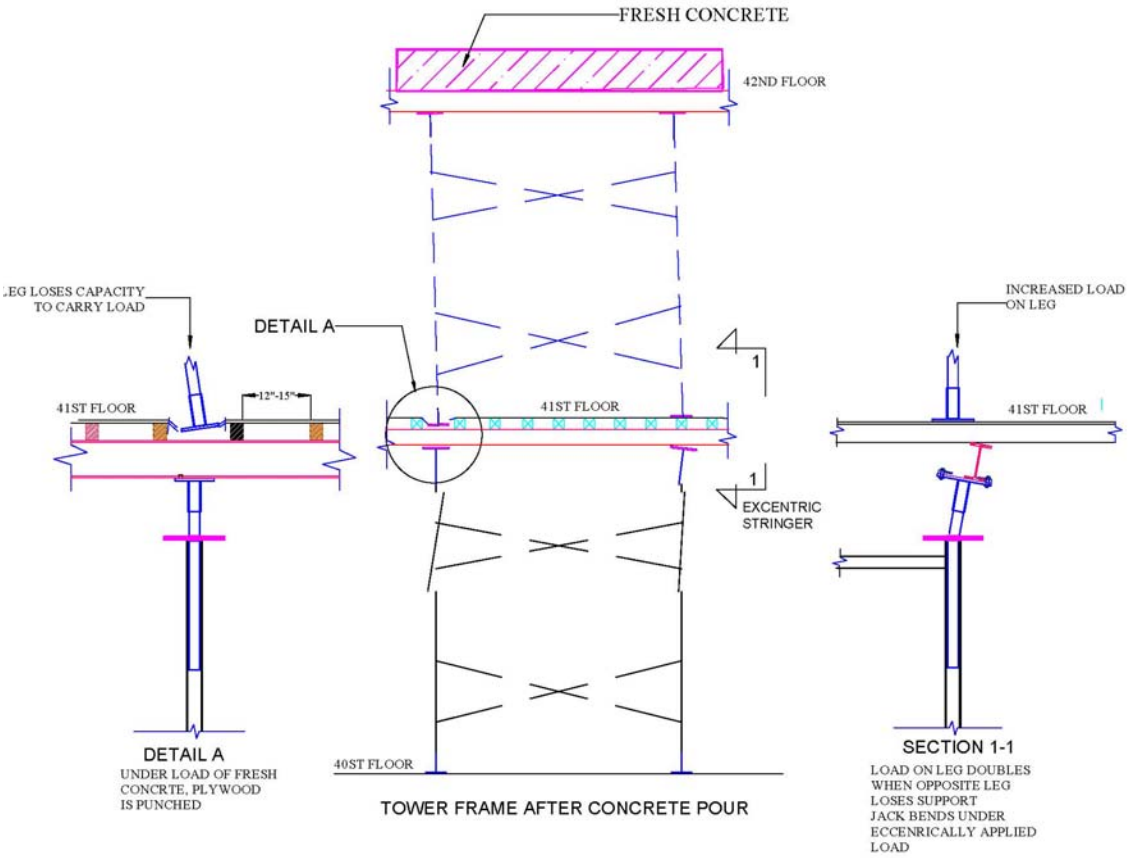


Figure 11 Scenario of Failure Following Concrete Pour

8.3 Conclusions

As presented in the previous chapter, whatever the initiating cause, the extent of the total failure was related to the two-floor stacked nature of the shores installation, as evidenced by the fact that the failure did not progress in any manner past the edge of the already poured 41st floor.

The two-floor stacked installation had not been engineered. The lack of engineering and common safe installation methods resulted in a structure that was not sufficiently stable. The structure lacked the capability to properly transfer vertical or lateral forces and lacked proper lateral restraint. Thus, the structure was susceptible to collapse.

The Building Code is specific as to the need to follow engineered drawings and to perform inspection for these types of structures. Both were not present.

APPENDIX A	Structural Calculations
APPENDIX B	Wood Formwork Testing and Report - Wood Advisory Services
APPENDIX C	Report on Patent Aluminum Shores Tests - ATLSS Lehigh University
APPENDIX D	Documentation and Preparatory Documents for Shoring Layout

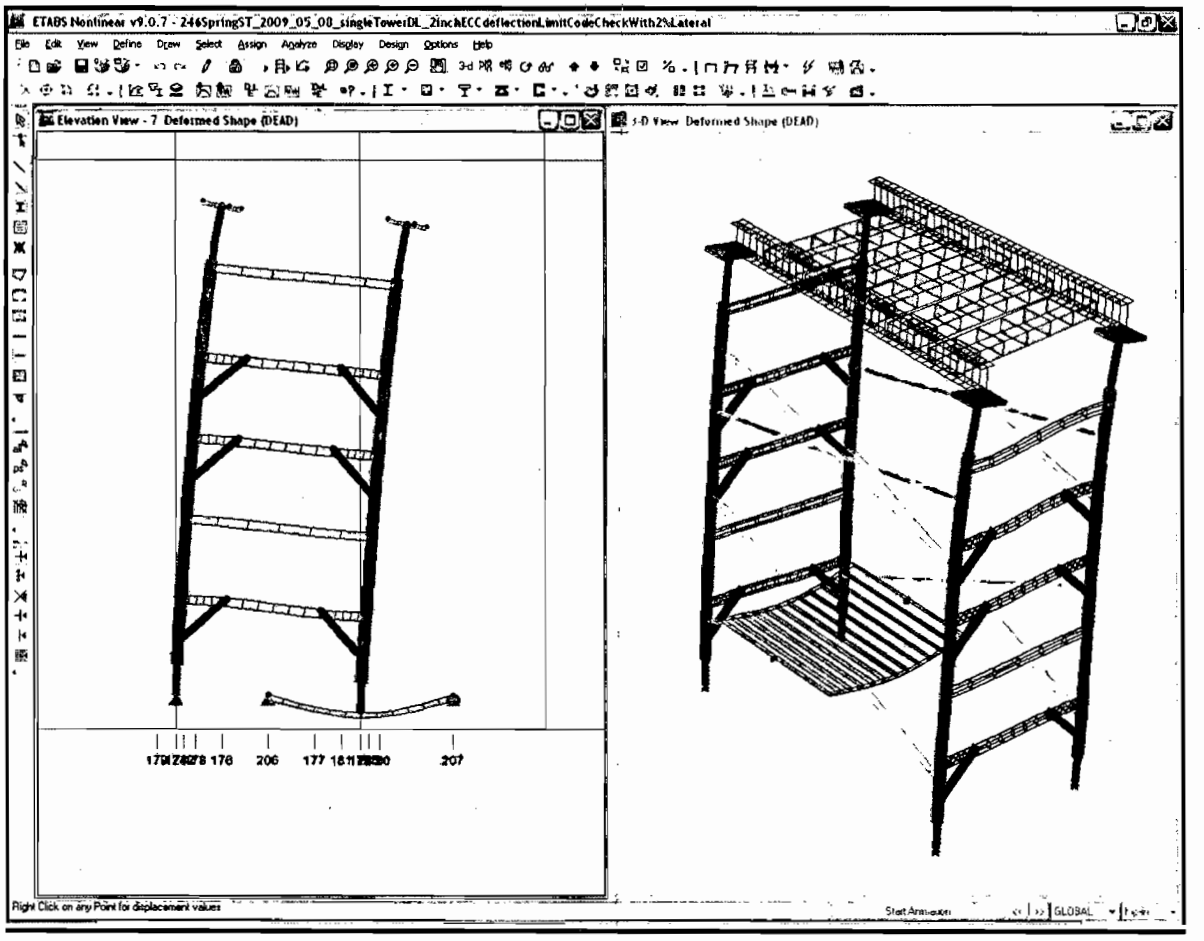
APPENDIX “A”
Structural Calculations

246 Spring Street

Structural Calculations

By: Guo Zhan Wu

Date: 2009/09/04



APPENDIX A

APPENDIX A

246 Spring Street
Table of Content - Structural Calculations

1 Computer Model-Description of computer analysis and loading condition

2 Patent Construction System Layout Drawing-

2.1 Global Check-Dead Weight Of Concrete And Formwork.

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3.2 Single Tower - Allowable Stress Code Check

3.2.1 Allowable Stress Check - Condition 1, Load Case A.

3.2.2 Allowable Stress Check - Condition 2, Load Case B.

3.3 Single Tower - Ultimate Condition.

3.3.1 Ultimate Condition 1 - Load Case A, And The Bottom Shoring Leg Support Is Flexible.

3.3.2 Ultimate Condition 2 - Load Case B, And The Bottom Shoring Leg Support Is Flexible.

3.3.3 Ultimate Condition 3 - Load Case A, And The Bottom Shoring Leg Support Is With .8" Deflection Limit.

3.3.4 Ultimate Condition 4 – Load Case B, And The Bottom Shoring Leg Support Is With 0.8" Deflection Limit.

3.4 Wood joists ultimate condition

3.5 Snap Tie Verification

4 Validity of Computer Analysis

4.1.1 **On One Side Of Frame, Load Case B. The Total Weight Is In The Order Of 55 Kips In The Middle Of the Shoring Tower.**

4.1.2 **Load Case A. The Total Weight Is In The Order Of 150 Kips (150 Kips / 4 Top Plates = 37.5 Kips On Each Top Plate).**

Appendix:

Aluminum Shoring Test Report From "ATLSS Multidirectional Laboratory of LeHigh University"

Appendix A –
Structural Calculations:

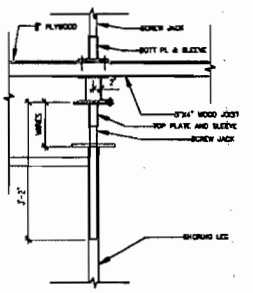
The Structural calculations and analysis were performed to verify the code compliance and possible modes of collapse of the formwork and supporting system. Hand and computer calculations were used to verify code adequacy of the drawings prepared by D'Alessio for Patent (*Patent Construction System Layout Drawing*) and the system as reconstructed by the investigation. and depicted in the *Collapsed Zone Reconstruction Shoring Layout*

1 Computer Model

The computer analysis used ETAB v9 with the following inputs:

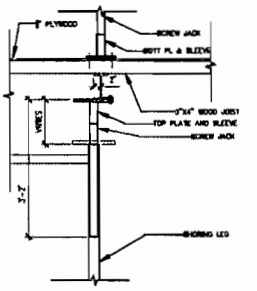
- Geometry: Screw jack top extension is 21 in, and bottom extension is 11 inch. Typical frame is 4 feet wide by 6 feet height in code check for Patent Construction System Layout Drawing. The typical frames for reconstruction shoring layout are consisted of 4 feet wide by 3'-6" height and 4 feet wide by 5'-3" height.
- Section Properties:
  a. 2" Ø "Screw Jack".
  b. 3" Ø "Shoring leg" with wall thickness =.15" (approximate cross-section for patent extrusion shoring leg)
  c. "Horizontal Rung" is 1.25" x 2.25" outside dimension with wall thickness = .075".
  d. "Knee brace" is 1.25" x 1.75" outside dimension with wall thickness = .075".
  e. "Stringer" is extrusion I – beam with approximate 4" flange width and flange thickness of .3", and 7.5" depth and web thickness of .15".
- Material Properties of Elements:
  a. Screw Jack is made of steel <u>Grade 1020 per ASTM A512 or A513</u> with Fy = 55ksi and E = 29000 ksi as <u>per "Patent Shoring System Manufacturer's Shop Drawings"</u>.
  b. Shoring Legs, horizontal members, stringers and knee braces are made of Aluminum 6061-T6, yield strength Fy = 35 ksi, <u>ultimate tensile strength Fu = 38 ksi</u> and E = 10100 ksi.

- Sketches - Eccentric placement of loads:



LOAD CASE A-CENTERED LOADED

SCALE: N.T.S.



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2 Patent Construction System Layout Drawing-

2.1 Global Check-Dead Weight Of Concrete And Formwork.

In the north east corner of 42nd floor which is 2 story high recess area approximately 26 feet by 28 feet, the total number of shoring frames is 23. Total weight of the concrete and formwork is 223 kips and representing 9.7 kips on each frame; this load is less than allowable load which is 20kips per frame (10kips per leg) as indicated in Patent Shoring System drawing 4607K070.

Project: 246 SPRING STREET Date: 6/8/09 Page: 1 of ____

Subject: _____ Engr: _____ Checked By: _____ Date: _____

PATENT DESIGN DWG CHECK: **20KA**

1. GLOBAL CHECK: FRAME CAPACITY = 20 K

NUMBER OF FRAMES IN EAST-WEST DIRECTION:

6

NUMBER OF FRAMES IN NORTH-SOUTH DIRECTION:

17

TOTAL AREA = 26' X 28' = 728 SF

a. 9" SLAB + 10 psf FORMWORK + 50 psf LL = 173 psf

WEIGHT = 173 X 728 = 126 K

b. BEAM ELEMENT:

- 54" X 42" X 26' LONG GIRDER

$$\frac{54}{12} \times \left(\frac{42'-9''}{12}\right) \times 26' \times 150 \text{ pcf} = 48.3 \text{ K}$$

- 36" X 42" X (29' − 4.5') GIRDER

$$\frac{36}{12} \times \left(\frac{42'-9''}{12}\right) \times 24.5' \times 150 \text{ pcf} = 30.3 \text{ K}$$

- 30" X 36" X (26' − 4.5') GIRDER

$$\frac{30}{12} \times \frac{(36''-9'')}{12} \times 21.5' \times 150 \text{ pcf} = 18.2 \text{ K}$$

TOTAL = 126 + 48.3 + 30.3 + 18.2 = 223 K

LOAD ON EACH FRAME = 223 K/(6+17) = 9.7 K < 20 K

∴ PATENT DESIGN DWG PASS CODE CHECK.

