NYC Buildings

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 40^{th} 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.

TABLE OF FIGURES AND PHOTOS

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

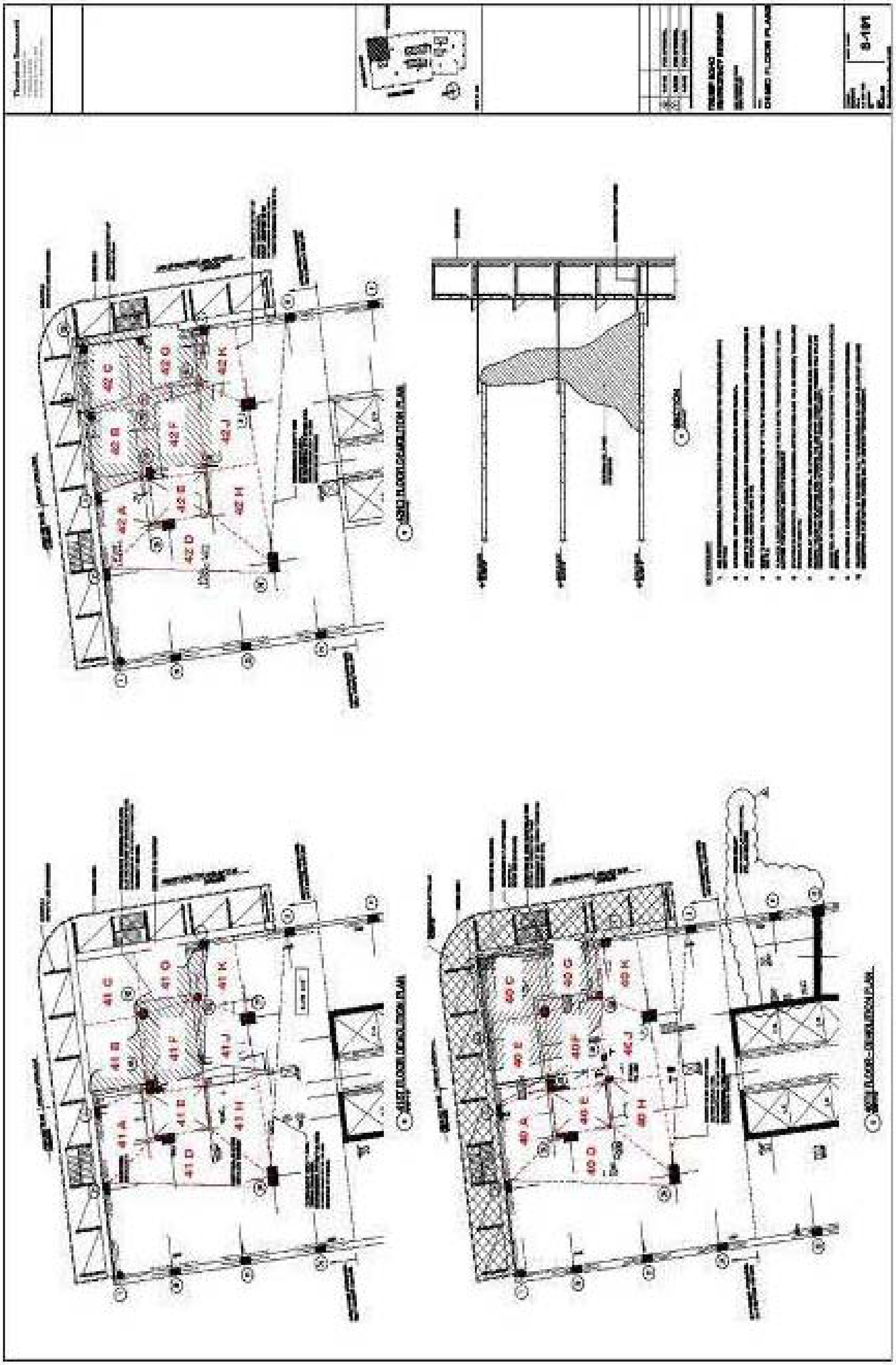

Figure 1 Evidence Collection Zones

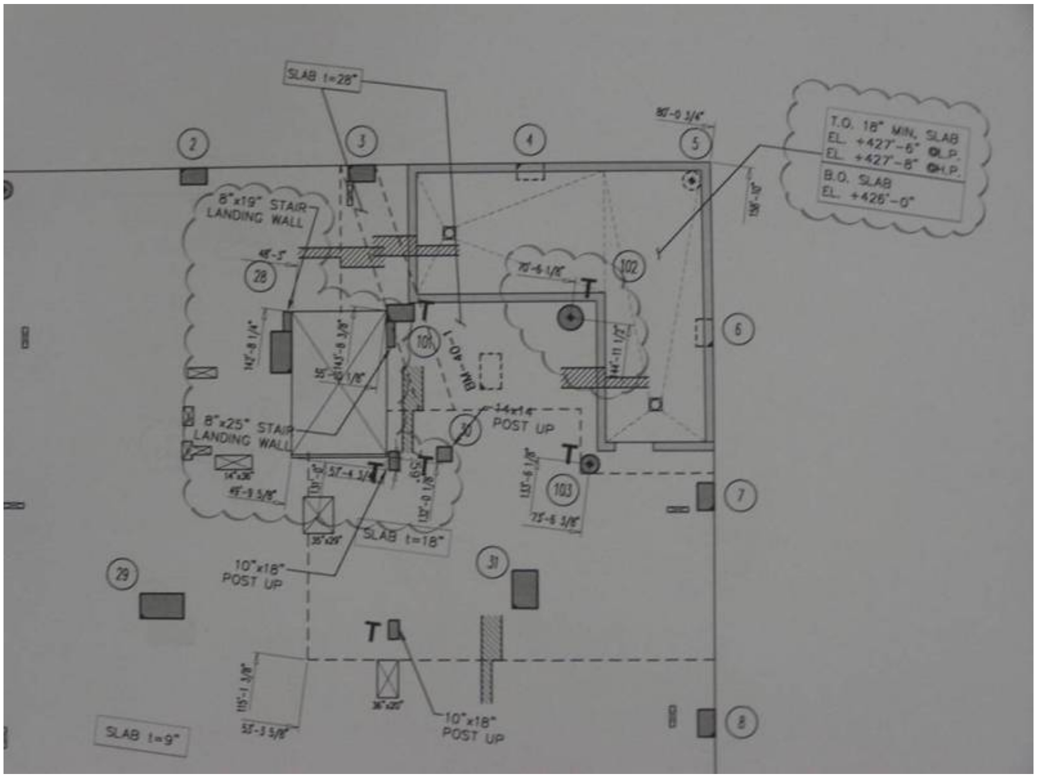

Photo 5 Partial Plan Concrete at 40th Floor by DeSimone Consulting Engineers

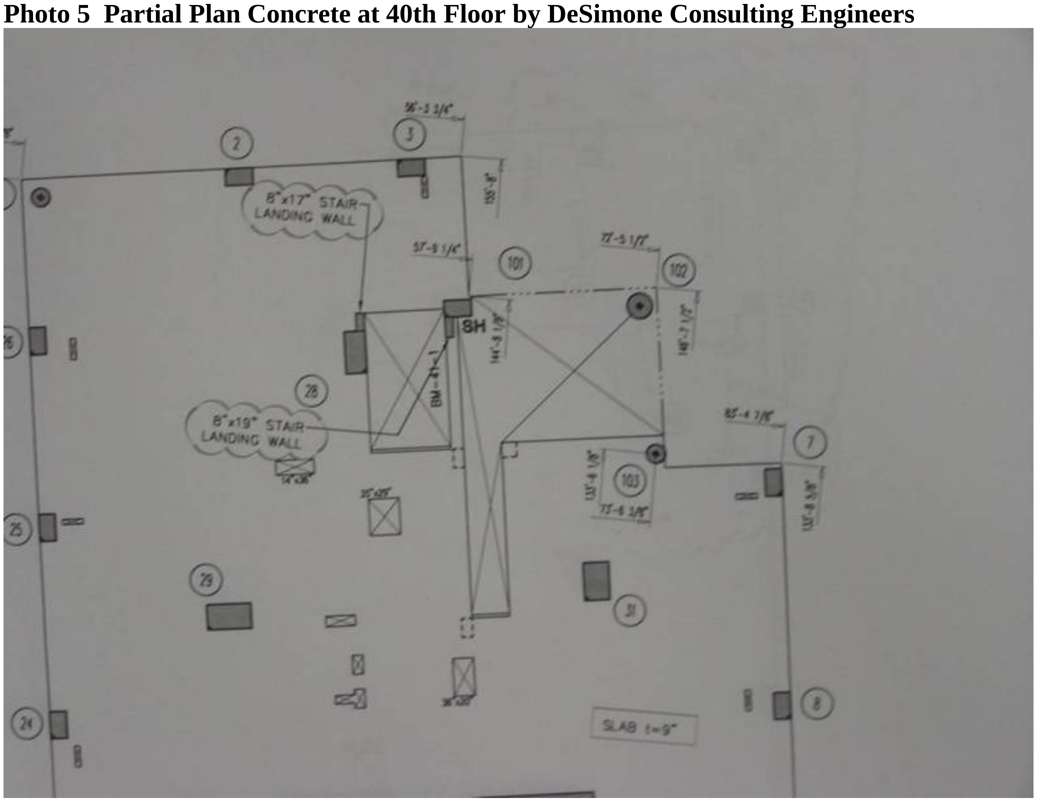

Photo 6 Partial Plan Concrete Floor at 41st Floor by DeSimone Consulting Engineers -

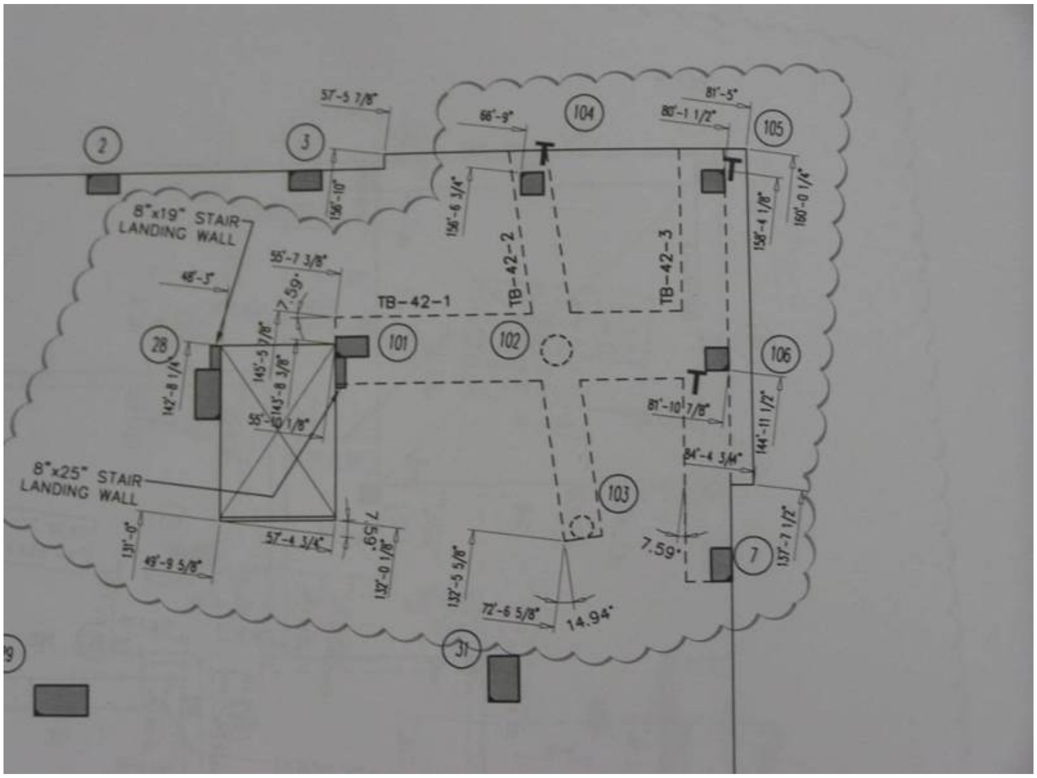

Photo 7 Partial Plan Concrete at 42nd Floor by DeSimone Consulting Engineers.

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.

<u>Braces</u>

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

<u>Adjustment Screw Extension Legs</u>

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).

<u>Top Plates</u>

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).

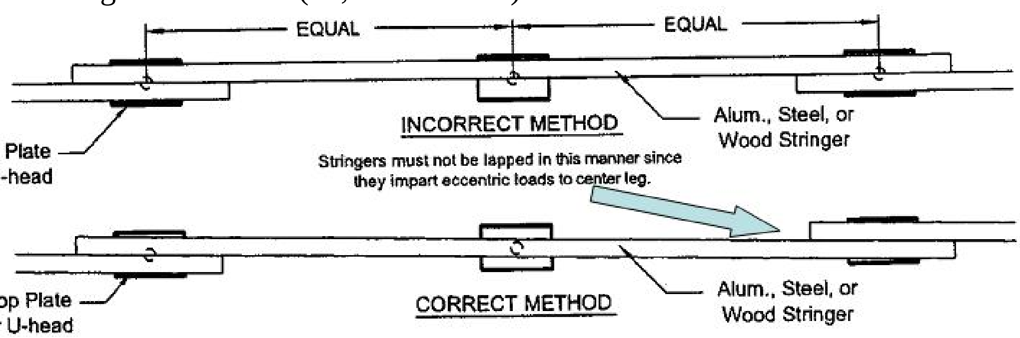

Figure 2 Stringer Setting - Patent Instructions

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).


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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).

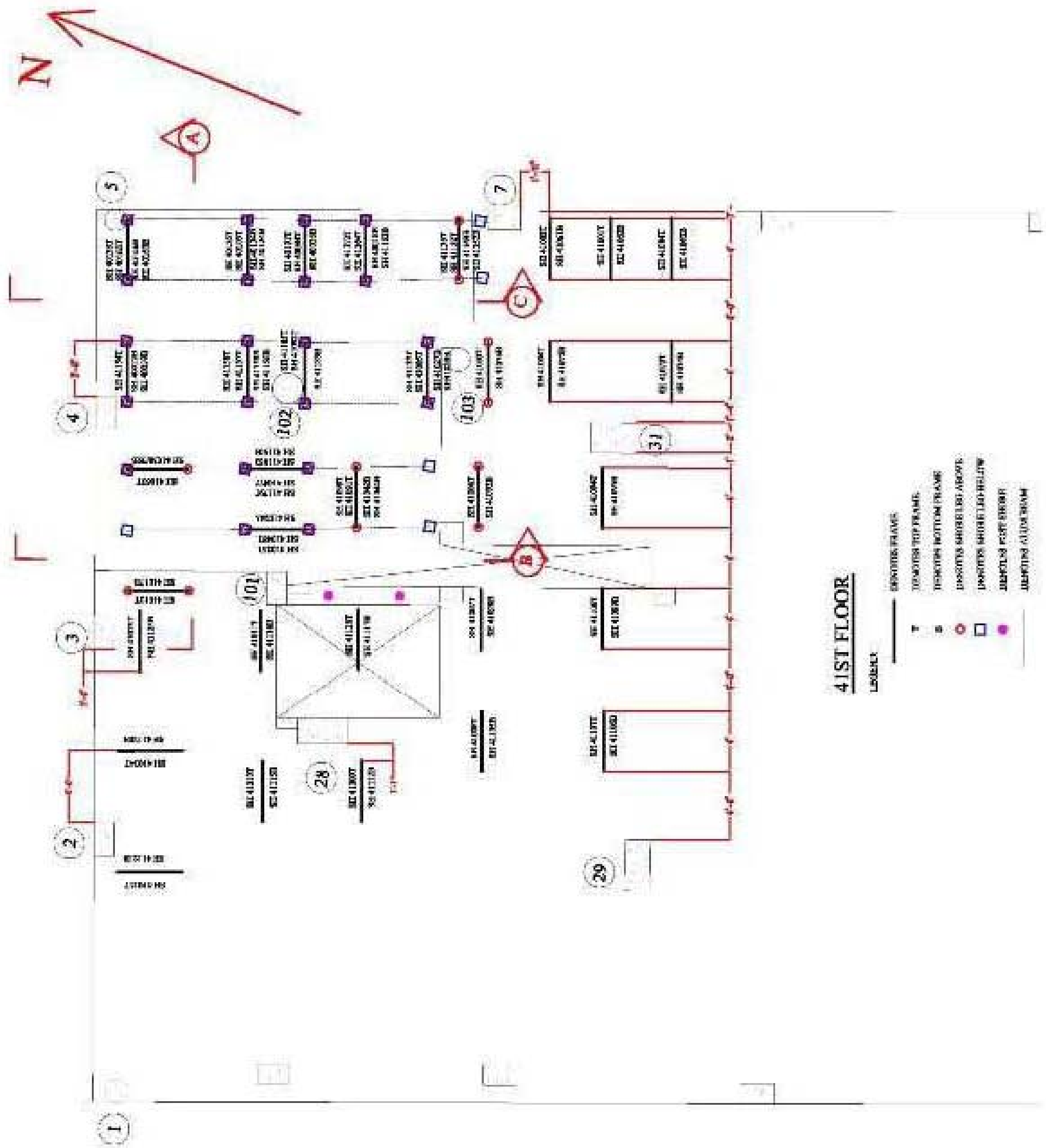


Figure 3 Plan Shoring at 41st Fl. Reconstruction

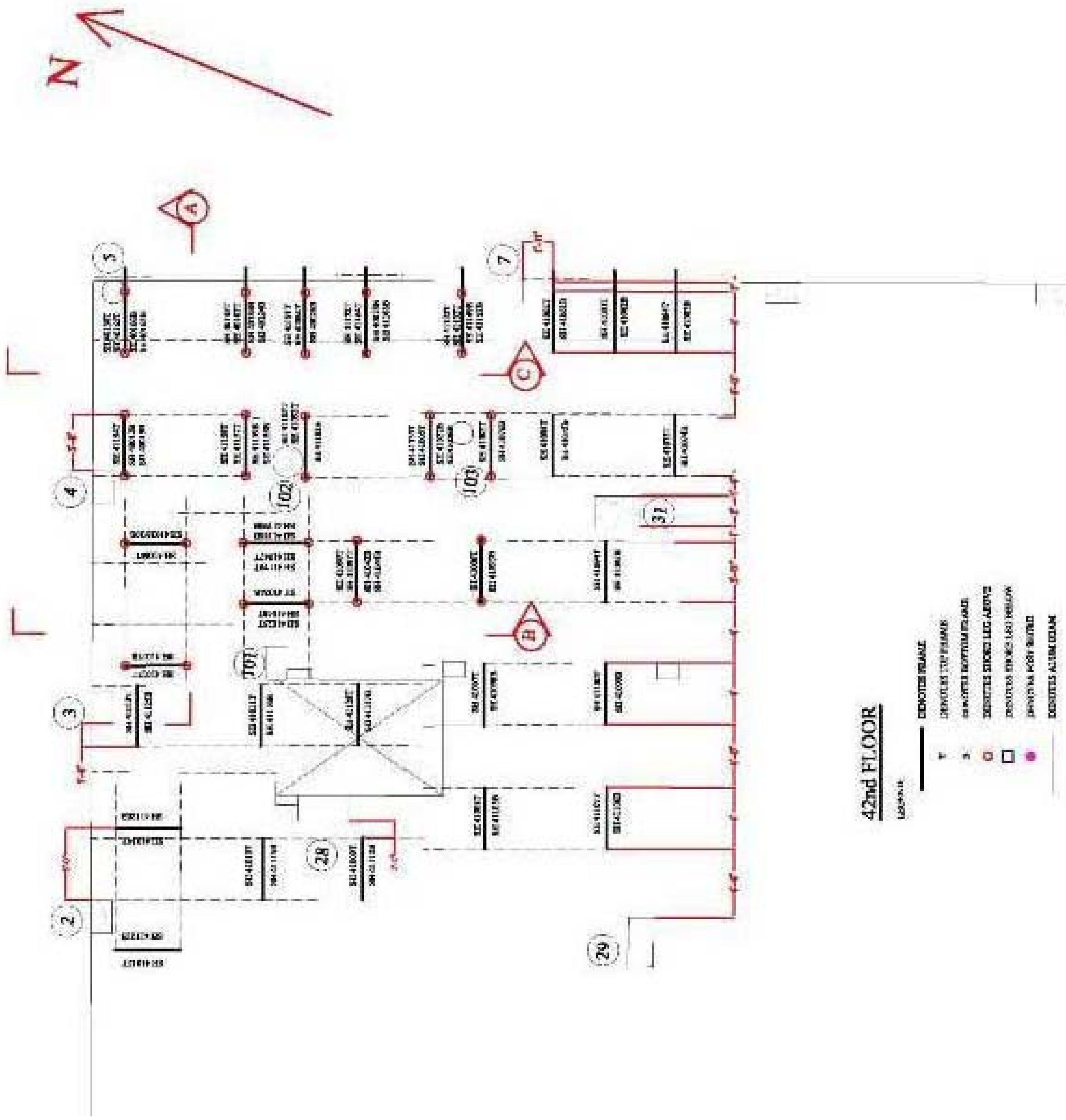


Figure 4 Plan Shoring at 42nd Fl. Reconstruction

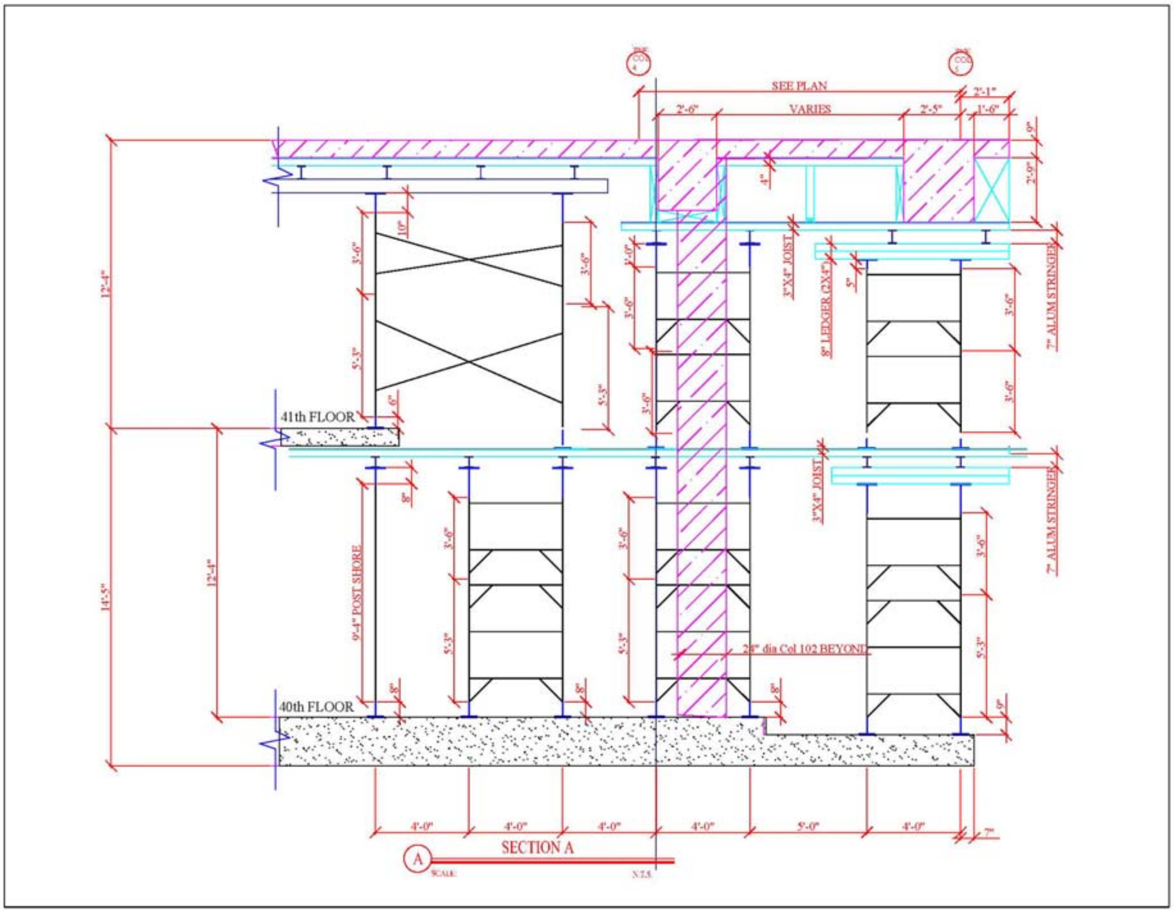


Figure 5 Section A Reconstruction Shoring (Looking North)

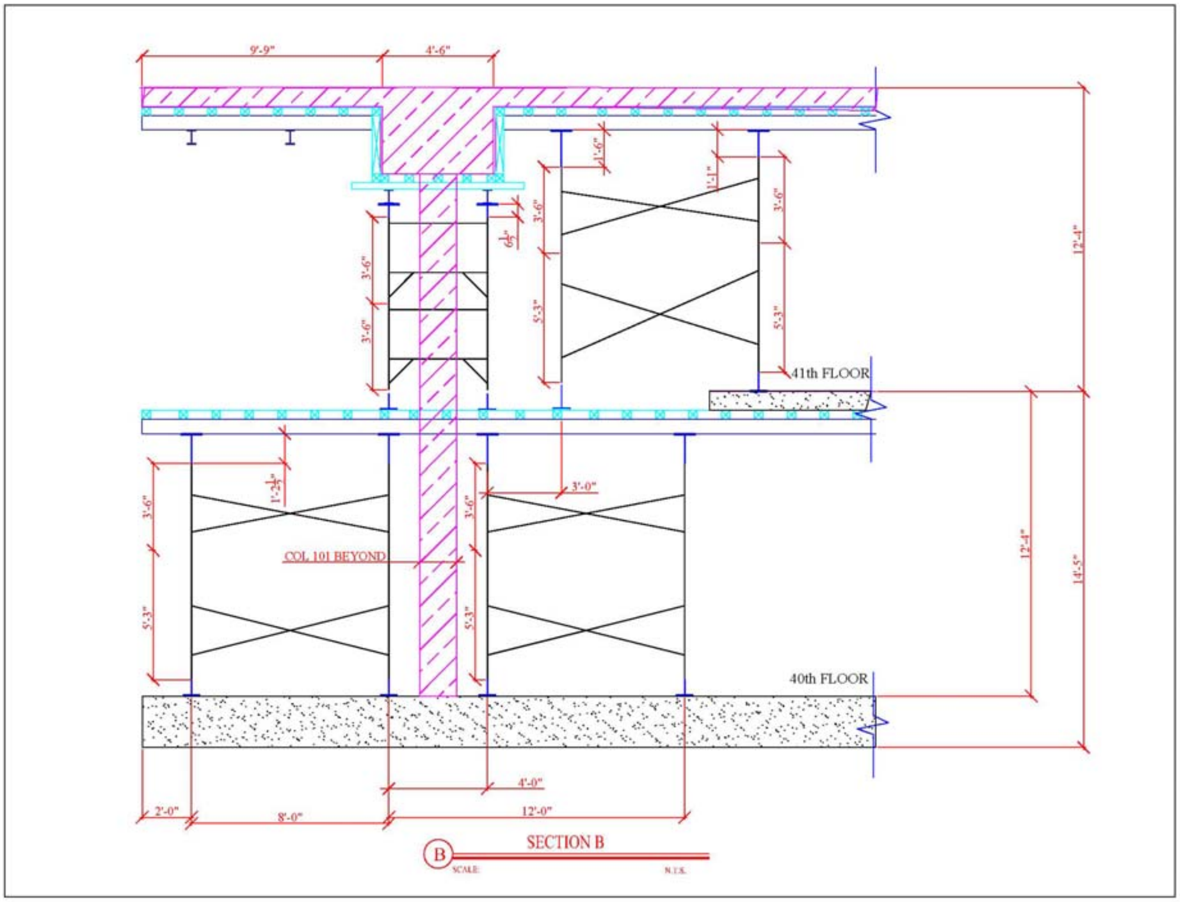


Figure 6 Section B. Reconstruction Shoring (Looking East)

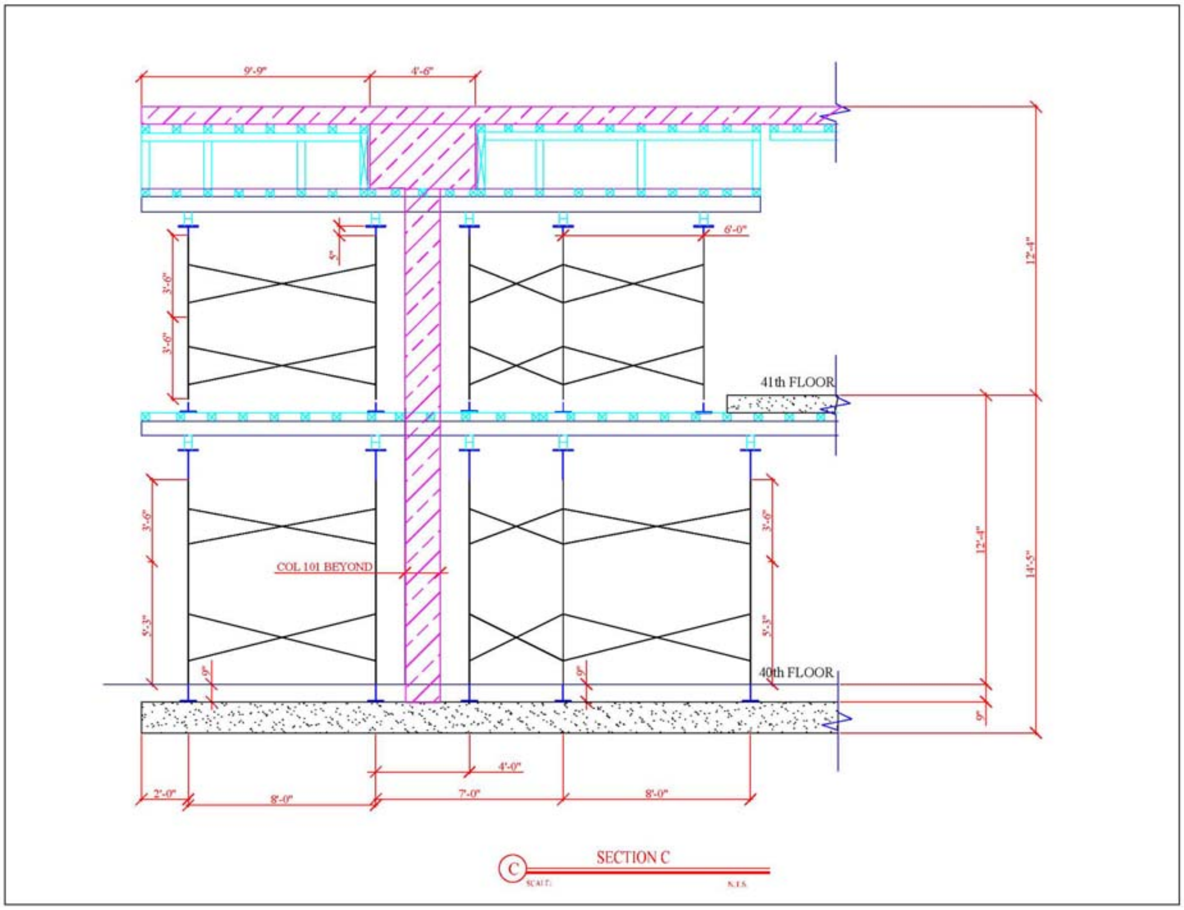


Figure 7 Section C. Reconstruction Shoring (Looking East)

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.

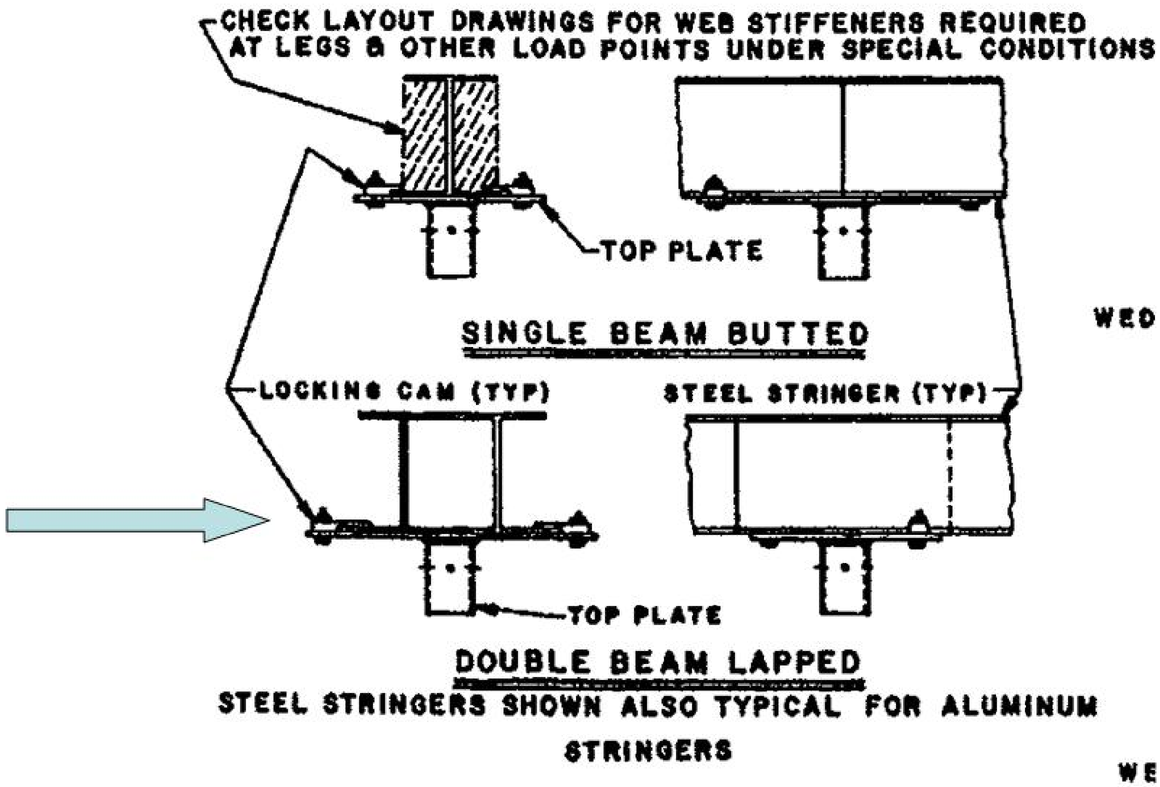


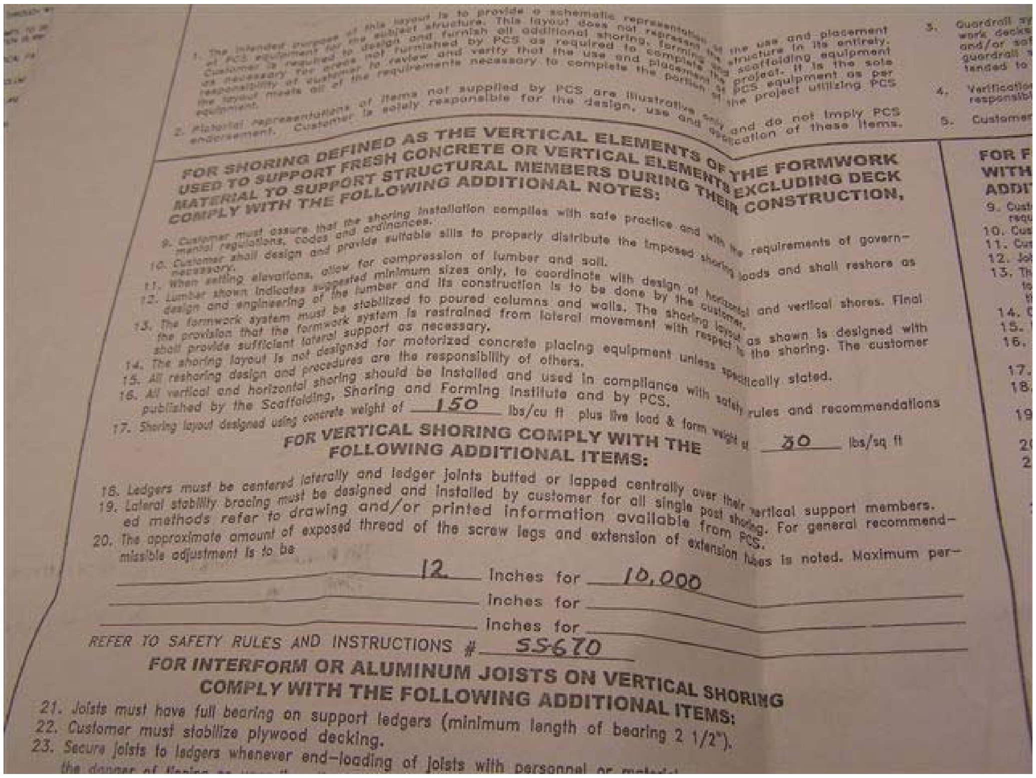
Figure 8 Lapping of Stringers - Patent Instructions

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).



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 50 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 material ...

Photo 19 Exposed Thread of Adjustment Screw Leg Instruction by Patent

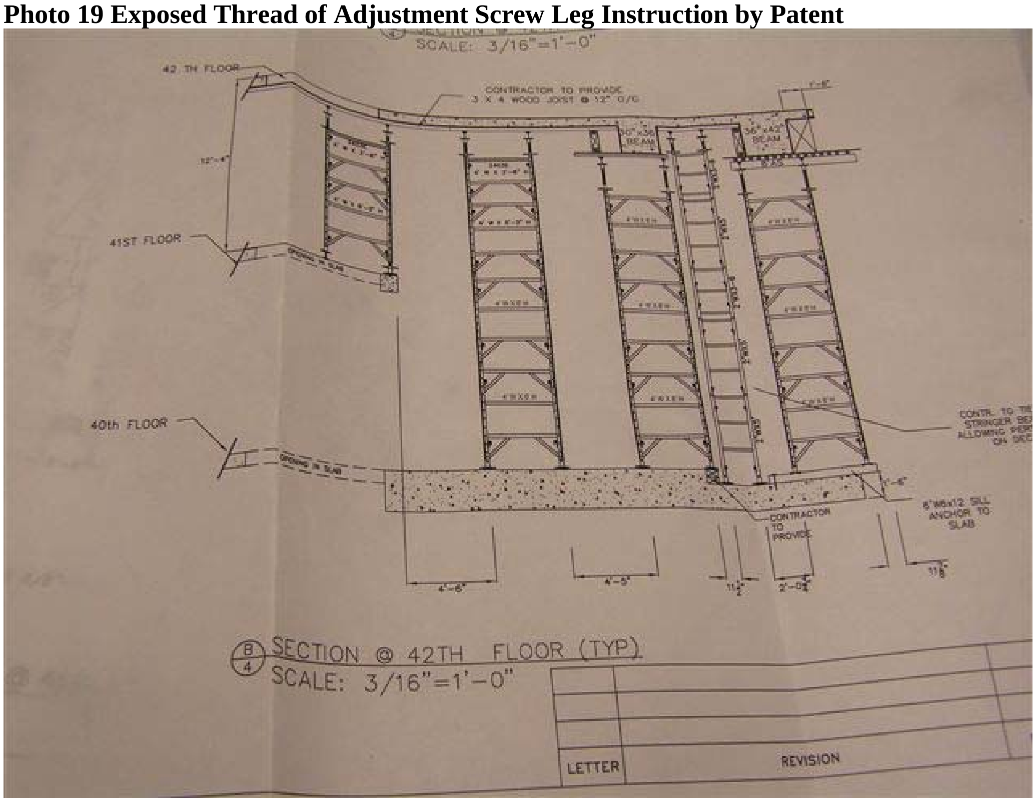


Figure 9 Plan of Shoring by Patent

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) <u>Substandard wood for formwork</u>
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) <u>Lack of design for wind and for horizontal loads</u>
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) <u>Lack of standard practice measures such as lateral bracing, including lack of attachment of the system to the 41st floor already-poured concrete</u>
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) <u>Lack of sufficient nailing</u>
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) <u>Eccentric application of loads</u>
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.

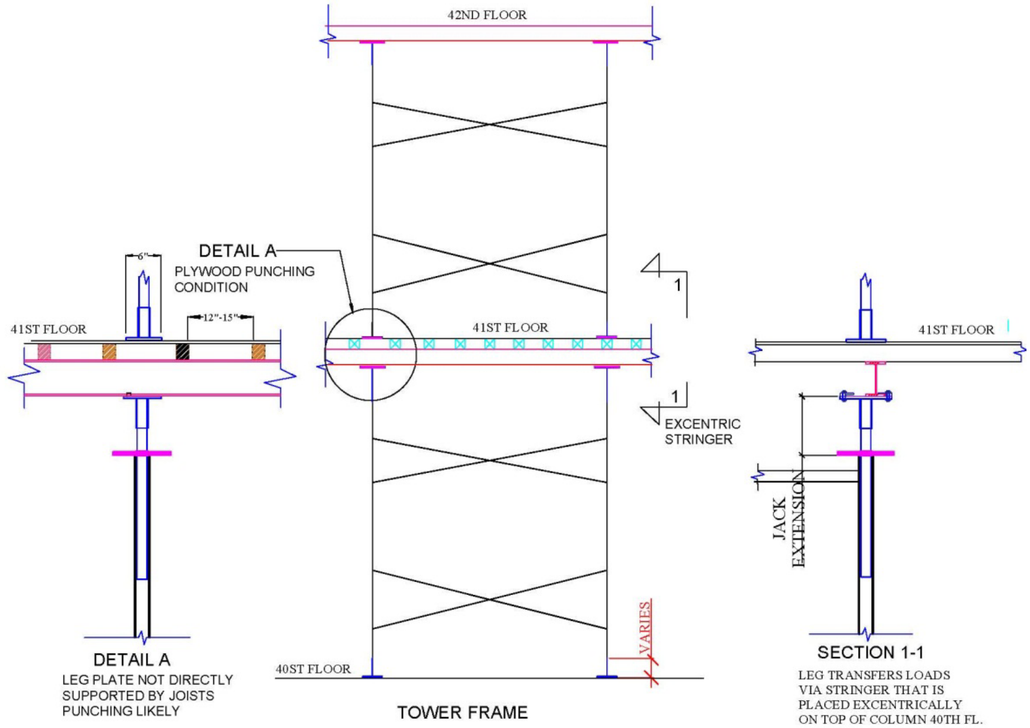


Figure 10 Transfer of Loads at "Mud Floor" – Before Loading and Collapse

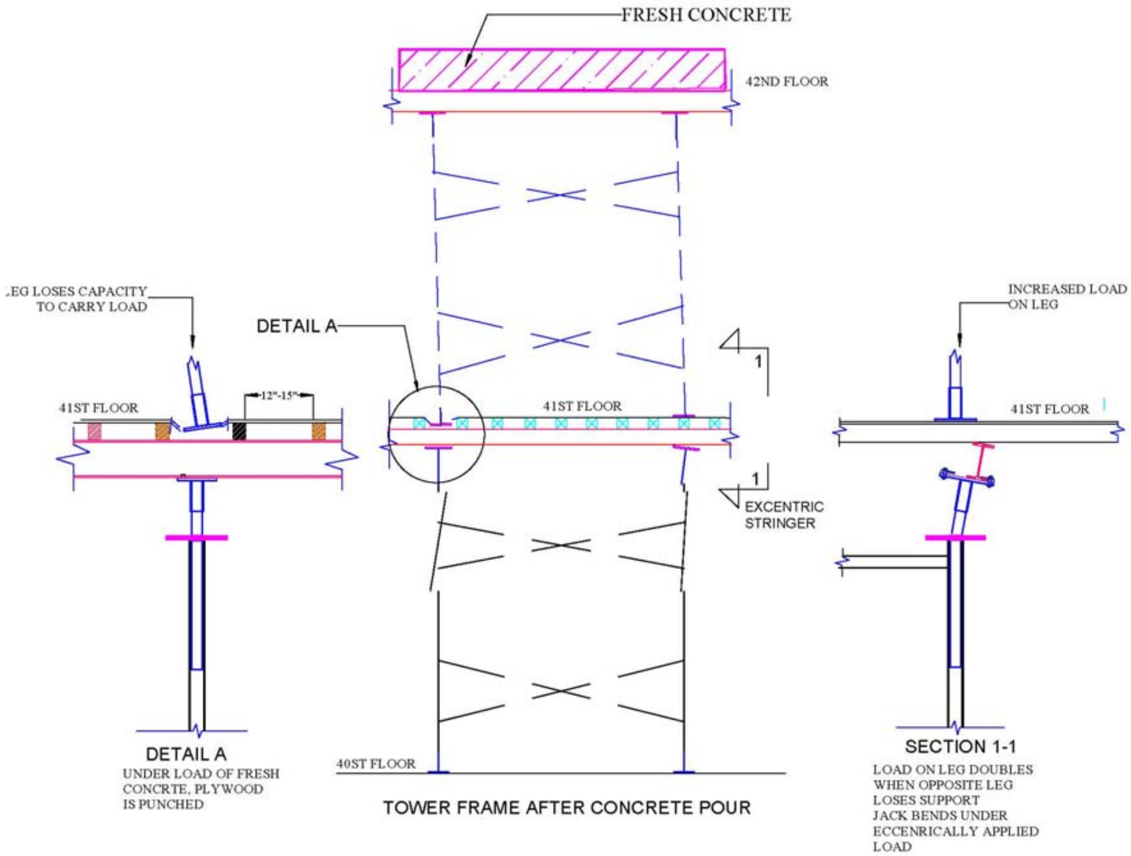


Figure 11 Scenario of Failure Following Concrete Pour

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

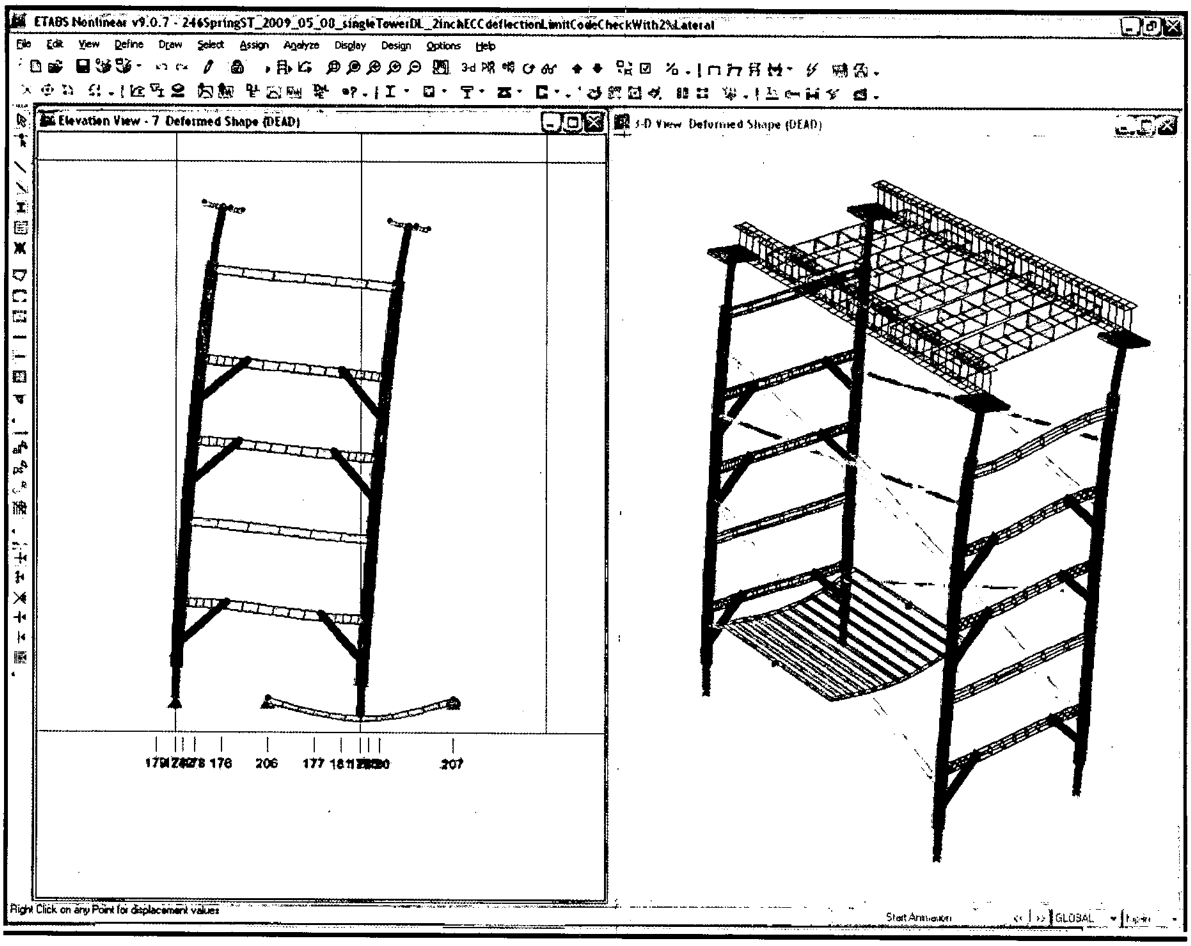
APPENDIX “A”
Structural Calculations

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

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:

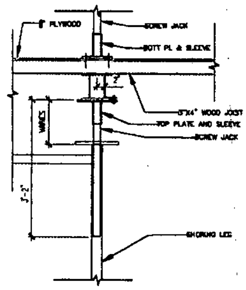
The Structural calculations and analysis were performed to verify the code compliance and possible modes of collapse of the formwork and supporting system. Hand and computer calculations were used to verify code adequacy of the drawings prepared by D'Alessio for Patent (*Patent Construction System Layout 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:

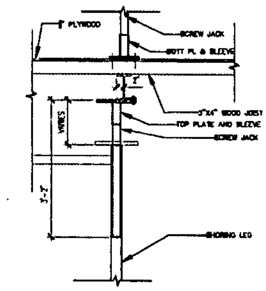
- Geometry: Screw jack top extension is 21 in, and bottom extension is 11 inch. Typical frame is 4 feet wide by 6 feet height in code check for Patent Construction System Layout Drawing. The typical frames for reconstruction shoring layout are consisted of 4 feet wide by 3'-6" height and 4 feet wide 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.



LOAD CASE B–2"ECC LOADED

SCALE: N.T.S.

2 Patent Construction System Layout Drawing-

2.1 Global Check-Dead Weight Of Concrete And Formwork.

In the north east corner of 42^{nd} 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.

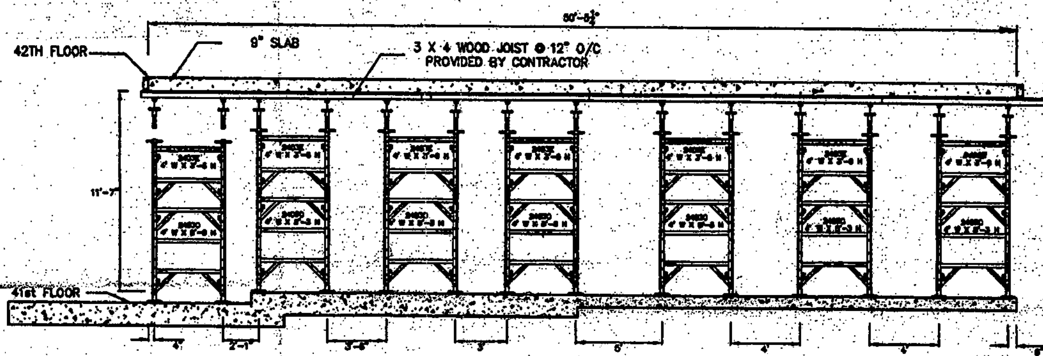
- 2'-6" x 8' TOWER

4' x 12' AREA = 48 SF

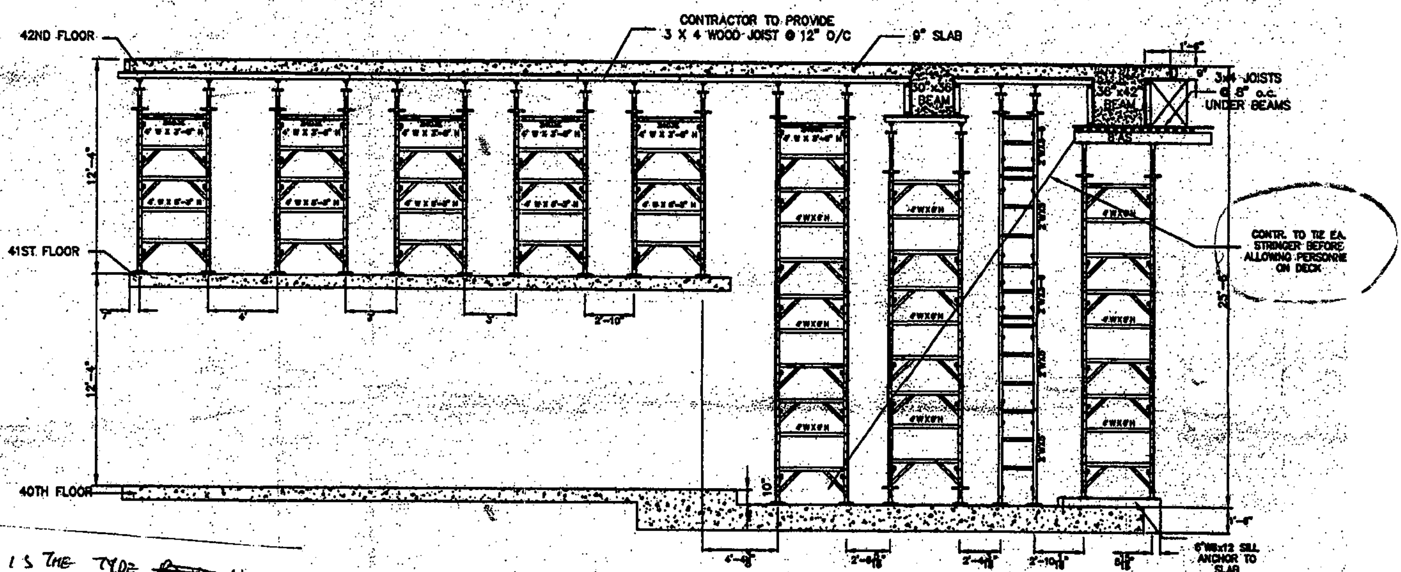
9" SLAB + 10 psf FORMWORK + 50 psf LL = 173 psf

WEIGHT = 173 x 48 = 8.3 KIPS < 40 K

∴ PATENT DRAWINGS PASS CODE CHECK.

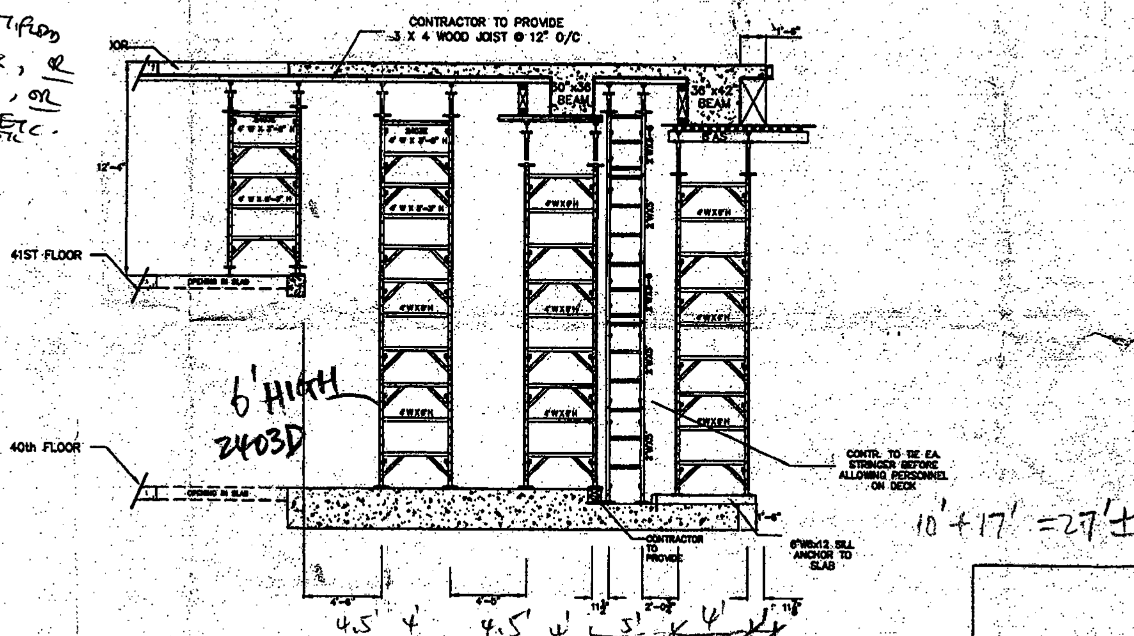


SECTION @ 42TH FLOOR (TYP)
SCALE: 3/16"=1'-0"



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

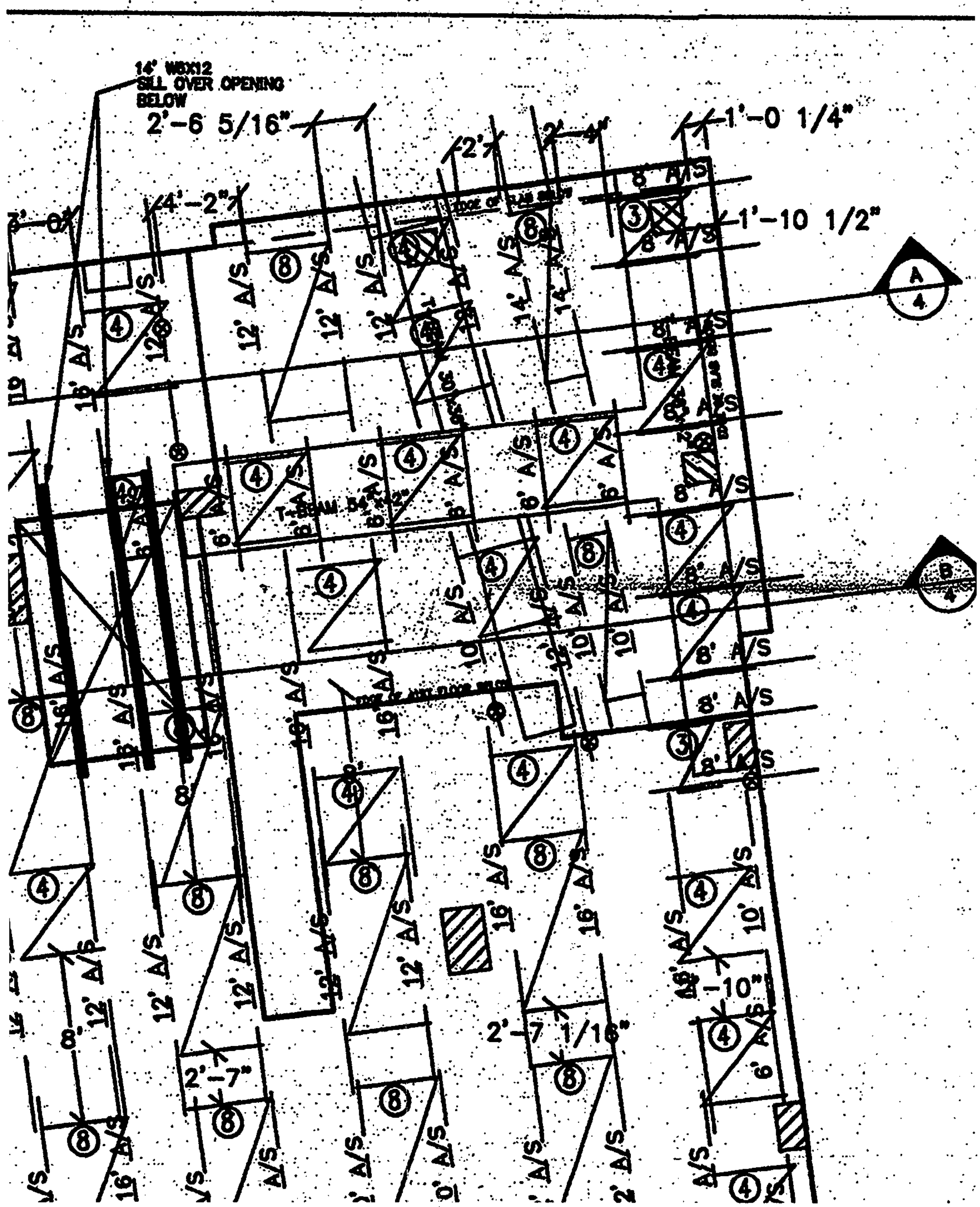
SECTION @ 42TH FLOOR (TYP)
SCALE: 3/16"=1'-0"



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

LETTER	REVISION	DATE	APP'D

FORMWORK SUPPORTING 42ND FLOOR



26'X28' COLLAPSE ZONE AREA

2.1

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

+4 kips

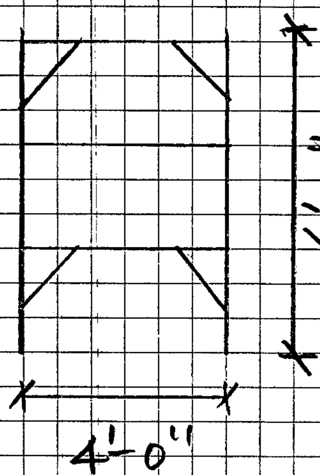
The typical shoring tower (2 frames) under beam-formwork is 4 feet by 4 ????feet. The tributary area is 64 SF, and total load on the shoring tower is 26kips (13kips per frame or 6.5kips per leg); this load is less than allowable load which is 20kips x 2 frames = 40 kips. Even with consideration of load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

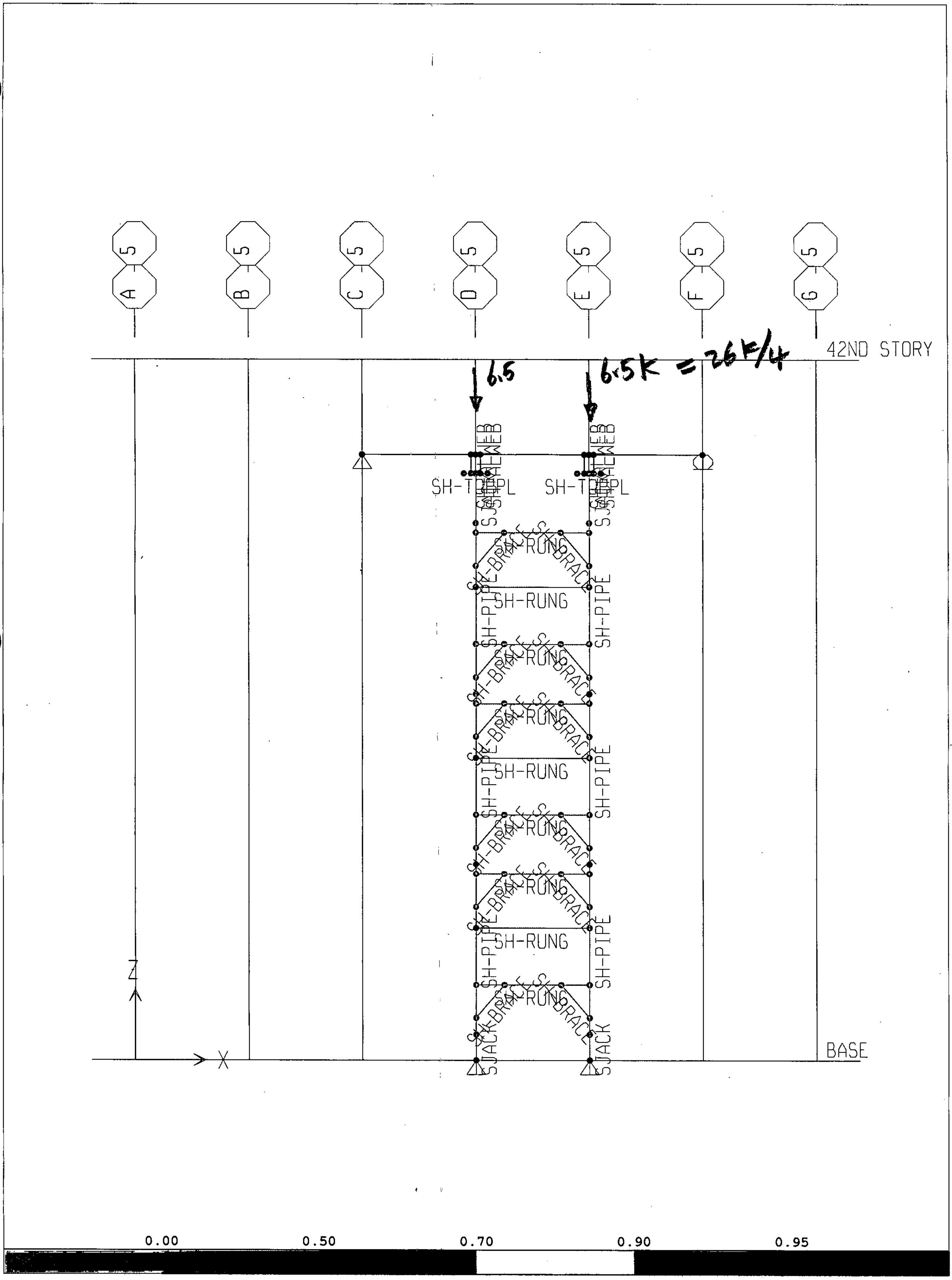
SUMMARY of RESULT - ETAB MODEL
"246SPRINGST_2DCODECHECK TYP PATENT SHORING"

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

∴ PATENT SHORING DWGS PASS CODE CHECK.

ETABS

City of New York



ETABS v9.0.7 - File: 246SpringST_2DCodeCheckTypPatentShoring - June 9,2009 15:24
Elevation View - 5 Steel Design Sections (AISC-ASD89) - Kip-in Units