Project: 246 SPRING STREET Date: 6/8/09 Page: 2 of ___

Subject: ___ Engr: ___ Checked By: ___ Date: ___

PATENT DESIGN DWG CHECK = 20 KA

2. LOCAL CHECK :

- 4'x4' TOWER UNDER 54"x42" TRANSFER BEAM FORMWORK

a. AREA OF SLAB = 8'x8' = 64 SF

9" SLAB + 10 psf FORMWORK + 50 psf LL = 173 psf

WEIGHT = 173 psf x 64 SF = 11.2 K

b. BEAM 54"x42"

AREA OF BEAM = $\frac{54''}{12}$ x 8' = 36 SF

WEIGHT = $\left(\frac{42''-9''}{12}\right)$ x .15 x 36 SF

= 14.85 K

TOTAL = 14.85 + 11.2 = 26 K

EACH TOWER HAS 4 LEGS AND EA LEG CAPACITY IS 10K, TYP. TOWER CAPACITY IS 4x10K = 40K.

- 2'-6" x 8' TOWER

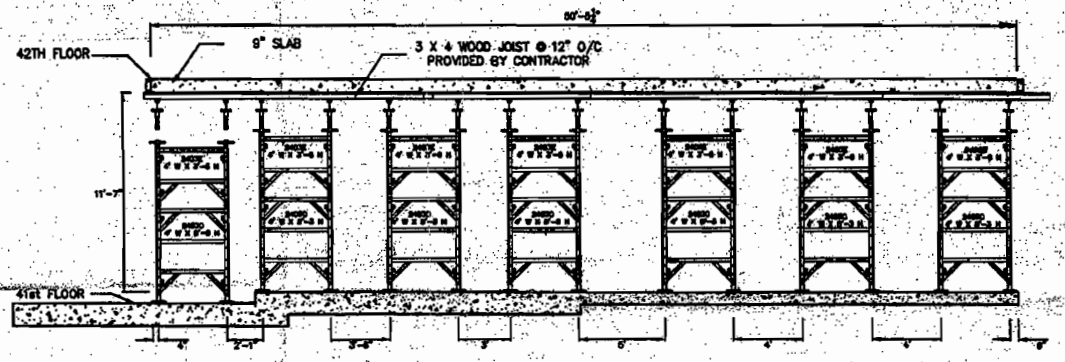
4' x 12' AREA = 48 SF

9" SLAB + 10 psf FORMWORK + 50 psf LL = 173 psf

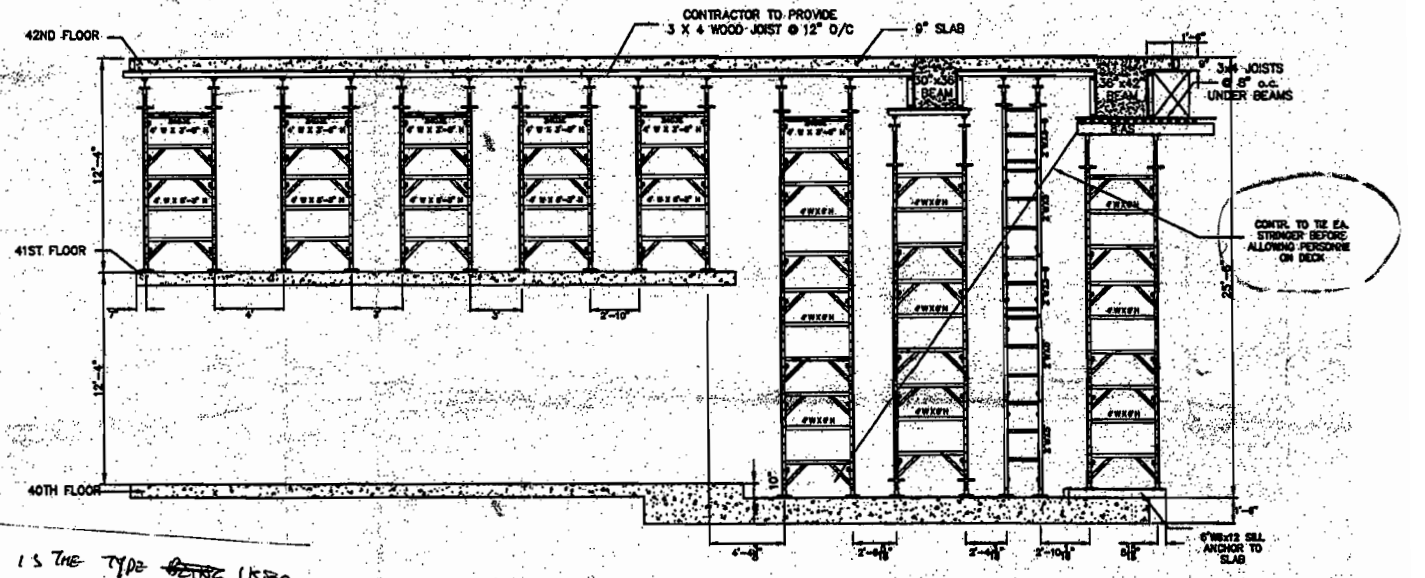
WEIGHT = 173 x 48 = 8.3 kips < 40 K

∴ PATENT DRAWINGS PASS CODE CHECK.

3

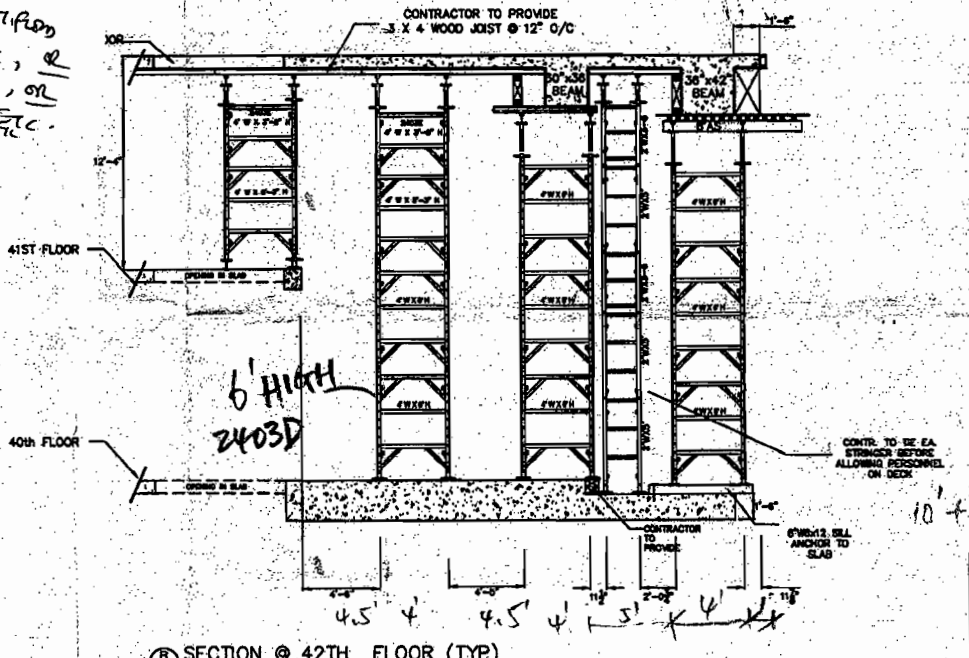


A/2 SECTION @ 42TH FLOOR (TYP)
SCALE: 3/16"=1'-0"



A/4 SECTION @ 42TH FLOOR (TYP)
SCALE: 3/16"=1'-0"

WHAT IS THE TYPE ~~BEING~~ USED @ 42ND FLOOR
AS VARIOUS TYPES
HAVE BEEN IDENTIFIED
SUCH AS VERTICAL SHORING, OR
TROUBLE SAVER SECTIONAL, OR
SPRINT SCAFFOLDING, ETC.
WHAT IS 2403 E / 2403 C ETC.

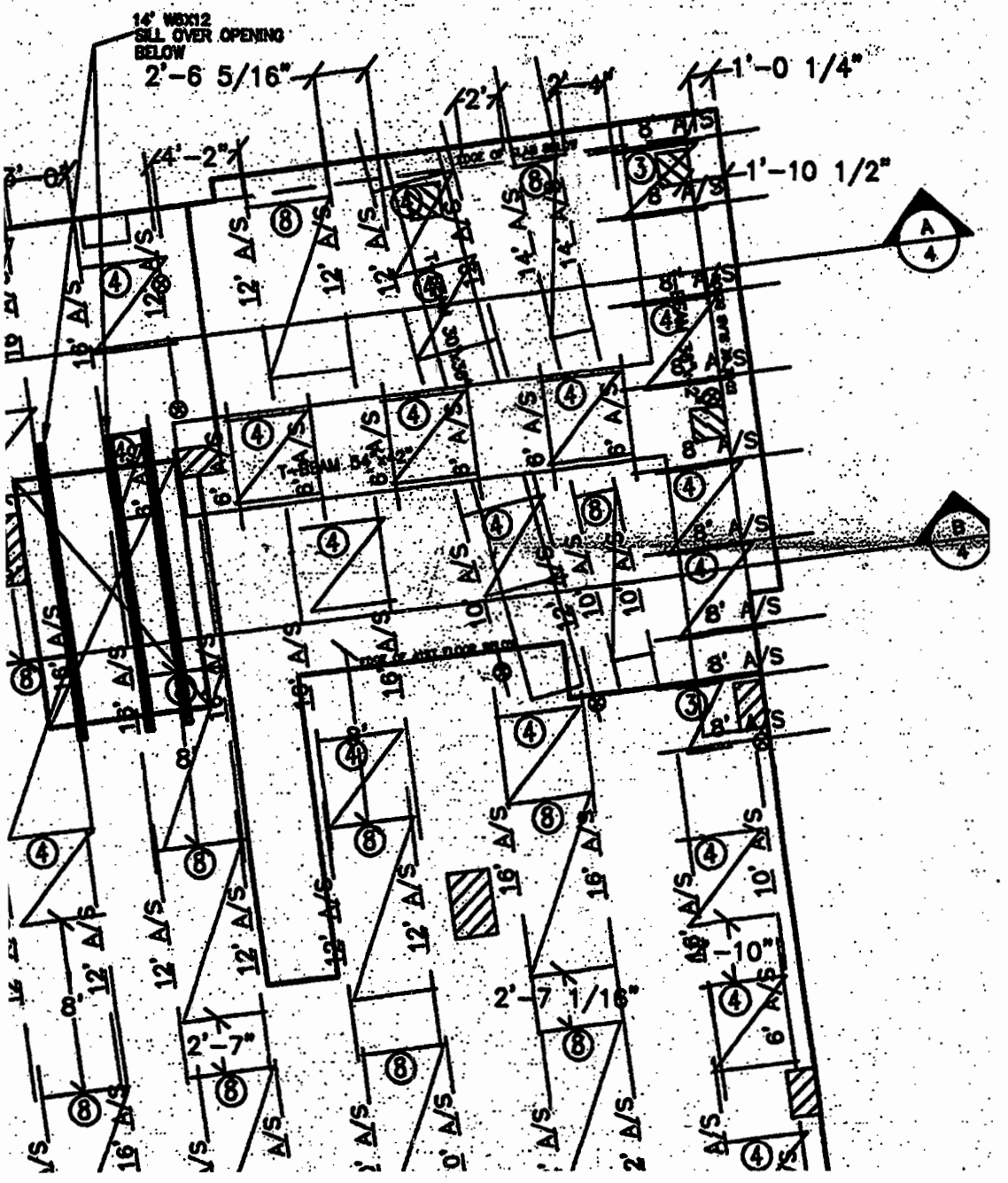


10' + 17' = 27' ±

B/4 SECTION @ 42TH FLOOR (TYP)
SCALE: 3/16"=1'-0"

LETTER	REVISION	DATE	APP'D

FORMWORK SUPPORTING 42ND FLOOR



26'X28' COLLAPSE ZONE AREA

2.2 *Individual Shoring Tower Check-Dead Weight Of Concrete And Formwork*

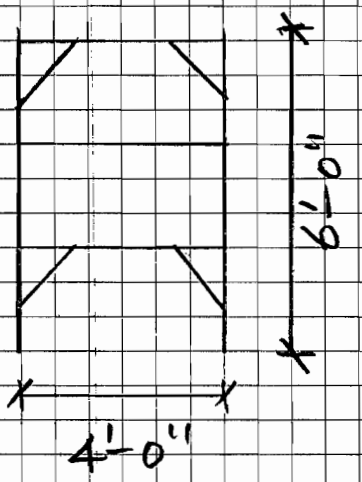
The typical shoring tower (2 frames) under beam-formwork is 4 feet by 4 ????feet. The tributary area is 64 SF, and total load on the shoring tower is 26kips (13kips per frame or 6.5kips per leg); this load is less than allowable load which is 20kips x 2 frames = 40 kips. Even with consideration of load increase due to continuity over support will be less than allowable load.

Project: 246 SPRING STREET Date: 6/9/09 Page: 1 of ___

Subject: _____ Engr: _____ Checked By: _____ Date: _____

PATENT DESIGN DWG CHECK = 20KA

3. COMPUTER ANALYSIS =

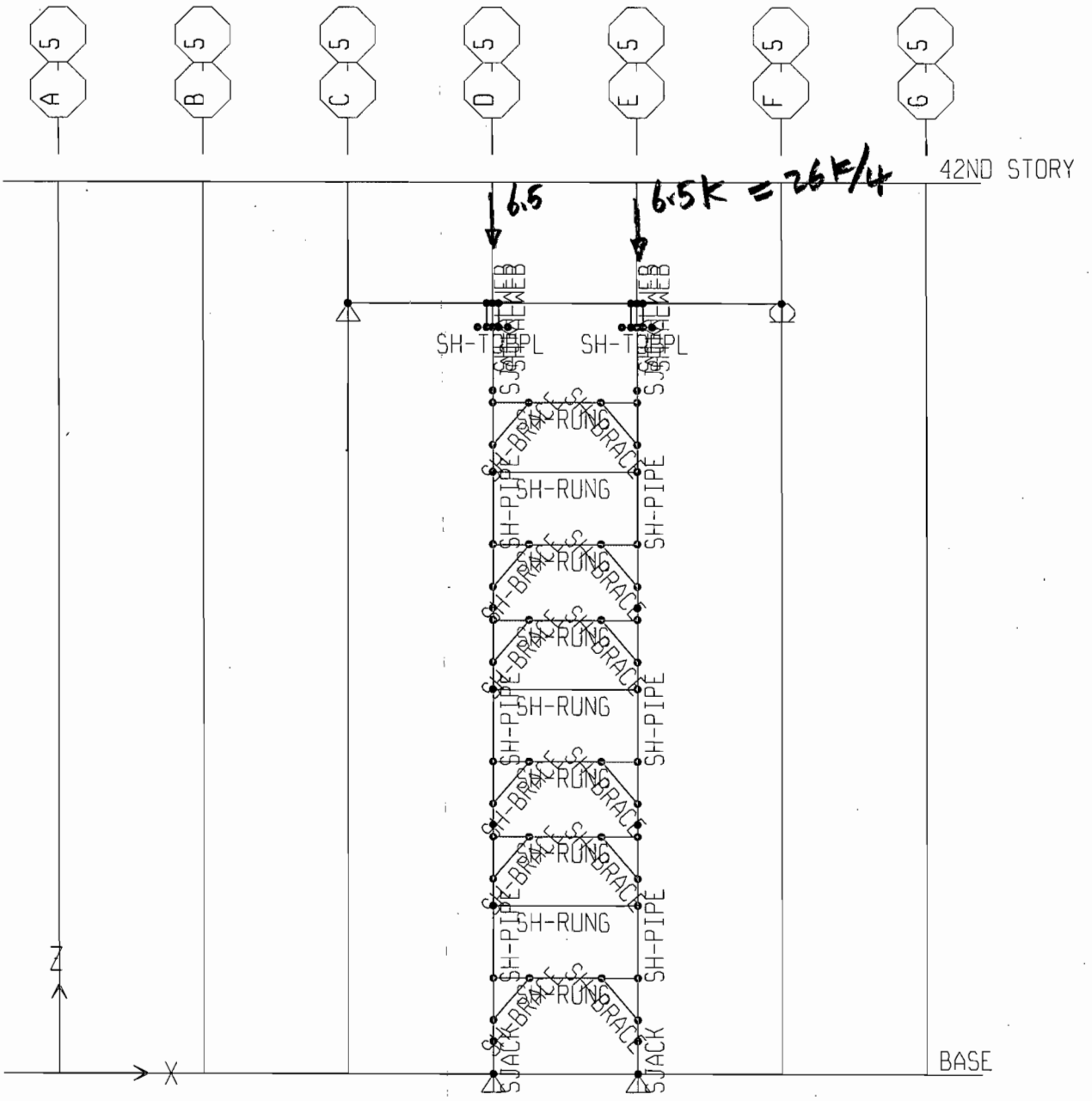


TYPICAL 6' SHORING FRAME.

SUMMARY OF RESULT - ETAB MODEL
"246SPRINGST_2DCODECHECK TYP PATENT SHORING"

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = .489 < 1 \quad \text{(OK)}$$

∴ PATENT SHORING DWGS PASS CODE CHECK.



0.00 0.50 0.70 0.90 0.95

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 42ND STORY  Element: C33-25  Station Loc: 21.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=72.000
A=1.343  i22=1.367  i33=1.367
s22=0.912  s33=0.912  r22=1.009  r33=1.009
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   0.489 = 0.479 + 0.010 + 0.000

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2     -6.500       0.224       0.000      -0.020       0.000

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             4.840      10.105      21.000

                         fb          Fb          Fe        Cm         K         L        Cb
                     Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending     0.246      23.100      91.929     0.850     1.000     0.333     1.000
    Minor Bending     0.000      23.100      10.214     1.000     1.000     1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.027      14.000       0.002
    Minor Shear       0.000      14.000       0.000
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 42ND STORY  Element: C33-23  Station Loc: 21.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=21.000
A=1.279  i22=0.509  i33=0.509
s22=0.509  s33=0.509  r22=0.631  r33=0.631
E=29000.000  fy=55.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   0.202 = 0.174 + 0.028 + 0.000

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2     -6.486      -0.536       0.000       0.005       0.000

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             5.071      29.179      33.000