3

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
```

4

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 ___

Subject: WOOD JOIST CHECK PER PATENT DWG 4607K070 Engr: GW Checked By: ___ Date: ___

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

CONSTRUCTION LIVE LOAD

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

CONC

$$= .383\ klf$$

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

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

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

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

$S_x = 5.1\ in^3$

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

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

$A_v = 8.75\ in^2$

customer, or, at the customer's option, be obtained through Patent Construction Systems or other suppliers. Patent Construction Systems will, at customer's request, consult on alternate means of access.

DURING USE OF EQUIPMENT ALWAYS FOLLOW SEPARATE SAFETY RULES & INSTRUCTIONS AS INDICATED IN EACH SPECIFIC SECTION.

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.

TITLE: 20KA SHORING LAYOUT **DATE:** 12/14/2007

PROJECT: SOHO HOTEL

LOCATION: 246 SPRING STREET NYC

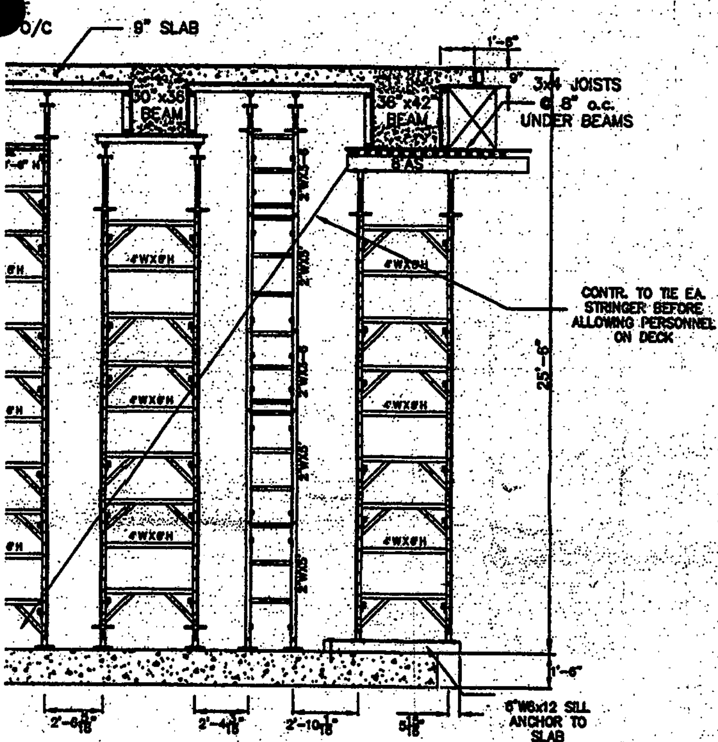
CUSTOMER: DI FAMA CONCRETE

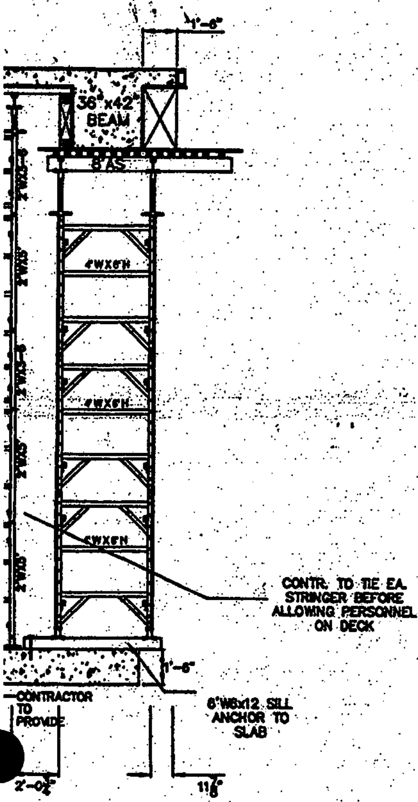
DRAWING NUMBER: 4607K070

SHEET NO. 1

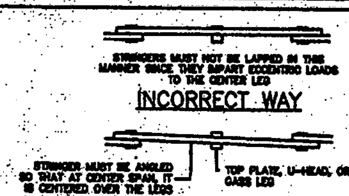
Patent Construction Systems
Harsco Corporation
PATENT CONSTRUCTION SYSTEMS
1800 Lower Road
Linden, New Jersey 07036
(732) 396-1289

3





Stringer Lapping Detail



LEGEND

- #A/S - DENOTES ALUMINUM STRINGER LENGTH
- #J - DENOTES J400 (JOIST) LENGTH
- #S - DENOTES STEEL (W6X12) STRINGER LENGTH
- W/S - DENOTES WOOD STRINGER (by Contractor)
- ⊗ - DENOTES PCS STEEL POLE SHORE

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

RECEIVED JAN 16 2008 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT

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

DRAWN: WME	CHECKED:	APPROVED:	DATE: 9/25/07	SCALE: 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:

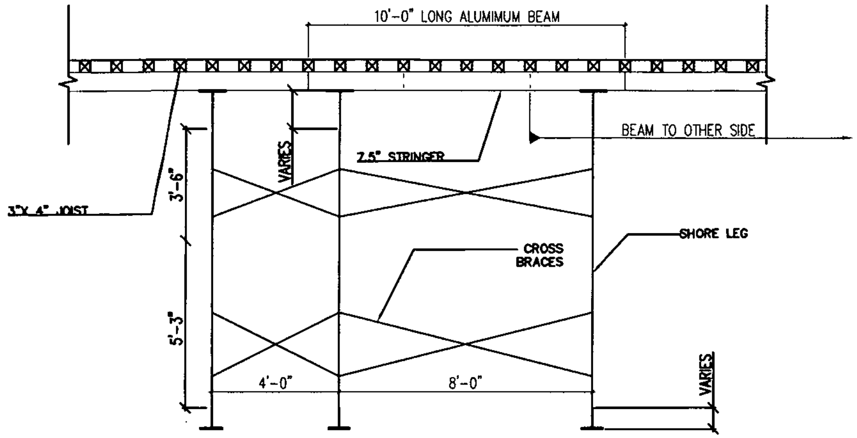
Load Take Down

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**

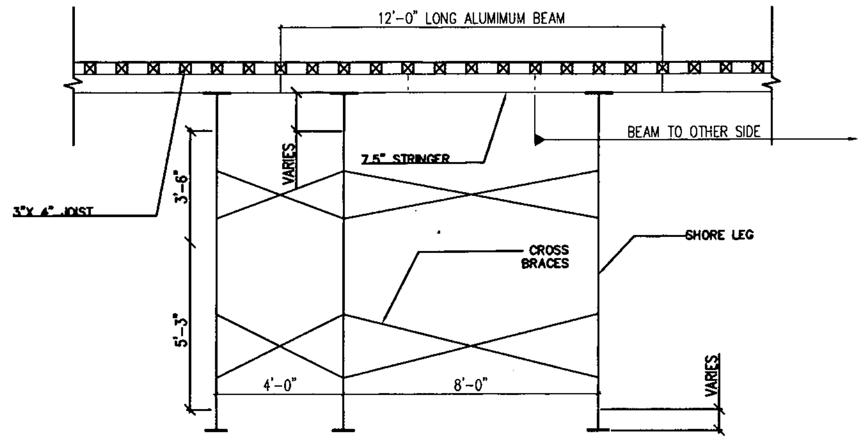
Typical 4' x 4' shoring tower (4 legs)

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**



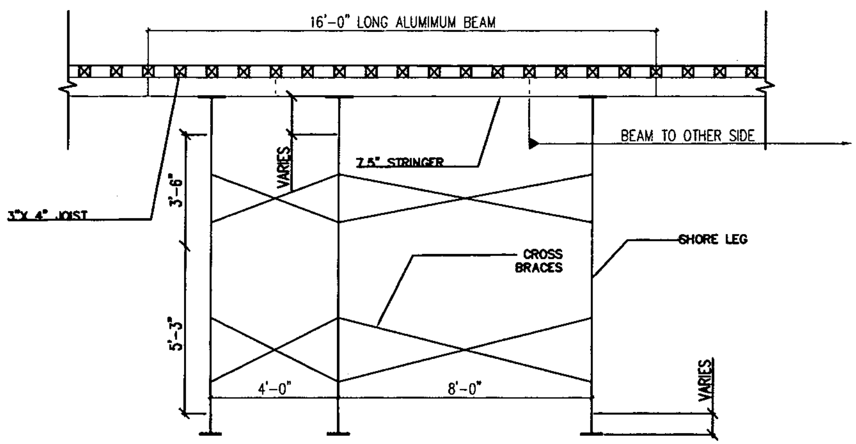
LOAD OVER 10'-0" LONG BEAM- SIMPLE SUPPORTED

SCALE: N.T.S.



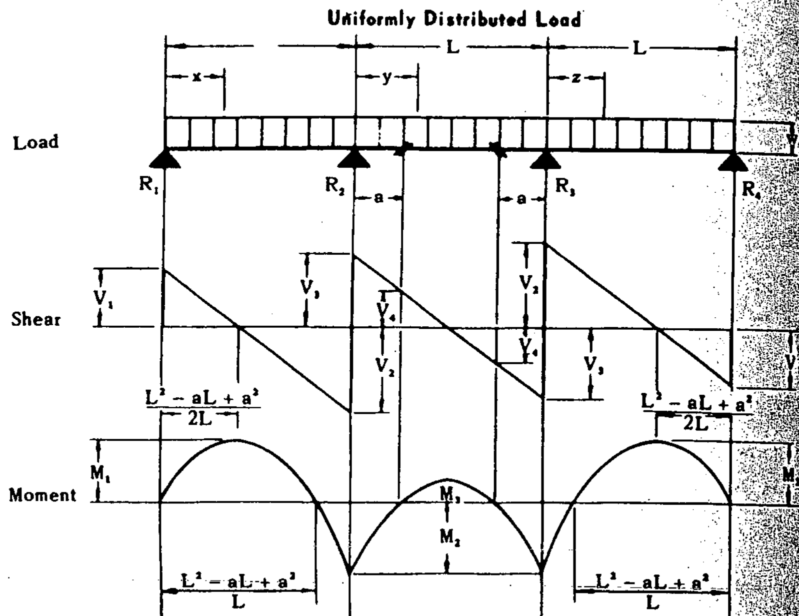
LOAD OVER 12'-0" LONG BEAM-SIMPLE SUPPORTED W/2' CANT.

SCALE: N.T.S.



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

SCALE: N.T.S.



Moment		0.086	−0.086	0.039	−0.086	0.086	wL^2
Shear	0.414		0.50 −0.586	0.280	−0.280	0.586 −0.50	−0.414	wL
Reaction	0.414		1.086		1.086		0.414	wL

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

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

General formulas are:

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

$$R_2 = R_3 = \frac{w}{2L}(2L^2 + aL - a^2) \qquad M_2 = -\frac{w}{2}(aL - a^2)$$

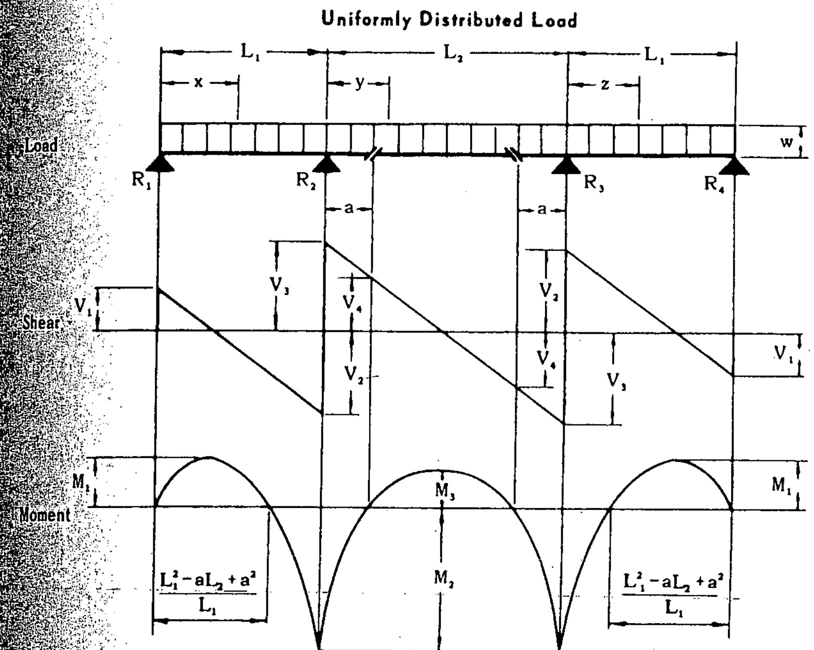
$$V_1 = \pm\frac{w}{2L}(L^2 - aL + a^2) \qquad M_3 = \frac{w}{8}(L - 2a)^2$$

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

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

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

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



Moment	0.086	−0.086		−0.086	0.086	wL_1^2
Shear	0.414 wL_1	+0.500 wL_2 −0.586 wL_1		+0.586 wL_1 −0.500 wL_2		
Reaction	0.414	1.086		1.086	0.414	wL_1

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:

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

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

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

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

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

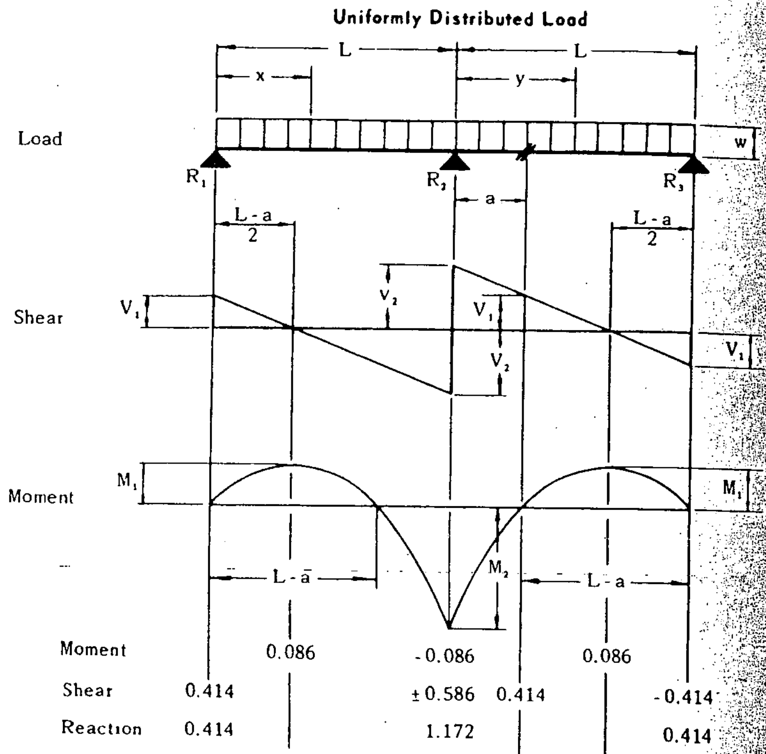


FIGURE 7.23. Cantilevered beam diagram. Two equal spans.

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

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

General formulas are:

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

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

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

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

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

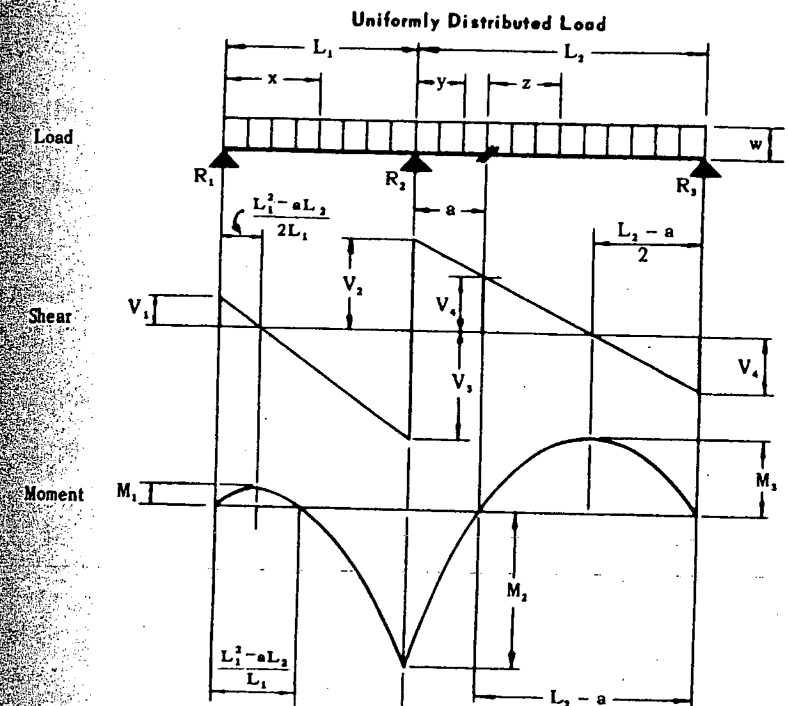


FIGURE 7.24. Cantilevered beam diagram: Two unequal spans.

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:

$$[illegible] = \frac{w}{2L_1}(L_1^2 - aL_2) \qquad V_3 = -\frac{w}{2L_1}(L_1^2 - aL_2) \qquad M_2 = -\frac{wL_2 a}{2}$$

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

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

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

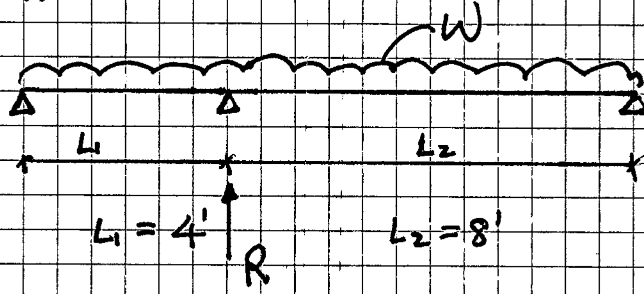
$$[illegible] = \frac{w}{2}(L_2 + a) \qquad V_z = \frac{w}{2}(L_2 - a) - wz \qquad M_z = \frac{w}{2}(L_2 - a - z)$$

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

Project: ______ Date: ______ Page: ____ of ____

Subject: ______ Engr: ______ Checked By: ______ Date: ______

12' LONG BEAM:



2 SPAN CONTINUOUS SUPPORT

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

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

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

$$= 6.516\, W$$

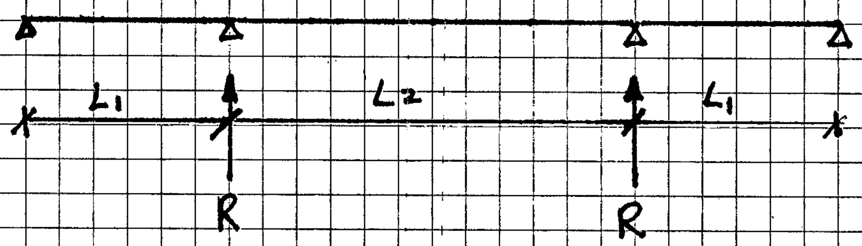
$$R_{\text{SIMPLE SUPPORTED}} = W(L_1 + L_2)/2 = 6W$$

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

Project: ______________________ Date: __________ Page: ____ of ____

Subject: ______________________ Engr: __________ Checked By: _______ Date: _________

16' LONG BEAM



3 SPAN CONTINUOOS SUPPORT

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

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

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

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

$$= 6.34\,w$$

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

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

7

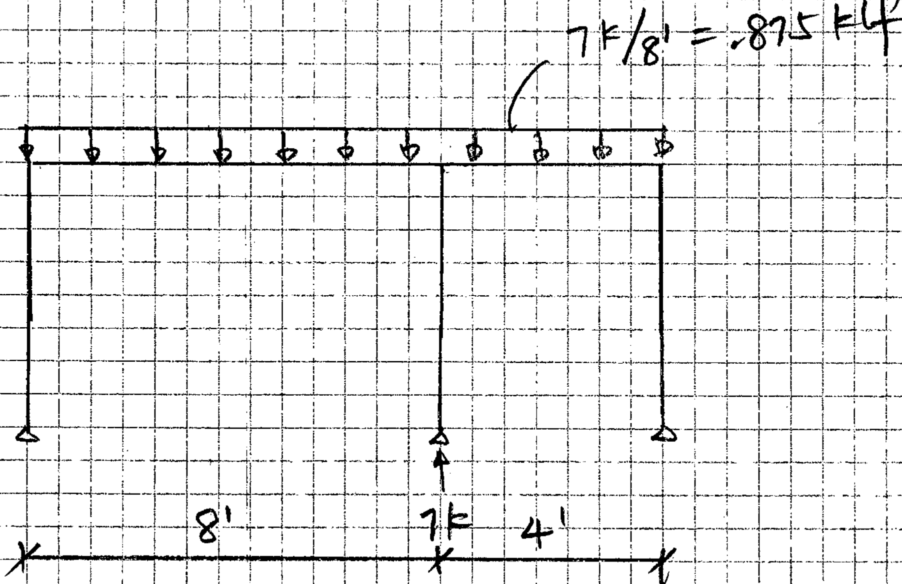
• REACTION DISTRIBUTION DUE TO LOOSE SUPPORT

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

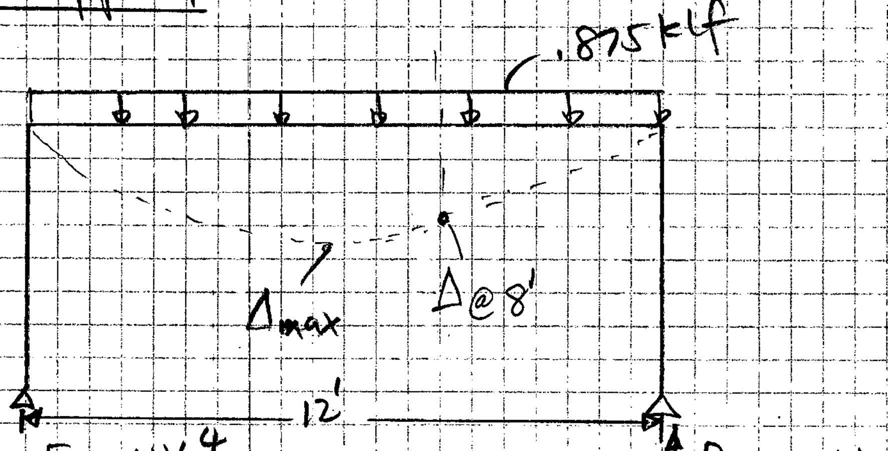
Subject: DEFLECTION @ ALUMINUM STRINGERS Engr: ____ Checked By: ____ Date: ____

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

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

• R_B TOTAL $= \frac{7^k}{2} + 7^k = 10.5$ K

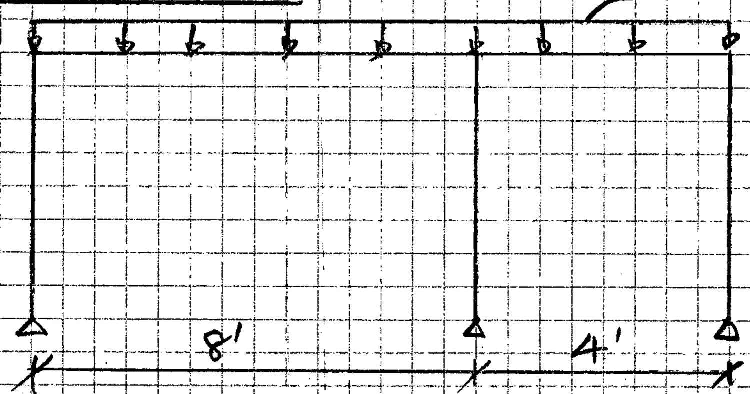
Project: 246 Spring Street Date: ______ Page: ___ of ___

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

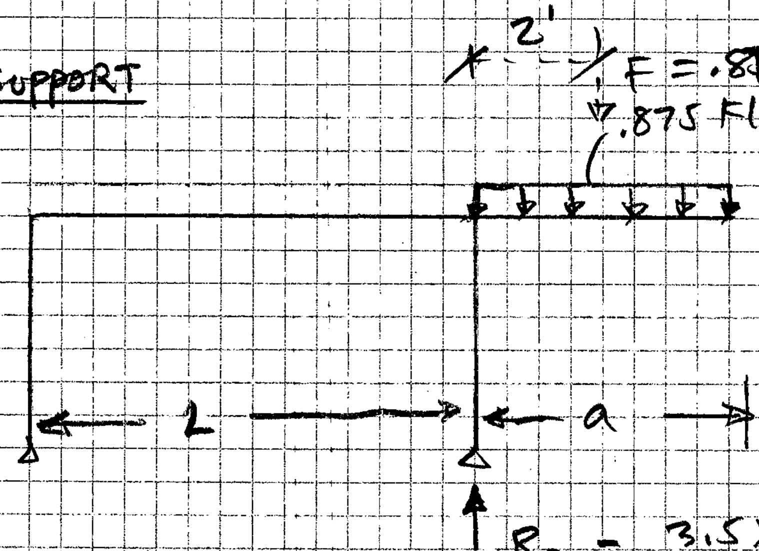
③ CONDITION 2 - OUTER SUPPORT IS LOOSE

ORIGINAL CONDITION

$\omega = 7^k/8' = .875\ klf$



LOOSE SUPPORT



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

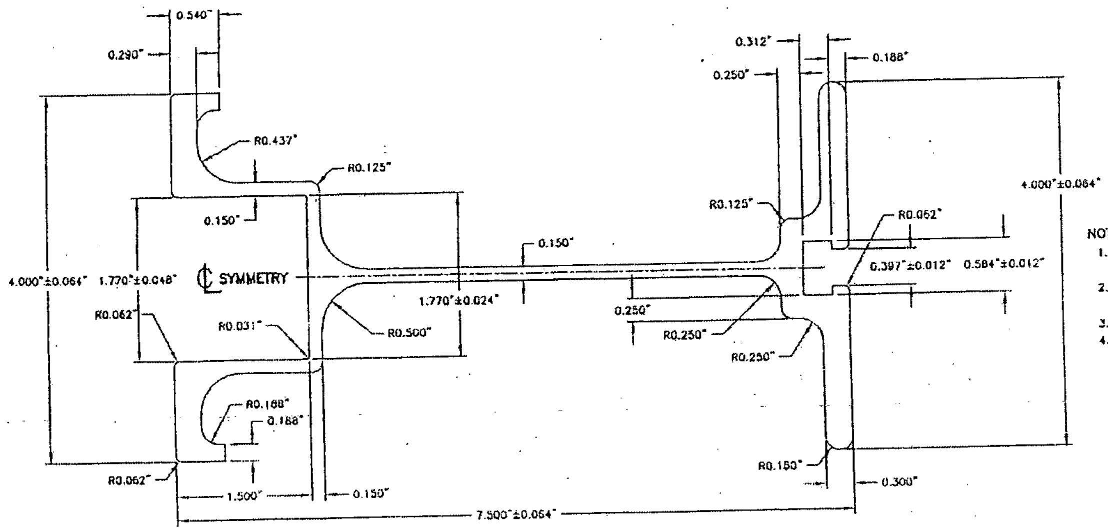
$$\Delta_{max} \stackrel{?}{=} \frac{\omega a^3}{24EI}(4L+3a) = \frac{.875 \times 4^3 \times 1728}{24 \times 10100 \times 34.8}(4\times8+3\times4) = .5''$$

$E = 10100\ ksi$

$I = 34.8\ in^4$

$R_{B\,TOTAL} = R_B + 7k = 11.375\ k$

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

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

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

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)261-5600

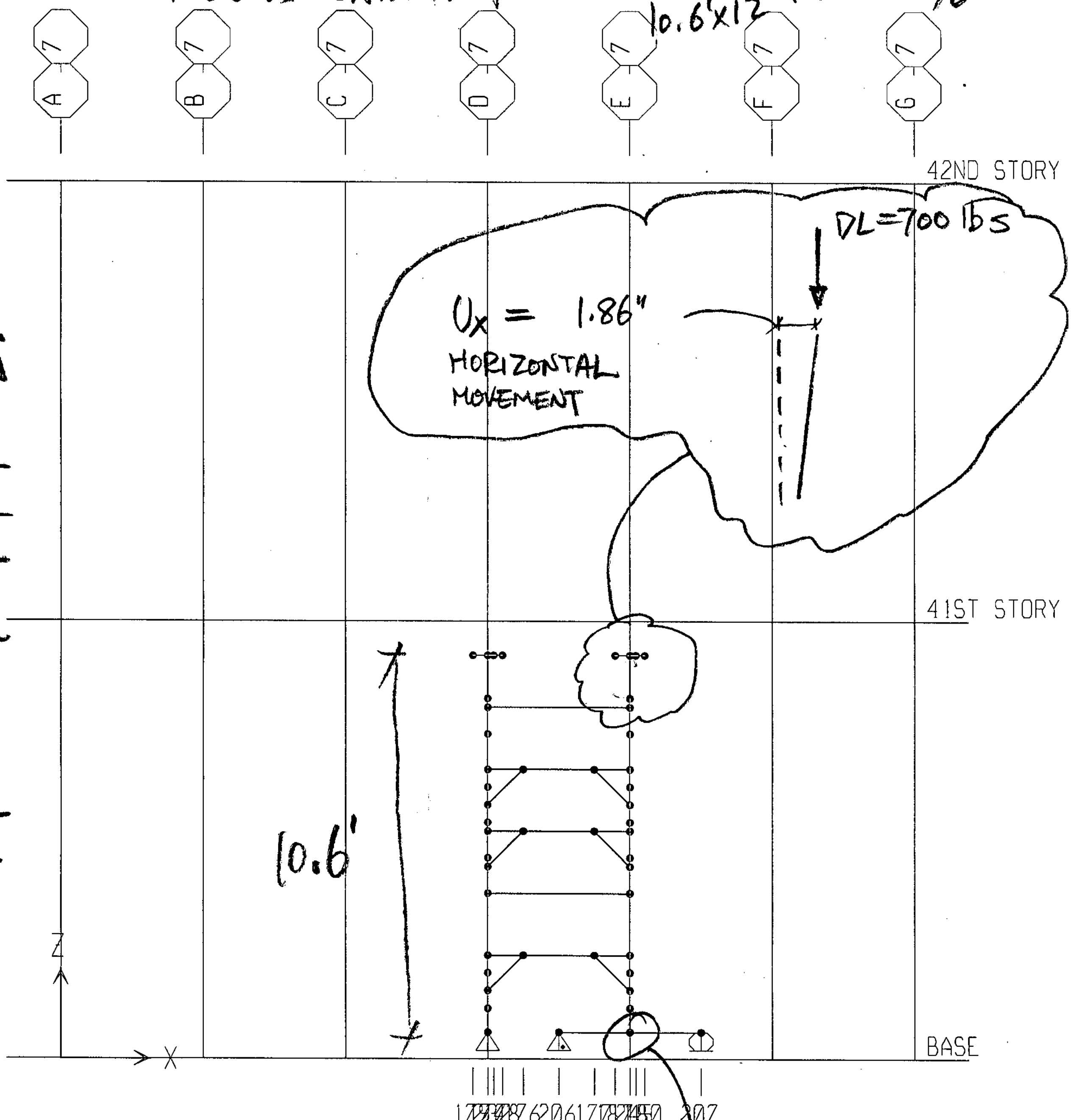
EXTRUSION PROFILE
ALUMINUM STRINGER
INTERFORM

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 = $\frac{1.86"}{10.6' \times 12}$ DL = 1.5% DL

- DEVELOPING 2% LATERAL FORCE BY PUNCHING OF PLYWOOD.



.8" DEFLECTION LIMIT DUE TO PLYWOOD PUNCH.

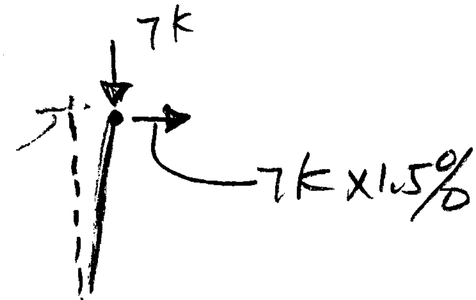
ETABS v9.0.7 File:246SPRINGST_2009_05_08_SINGLETOWERDL_2INCHECCDEFLECTIONLIMIT Units:Kip-in July 22, 2009 18:01

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

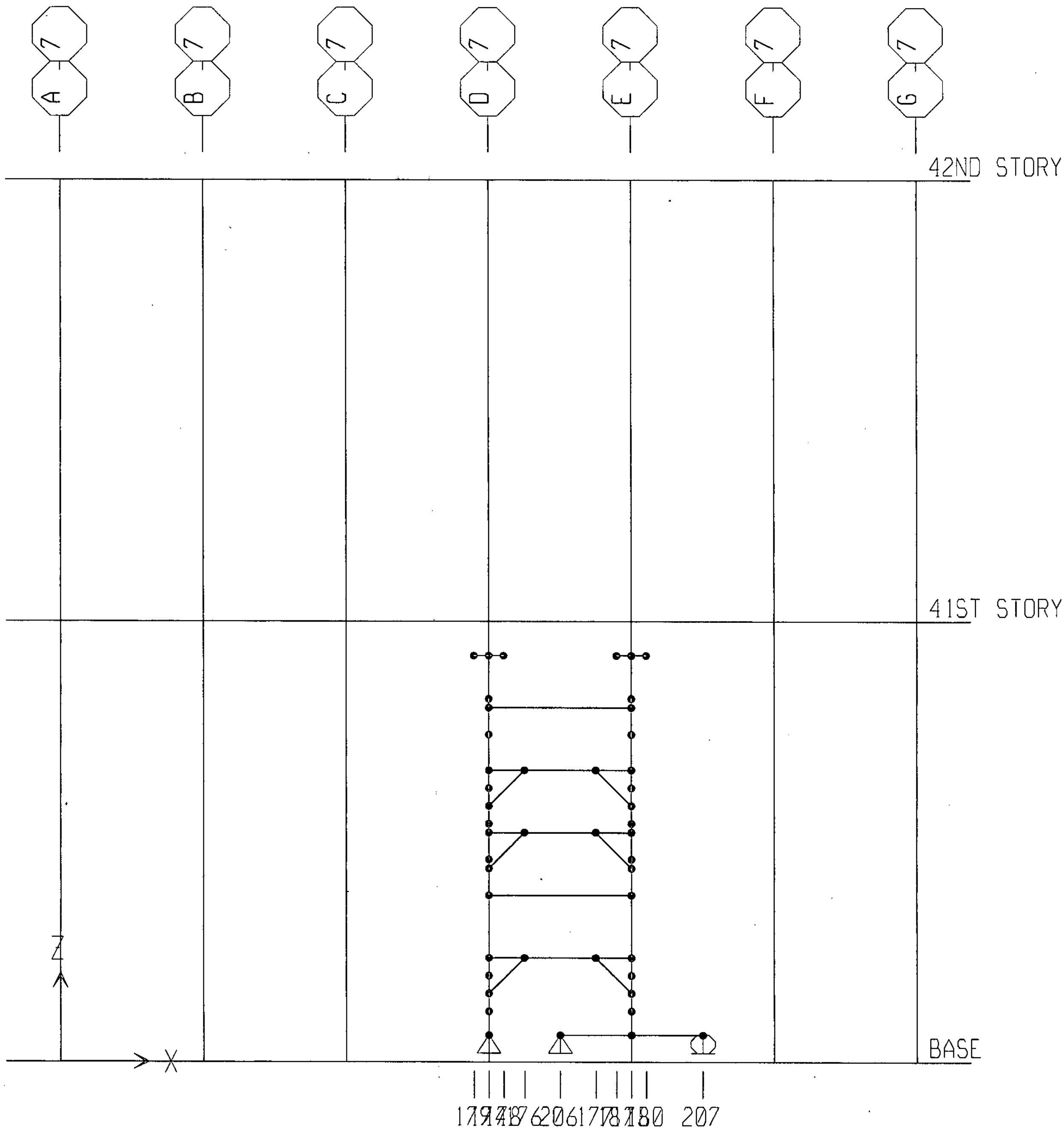
HORIZONTAL DEFLECTION ON THE TOP CAUSED BY THE DEAD LOAD AND 18" LIMITED SUPPORT DUE TO PUNCHING OF PLYWOOD,

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



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: ________

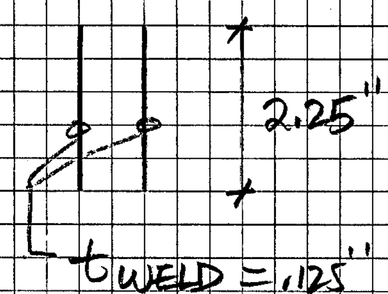
HORIZONTAL RUNG WELD TO SHORE LEG

- 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.

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 $\frac{1}{16}$ 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 $\frac{1}{50}$ 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 $\frac{1}{16}$ 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 $\frac{1}{16}$ 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 sacrifice in ductility of the weld. Another disadvantage is the warping which often accompanies the heat-treating procedure.

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 6063-T6	4043	17	11	11	11	11.5
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 $\frac{3}{8}$ in. or more. For smaller thicknesses, the strengths listed for 5356 and 5556 filler wire may also be used for 4043 filler.

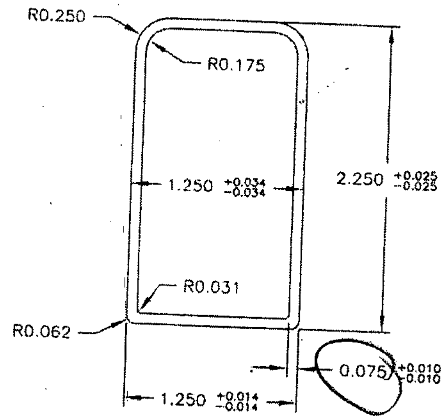
Factor of Safety. Although the tensile strengths across butt welds listed in Table 13 are equal to the ASME weld-qualification test values, they are not generally considered to have the same reliability as the minimum mechanical properties of the parent metal. Accordingly, it has been recommended[3] 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 IN^2

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) 201-5500

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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3004-H34 or 24	4043	22	11	11	14	11.5
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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[f]
6063-T5 6063-T6	4043	17	11	11	11	11.5
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.

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

Subject: ____ Engr: ____ Checked By: ____ Date: ____

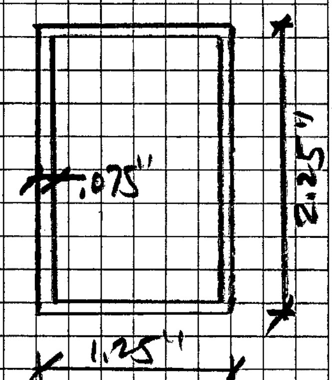
HORIZONTAL RUNG MEMBER CAPACITY

$F_u = 38$ 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

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

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

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

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

$= 5.4$ Kips

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

72

Table 3.3-1
MINIMUM MECHANICAL PROPERTIES FOR ALUMINUM ALLOYS

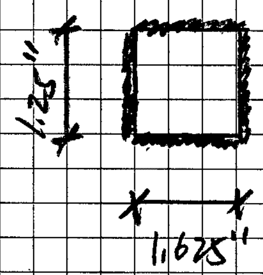
ALLOY AND TEMPER	PRODUCT	THICKNESS RANGE IN.	TENSION F_{tu}† ksi	TENSION F_{ty}† ksi	COMPRESSION F_{cy} ksi	SHEAR F_{su} ksi	SHEAR F_{sy} ksi	COMPRESSIVE MODULUS OF ELASTICITY‡ E ksi
5052-O	Sheet & Plate	0.006-3.000	25	9.5	9.5	16	5.5	10,200
-H32	Sheet & Plate	All	31	23	21	19	13	10,200
-H34	Cold Fin. 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.

WELD CONNECTION FROM BRACE TO SHORING LEG.

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



TOTAL LENGTH OF WELDS

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

$= 4.25''$

$F_v = 11.5\ \text{ksi}$

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

$= 6.5\ \text{kips}$

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

22

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
```