                         fb          Fb          Fe      Cm       K       L      Cb
                     Stress   Allowable   Allowable  Factor  Factor  Factor  Factor
    Major Bending     1.051      36.300     134.847   0.926   1.000   1.000   1.094
    Minor Bending     0.000      36.300     134.847   1.000   1.000   1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.006      22.000       0.000
    Minor Shear       0.000      22.000       0.000
```

2.3 Manual Calculation In Code Check For 3" X 4" Wood Joist:

Wood joist under beam formwork is spaced at 8 in o.c, and simple supported at 4'-0".
Actual bending stress fb = 1647 psi which is equal to allowable bending stress Fb = 1640 psi as shown in Patent drawing 4607K070.
Actual shear stress fv = 120 psi which is less than horizontal stress Fv = 180 psi as shown in Patent drawing 4607K070.

Project: 246 Spring Street Date: 1/7/2009 Page: 1 of 8

Subject: WOOD JOIST CHECK Engr: GW Checked By: _____ Date: _____

PER PATENT DWG 4607K070

4. 3"X4" JOIST – SHORT TERM LOADING, $C_D = 1.25$ FOR 7 DAYS.
JOIST SPAN = 4'-0", JOIST SPACING = 8"
UNDER 42" BEAM FORMWORK

$$w = \frac{42''}{12} \times \frac{8''}{12} \times (150\ pcf) + 50\ psf \times \frac{8''}{12}$$

CONC

CONSTRUCTION LIVE LOAD

$$= .383\ klf$$

$$M_{max} = \frac{wL^2}{8} = \frac{.383 \times 4^2}{8} = .77\ k\text{-}ft$$

$$V_{max} = \frac{wL}{2} = \frac{.383 \times 4}{2} = .77\ k$$

- $F_b = C_D \times 1640\ psi = 2050\ psi$

$$f_b = \frac{M_{max}}{S_x} = .77 \times 1000 \times 12 / 5.1 = 1812\ psi < F_b \quad (ok)$$

$$S_x = 5.1\ in^3$$

- $F_v = C_D \times 180\ psi = 225\ psi$

$$f_v = 3V_{max} / 2A_v = 132\ psi < F_v \quad (ok)$$

$$A_v = 8.75\ in^2$$

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LUMBER DESIGN VALUES

Suggested lumber details shown are based on the use of lumber with allowable unit sresses increased per ANSI/AF&PA NDS – 1997 for short term loading to the limiting values below:

Extreme fiber stress in bending	**1,640 PSI**
Horizontal shear	**180 PSI**
Compression perp. to grain	**625 PSI**
Compression parallel to grain	**1,350 PSI**
Modulus of elasticity	**1,600,000 PSI**

Face grain of plywood must run at right angles to its support. Plywood suggested in layout assumed to be A.P.A. plyform Class I, B-B exterior type PS 1-95 or equal in "as new" condition. Customer must make suitable allowances for lower grades or condition of plywood used.

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TITLE: 20KA SHORING LAYOUT	**DATE:** 12/14/2007
PROJECT: SOHO HOTEL	
LOCATION: 246 SPRING STREET NYC	
CUSTOMER: DI FAMA CONCRETE	

Patent

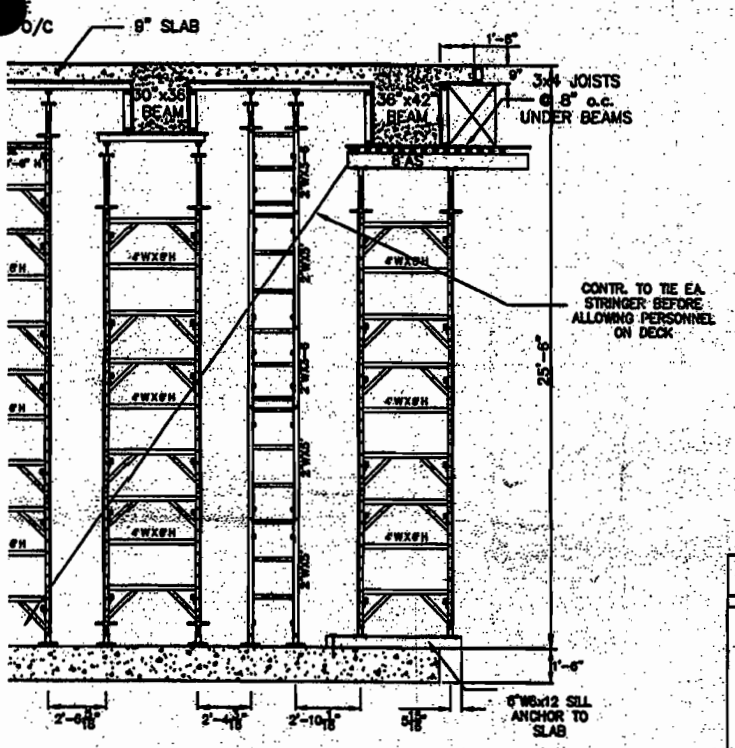
Patent Construction Systems
Harsco Corporation
PATENT CONSTRUCTION SYSTEMS
1800 Lower Road
Linden, New Jersey 07036
(732) 396-1289

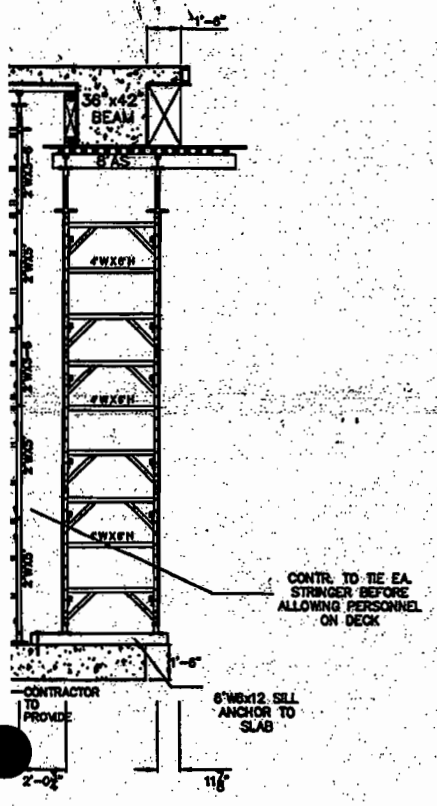
DRAWING NUMBER: 4607K070

SHEET NO. 1

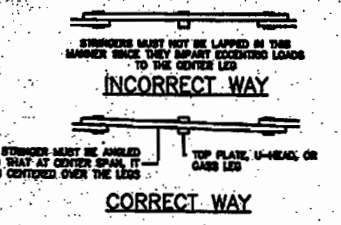
RECEIVED JAN 16 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT
RECEIVED JA 1 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT
RECEIVED JA 1 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT

3





Stringer Lapping Detail



LEGEND

#A/S - DENOTES ALUMINUM STRINGER LENGTH
#J - DENOTES J400 (JOIST) LENGTH
#S - DENOTES STEEL (W8X12) STRINGER LENGTH
W/S - DENOTES WOOD STRINGER (by Contractor)
⊗ - DENOTES PCS STEEL POLE SHORE

RECEIVED JAN 16 2008 DEPARTMENT OF BUILDINGS PLAN EXAMINING UNIT

ALL SHORING TOWERS ARE CENTERED UNDER CONCRETE BEAMS, BETWEEN COLUMN LINES, UNLESS OTHERWISE NOTED.

FOR IMPORTANT GENERAL NOTES AND ACCESS RECOMMENDATIONS APPLYING TO THIS PROJECT BE CERTAIN TO REFER TO SHEET NO. 1. DO NOT PROCEED WITH THE ERECTION OR USE OF THIS EQUIPMENT UNLESS YOU HAVE READ AND UNDERSTOOD THE SAFETY INFORMATION CONTAINED IN BOOKLET # SS-670 ATTACHED TO THESE DRAWINGS. IF YOU DO NOT HAVE THIS BOOKLET CALL 1-800-969-5600, EXT. 500221

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DRAWN:	CHECKED:	APPROVED:	DATE:	SCALE:
WMB			9/25/07	AS NOTED

TITLE:	20KA SHORING LAYOUT
PROJECT:	SOHO HOTEL
LOCATION:	246 SPRING STREET, NYC
CUSTOMER:	DI FAMA CONCRETE

patent
Patent Construction Systems

DRAWING NUMBER
4607K070

3 Reconstruction Of Formwork –Engineerng Analysis

3.1 Dead Weight On Typical Shoring Leg:

- Typical shoring tower is 4 feet by 8 feet (4 legs).
  The load on each leg is in the order of 7000 lbs based on 10' long simple supported aluminum stringer, and 7600 lbs with consideration of 12' long 2 span continuous aluminum stringer over the support.

- In the plywood punching scenario, the support for the shoring leg would be loose; the deflection in the point of loose support would be in the range of .5" to 1", and the reaction redistributed to adjacent shoring legs would have increased in the range of 10.5 kips to 11.5 kips. Our calculation also shows the likelihood of developing 2% lateral force due to plywood punching.

- Capacity of Aluminum welds, horizontal rungs, knee braces, and cross braces

- Capacity of Screw Jack

Project: 246 Spring Street
Subject: Typical shoring load

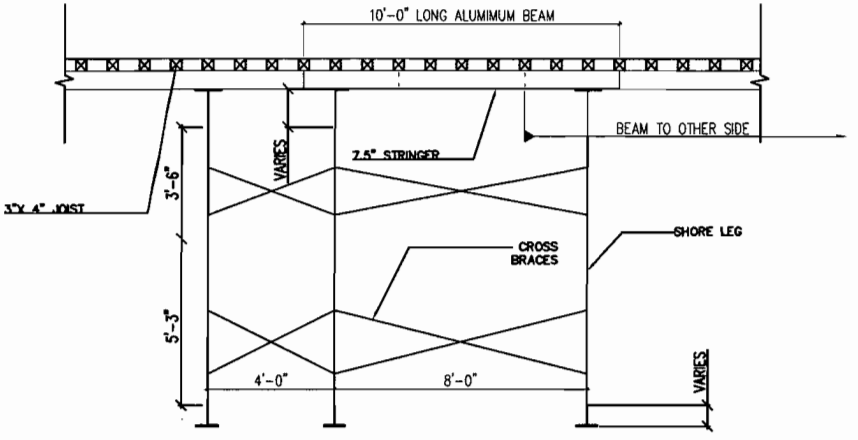
Engineer: JW
Checked by:

Date: 5/8/2009
Date:

Load Take Down

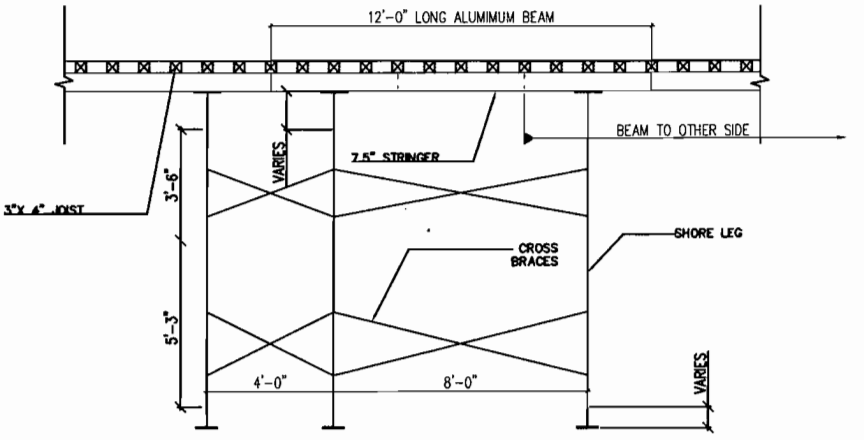
Typical 4' x 8' shoring tower (4 legs)									
Formwork	Trib Area (sf)	Formwork (psf)	Slab Trib Area (sf)	Slab (psf)	Beam Trib Area (SF)	Beam (psf)	Base Load on Ea Leg (lbs) Simple Supported	12' Long Stringer (lbs) (1.09 factor) 2-continuous spans	16' Long Stringer (lbs) (1.06 factor) 3-continuous spans
Supporting 9" Slab	96	10	96	112.5	0	525	**2940**	**3205**	**3116**
Supporting 36"x 42"Beam	96	25	60	112.5	36	525	**7013**	**7644**	**7433**

Typical 4' x 4' shoring tower (4 legs)									
Formwork	Trib Area (sf)	Formwork (psf)	Slab Trib Area (sf)	Slab (psf)	Beam Trib Area (SF)	Beam (psf)	Base Load on Ea Leg (lbs) Simple Supported	12' Long Stringer (lbs) (1.09 factor) 2-continuous spans	16' Long Stringer (lbs) (1.06 factor) 3-continuous spans
Supporting 54" x 42"Beam	64	25	28	112.5	36	525	**5913**	**6445**	**6267**



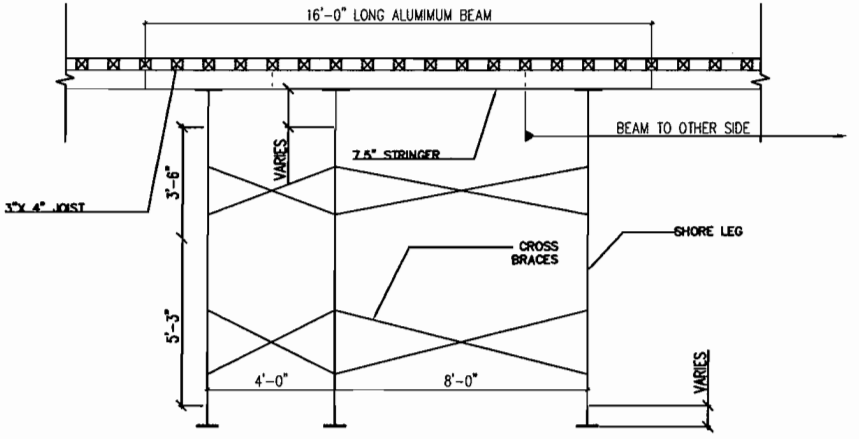
LOAD OVER 10'-0" LONG BEAM- SIMPLE SUPPORTED

SCALE: N.T.S.



LOAD OVER 12'-0" LONG BEAM-SIMPLE SUPPORTED W/2' CANT.

SCALE: N.T.S.



LOAD OVER 16'-0" LONG BEAM-CONT. SUPPORTED

SCALE: N.T.S.

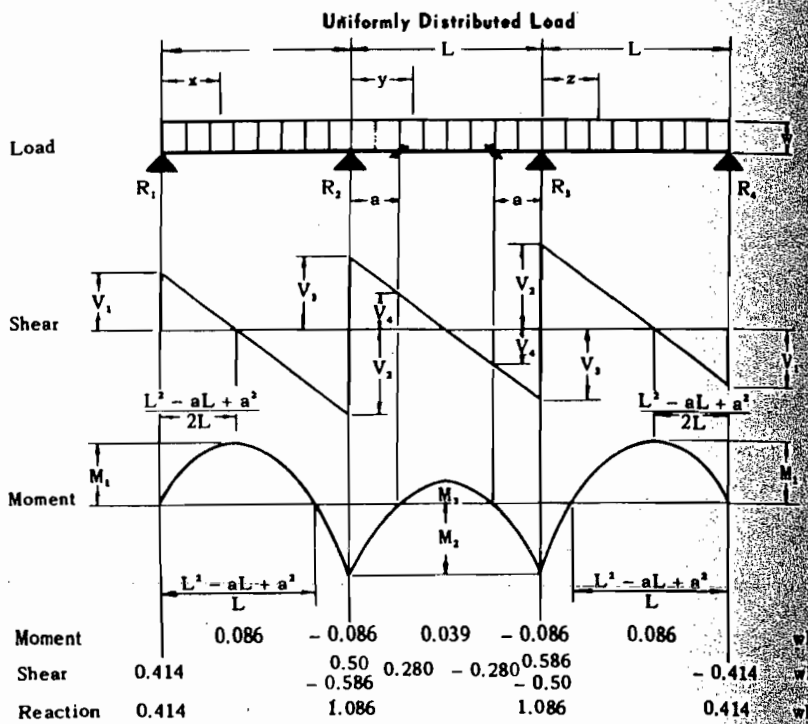


FIGURE 7.25. Cantilevered beam diagram: Three equal spans—single cantilever each end.

For the maximum positive and negative moments of the cantilevered portions of the beam to be equal, $a = 0.220L$, and the above coefficients may be applied to wL^2 and wL to find the respective critical values of moment, shear, and reaction. Maximum deflection in end spans will be

$$\Delta = 13.31\,\frac{wL^4}{EI} \quad \text{in.}$$

General formulas are:

$$R_1 = R_4 = \frac{w}{2L}(L^2 - aL + a^2) \qquad M_1 = -\frac{w}{8L^2}(L^2 - aL + a^2)^2$$

$$R_2 = R_1 = \frac{w}{2L}(2L^2 + aL - a^2) \qquad M_2 = -\frac{w}{2}(aL - a^2)$$

$$V_1 = \pm\frac{w}{2L}(L^2 - aL + a^2) \qquad M_3 = \frac{w}{8}(L - 2a)^2$$

$$V_2 = \pm\frac{w}{2L}(L^2 + aL - a^2) \qquad V_x = \frac{w}{2L}(L^2 - aL + a^2) - wx$$

$$V_3 = \pm\frac{wL}{2} \qquad V_y = \frac{w}{2}(L - 2y)$$

$$V_4 = \pm\frac{w}{2}(L - 2a) \qquad V_z = \frac{w}{2L}(L^2 + aL - a^2) - wz$$

$$M_x = \frac{wx}{2L}(L^2 - aL + a^2) - \frac{wx^2}{2} \qquad M_y = \frac{w}{2}(y - a)(L - y - a) \qquad M_z = \frac{w}{2}(L - z)\left(\frac{a}{L}\right.$$