$P'_{max} = 1.069\,k < V = 39.4\,k$

∴ WELD CAPACITY @ DIAGONAL IS ADEQUATE.

23

ETABS Steel Design

Engineer
Project
Subject

```
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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
```
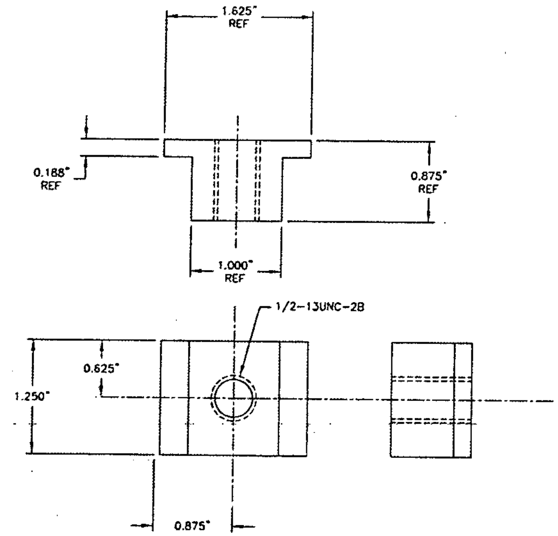

$P_{max} = 1.158\ k < V = 3.94\ k$

∴ WELD CAPACITY @ DIAGONAL IS ADEQUATE.

24

REVISIONS			
LTR	DESCRIPTION	DATE	APPROVED
C	REDRAWN. DIMENSIONS CHANGED TO DECIMAL. NOTES ADDED.	7-27-00	



NOTES:

1. BREAK ALL SHARP EDGES AND CORNERS.
2. DEBURR TAPPED HOLE.
3. FOR USE WITH CAST SLIDELOK 03-120 OR WITH TOGGLE ASSEMBLY 2403-4.

NEXT ASSY.	USED ON
	2403EP
	2403DP
	2403CP
	2403BP
	2403E
	2403D
	2403C
2403-7	2403B

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: EXTRUSION 2403-8A

FINISH:

CONTRACT NO.

PREPARED	S.A.S.	7-27-00
CHECKED		
APPROVED		

OTHER APPROVAL:

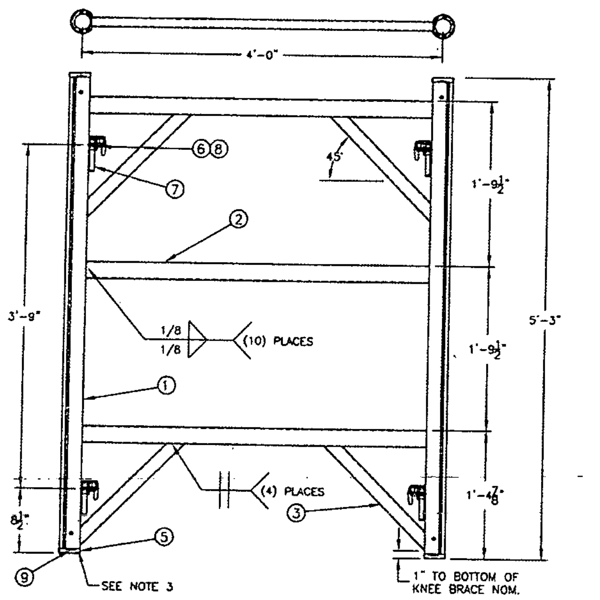
patent — Patent Construction Systems, Harsco Corporation

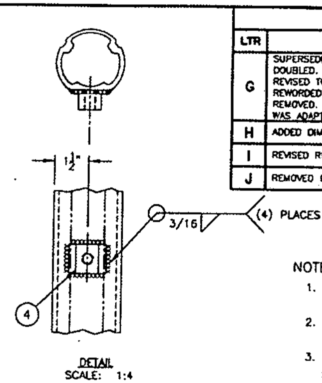
SLIDELOK ADAPTER
FRAME LEG SUB-ASSEMBLY
FRAME
20KA ALUMINUM SHORING

SIZE	CODE IDENT NO.	DRAWING NO.	REV.
B	45826	2403-8	C

SCALE FULL | UNIT WEIGHT 0.10 LBS. | SHEET 1 OF 1

Y:\ENGIN\PARTS\20KA\2403-8





REVISIONS			
LTR	DESCRIPTION	DATE	APPROVED
G	SUPERSEDES DRAWING 2403D REV F DATED 7-13-82. TOLERANCE BLOCK VALUES DOUBLED. ALTERNATE SLIDELOK ASSEMBLY CALLOUT AND NOTE REMOVED. DETAIL REVISED TO TWO VIEWS. 1/32" MISMATCH DIMENSION ADDED TO NOTE 1. NOTE 2 REWORDED. NOTE 3 ADDED. DUPLICATE DIMENSIONS FROM PART DETAIL DRAWINGS REMOVED. ITEM 4 WAS SLIDELOK ASSEMBLY 2403-10 OR 2403-7; ITEM 5 NAME WAS ADAPTOR CUP; ITEMS 6,7,8 AND 9 ADDED AS SEPARATE ITEMS.	7-27-00	
H	ADDED DIMENSIONS, PART CALLOUTS, WELD CALLOUTS	8-20-06	
I	REVISED RUNG DRAWING CALLOUT	8-21-06	
J	REMOVED 9/16" DIA. HOLES IN LEGS	7-20-07	

NOTES:

1. CUT AND MATCH LEGS AND RUNGS TO 1/32". DO NOT USE PARTS MISMATCHED BY MORE THAN 1/32".
2. SLIDELOK ADAPTOR, ITEM 4, MUST BE ORIENTED AS SHOWN IN DETAIL SO THAT SLIDELOK, ITEM 7, SLIDES VERTICALLY.
3. SEE DRAWING 2403-9A FOR ATTACHMENT PROCEDURE DETAILS FOR STEEL END CAP.

NO.	QTY.	DESCRIPTION	DWG. NO.
9	8	#6 X 3/8" LONG DRIVE SCREW, STEEL, ZINC PLATE SEE NOTE 3	
8	4	SPECIAL HEX NUT	03-122
7	4	SLIDELOK	03-120
6	4	1/2-13UNC-2A X 2" LONG THREADED ROD, STEEL, ASTM A307 OR SAE J429 GRADE 2, ZINC PLATE CHROMATE DIP	
5	4	STEEL END CAP SEE NOTE 3	2403-9
4	4	SLIDELOK ADAPTOR SEE NOTE 2 AND DETAIL	2403-8
3	4	DIAGONAL	2403-3C
2	3	RUNG	2403-2A
1	2	FRAME LEG	2403-1C

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:

FINISH:

CONTRACT NO.

PREPARED	S.A.S.	7-27-00
CHECKED		
APPROVED		

OTHER APPROVAL:

patent Patent Construction Systems
Harsco Corporation

5'-0" X 4'-0" WIDE FRAME WITH STEEL END CAPS
20KA ALUMINUM SHORING

SIZE	CODE IDENT NO.	DRAWING NO.	REV.
B	45826	2403C	J
SCALE NOTED	UNIT WEIGHT 36.5 LBS.	SHEET 1 OF 1	

Y:\ENGIN\PARTS\20KA\2403CREVJ

26

Project: 246 Spring Street
Subject: Screw Jack Capacity

Engineer: JW
Checked by:

Date: 12/23/2008
Date:

Design Component: Steel - Screw Jack
Design Method: Allowable Stress Design
Design Code: AISC - ASD 89

Buckling:

Fy =	55	ksi
E =	29000	ksi
K =	2	
L =	21	in
r =	0.638	in
A =	1.279	in^2
KL / r =	65.83072	
Cc =	102	
Fa =	23.23	ksi
Pa =	29.71	kips

Fixed-free assumption (K = 2)

Factor of Safety = 1.67
Buckling Strength Pultimate= 49.61 kips

Flextural: Eccentricity = 2 inch

Fy =	55	ksi
Sx =	0.51	in^3
Ma =	18.51	k-in

Mrupture = 28.05 k-in

Actual Load Input:

Pmax = 7 kips

Mmax = 14 k-in

Pmax / Pultimate + Mmax / Mrupture= 0.64

Tested Load Input:

Pmax = 14 kips

Mmax = 28 k-in

Pmax / Pultimate + Mmax / Mrupture= 1.28

Code Check (include 2% lateral load = 7kips x 2% = 140 lbs

Pmax = 7 kips

Mmax = 16.94 =M1+M2
M1 = 14 k-in
M2 = 2.94 k-in

Pmax / Pa + Mmax / Ma= 1.15

1.6

$\frac{1.15}{1.6}$

140 × 21

3 × 3/4

9/28

27

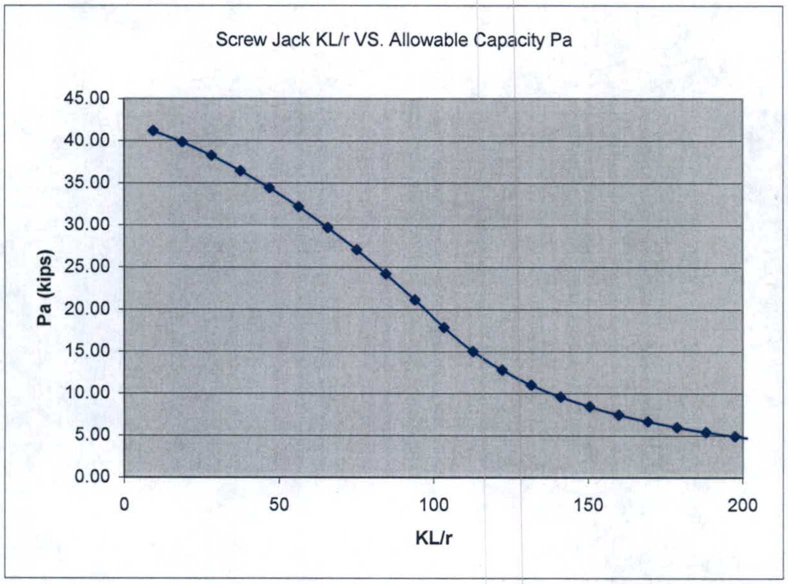
Project: 246 Spring Street	Engineer: JW	Date: 12/23/2008
Subject: Screw Jack Capacity	Checked by:	Date:

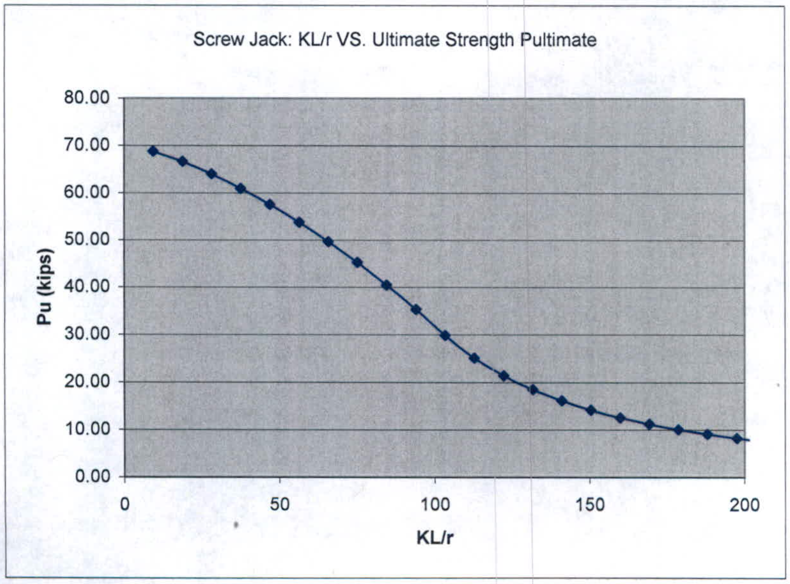
Design Component: Steel - Screw Jack
Design Method: Allowable Stress Design
Design Code: AISC - ASD 89 Factor of Safety = 1.67

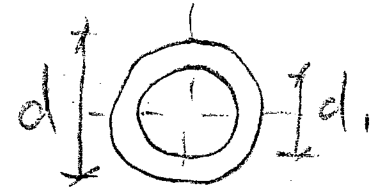
Fy = 55 ksi
E = 29000 ksi

r = 0.638 in
A = 1.279 in^2

Cc = 102

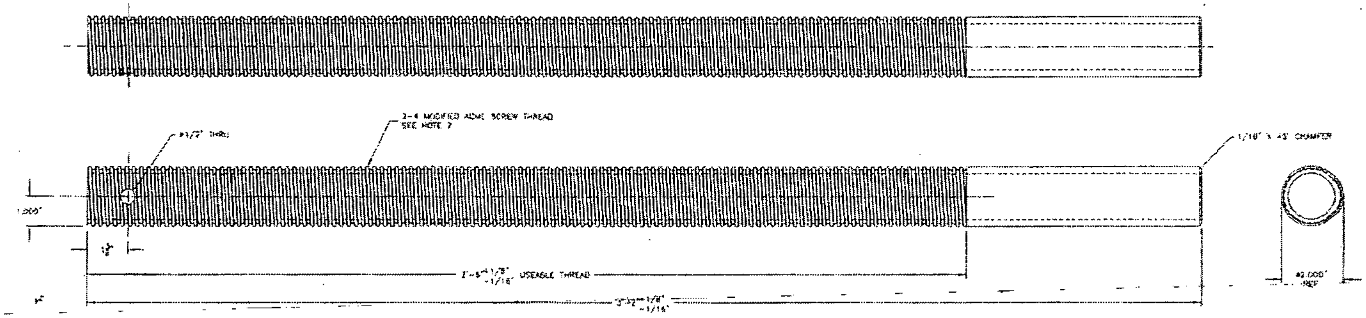






$$d \qquad d_1$$

REVISIONS			
LTR	DESCRIPTION	DATE	APPROVED
D	SUPERSEDES DRAWING 202EE-1 REVISION C [illegible]	4-3-98	



$$A = 3.1415 \times \left[2^2 - (2 - .23 \times 2)^2\right] / 4$$

$$= 1.279 \text{ in}^2$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .51 \text{ in}^4$$

$$S = \frac{\pi(d^4 - d_1^4)}{32\,d} = .51 \text{ in}^3$$

$$r = \sqrt{\frac{d^2 + d_1^2}{4}} = .638''$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6} = .725 \text{ in}^3$$

NOTES:

1. MATERIAL: 2.000" O.D. X 0.230" WALL COLD DRAWN BUTTWELD CARBON STEEL MECHANICAL TUBE MANDREL DRAWN, GRADE 1020, 55,000 PSI MINIMUM YIELD STRENGTH, PER ASTM A512 OR ERW CARBON STEEL TUBE, MANDREL DRAWN, GRADE 1020, 55,000 PSI MINIMUM YIELD, PER ASTM A513
2. SEE THREAD SPECIFICATION DRAWING 1HD-002.