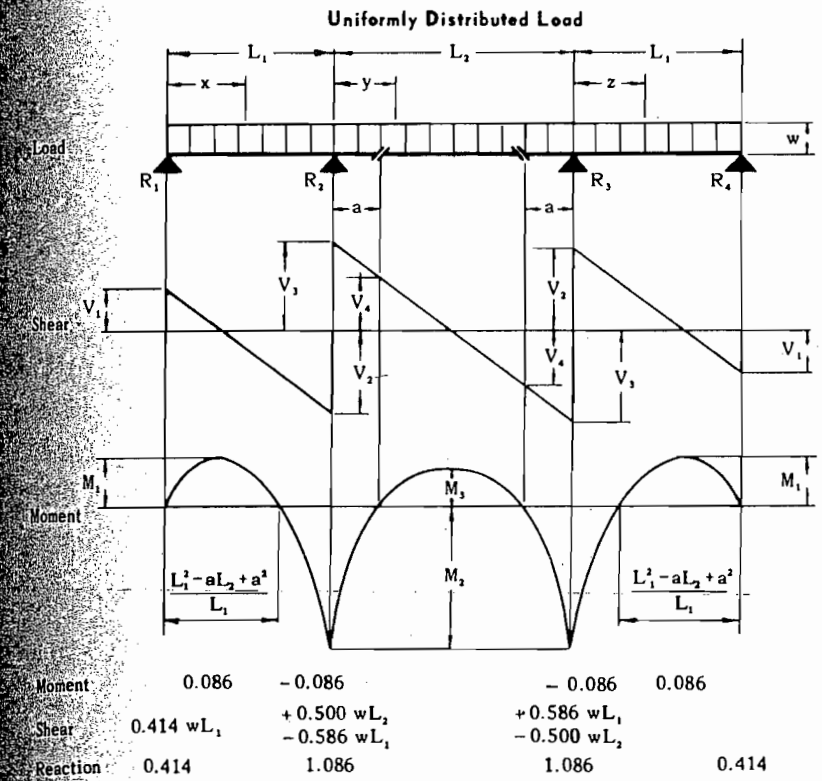


FIGURE 7.26. Cantilevered beam diagrams: Three spans—end spans equal—single cantilever each end.

For the maximum positive and negative moments of the cantilevered portions of the beam to be equal, $a = \frac{1}{2}(L_2 - \sqrt{L_2^2 - 0.688L_1^2})$ and the above coefficients may be applied to find the respective critical values of moment, shear, and reaction. Coefficients are omitted when calculation using the general formula is simpler. General formulas are:

$$\frac{w}{2L_1}(L_1^2 - aL_2 + a^2) \qquad V_4 = \pm\frac{w}{2}(L_2 - 2a) \qquad M_2 = -\frac{w}{2}(aL_2 - a^2)$$

$$\frac{w}{2L_1}(L_1 + a)(L_1 + L_2 - a) \qquad M_1 = \frac{w}{8L_1^2}(L_1^2 - aL_2 + a^2)^2 \qquad M_3 = \frac{w}{8}(L_2 - 2a)^2$$

$$\frac{w}{2L_1}(L_1^2 - aL_2 - a^2) \qquad V_x = \frac{w}{2L_1}(L_1^2 - aL_2 + a^2) - wx \qquad M_x = \frac{wx}{2L_1}(L_1^2 - aL_2 + a^2) - \frac{wx^2}{2}$$

$$\frac{w}{2L_1}(L_1^2 - aL_2 + a^2) \qquad V_y = \frac{w}{2}(L_2 - 2y) \qquad M_y = \frac{w}{2}(y - a)(L_2 - y - a)$$

$$\frac{wL_2}{2} \qquad V_z = \frac{w}{2L_1}(L_1^2 - aL_2 + a^2) - wz \qquad M_z = \frac{w}{2L_1}(L_1 z - aL_2)(L_1 - z)$$

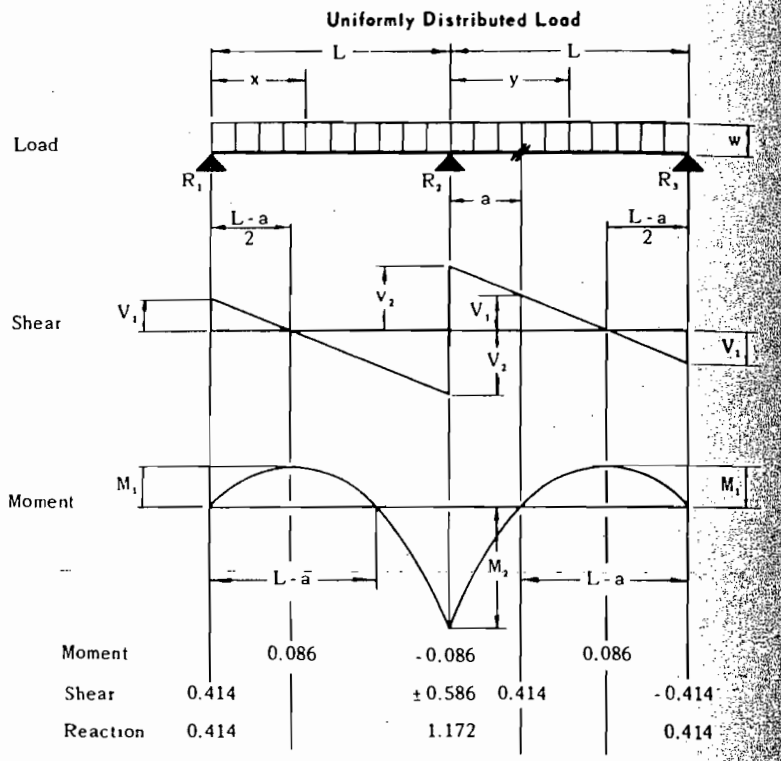


FIGURE 7.23. Cantilevered beam diagram. Two equal spans.

For maximum positive moment equal to maximum negative moment, $a = 0.172L$, and the above coefficients may be applied to wL^2 and wL to find the respective critical values of moment, shear, and reaction. Maximum deflection in either span will be

$$\Delta = 13.31\,\frac{wL^4}{EI} \quad \text{in.}$$

General formulas are:

$$R_1 = R_3 = \frac{w}{2}(L - a) \qquad V_y = \frac{w}{2}(L + a - 2y)$$

$$R_2 = w(L + a) \qquad M_1 = \frac{w}{8}(L - a)^2$$

$$V_1 = \pm\frac{w}{2}(L - a) \qquad M_2 = -\frac{wLa}{2}$$

$$V_2 = \pm\frac{w}{2}(L + a) \qquad M_x = \frac{wx}{2}(L - a - x)$$

$$V_x = \frac{w}{2}(L - a - 2x) \qquad M_y = \frac{w}{2}(y - a)(L - y)$$

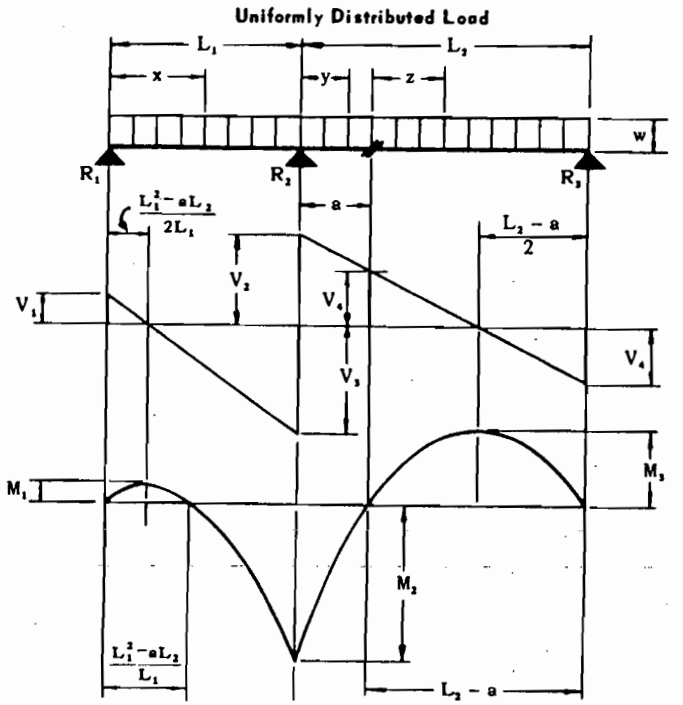


FIGURE 7.24. Cantilevered beam diagram: Two unequal spans.

For maximum positive and negative moments in the cantilevered portion to be equal, $a = 0.172L_1^2/L_2$. Under these conditions, $M_1 = M_2 = 0.086wL_1^2$, $R_1 = V_1 = 0.414wL_1$, and $V_3 = -0.586wL_1$. Other coefficients can be determined for the above or other values of a from the general formulas following:

$$R_1 = \frac{w}{2L_1}(L_1^2 - aL_2) \qquad V_3 = -\frac{w}{2L_1}(L_1^2 - aL_2) \qquad M_2 = -\frac{wL_2a}{2}$$

$$R_2 = \frac{w}{2L_1}(L_1 + a)(L_1 + L_2) \qquad V_4 = \pm\frac{w}{2}(L_2 - a) \qquad M_3 = \frac{w}{8}(L_2 - a)^2$$

$$R_3 = \frac{w}{2}(L_2 - a) \qquad V_x = \frac{w}{2L_1}(L_1^2 - aL_2) - wx \qquad M_x = \frac{wx}{2L_1}(L_1^2 - xL_1 - aL_2)$$

$$V_1 = \frac{w}{2L_1}(L_1^2 - aL_2) \qquad V_y = \frac{w}{2}(L_2 - a) - wy \qquad M_y = \frac{w}{2}(L_2 - y)(y - a)$$

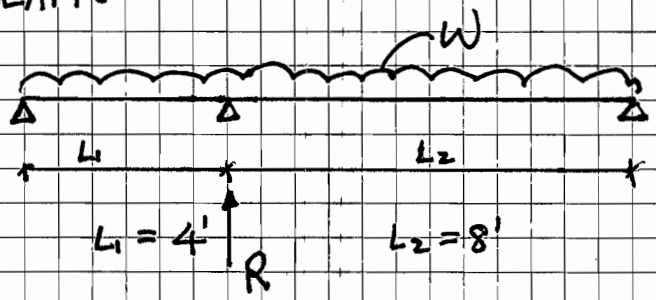
$$V_2 = \frac{w}{2}(L_2 + a) \qquad V_z = \frac{w}{2}(L_2 - a) - wz \qquad M_z = \frac{w}{2}(L_2 - a - z)$$

$$M_1 = \frac{w}{8L_1^2}(L_1^2 - aL_2)^2$$

Project: _____ Date: _____ Page: ____ of ____

Subject: _____ Engr: _____ Checked By: _____ Date: _____

12' LONG BEAM:



2 SPAN CONTINUOUS SUPPORT

$$a = .172\, L_1^2 / L_2 = .172 \times 4^2 / 8 = .344'$$

$$R = \frac{W}{2L_1}(L_1 + a)(L_1 + L_2)$$

$$= \frac{W}{2 \times 4}(4' + .344')(4' + 8')$$

$$= 6.516\, W$$

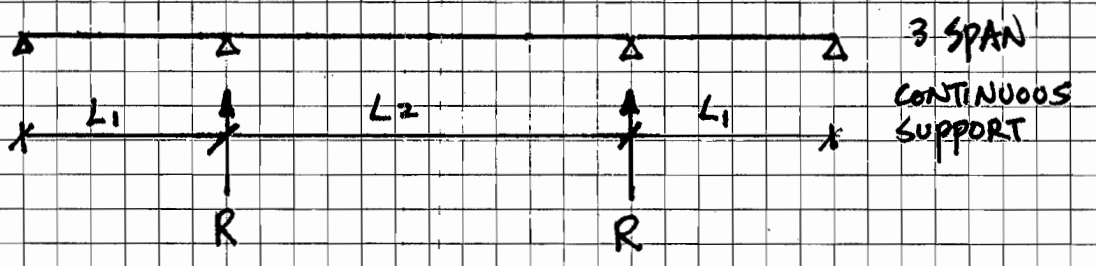
$$R_{\text{SIMPLE SUPPORTED}} = W(L_1 + L_2)/2 = 6W$$

$$\text{RATIO} = R / R_{\text{SIMPLE SUPPORTED}} = \frac{6.516}{6} = 1.09$$

Project: _____ Date: _____ Page: ____ of ____

Subject: _____ Engr: _____ Checked By: _____ Date: _____

16' LONG BEAM



$L_1 = 4'$; $L_2 = 8'$

$$a = \frac{1}{2}\left(L_2 - \sqrt{L_2^2 - .688 L_1^2}\right) = .36'$$

$$R = \frac{w}{2L_1}(L_1 + a)(L_1 + L_2 - a)$$

$$= \frac{w}{2 \times 4'}(4' + .36')(4' + 8' - .36')$$

$$= 6.34\, w$$

$$R_{\text{SIMPLE SUPPORTED}} = 6w$$

$$\text{RATIO} = 6.34/6 = 1.06$$

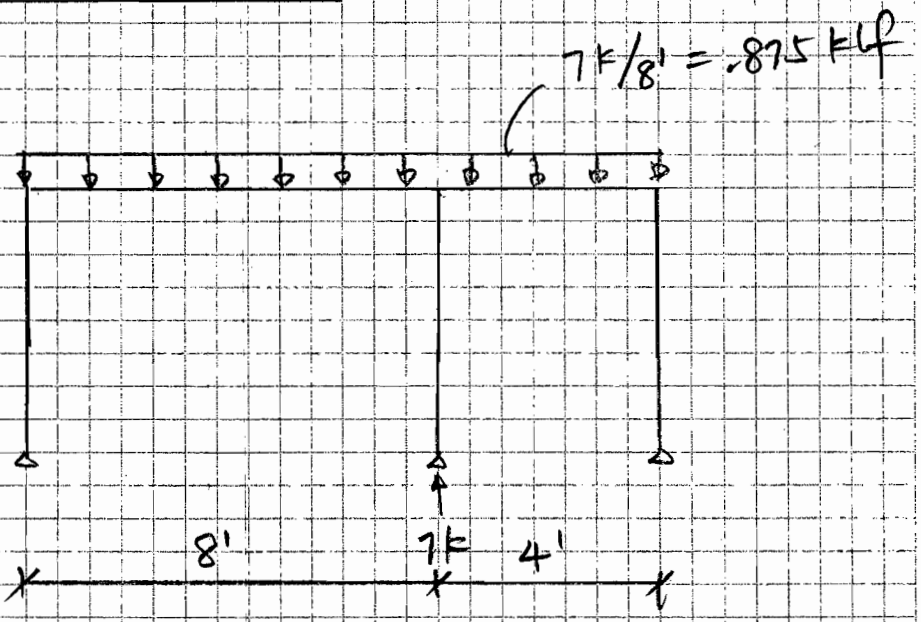
REACTION DISTRIBUTION DUE TO LOOSE SUPPORT

Project: 246 Spring Street Date: _____ Page: ____ of ____

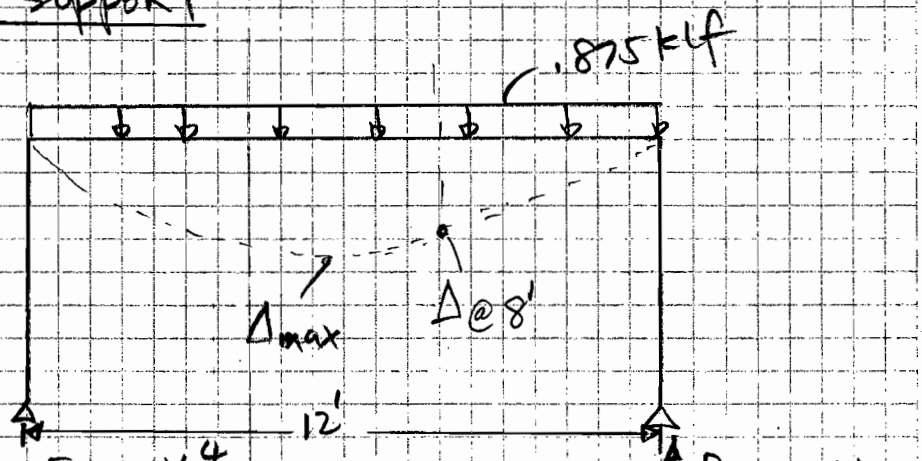
Subject: DEFLECTION @ ALUMINUM STRINGERS Engr: _____ Checked By: _____ Date: _____

① CONDITION 1 – MIDDLE SUPPORT IS LOOSE

ORIGINAL CONDITION



LOOSE SUPPORT



- $\Delta_{max} = \frac{5}{384} \frac{wL^4}{EI} = 1.16''$

$E = 10100$ ksi

$I = 34.8$ in^4

$\Delta_{@8'} = \frac{w \times 8'}{24EI}(12^3 - 2\times12\times8^2 + 8^3) \times 1728 = 1''$

- R_B TOTAL $= \frac{7k}{2} + 7k = 10.5$ k

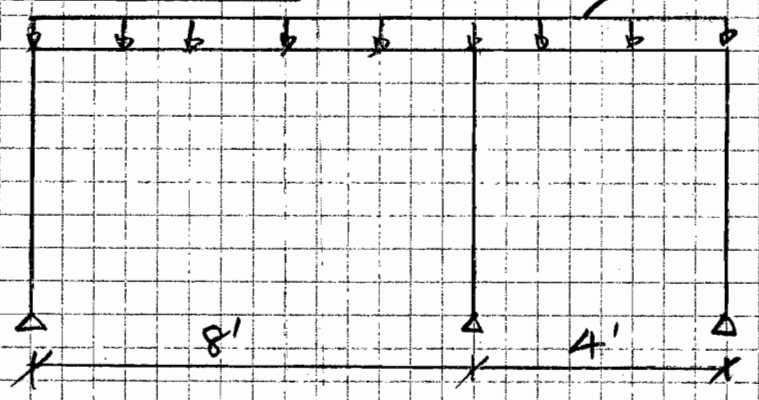
Project: 246 Spring Street Date: _____ Page: ____ of ____

Subject: DEFLECTION @ ALUMINUM STRINGER Engr: _____ Checked By: _____ Date: _____

② CONDITION 2 – OUTER SUPPORT IS LOOSE

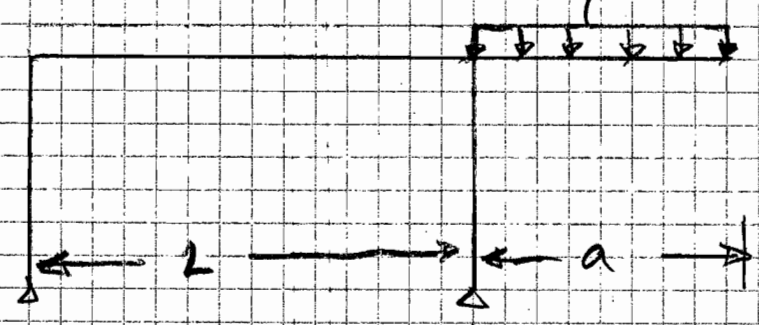
ORIGINAL CONDITION

$w = 7k/8' = .875\ k/f$



LOOSE SUPPORT

$F = .875 \times 4' = 3.5k$



$$R_B = \frac{3.5 \times (8' + 2')}{8'} = 4.375\ k$$

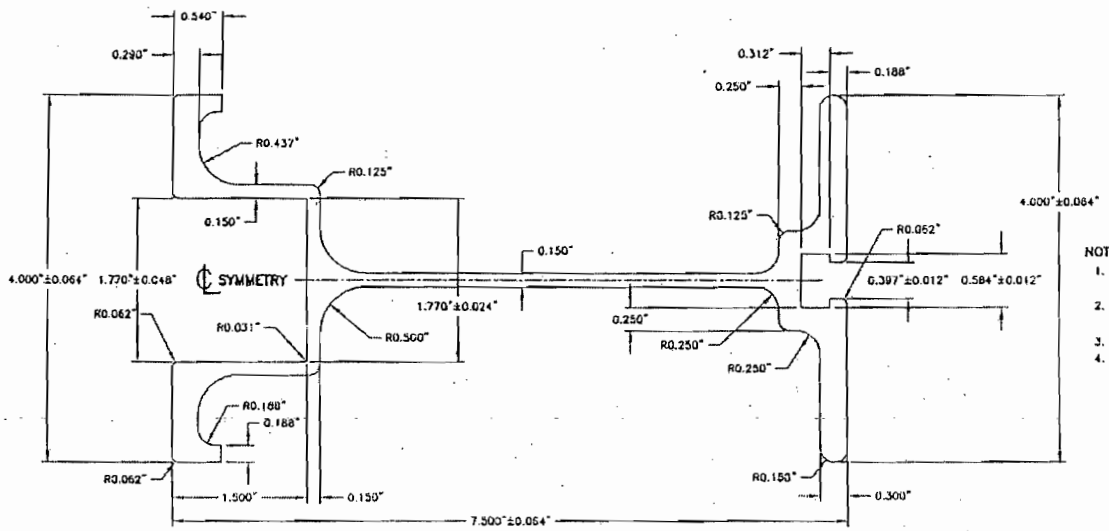
$$\Delta_{max} = \frac{wa^3}{24EI}(4L + 3a) = \frac{.875 \times 4^3 \times 1728}{24 \times 10100 \times 34.8}(4 \times 8 + 3 \times 4) = .5''$$

$E = 10100\ ksi$

$I = 34.8\ in^4$

$$R_{BTOTAL} = R_B + 7k = 11.375\ k$$

REVISIONS			
LTR	DESCRIPTION	DATE	APPROVED
D	REDRAWN. ADDED NOTES. REVISED SECTION PROPERTIES AND WEIGHT.	8-23-99	



NOTES:

1. STANDARD EXTRUSION TOLERANCES APPLY UNLESS OTHERWISE NOTED.
2. BOLT SLOT AT BOTTOM SIZED FOR 3/8" HEX HEAD BOLT.
3. SHAPE IS SYMMETRIC ABOUT CENTERLINE AS SHOWN.
4. SECTION PROPERTIES: (CALCULATED)
   AREA: 3.8310 SQ IN
   MOMENTS OF INERTIA: Ix = 34.854
   Iy = 3.921
   SECTION MODULUS: Sx top = 8.954
   Sy = 1.960
   RADIUS OF GYRATION: Rx = 3.016
   Ry = 1.012
   NEUTRAL AXIS LOCATION: 3.8926 IN FROM TOP

$$y_{TOP} = \frac{I}{S_{x\,TOP}} = \frac{34.9}{8.954} = 3.9''$$

$$S_{x\,BOTT} = \frac{I}{(7.5''-3.9'')} = 9.7\ in^3$$

THIS DRAWING IS LOANED WITH THE EXPRESSED AGREEMENT THAT THE DRAWING AND INFORMATION THEREIN CONTAINED ARE THE PROPERTY OF PATENT CONSTRUCTION SYSTEMS, HARSCO CORPORATION AND WILL NOT BE REPRODUCED, COPIED OR OTHERWISE DISPOSED OF, DIRECTLY OR INDIRECTLY, AND WILL NOT BE USED IN WHOLE OR IN PART TO ASSIST IN MAKING OR TO FURNISH ANY INFORMATION FOR THE MAKING OF DRAWINGS, PRINTS OR OTHER REPRODUCTIONS HEREOF, OR FOR THE MAKING OF APPARATUS OR PARTS [illegible], EXCEPT UPON WRITTEN [illegible] OF PATENT CONSTRUCTION [illegible] CORPORATION FIRST [illegible] SPECIFIC AS TO EACH [illegible] OF THIS DRAWING [illegible] IN ACCEPTANCE [illegible]

NEXT ASSY.	USED ON
APPLICATION	

DO NOT SCALE DRAWING
UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN INCHES
TOLERANCES ON:
FRACTIONS: 0–24" ± 1/32
24" AND UP ± 1/16
DECIMALS: ± .010
ANGLES: ± 1° –0'

MATERIAL: 6061-T6 ALUMINUM

FINISH:

CONTRACT NO.

PREPARED	S.A.S.	8-23-99
CHECKED		
APPROVED		

OTHER APPROVAL:

Patent Construction Systems
a harsco company
Patent Construction Systems, Harsco Corporation
Executive office
One Mack Centre Drive
Paramus, NJ USA 07652
(201)261-5800

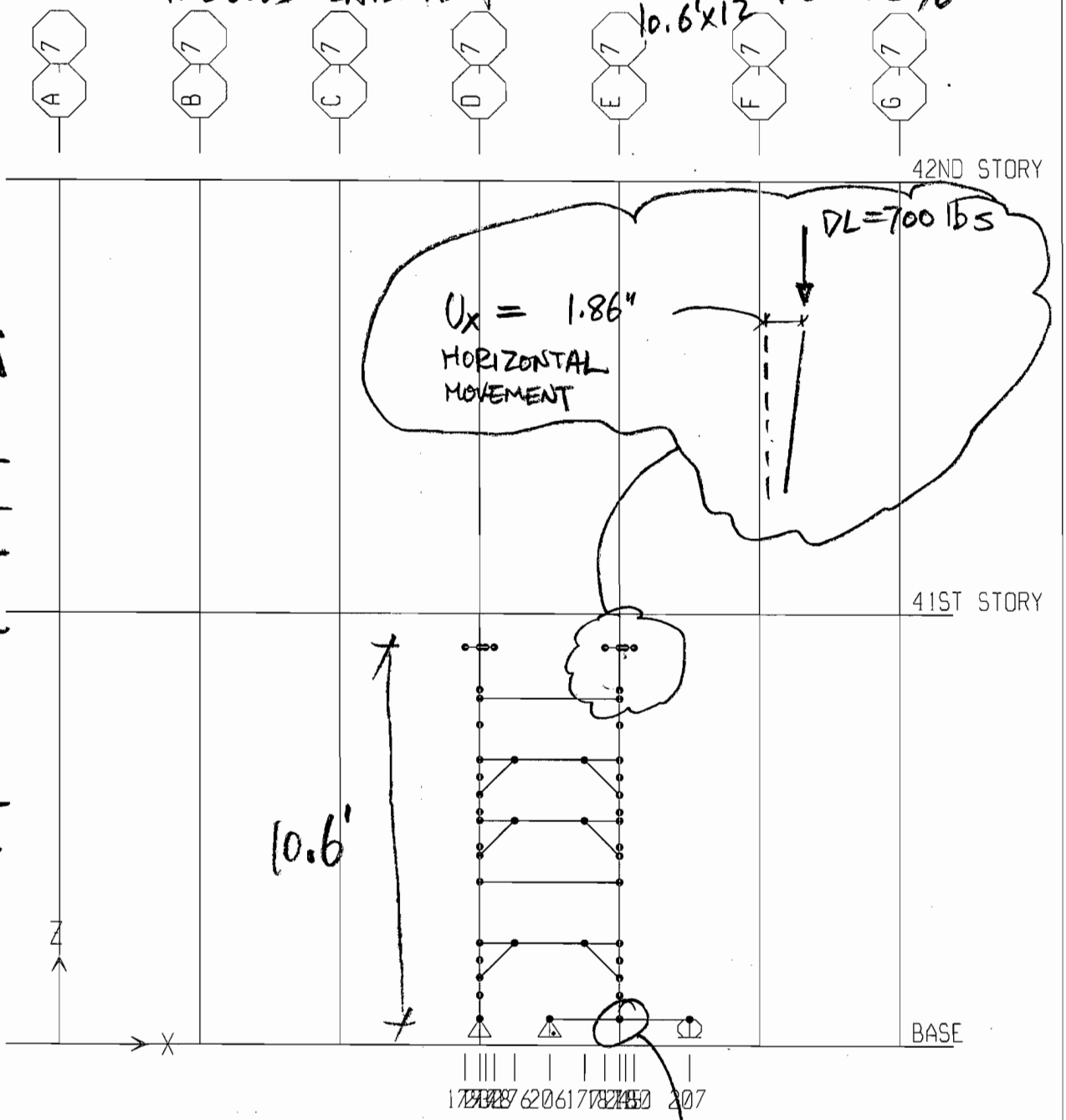
EXTRUSION PROFILE
ALUMINUM STRINGER
INTERFORM

SIZE	CODE IDENT NO.	DRAWING NO.	REV.
B	45826	46001	D

SCALE FULL | UNIT WEIGHT 4.505 LBS/FT | SHEET 1 OF 1

- LOADING CONDITION 2" ECCENTRIC
- BASED ON GEOMETRY
  CALC SHOWS 1.5% INDUCED LATERAL FORCE.

- INDUCED LATERAL FORCE $= \frac{1.86''}{10.6' \times 12} \cdot DL = 1.5\%\ DL$