202EE	202EE
NEXT ASSY.	USED ON
APPLICATION	

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: SEE NOTE 1

FINISH:

CONTRACT NO.

PREPARED	S.A.S.	4-3-98
CHECKED		
APPROVED		

OTHER APPROVAL:

patent — Patent Construction Systems, Harsco Corporation

SCREW
24" ADJUSTABLE SCREW LEG
20K SHORING

SIZE	CODE IDENT NO.	DRAWING NO.	REV.
C	45826	202EE-1	D

SCALE 7:18 | UNIT WEIGHT 8.73 LBS. | SHEET 1 OF 1

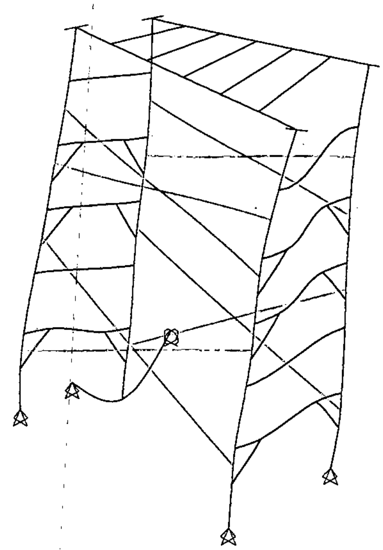
4/14/09

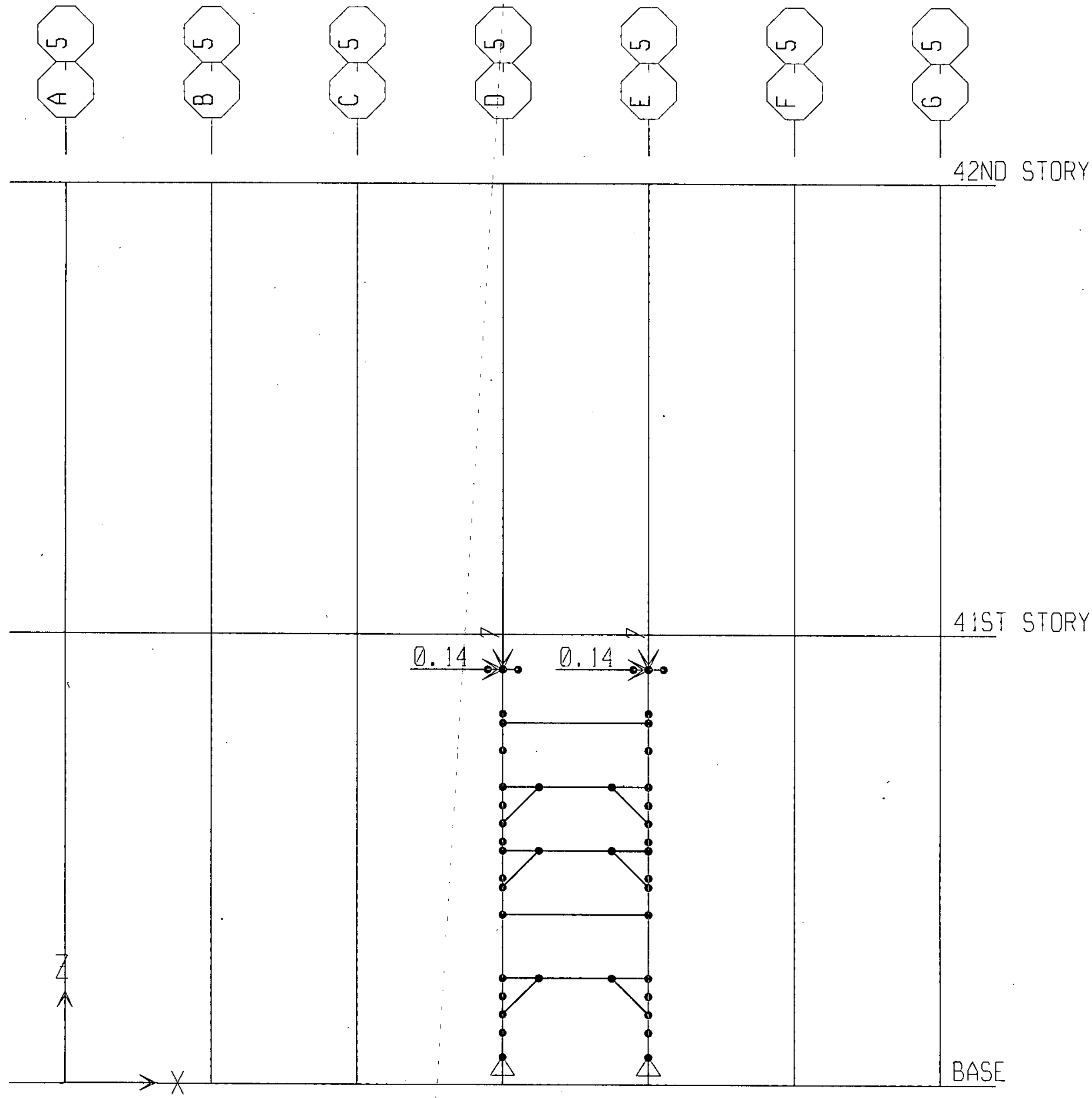
3.2 Single Tower - Allowable Stress Code Check

These computer analyses combine deadweight of concrete and formwork with a lateral force equivalent to 2% of the vertical load and a .8 deflection under one leg only. The 0.8 deflection was added as a result of the testing observations. Such deflection was likely at the 41st floor level in one leg as the plywood underneath was being punched. The vertical load on each leg is in the order of 7000 lbs as described above in section 14.2.1. The overall frame is made of 5'-3" bottom frame and 3'-6" top frame, and the frame is 8' apart. Two loading conditions have been considered and studied for the effect 0.8" deflection to all members in the tower.

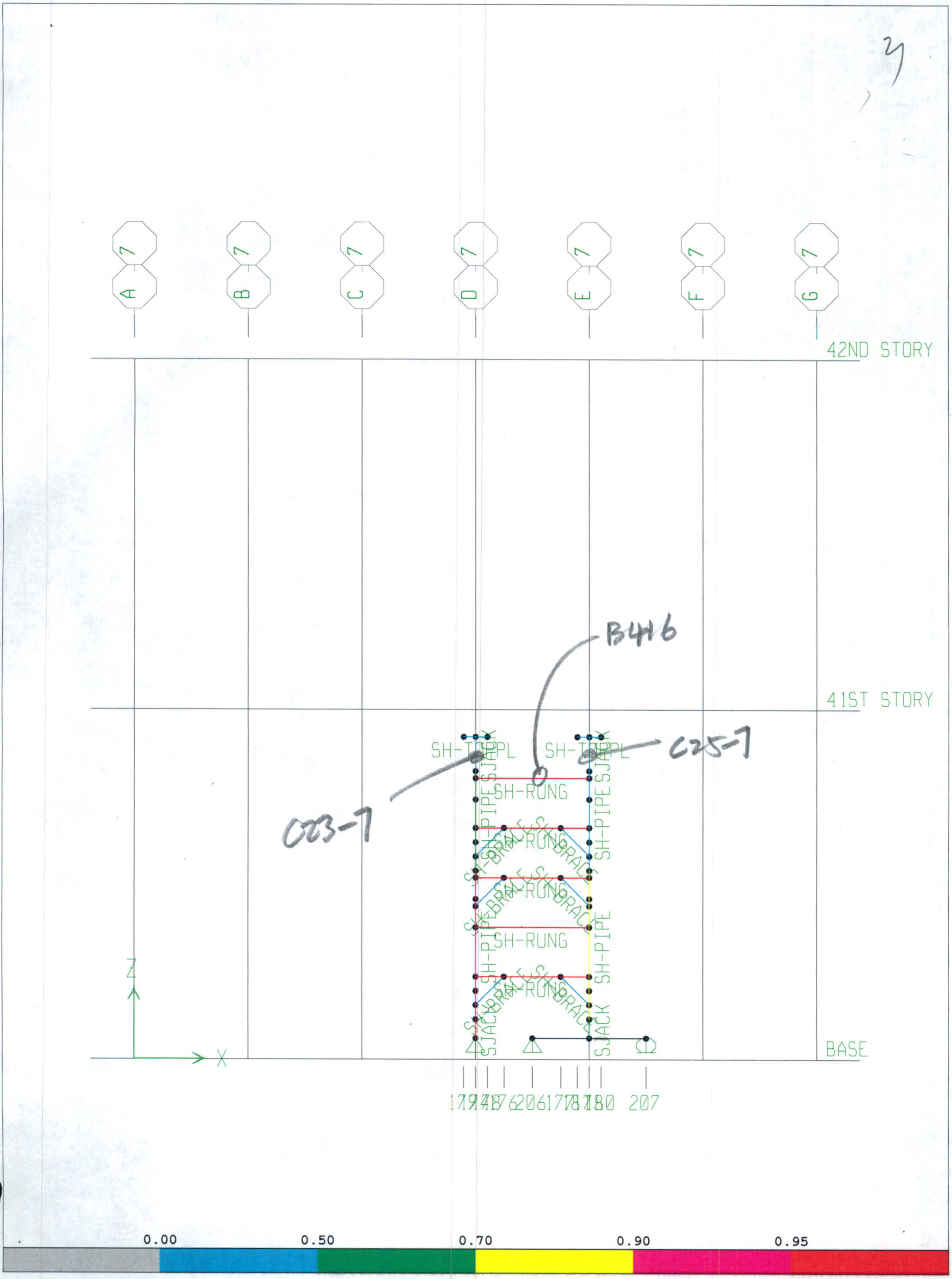
3.2.1 Allowable Stress Check - Condition 1, Load Case A.

- The model file is "246SpringST_2009_05_08_singleTowerDL_NoECCdeflectionLimitCodeCheckWith2%Lateral".
- A vertical load of 7,000 lbs and <u>a horizontal load of 140 lbs</u> placed at the center of top plate.
  Result: shore failure
- Screw jack allowable stresses produced by bending and compression are about 60% of the allowable.
- Aluminum shoring leg under bending and compression was stressed at about 90% of allowable.
- Horizontal members and connections from horizontal members to shoring legs do not meet allowable stress ratio combination. The ratio actual versus allowable is 2.
- Stress ratio in knee brace is insignificant.

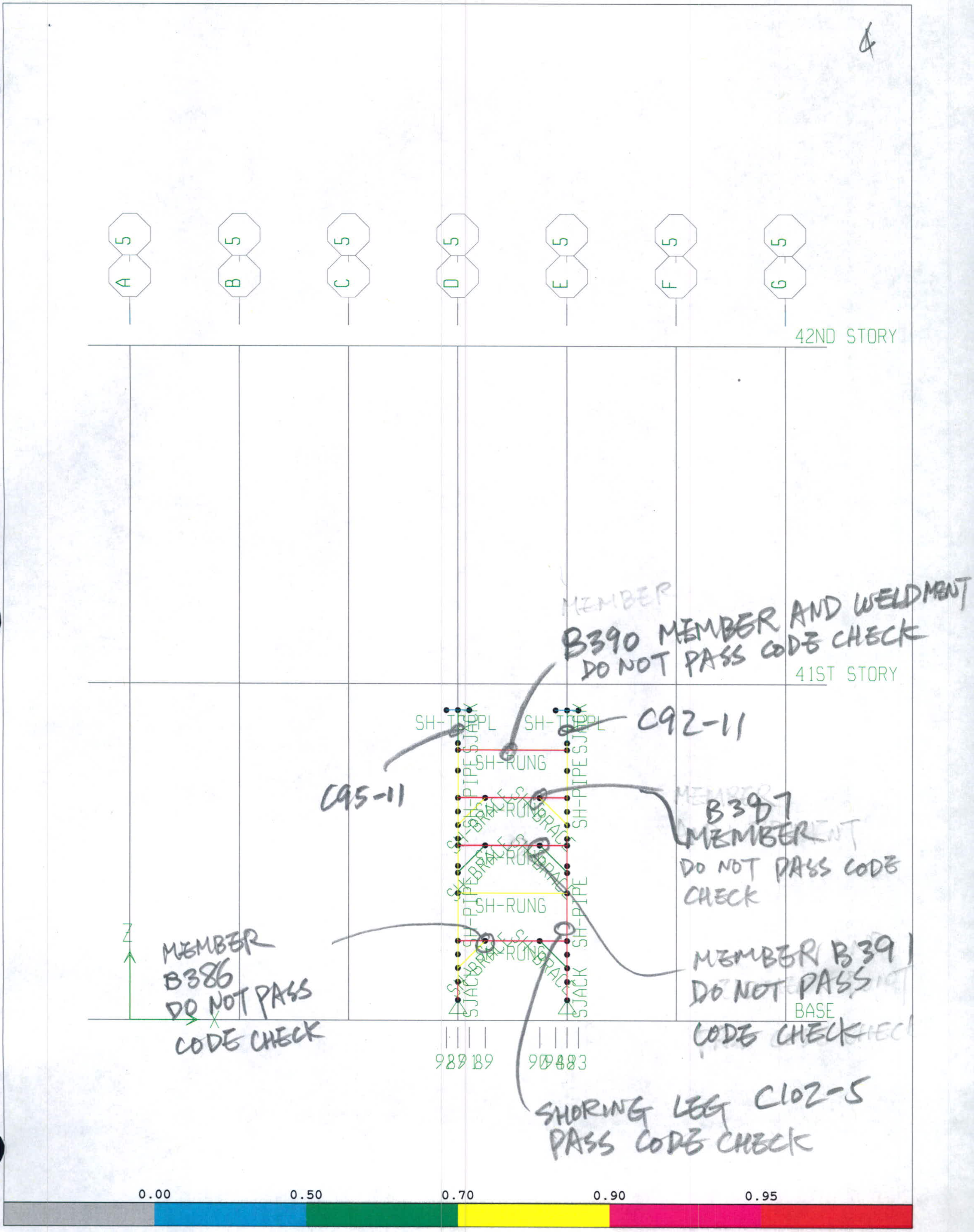




ETABS v9.0.7 - File: 246SpringST_2009_05_08_singleTowerDL_NoECCdeflectionLimitCodeCheckWith2%Lateral - June 11,2009 18:42
Elevation View - 5 Point Loads (DEAD) - Kip-in Units


7
A
B
C
D
E
F
G
42ND STORY
41ST STORY
BASE
B416
C25-7
C23-7
SH-TOPPL
SJACK
SH-PIPE
SH-RUNG
SH-BRACE
Z
X
207
0.00
0.50
0.70
0.90
0.95

4



A 5
B 5
C 5
D 5
E 5
F 5
G 5
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-RUNG
SH-PIPE
SJACK
SH-BRACE
Z
X
B390 MEMBER AND WELDMENT DO NOT PASS CODE CHECK
C92-11
C95-11
B397 MEMBER DO NOT PASS CODE CHECK
MEMBER B386 DO NOT PASS CODE CHECK
MEMBER B391 DO NOT PASS CODE CHECK
SHORING LEG C102-5 PASS CODE CHECK
0.00
0.50
0.70
0.90
0.95

LOC:0 LOC: 48

START 48" END

MEMBER DESIGNATION

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK     Units: Kip-in   (Summary for Combo and Station)
Level: 41ST STORY  Element: B416  Station Loc: 0.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  0.849 = 0.002 + 0.428 + 0.419

STRESS CHECK FORCES & MOMENTS
                            P          M33          M22           V2          V3
    Combo  DSTLS2       0.026       -2.697       -1.875       -0.113      -0.078

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                           fa           Fa           Ft
                       Stress    Allowable    Allowable
    Axial               0.051        5.987       21.000

                           fb           Fb           Fe       Cm        K        L       Cb
                       Stress    Allowable    Allowable   Factor   Factor   Factor   Factor
    Major Bending       8.987       21.000       15.165    1.000    1.000    1.000    2.300
    Minor Bending       8.790       21.000        5.987    1.000    1.000    1.000

SHEAR DESIGN
                           fv           FV       Stress
                       Stress    Allowable        Ratio
    Major Shear         0.334       14.000        0.024
    Minor Shear         0.417       14.000        0.030
```

TYPE OF ASSIGNED MEMBER

MEMBER CAPACITY

$M_a = 6.4$ K-in > $M_{33} = 2.697$ K-in (OK)

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-7  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.312 = 0.168 + 0.144 + 0.004

STRESS CHECK FORCES & MOMENTS
                           P          M33         M22          V2          V3
    Combo  DSTLS2       -7.075      -2.671       0.076      -0.118       0.007

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                          fa          Fa          Ft
                      Stress   Allowable   Allowable
    Axial              5.532      32.025      33.000

                          fb          Fb          Fe        Cm         K         L        Cb
                      Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending      5.245      36.300    1213.622     0.877     1.000     0.483     1.167
    Minor Bending      0.149      36.300    1213.622     0.850     1.000     0.483

SHEAR DESIGN
                          fv          FV      Stress
                      Stress   Allowable       Ratio
    Major Shear        0.155      22.000       0.007
    Minor Shear        0.009      22.000       0.000
```

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C23-7  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.334 = 0.168 + 0.166 + 0.001

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2     -7.075       -3.071       0.020      -0.142       0.004

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                         fa           Fa          Ft
                     Stress    Allowable   Allowable
    Axial             5.532       32.025      33.000

                         fb           Fb          Fe      Cm       K       L      Cb
                     Stress    Allowable   Allowable  Factor  Factor  Factor  Factor
    Major Bending     6.029       36.300    1213.622   0.871   1.000   0.483   1.177
    Minor Bending     0.040       36.300    1213.622   0.850   1.000   0.483

SHEAR DESIGN
                         fv           FV      Stress
                     Stress    Allowable       Ratio
    Major Shear       0.187       22.000       0.009
    Minor Shear       0.005       22.000       0.000
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in   (Summary for Combo and Station)
Level: 41ST STORY  Element: B390   Station Loc: 48.000   Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.590 = 0.002 + 1.176 + 0.411

STRESS CHECK FORCES & MOMENTS
                        P         M33        M22        V2         V3
    Combo  DSTLS2      0.025     -7.414      1.843      0.309     -0.077

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                       fa         Fa          Ft
                    Stress    Allowable   Allowable
    Axial            0.050      5.987      21.000

                       fb         Fb          Fe        Cm       K        L       Cb
                    Stress    Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending   24.706     21.000      15.165     1.000    1.000    1.000    2.300
    Minor Bending    8.641     21.000       5.987     1.000    1.000    1.000

SHEAR DESIGN
                       fv         FV        Stress
                    Stress    Allowable     Ratio
    Major Shear      0.916     14.000       0.065
    Minor Shear      0.409     14.000       0.029
```

MEMBER CAPACITY:

$M_a = 6.4$ k-in $< M_{33} = 7.414$ k-in

ETABS Steel Design

Engineer________________
Project________________
Subject________________

a

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C92-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.552 = 0.168 + 0.385 + 0.004

STRESS CHECK FORCES & MOMENTS
                              P         M33         M22          V2          V3
    Combo  DSTLS2        -7.075       7.114      -0.076       0.421      -0.007

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 5.532      32.025      33.000

                             fb          Fb          Fe        Cm        K         L        Cb
                         Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending        13.968      36.300    1213.622     0.850     1.000     0.483     1.238
    Minor Bending         0.149      36.300    1213.622     0.850     1.000     0.483

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           0.556      22.000       0.025
    Minor Shear           0.009      22.000       0.000
```

10

ETABS Steel Design

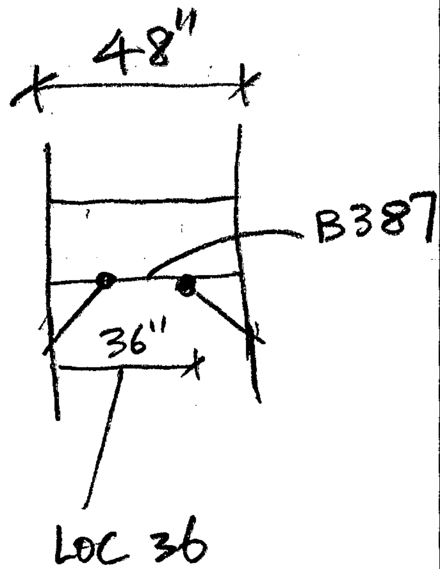
Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C95-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame  Classification:  Compact