- DEVELOPING 2% LATERAL FORCE BY PUNCHING OF PLYWOOD.

.8" DEFLECTION LIMIT DUE TO PLYWOOD PUNCH.

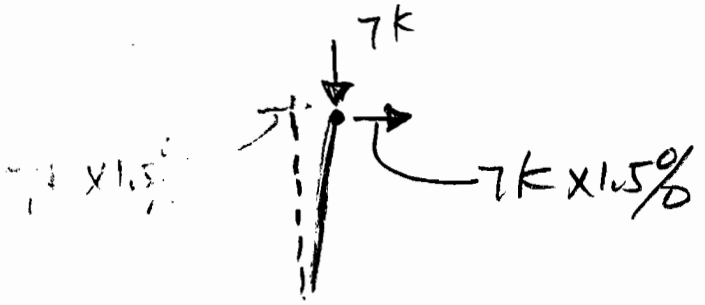
ETABS v9.0.7 File:246SPRINGST_2009_05_08_SINGLETOWERDL_2INCHECCDEFLECTIONLIMIT Units:Kip-in July 22, 2009 18:01

P O I N T D I S P L A C E M E N T S

STORY	POINT	LOAD	UX	UY	UZ	RX	RY	RZ
41ST STORY	175-12	DEAD	1.8592	0.9090	-0.8942	-0.00874	0.03393	0.00000

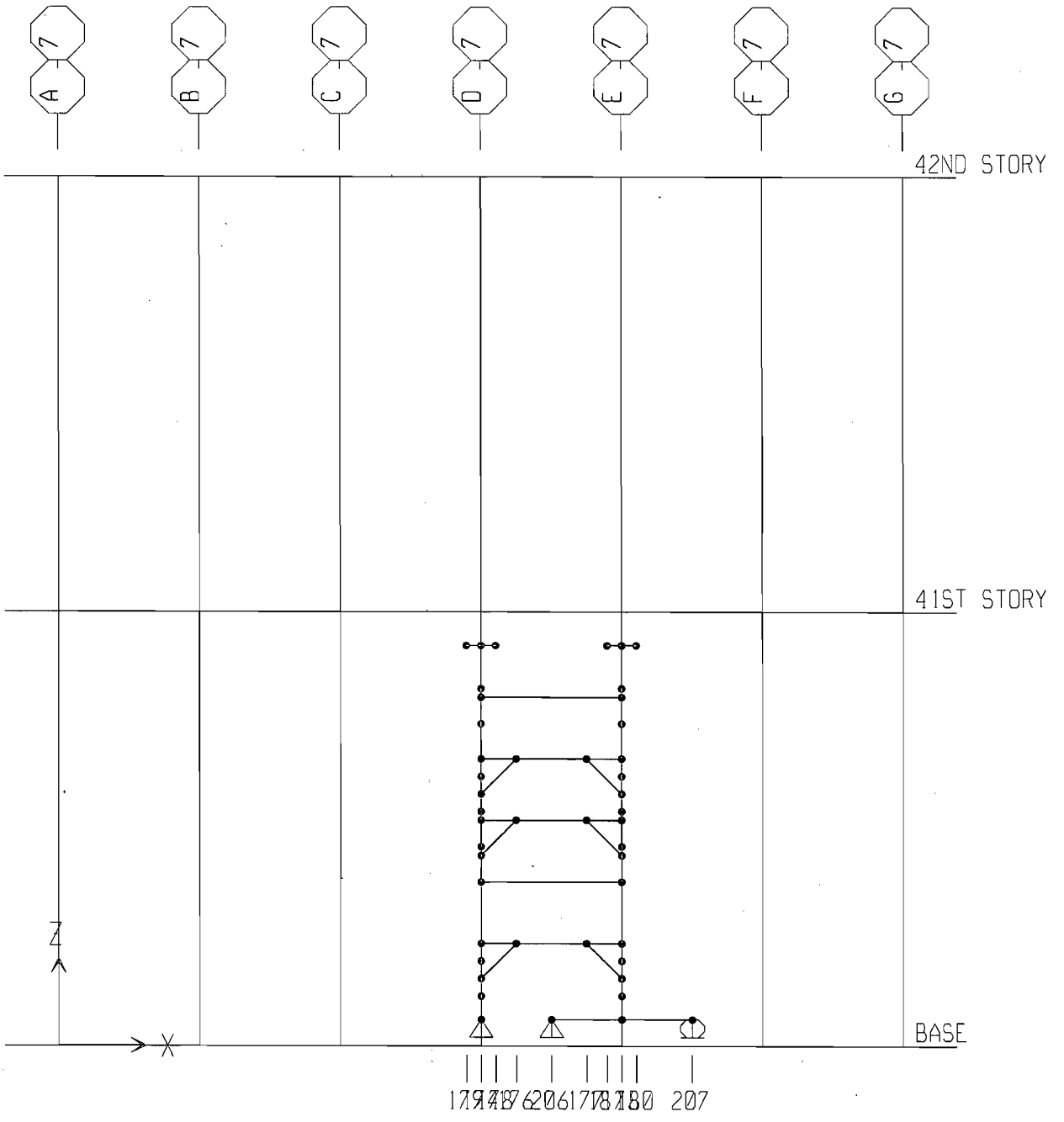
HORIZONTAL DEFLECTION ON THE TOP CAUSED BY THE DEAD LOAD AND 18" LIMITED SUPPORT DUE TO PUNCHING OF PLYWOOD,

$$7\ \text{kip} \times \frac{1.86''}{10.6' \times 12} = 1.5\%$$



BASED ON GEOMETRY

12



P O I N T D I S P L A C E M E N T S

STORY	POINT	LOAD	UX	UY	UZ	RX	RY	RZ
41ST STORY	175-12	DEAD	1.0567	0.8401	-0.8270	-0.00806	0.00196	0.00000

• CAPACITY OF ALUMINUM WELD, RUNG AND KNEE BRACE

Project: 246 Spring Street Date: 1/5/2009 Page: ____ of ____

Subject: _____ Engr: _____ Checked By: _____ Date: _____

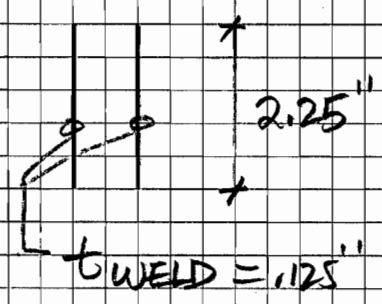
HORIZONTAL RUNG WELD TO SHORE LEG

• WELD CAPACITY: FACTOR OF SAFETY = 1.65

$t_{WELD} = \frac{1}{8}'' = .125''$ BUTT WELD

ER4043 → TENSILE STRENGTH = 24 ksi
SHEAR STRENGTH = 15 ksi

$S_x = 2 \times (.125 \times .707) \times 2.25^2/6$
$= .15 \text{ in}^3$



$F_{TS} = 24$ ksi (AT FAILURE)

$M_{ULTIMATE} = 3.6$ k-in
(FRACTURE MOMENT)

$M_{allowable} = 3.6/1.65 = 2.2$ k-in

$F_V = 15$ ksi (AT FAILURE)

$V_{ULTIMATE} = (.125'' \times .707) \times 2.25'' \times 2 \times 15$ ksi
$= 6$ kips

$V_{allowable} = 6/1.65 = 3.64$ k

Section 1. General

1.1 Scope

This *Specification* shall apply to the design of aluminum alloy load carrying members.

1.2 Materials

The materials to which this *Specification* applies are aluminum alloys registered with the Aluminum Association. Those frequently used for structural members are listed in Table 3.3-1, Section 3, Minimum Mechanical Properties. Applicable ASTM specifications are designations B209, B210, B211, B221, B241, B247, B308, and B429.

1.3 Safety Factors

1.3.1 Building Type Structures

Basic allowable tensile stresses for buildings, structural supports for highway signs, luminaires, traffic signals and similar structures shall be the lesser of the minimum yield strength divided by a factor of safety of 1.65, or the minimum ultimate tensile strength divided by a factor of safety of 1.95. Other allowable stresses for buildings and similar structures shall be based upon the factors of safety shown in Table 3.4-1.

1.3.2 Bridge Type Structures

Basic allowable tensile stresses for bridge type structures shall be the lesser of the minimum yield strength divided by a factor of safety of 1.85, or the minimum ultimate tensile strength divided by a factor of safety of 2.2. Other allowable stresses for bridge and similar structures shall be based upon the factors of safety shown in Table 3.4-1.

1.3.3 Other Type Structures

Where it is customary or standard practice to use factors of safety other than those given in Sections 1.3.1 or 1.3.2, the general formulas in Table 3.4-3 shall be permitted to be used with the desired factors of safety substituted for n_u, n_y or n_a.

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Sold to:NEW YORK DEPT OF BUILDING, 01718631
2008/9/16 22:21:7 GMT

STRUCTURAL ENGINEERING HANDBOOK — GAYLOAD & GAYLORD

Holes. In structures such as transmission towers and electrical substations, it is common practice to punch or drill bolt holes 1/16 in. larger than the nominal bolt diameter. In other classes of structures, where the small amount of slip that may occur with oversized holes is not permissible, bolt holes may be drilled or reamed to give a driving fit or a small clearance, such as 1/50 in. on the diameter. Allowable bearing stresses on bolts are determined in the same way as for rivets, except that the allowable bearing load on a bolt is based on the nominal diameter of the bolt rather than on the hole diameter, as in the case of rivets. Allowable shear and bearing stresses on bolts are independent of the hole clearance, as long as the hole diameter is not more than 1/16 in. greater than the nominal diameter of the bolt.

Tightening. Bolts function best when properly tightened. No definite rules can be specified for tightening torques, since the proper torque depends on the friction developed in the threads and other bearing surfaces, which in turn is greatly influenced by the degree of lubrication, accidental or intentional. One recommendation that is often made with regard to tightening aluminum-alloy bolts is as follows: Tighten several bolts of any given size and type to the breaking point under the same conditions of lubrication that will be encountered on the job, and then use 70 or 80 percent of the lowest torque obtained in these tests for tightening all bolts of this size and type on the job. The 70 percent value should be used for "temporary" bolts, or those that may need to be removed occasionally, while the 80 percent value applies to "permanent" bolts. The use of a good lubricant on the threads and all bearing surfaces is recommended. A lubricant having a white-lead base, prepared for threaded fittings and meeting the requirements of Military Specification JAN-A-6669, will give excellent results both as a lubricant and as a protective coating on the contacting surfaces.

25. Welded Connections. Aluminum alloys can be joined by arc welding, resistance welding, gas welding, or brazing. Arc cutting is also used on aluminum. The most commonly used welding process employs a shield of inert gas such as argon or helium to inhibit oxide formation during welding. These processes have almost entirely displaced the use of flux-coated electrodes for welding aluminum. The electrode used in the inert-gas welding processes may be either consumable metal (MIG welding) or tungsten (TIG welding). Both processes make superior joints from the standpoint of consistent weld soundness, both can be used for welding in any position, and neither requires postweld cleaning. Higher welding speeds are attained with the consumable-electrode process, which is preferred for most structural welding. The process can be employed on metal 1/16 in. or more in thickness. Tungsten-electrode welding is generally used for material in the thickness range from 0.050 to 0.250 in.

The most commonly used structural aluminum alloys are all readily weldable. Some of these alloys are listed in Table 13, which gives data on weld strengths. Butt welds made in aluminum alloys in the annealed condition are usually 100 percent efficient; that is, the joint is at least as strong as the parent metal. When butt welds are made in aluminum alloys in the strain-hardened or heat-treated tempers, however, the heat of welding softens the metal on each side of the weld so that it is not so strong as the parent metal. Butt welds in the non-heat-treatable alloys have approximately the strength of annealed material, even though the welded parts may have been strain-hardened prior to welding. The strength of a butt weld in heat-treated material is usually intermediate between the strength of the parent metal and the strength of the fully annealed material. Table 13 shows the static tensile strengths of butt welds in various aluminum alloys. These strength values are equal to the minimum strengths required in the ASME weld qualification tests.

The yield strength measured across butt welds made in cold-worked or heat-treated aluminum alloys depends on the gage length used in the measurement, the yield strength increasing with gage length. The yield-strength value determined at 0.2 percent offset on a 10-in. gage length is considered to be applicable to the design of many welded structures.[3] Minimum expected values for the 10-in. gage-length yield strength across butt welds in various alloys are listed in Table 13.

Fillet welds, as well as butt welds, are used in aluminum-alloy construction, and Table 13 gives minimum expected strengths of fillet welds made with various filler-metal alloys.

FILLET WELDS

Heat-affected Zone. The strength of the heat-affected material in the vicinity of a weld has a minimum value in a narrow zone adjacent to the weld. Outside this zone, the strength increases until it reaches the strength of the unaffected parent metal at a short distance from the weld. Methods of accounting for the reduced strength of heat-affected zones in design are discussed in Arts. 5, 11, and 12.

Welded assemblies are sometimes of such size and shape that they can be re-heat-treated after welding, assuming they are built of a heat-treatable alloy. This procedure greatly increases the strength of the welded material but results in some

Table 13. Minimum[a] Strengths (ksi) for Welded Joints (TIG or MIG Welding with No Postweld Heat Treatment)

Parent metal	Filler wire[b]	Strength of butt welds				Shear strength of fillet welds[e]
		TS[c]	TYS[d]	CYS[d]	SS	
3003-H14 or 24	1100	14	7	7	10	7.5
3004-H34 or 24	4043	22	11	11	14	11.5
5083-H111	5356	39	21	20	23	17
5083-H321	5356	40	24	24	24	17
5086-H111	5356	35	18	17	21	17
5086-H34	5356	35	19	19	21	17
5454-H111	5554	31	16	15	19	17
5454-H34	5554	31	16	16	19	17
5456-H111	5556	41	24	22	24	20
5456-H321	5556	42	26	24	25	20
6061-T6	5556	24	20	20	15	20
6061-T6	5356	24	20	20	15	17
6061-T6	4043	24	15[f]	15[f]	15	11.5
6063-T5	4043	17	11	11	11	11.5
6063-T6						
6070-T6	5556	28	24	24	17	20

TS = tensile strength, ksi
TYS = tensile yield strength, ksi
CYS = compressive yield strength, ksi
SS = shear strength, ksi

[a] These are minimum expected strength values to be used as basis for design. Typical or average strength values are appreciably higher.

[b] Filler wires listed are commonly used. They do not necessarily represent recommended filler wires for all applications.

[c] These are ASME weld-qualification test-requirement values. The design strength is often considered as 90 percent of these values.

[d] Yield strength across a butt weld corresponds to 0.2 percent set on a 10-in. gage length.

[e] Applicable to throat area of fillet. For double fillet welds stressed in transverse shear, the strengths are somewhat higher than the values listed.

[f] These values apply for thicknesses of 3/8 in. or more. For smaller thicknesses, the strengths listed for 5356 and 5556 filler wire may also be used for 4043 filler.

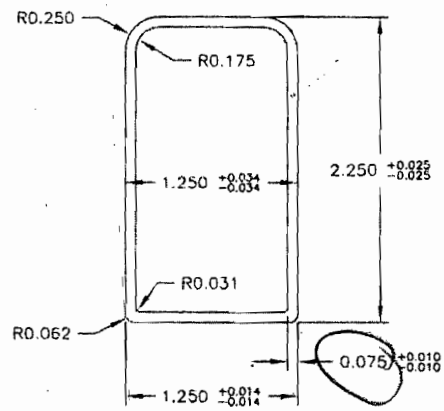
sacrifice in ductility of the weld. Another disadvantage is the warping which often accompanies the heat-treating procedure.

Factor of Safety. Although the tensile strengths across butt welds listed in Table 13 are equal to the ASME weld-qualification test values, they are not generally considered to have the same reliability as the minimum mechanical properties of the parent metal. Accordingly, it has been recommended[3] that a factor of 0.9 be applied to these minimum strength values before dividing by the factor of safety on ultimate strength to obtain allowable stresses. The factor of safety on yielding is applied to the 10-in. gage-length yield strength across butt welds, and the allowable stress is chosen as the lower of the two values obtained from the yield and ultimate strengths. This procedure was followed in establishing the allowable stresses across butt welds in the

17

APPLICATION	
NEXT ASSY.	USED ON
2403-2A	2403B
2403-2A	2403C
2403-2A	2403D
2403-2A	2403E

REVISIONS			
LTR	DESCRIPTION	DATE	APPROVED
C	REDRAWN	1-30-07	



AREA = 0.451 IN²

MATERIAL – ALUMINUM ALLOY 6061-T6

NO.	QTY.	DESCRIPTION	DWG. NO.

DO NOT SCALE DRAWING

UNLESS OTHERWISE SPECIFIED DIMENSIONS ARE IN INCHES

TOLERANCES ON:

FRACTIONS: 0-24" ± 1/32
24" AND UP ± 1/16
DECIMALS: ± .010
ANGLES: ± 1° -0'

Patent

Patent Construction Systems
Harsco Corporation
One Mack Centre Drive - Paramus, NJ USA 07652 - (201) 261-9600

RUNG EXTRUSION
RUNG
20KA ALUMINUM FRAME
20KA SHORING SYSTEM

PREPARED	TPK	1-30-07
CHECKED		
APPROVED		
OTHER APPROVAL		

SIZE	CODE IDENT NO.	DRAWING NO.	REV.
A	45826	2403-2	C

SCALE	FULL	UNIT WEIGHT 0.59 LBS/FT	SHEET 1 OF 1

FIELD MEASUREMENT
3/32" = .09375"
ACCEPTABLE
* FIELD MEASUREMENT IS APPROXIMATE

Holes. In structures such as transmission towers and electrical substations, it is common practice to punch or drill bolt holes 1/16 in. larger than the nominal bolt diameter. In other classes of structures, where the small amount of slip that may occur with oversized holes is not permissible, bolt holes may be drilled or reamed to give a driving fit or a small clearance, such as 1/50 in. on the diameter. Allowable bearing stresses on bolts are determined in the same way as for rivets, except that the allowable bearing load on a bolt is based on the nominal diameter of the bolt rather than on the hole diameter, as in the case of rivets. Allowable shear and bearing stresses on bolts are independent of the hole clearance, as long as the hole diameter is not more than 1/16 in. greater than the nominal diameter of the bolt.

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5086-H111	5356	35	18	17	21	17
5086-H34	5356	35	19	19	21	17
5454-H111	5554	31	16	15	19	17
5454-H34	5554	31	16	16	19	17
5456-H111	5556	41	24	22	24	20
5456-H321	5556	42	26	24	25	20
6061-T6	5556	24	20	20	15	20
6061-T6	5356	24	20	20	15	17
6061-T6	4043	24	15[f]	15[f]	15	11.5[f]
6063-T5	4043	17	11	11	11	11.5
6063-T6						
6070-T6	5556	28	24	24	17	20

TS = tensile strength, ksi
TYS = tensile yield strength, ksi
CYS = compressive yield strength, ksi
SS = shear strength, ksi

[a] These are minimum expected strength values to be used as basis for design. Typical or average strength values are appreciably higher.

[b] Filler wires listed are commonly used. They do not necessarily represent recommended filler wires for all applications.

[c] These are ASME weld-qualification test-requirement values. The design strength is often considered as 90 percent of these values.

[d] Yield strength across a butt weld corresponds to 0.2 percent set on a 10-in. gage length.

[e] Applicable to throat area of fillet. For double fillet welds stressed in transverse shear, the strengths are somewhat higher than the values listed.

[f] These values apply for thicknesses of 3/8 in. or more. For smaller thicknesses, the strengths listed for 5356 and 5556 filler wire may also be used for 4043 filler.

sacrifice in ductility of the weld. Another disadvantage is the warping which often accompanies the heat-treating procedure.