L=14.500
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   0.533 = 0.168 + 0.365 + 0.001

STRESS CHECK FORCES & MOMENTS
                              P          M33          M22           V2           V3
    Combo  DSTLS2        -7.075        6.748       -0.020        0.399       -0.004

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                             fa           Fa           Ft
                         Stress    Allowable    Allowable
    Axial                 5.532       32.025       33.000

                             fb           Fb           Fe       Cm        K        L       Cb
                         Stress    Allowable    Allowable   Factor   Factor   Factor   Factor
    Major Bending        13.249       36.300     1213.622    0.850    1.000    0.483    1.237
    Minor Bending         0.040       36.300     1213.622    0.850    1.000    0.483

SHEAR DESIGN
                             fv           FV       Stress
                         Stress    Allowable        Ratio
    Major Shear           0.526       22.000        0.024
    Minor Shear           0.005       22.000        0.000
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK   Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B387  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame    Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.043 = 0.006 + 1.196 + 0.841

STRESS CHECK FORCES & MOMENTS
                              P         M33         M22          V2          V3
    Combo  DSTLS2        -0.018      -7.536       3.768       0.624      -0.314

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 0.036       5.987      21.000

                             fb          Fb          Fe          Cm           K           L          Cb
                         Stress   Allowable   Allowable      Factor      Factor      Factor      Factor
    Major Bending        25.111      21.000      60.662       0.850       1.000       0.500       1.000
    Minor Bending        17.669      21.000       5.987       0.850       1.000       1.000

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           1.850      14.000       0.132
    Minor Shear           1.675      14.000       0.120
```



12

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B391  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.598 = 0.001 + 0.985 + 0.611

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22          V2          V3
    Combo  DSTLS2      -0.004       -6.210       2.737       0.514      -0.228

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                           fa           Fa          Ft
                       Stress    Allowable   Allowable
    Axial               0.007        5.987      21.000

                           fb           Fb          Fe       Cm        K        L       Cb
                       Stress    Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending      20.694       21.000      60.662    0.850    1.000    0.500    1.000
    Minor Bending      12.835       21.000       5.987    0.850    1.000    1.000

SHEAR DESIGN
                           fv           FV      Stress
                       Stress    Allowable       Ratio
    Major Shear         1.522       14.000       0.109
    Minor Shear         1.216       14.000       0.087
```

13

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B386  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.429 = 0.003 + 1.207 + 0.219

STRESS CHECK FORCES & MOMENTS
                         P          M33         M22         V2          V3
    Combo  DSTLS2     -0.009      -7.605      -0.981       0.630       0.082

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                        fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.018       5.987      21.000

                        fb          Fb          Fe         Cm        K        L       Cb
                     Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending    25.344      21.000      60.662     0.850    1.000    0.500    1.000
    Minor Bending     4.598      21.000       5.987     0.850    1.000    1.000

SHEAR DESIGN
                        fv          FV      Stress
                     Stress   Allowable      Ratio
    Major Shear       1.867      14.000      0.133
    Minor Shear       0.436      14.000      0.031
```

14

ETABS Steel Design

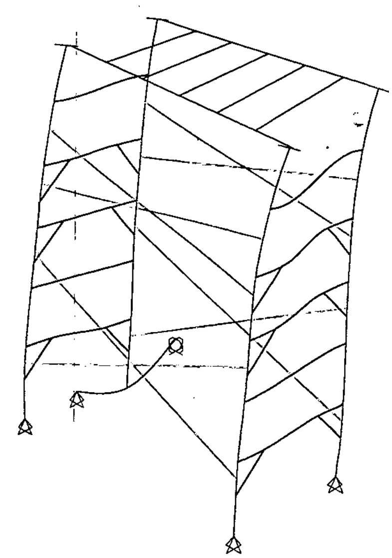
Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-5  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   0.973 = 0.418 + 0.024 + 0.555

STRESS CHECK FORCES & MOMENTS
                            P         M33         M22          V2          V3
    Combo  DSTLS2      -9.268      -0.553      11.025      -0.958      -0.591

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1)
                           fa          Fa          Ft
                       Stress   Allowable   Allowable
    Axial               6.901      16.510      21.000

                           fb          Fb          Fe        Cm         K         L        Cb
                       Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending       0.606      23.100     120.071     0.850     1.000     0.333     1.000
    Minor Bending      12.094      23.100      34.813     0.850     1.000     0.619

SHEAR DESIGN
                           fv          FV      Stress
                       Stress   Allowable       Ratio
    Major Shear         1.322      14.000       0.094
    Minor Shear         0.815      14.000       0.058
```

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

- The model file is "246SpringST_2009_05_08_singleTowerDL_2inchECCdeflectionLimitCodeCheckWith2%Lateral".
- A vertical load of 7,000 lbs and <u>a horizontal load of 140 lbs</u> placed at <u>2" away from</u> the center of top plate.

  Result: screw jack and shore failure
- Screw jack allowable stresses produced by bending and compression are about 140% of the allowable. Overstress condition.
- Aluminum shoring leg under bending and compression was stressed at about 140 % of allowable. Overstress condition.
- Horizontal members and connections from horizontal members to shoring legs do not meet allowable stress ratio combination. The ratio actual versus allowable is 2.62 Overstress condition.
- Stress ratio in knee brace is insignificant.

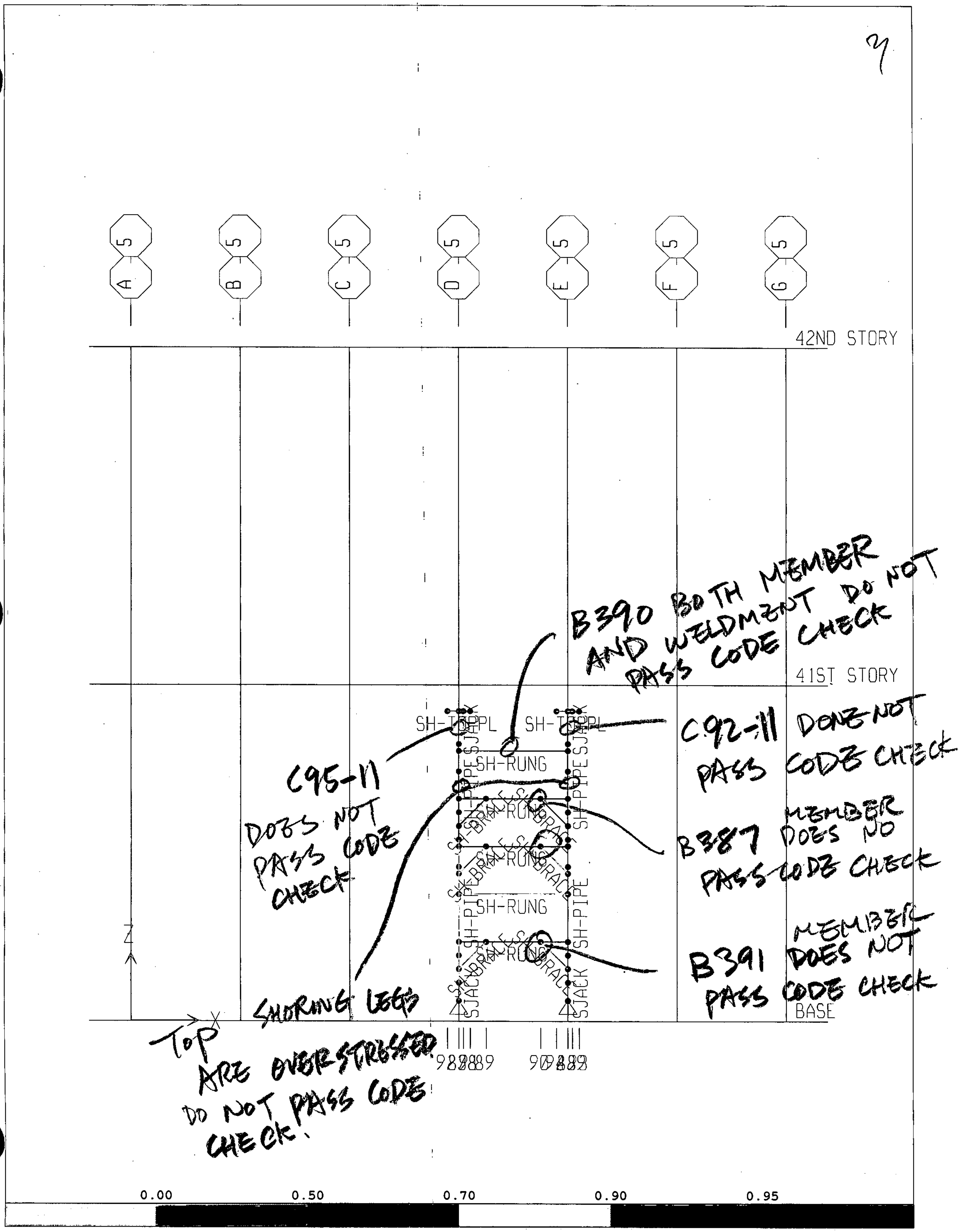


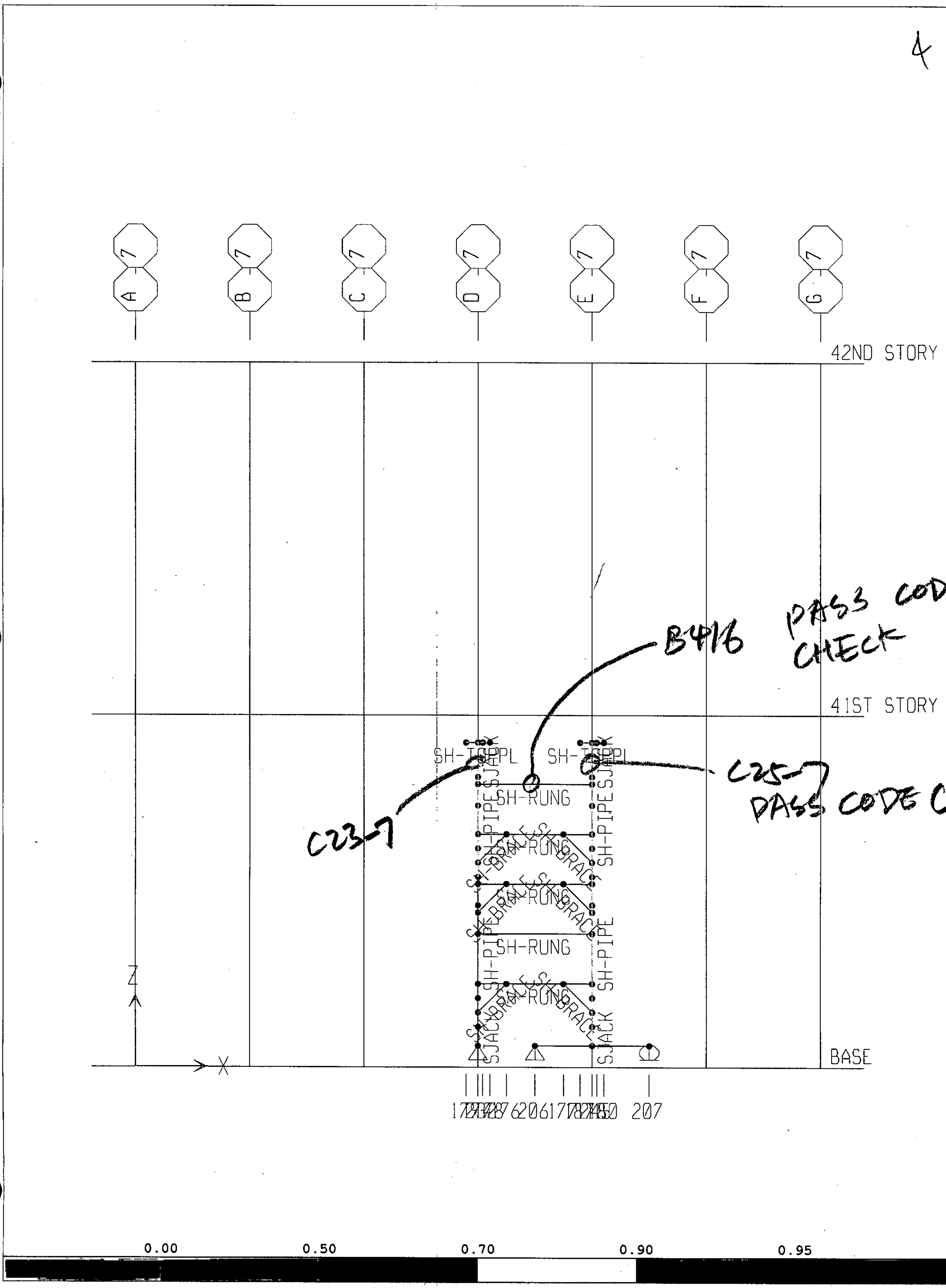
ETABS Steel Design

Engineer__________
Project__________
Subject__________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C92-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   1.335 = 0.168 + 1.167 + 0.005

STRESS CHECK FORCES & MOMENTS
                              P         M33         M22          V2          V3
    Combo  DSTLS2        -7.075      21.581      -0.097       0.441      -0.005

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 5.532      30.637      33.000

                             fb          Fb          Fe       Cm        K        L       Cb
                         Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending        42.372      36.300     282.842    0.881    1.000    1.000    1.160
    Minor Bending         0.191      36.300     282.842    0.850    1.000    1.000

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           0.583      22.000       0.027
    Minor Shear           0.007      22.000       0.000
```

7


A-5
B-5
C-5
D-5
E-5
F-5
G-5
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-TOPPL
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-PIPE
SJACK
BRACE
Z
X
B390 BOTH MEMBER AND WELDMENT DO NOT PASS CODE CHECK
C92-11 DOES NOT PASS CODE CHECK
C95-11 DOES NOT PASS CODE CHECK
B387 MEMBER DOES NO PASS CODE CHECK
B391 MEMBER DOES NOT PASS CODE CHECK
SHORING LEGS ARE OVERSTRESSED DO NOT PASS CODE CHECK.
Top
0.00
0.50
0.70
0.90
0.95


4
A-7
B-7
C-7
D-7
E-7
F-7
G-7
42ND STORY
41ST STORY
BASE
B416 PASS CODE CHECK
C25-7 PASS CODE CHECK
C23-7
SH-RUNG
SH-PIPE
SJACK
Z
X
207
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: 41ST STORY   Element: B390   Station Loc: 48.000   Section ID: SH-RUNG
Element Type: Moment Resisting Frame   Classification: Compact

L=48.000
A=0.503 i22=0.133 i33=0.338
s22=0.213 s33=0.300 r22=0.515 r33=0.820
E=10100.000 fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is 2.697 = 0.003 + 2.252 + 0.442

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS2	0.027	-14.191	1.981	0.592	-0.083

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H2-1)

	fa Stress	Fa Allowable	Ft Allowable
Axial	0.053	5.987	21.000

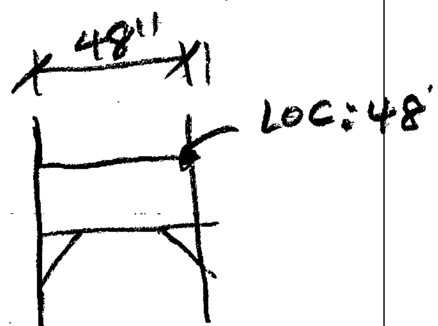
	fb Stress	Fb Allowable	Fe Allowable	Cm Factor	K Factor	L Factor	Cb Factor
Major Bending	47.290	21.000	15.165	1.000	1.000	1.000	2.300
Minor Bending	9.288	21.000	5.987	1.000	1.000	1.000	

SHEAR DESIGN

	fv Stress	FV Allowable	Stress Ratio
Major Shear	1.753	14.000	0.125
Minor Shear	0.440	14.000	0.031

MEMBER CAPACITY:

$M_a = 6.4$ k-in $< M_{33} = 14.2$ k-in
(NG)



6

ETABS Steel Design

Engineer__________
Project__________
Subject__________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B387  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame  Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.619 = 0.006 + 1.708 + 0.904

STRESS CHECK FORCES & MOMENTS
                        P          M33          M22          V2          V3
    Combo  DSTLS2    -0.019      -10.766       4.050       0.894      -0.337

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                       fa          Fa          Ft
                   Stress   Allowable   Allowable
    Axial           0.038       5.987      21.000

                       fb          Fb          Fe       Cm        K        L       Cb
                   Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending  35.876      21.000      60.662    0.850    1.000    0.500    1.000
    Minor Bending  18.989      21.000       5.987    0.850    1.000    1.000

SHEAR DESIGN
                       fv          FV      Stress
                   Stress   Allowable       Ratio
    Major Shear     2.647      14.000       0.189
    Minor Shear     1.800      14.000       0.129
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B391  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.813 = 0.001 + 1.155 + 0.656

STRESS CHECK FORCES & MOMENTS
                                  P        M33        M22         V2          V3
    Combo  DSTLS2            -0.003     -7.279      2.939      0.603      -0.245

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 0.007       5.987      21.000

                             fb          Fb          Fe       Cm        K        L       Cb
                         Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending        24.257      21.000      60.662    0.850    1.000    0.500    1.000
    Minor Bending        13.783      21.000       5.987    0.850    1.000    1.000

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           1.786      14.000       0.128
    Minor Shear           1.306      14.000       0.093
```

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B386  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.523 = 0.003 + 1.284 + 0.236

STRESS CHECK FORCES & MOMENTS
                           P         M33         M22          V2          V3
    Combo  DSTLS2     -0.009      -8.090      -1.057       0.670       0.088

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                          fa          Fa          Ft
                      Stress   Allowable   Allowable
    Axial              0.018       5.987      21.000

                          fb          Fb          Fe       Cm       K       L      Cb
                      Stress   Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending     26.957      21.000      60.662    0.850   1.000   0.500   1.000
    Minor Bending      4.959      21.000       5.987    0.850   1.000   1.000

SHEAR DESIGN
                          fv          FV      Stress
                      Stress   Allowable       Ratio
    Major Shear        1.986      14.000       0.142
    Minor Shear        0.470      14.000       0.034
```

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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: C95-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   1.314 = 0.168 + 1.146 + 0.002

STRESS CHECK FORCES & MOMENTS
                           P          M33         M22          V2          V3
    Combo  DSTLS2      -7.075      21.186      -0.032       0.417      -0.002

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                           fa          Fa          Ft
                       Stress   Allowable   Allowable
    Axial               5.532      30.637      33.000

                           fb          Fb          Fe         Cm        K        L       Cb
                       Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending      41.597      36.300     282.842     0.886    1.000    1.000    1.153
    Minor Bending       0.063      36.300     282.842     0.850    1.000    1.000

SHEAR DESIGN
                           fv          FV      Stress
                       Stress   Allowable       Ratio
    Major Shear         0.551      22.000       0.025
    Minor Shear         0.002      22.000       0.000
```

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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: C92-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   1.335 = 0.168 + 1.167 + 0.005

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      -7.075       21.581      -0.097       0.441      -0.005

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             5.532      30.637      33.000

                         fb          Fb          Fe       Cm       K        L       Cb
                     Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending    42.372      36.300     282.842    0.881    1.000    1.000    1.160
    Minor Bending     0.191      36.300     282.842    0.850    1.000    1.000

SHEAR DESIGN
                         fv          Fv      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.583      22.000       0.027
    Minor Shear       0.007      22.000       0.000
```

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-3  Station Loc: 39.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.339 = 0.251 + 1.088 + 0.005

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22          V2          V3
    Combo  DSTLS2      -7.075       22.905      -0.113       0.441      -0.005

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                           fa           Fa          Ft
                       Stress    Allowable   Allowable
    Axial               5.268       19.050      21.000

                           fb           Fb          Fe       Cm       K        L       Cb
                       Stress    Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending      25.126       23.100     120.071    0.850    1.000    0.500    1.000
    Minor Bending       0.124       23.100     163.430    0.850    1.000    0.429

SHEAR DESIGN
                           fv           FV      Stress
                       Stress    Allowable       Ratio
    Major Shear         0.609       14.000       0.043
    Minor Shear         0.007       14.000       0.001
```

12

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C100-3  Station Loc: 39.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.316 = 0.251 + 1.066 + 0.002

STRESS CHECK FORCES & MOMENTS
                         P          M33         M22          V2          V3
    Combo  DSTLS2     -7.075      22.438      -0.038       0.417      -0.002

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                        fa          Fa          Ft
                    Stress   Allowable   Allowable
    Axial            5.268      19.050      21.000

                        fb          Fb          Fe        Cm       K       L      Cb
                    Stress   Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending   24.614      23.100     120.071     0.850   1.000   0.500   1.000
    Minor Bending    0.041      23.100     163.430     0.850   1.000   0.429

SHEAR DESIGN
                        fv          FV      Stress
                    Stress   Allowable       Ratio
    Major Shear      0.576      14.000       0.041
    Minor Shear      0.002      14.000       0.000
```

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-7  Station Loc: 14.500  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.874 = 0.167 + 0.706 + 0.001

STRESS CHECK FORCES & MOMENTS
                          P          M33          M22          V2          V3
    Combo  DSTLS2    -7.069       13.057        0.020      -0.136        0.005

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-2)
                         fa          Fa           Ft
                     Stress   Allowable    Allowable
    Axial             5.527      30.637       33.000

                         fb          Fb           Fe        Cm        K        L       Cb
                     Stress   Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending    25.636      36.300      282.842     0.939    1.000    1.000    1.075
    Minor Bending     0.039      36.300      282.842     0.850    1.000    1.000

SHEAR DESIGN
                         fv          FV       Stress
                     Stress   Allowable        Ratio
    Major Shear       0.180      22.000        0.008
    Minor Shear       0.007      22.000        0.000
```

14

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C23-7  Station Loc: 14.500  Section ID: SJACK
Element Type:  Moment Resisting Frame  Classification:  Compact

L=14.500
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   0.871 = 0.167 + 0.703 + 0.000

STRESS CHECK FORCES & MOMENTS
                           P         M33         M22          V2          V3
    Combo  DSTLS2     -7.069      13.003       0.007      -0.162       0.002

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                          fa          Fa          Ft
                      Stress   Allowable   Allowable
    Axial              5.527      30.637      33.000

                          fb          Fb          Fe        Cm         K         L        Cb
                      Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending     25.531      36.300     282.842     0.928     1.000     1.000     1.091
    Minor Bending      0.013      36.300     282.842     0.850     1.000     1.000

SHEAR DESIGN
                          fv          FV      Stress
                      Stress   Allowable       Ratio
    Major Shear        0.214      22.000       0.010
    Minor Shear        0.002      22.000       0.000
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B416  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame  Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  0.989 = 0.003 + 0.536 + 0.450

STRESS CHECK FORCES & MOMENTS
                              P          M33         M22          V2          V3
    Combo  DSTLS2         0.027       -3.381       2.015       0.141      -0.084

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H2-1)
                             fa           Fa          Ft
                         Stress    Allowable   Allowable
    Axial                 0.055        5.987      21.000

                             fb           Fb          Fe       Cm        K        L       Cb
                         Stress    Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending        11.265       21.000      15.165    1.000    1.000    1.000    2.300
    Minor Bending         9.447       21.000       5.987    1.000    1.000    1.000