Factor of Safety. Although the tensile strengths across butt welds listed in Table 13 are equal to the ASME weld-qualification test values, they are not generally considered to have the same reliability as the minimum mechanical properties of the parent metal. Accordingly, it has been recommended[3] that a factor of 0.9 be applied to these minimum strength values before dividing by the factor of safety on ultimate strength to obtain allowable stresses. The factor of safety on yielding is applied to the 10-in. gage-length yield strength across butt welds, and the allowable stress is chosen as the lower of the two values obtained from the yield and ultimate strengths. This procedure was followed in establishing the allowable stresses across butt welds in the

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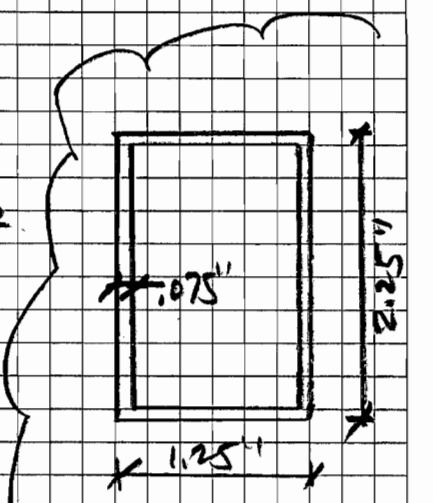
HORIZONTAL RUNG MEMBER CAPACITY

$F_u = 38$ ksi ;

$F_y = 35$ ksi ; $F_v = 24$ ksi

$A_w \cong 2.25'' \times 2 \times .075'' = .3375 \text{ in}^2$

$S_x \cong .3 \text{ in}^3$



HORIZONTAL RUNG SECTION

$M_{ultimate} = 35 \times .3 = 10.5$ k-in

$M_{allowable} = \frac{10.5}{1.65} = 6.4$ k-in

$V_{ULTIMATE} = \frac{2}{3} A_w \times F_v$

$= \frac{2}{3} \times .3375 \times 24$

$= 5.4$ kips

$V_{allowable} = 5.4 / 1.65 = 3.3$ k

Table 3.3-1
MINIMUM MECHANICAL PROPERTIES FOR ALUMINUM ALLOYS

ALLOY AND TEMPER	PRODUCT	THICKNESS RANGE IN.	TENSION		COMPRESSION	SHEAR		COMPRESSIVE MODULUS OF ELASTICITY‡
			F_{tu}† ksi	F_{ty}† ksi	F_{cy} ksi	F_{su} ksi	F_{sy} ksi	E ksi
5052-O	Sheet & Plate	0.006-3.000	25	9.5	9.5	16	5.5	10,200
-H32	Sheet & Plate	All	31	23	21	19	13	10,200
-H34	Cold Fin. Rod & Bar Drawn Tube	All	34	26	24	20	15	10,200
-H36	Sheet	0.006-0.162	37	29	26	22	17	10,200
5083-O	Extrusions	up thru 5.000	39	16	16	24	9	10,400
-H111	Extrusions	up thru 0.500	40	24	21	24	14	10,400
-H111	Extrusions	0.501-5.000	40	24	21	23	14	10,400
-O	Sheet & Plate	0.051-1.500	40	18	18	25	10	10,400
-H116	Sheet & Plate	0.188-1.500	44	31	26	26	18	10,400
-H321	Sheet & Plate	0.188-1.500	44	31	26	26	18	10,400
-H116	Plate	1.501-3.000	41	29	24	24	17	10,400
-H321	Plate	1.501-3.000	41	29	24	24	17	10,400
5086-O	Extrusions	up thru 5.000	35	14	14	21	8	10,400
-H111	Extrusions	up thru 0.500	36	21	18	21	12	10,400
-H111	Extrusions	0.501-5.000	36	21	18	21	12	10,400
-O	Sheet & Plate	0.020-2.000	35	14	14	21	8	10,400
-H112	Plate	0.250-0.499	36	18	17	22	10	10,400
-H112	Plate	0.500-1.000	35	16	16	21	9	10,400
-H112	Plate	1.001-2.000	35	14	15	21	8	10,400
-H112	Plate	2.001-3.000	34	14	15	21	8	10,400
-H116	Sheet & Plate	All	40	28	26	24	16	10,400
-H32	Sheet & Plate Drawn Tube	All	40	28	26	24	16	10,400
-H34	Sheet & Plate Drawn Tube	All	44	34	32	26	20	10,400
5154-H38	Sheet	0.006-0.128	45	35	33	24	20	10,300
5454-O	Extrusions	up thru 5.000	31	12	12	19	7	10,400
-H111	Extrusions	up thru 0.500	33	19	16	20	11	10,400
-H111	Extrusions	0.501-5.000	33	19	16	19	11	10,400
-H112	Extrusions	up thru 5.000	31	12	13	19	7	10,400
-O	Sheet & Plate	0.020-3.000	31	12	12	19	7	10,400
-H32	Sheet & Plate	0.020-2.000	36	26	24	21	15	10,400
-H34	Sheet & Plate	0.020-1.000	39	29	27	23	17	10,400
5456-O	Sheet & Plate	0.051-1.500	42	19	19	26	11	10,400
-H116	Sheet & Plate	0.188-1.250	46	33	27	27	19	10,400
-H321	Sheet & Plate	0.188-1.250	46	33	27	27	19	10,400
-H116	Plate	1.251-1.500	44	31	25	25	18	10,400
-H321	Plate	1.251-1.500	44	31	25	25	18	10,400
-H116	Plate	1.501-3.000	41	29	25	25	17	10,400
-H321	Plate	1.501-3.000	41	29	25	25	17	10,400
6005-T5	Extrusions	up thru 1.000	38	35	35	24	20	10,100
6061-T6, T651	Sheet & Plate	0.010-4.000	42	35	35	27	20	10,100
-T6, T6510, T6511	Extrusions	All	38	35	35	24	20	10,100
-T6, T651	Cold Fin. Rod & Bar	up thru 8.000	42	35	35	25	20	10,100
-T6	Drawn Tube	0.025-0.500	42	35	35	27	20	10,100
-T6	Pipe	All	38	35	35	24	20	10,100
6063-T5	Extrusions	up thru 0.500	22	16	16	13	9	10,100
-T5	Extrusions	0.500-1.000	21	15	15	12	8.5	10,100
-T6	Extrusions & Pipe	All	30	25	25	19	14	10,100
6066-T6, T6510, T6511	Extrusions	All	50	45	45	27	26	10,100
6070-T6, T62	Extrusions	up thru 2.999	48	45	45	29	26	10,100
6105-T5	Extrusions	up thru 0.500	38	35	35	24	20	10,100
6351-T5	Extrusions	up thru 1.000	38	35	35	24	20	10,100
6463-T6	Extrusions	up thru 0.500	30	25	25	19	14	10,100

† F_{tu} and F_{ty} are minimum specified values (except F_{ty} for 1100-H12, -H14 Cold Finished Rod and Bar and Drawn Tube, Alclad 3003-H18 Sheet and 5050-H32, -H34 Cold Finished Rod and Bar which are minimum expected values); other strength properties are corresponding minimum expected values.

‡Typical values. For deflection calculations an average modulus of elasticity is used; this is 100 ksi lower than values in this column.

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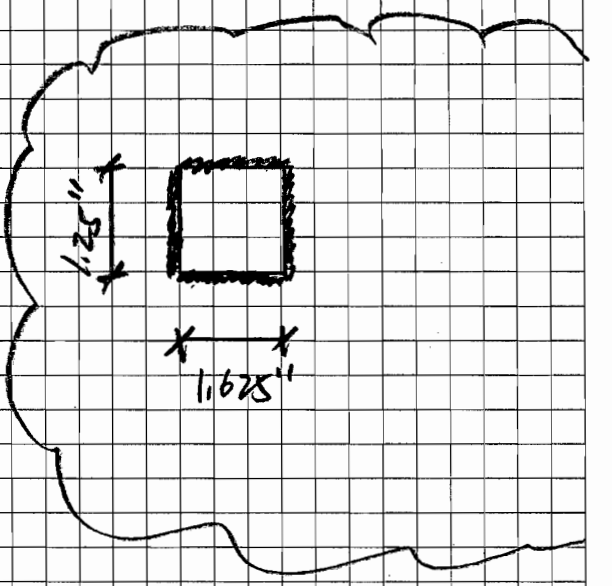
WELD CONNECTION FROM BRACE TO SHORING LEG.

$t_w = \frac{3}{16} = .188''$



TOTAL LENGTH OF WELDS

$= (2 \times 1.25'' + 1.625 \times 2 - 8 \times t_w)$

$= 4.25''$

$F_v = 11.5$ ksi

$V = (4.25'' \times .707 \times .188'') \times F_v$

$= 6.5$ kips

$V_{allowable} = \frac{6.5}{1.65} = 3.94$ k

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ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: D221  Station Loc: 51.810  Section ID: S-CBRACE
Element Type:  Moment Resisting Frame   Classification:  Non-Compact

L=103.619
A=0.234  i22=0.022  i33=0.022
s22=0.031  s33=0.031  r22=0.304  r33=0.304  alpha=45.000
E=29000.000  fy=50.000
RLLF=1.000

    Stress Check Message - l/r > 300

P-M33-M22 Demand/Capacity Ratio is   0.239 = 0.152 + 0.012 + 0.075

STRESS CHECK FORCES & MOMENTS
                          P)          M33          M22           V2           V3
    Combo  DSTLS2      1.069       -0.053        0.017       -0.002    9.636E-04

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                          fa           Fa           Ft
                      Stress    Allowable    Allowable
    Axial              4.563        0.532       30.000

                          fb           Fb           Fe       Cm        K        L       Cb
                      Stress    Allowable    Allowable   Factor   Factor   Factor   Factor
    Major Bending      0.358       16.421        5.157    1.000    1.000    0.500    1.000
    Minor Bending      2.481       33.000        0.532    1.000    1.000    1.000

SHEAR DESIGN
                          fv           FV       Stress
                      Stress    Allowable        Ratio
    Major Shear        0.017       20.000        0.001
    Minor Shear        0.008       20.000        0.000
```

$P_{max} = 1.069k < V = 3.94k$

∴ WELD CAPACITY @ DIAGONAL IS ADEQUATE.

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: D221  Station Loc: 51.810  Section ID: S-CBRACE
Element Type:  Moment Resisting Frame   Classification:  Non-Compact

L=103.619
A=0.234  i22=0.022  i33=0.022
s22=0.031  s33=0.031  r22=0.304  r33=0.304  alpha=45.000
E=29000.000  fy=50.000
RLLF=1.000

    Stress Check Message - l/r > 300

P-M33-M22 Demand/Capacity Ratio is   0.258 = 0.165 + 0.014 + 0.079

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      1.158       -0.056       0.021      -0.002       0.001

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             4.943       0.532      30.000

                         fb          Fb          Fe       Cm        K        L       Cb
                     Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending     0.422      16.421       5.157    1.000    1.000    0.500    1.000
    Minor Bending     2.615      33.000       0.532    1.000    1.000    1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.017      20.000       0.001
    Minor Shear       0.008      20.000       0.000
```

$P_{max} = 1.158\ k < V = 3.94\ k$

∴ WELD CAPACITY @ DIAGONAL IS ADEQUATE.