SHEAR DESIGN
                             fv           FV      Stress
                         Stress    Allowable       Ratio
    Major Shear           0.418       14.000       0.030
    Minor Shear           0.448       14.000       0.032
```

16

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B390  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.697 = 0.003 + 2.252 + 0.442

STRESS CHECK FORCES & MOMENTS
                              P          M33         M22          V2          V3
    Combo  DSTLS2         0.027      -14.191       1.981       0.592      -0.083

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                             fa           Fa          Ft
                         Stress    Allowable   Allowable
    Axial                 0.053        5.987      21.000

                             fb           Fb          Fe         Cm        K        L        Cb
                         Stress    Allowable   Allowable    Factor   Factor   Factor    Factor
    Major Bending        47.290       21.000      15.165     1.000    1.000    1.000     2.300
    Minor Bending         9.288       21.000       5.987     1.000    1.000    1.000

SHEAR DESIGN
                             fv           FV      Stress
                         Stress    Allowable       Ratio
    Major Shear           1.753       14.000       0.125
    Minor Shear           0.440       14.000       0.031
```

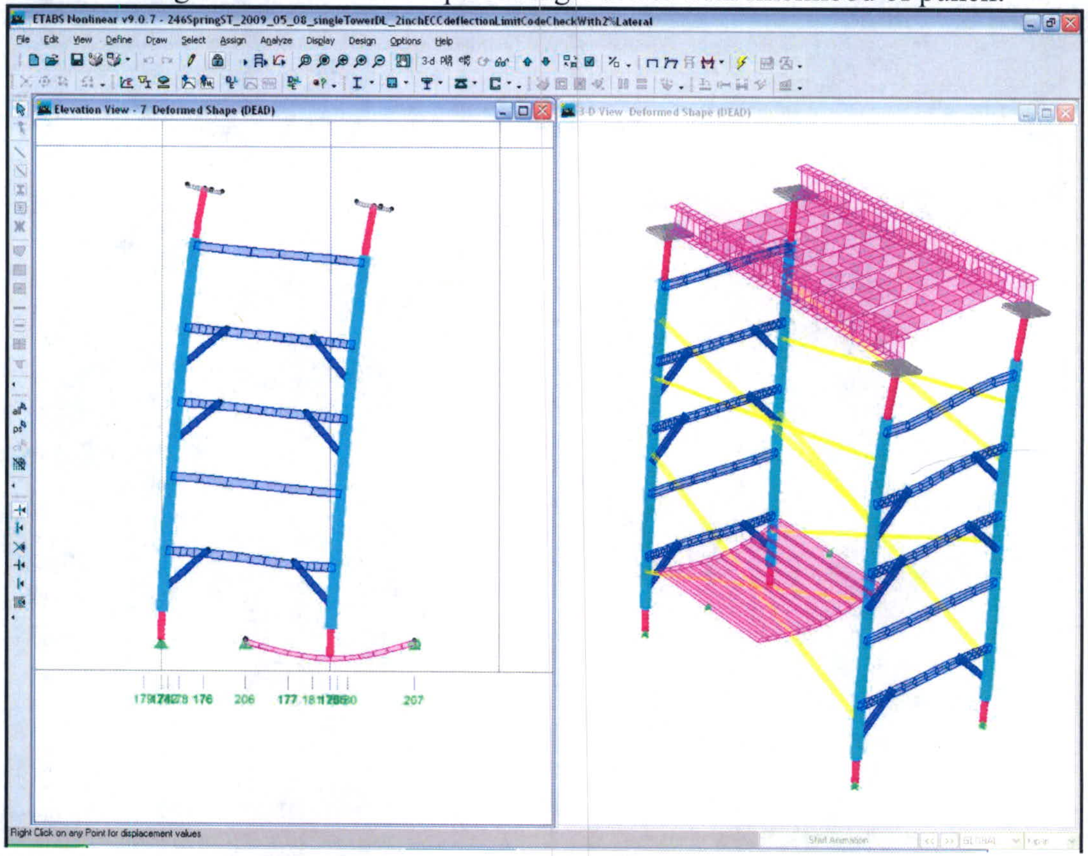
MEMBER CAPACITY:

$M_a = 6.4$ k-in $< M_{33} = 14.2$

3.3 Single Tower - Ultimate Condition.

The various computer models consider dead weight assumed existing at the time of the collapse (no construction live load or horizontal load.) only. The vertical load on each leg is in the order of 7000 lbs as described above in section 14.2.1. The overall frame is made of 5'-3" bottom frame and 3'-6" top frame, and the frame is 8'-0" apart. Four loading conditions have been considered and studied for the effect ".8" limited deflection on leg support" and "flexible leg support" to all members in the tower. The calculations use ultimate strength of materials.

Below is the general deflected shape of single tower with likelihood of punch:



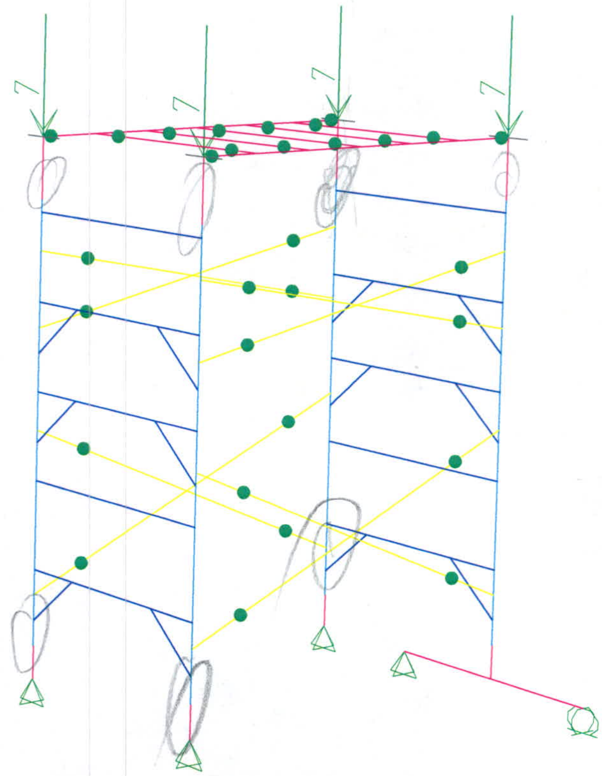
3.3.1 Ultimate Condition 1 - Load Case A, And The Bottom Shoring Leg Support Is Flexible.

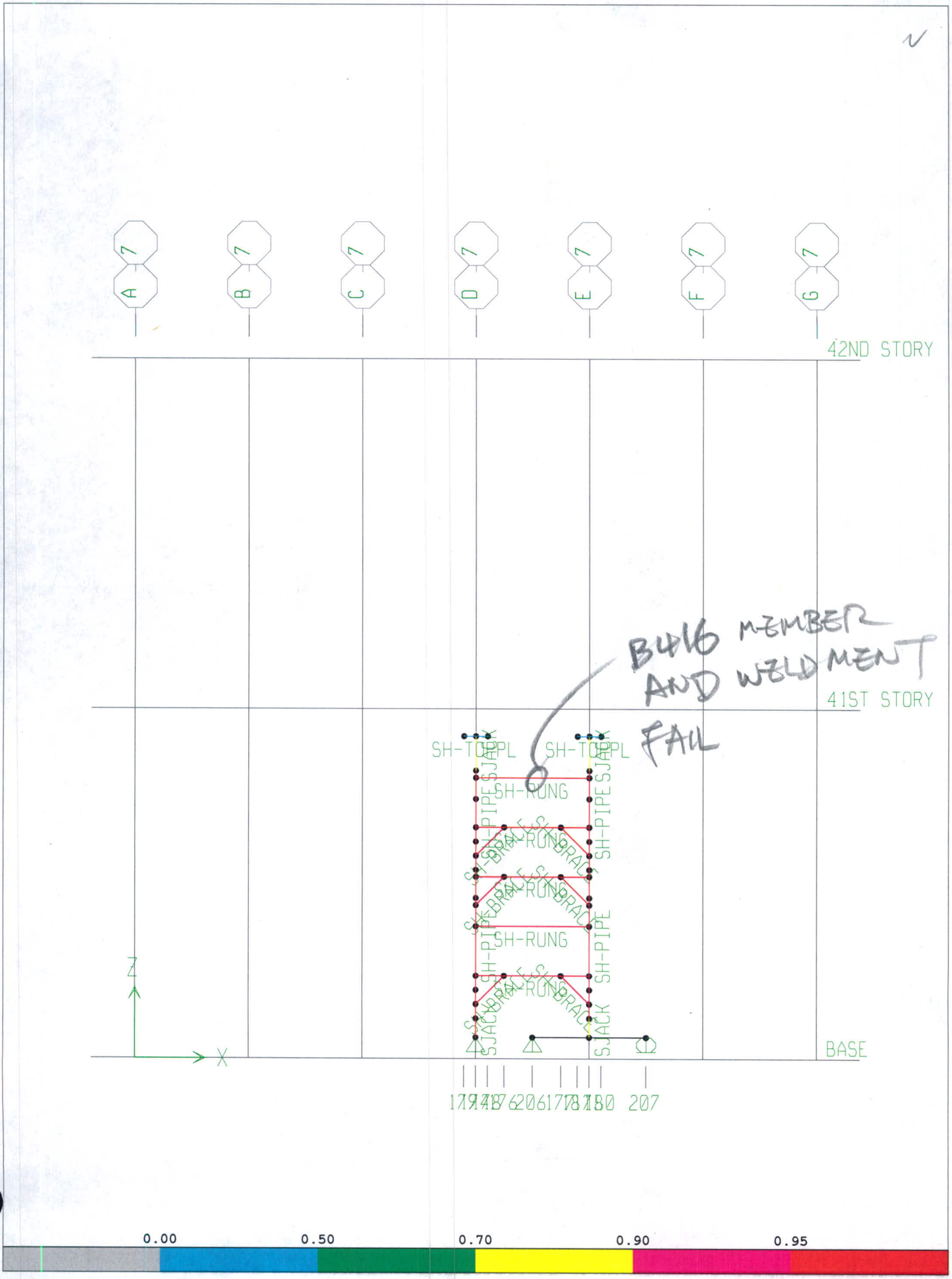
- The model file is "246SpringST_2009_05_08_singleTowerDL_NoECC".

Result: shore failure

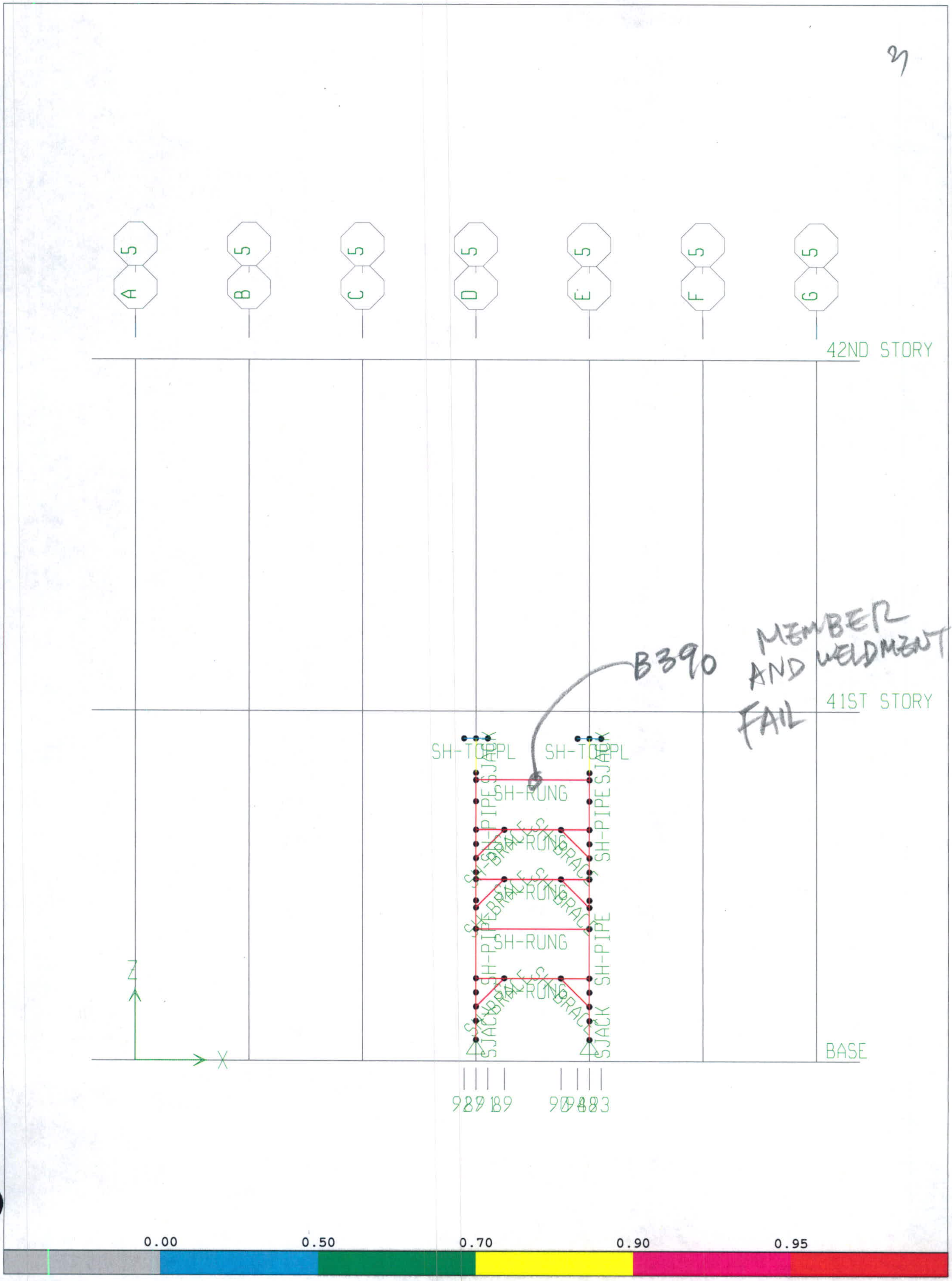
- Screw jacks do not fail.
- Shoring leg across from the flexible support leg may or may not fail in combination of axial compression and bending. Other 3 legs do not fail in buckle or bending.

- Horizontal members fail in bending, and connections from horizontal members to shoring leg fail in weldment.
- Knee brace does not fail .
- The downward deflection at the flexible support is 1.93".




A 7
B 7
C 7
D 7
E 7
F 7
G 7
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-TOPPL
SJACK
SH-PIPE
SH-RUNG
SH-BRACE
Z
X
207
B416 MEMBER AND WELDMENT FAIL
0.00
0.50
0.70
0.90
0.95

3


A 5
B 5
C 5
D 5
E 5
F 5
G 5
42ND STORY
41ST STORY
BASE
B390 MEMBER AND WELDMENT FAIL
SH-TOPPL
SH-TOPPL
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-PIPE
SH-BRACE
SJACK
Z
X
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: 41ST STORY  Element: B416  Station Loc: 0.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.774 = 0.006 + 1.850 + 0.919

STRESS CHECK FORCES & MOMENTS
                          P          M33          M22          V2          V3
    Combo  DSTLS2     0.059      -11.658       -4.115      -0.485      -0.171

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                         fa           Fa           Ft
                     Stress    Allowable    Allowable
    Axial             0.117        5.987       21.000

                         fb           Fb           Fe        Cm        K        L       Cb
                     Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending    38.848       21.000       15.165     1.000    1.000    1.000    2.300
    Minor Bending    19.297       21.000        5.987     1.000    1.000    1.000

SHEAR DESIGN
                         fv           FV       Stress
                     Stress    Allowable        Ratio
    Major Shear       1.438       14.000        0.103
    Minor Shear       0.914       14.000        0.065
```

MEMBER FAILURE STRENGTH

Multimate = 10.5 k-in < M_{33} (FAIL)

WELD FAILURE STRENGTH

Multmate = 3.6 k-in < M_{33} (FAIL)

ETABS Steel Design

Engineer__________
Project__________
Subject__________

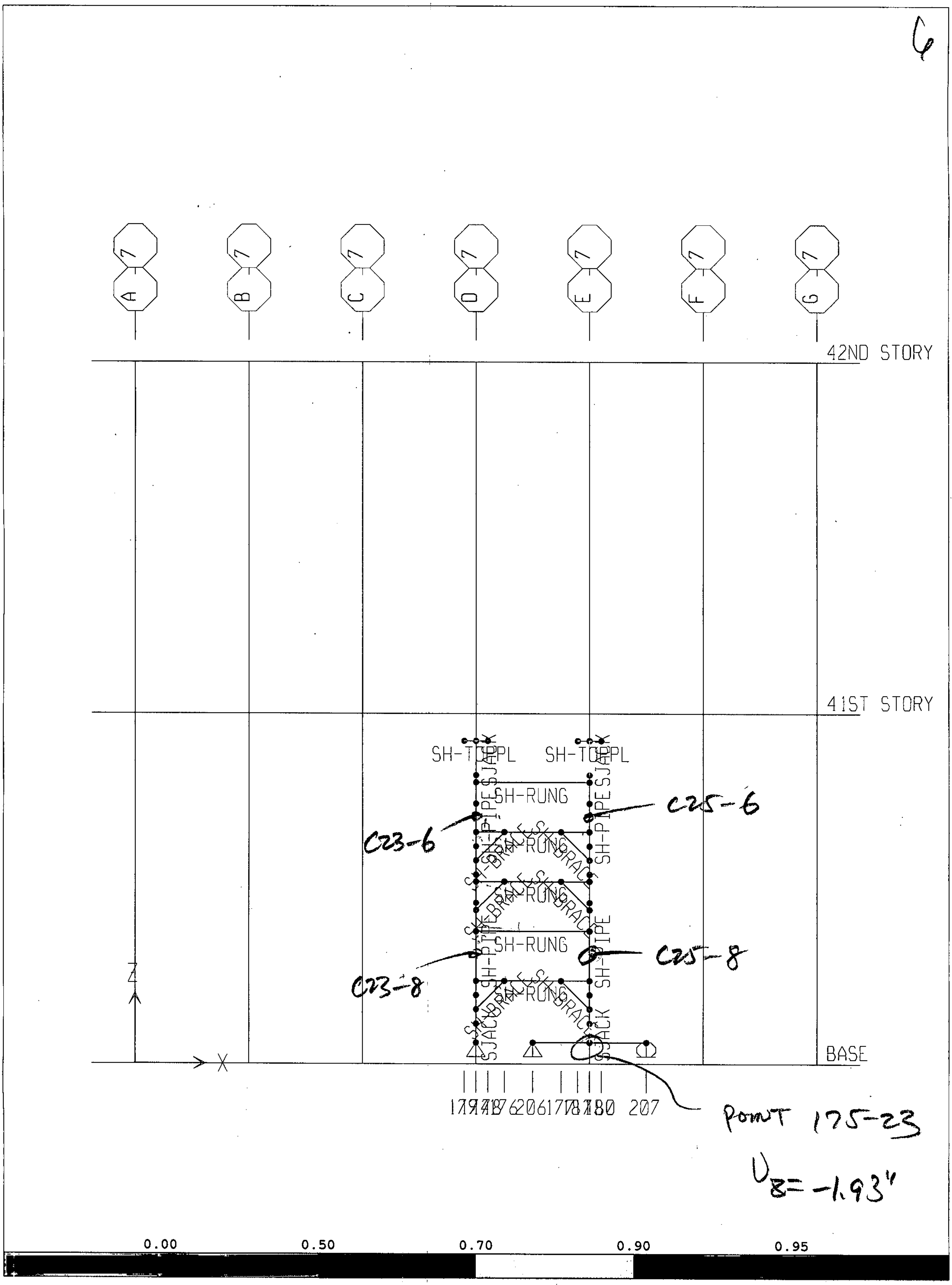
```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B390  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.757 = 0.005 + 1.849 + 0.903

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22         V2          V3
    Combo  DSTLS2       0.057      -11.651       4.045       0.485      -0.169

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                           fa           Fa          Ft
                       Stress    Allowable   Allowable
    Axial               0.114        5.987      21.000

                           fb           Fb          Fe        Cm        K        L       Cb
                       Stress    Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending      38.825       21.000      15.165     1.000    1.000    1.000    2.300
    Minor Bending      18.967       21.000       5.987     1.000    1.000    1.000

SHEAR DESIGN
                           fv           FV      Stress
                       Stress    Allowable       Ratio
    Major Shear         1.437       14.000       0.103
    Minor Shear         0.899       14.000       0.064
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-7  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.754 = 0.168 + 0.586 + 0.006

STRESS CHECK FORCES & MOMENTS
                              P         M33         M22         V2          V3
    Combo  DSTLS2        -7.075     -10.832       0.108      -0.594       0.008

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 5.532      32.025      33.000

                             fb          Fb          Fe        Cm        K        L       Cb
                         Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending        21.269      36.300    1213.622     0.850    1.000    0.483    1.217
    Minor Bending         0.211      36.300    1213.622     0.850    1.000    0.483

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           0.784      22.000       0.036
    Minor Shear           0.011      22.000       0.001
```

6


A-7
B-7
C-7
D-7
E-7
F-7
G-7
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-TOPPL
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-PIPE
SH-PIPE
SJACK
SJACK
Z
X
207
C23-6
C25-6
C23-8
C25-8
POINT 175-23
U_z = -1.93"
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: 41ST STORY  Element: C25-8  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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  1.279 = 0.144 + 0.114 + 1.130

STRESS CHECK FORCES & MOMENTS
                          P           M33          M22          V2           V3
    Combo  DSTLS2      -4.064        2.405       23.786        1.755        1.102

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                         fa           Fa           Ft
                     Stress    Allowable    Allowable
    Axial             3.026       16.510       21.000

                         fb           Fb           Fe       Cm        K        L       Cb
                     Stress    Allowable    Allowable   Factor   Factor   Factor   Factor
    Major Bending     2.638       23.100      120.071    0.850    1.000    0.333    1.000
    Minor Bending    26.093       23.100       34.813    0.850    1.000    0.619

SHEAR DESIGN
                         fv           FV       Stress
                     Stress    Allowable        Ratio
    Major Shear       2.419       14.000        0.173
    Minor Shear       1.519       14.000        0.108
```

4

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C23-8  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   1.772 = 0.498 + 0.090 + 1.271

STRESS CHECK FORCES & MOMENTS
                           P          M33          M22           V2           V3
    Combo  DSTLS2    -11.053        2.077      -24.036        1.566        1.289

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1)
                          fa           Fa           Ft
                      Stress    Allowable    Allowable
    Axial              8.230       16.510       21.000

                          fb           Fb           Fe        Cm        K        L       Cb
                      Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending      2.278       23.100      120.071     0.850    1.000    0.333    1.000
    Minor Bending     26.368       23.100       34.813     0.850    1.000    0.619

SHEAR DESIGN
                          fv           FV       Stress
                      Stress    Allowable        Ratio
    Major Shear        2.160       14.000        0.154
    Minor Shear        1.777       14.000        0.127
```

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-6  Station Loc: 18.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.051 = 0.227 + 0.706 + 0.424

STRESS CHECK FORCES & MOMENTS
                             P         M33         M22          V2          V3
    Combo  DSTLS2       -6.399     -14.873       8.929      -0.659       0.907

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                            fa          Fa          Ft
                        Stress   Allowable   Allowable
    Axial                4.764      19.050      21.000

                            fb          Fb          Fe        Cm       K       L      Cb
                        Stress   Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending       16.316      23.100     120.071     0.850   1.000   0.500   1.000
    Minor Bending        9.795      23.100     163.430     0.850   1.000   0.429

SHEAR DESIGN
                            fv          FV      Stress
                        Stress   Allowable       Ratio
    Major Shear          0.909      14.000       0.065
    Minor Shear          1.251      14.000       0.089
```

10

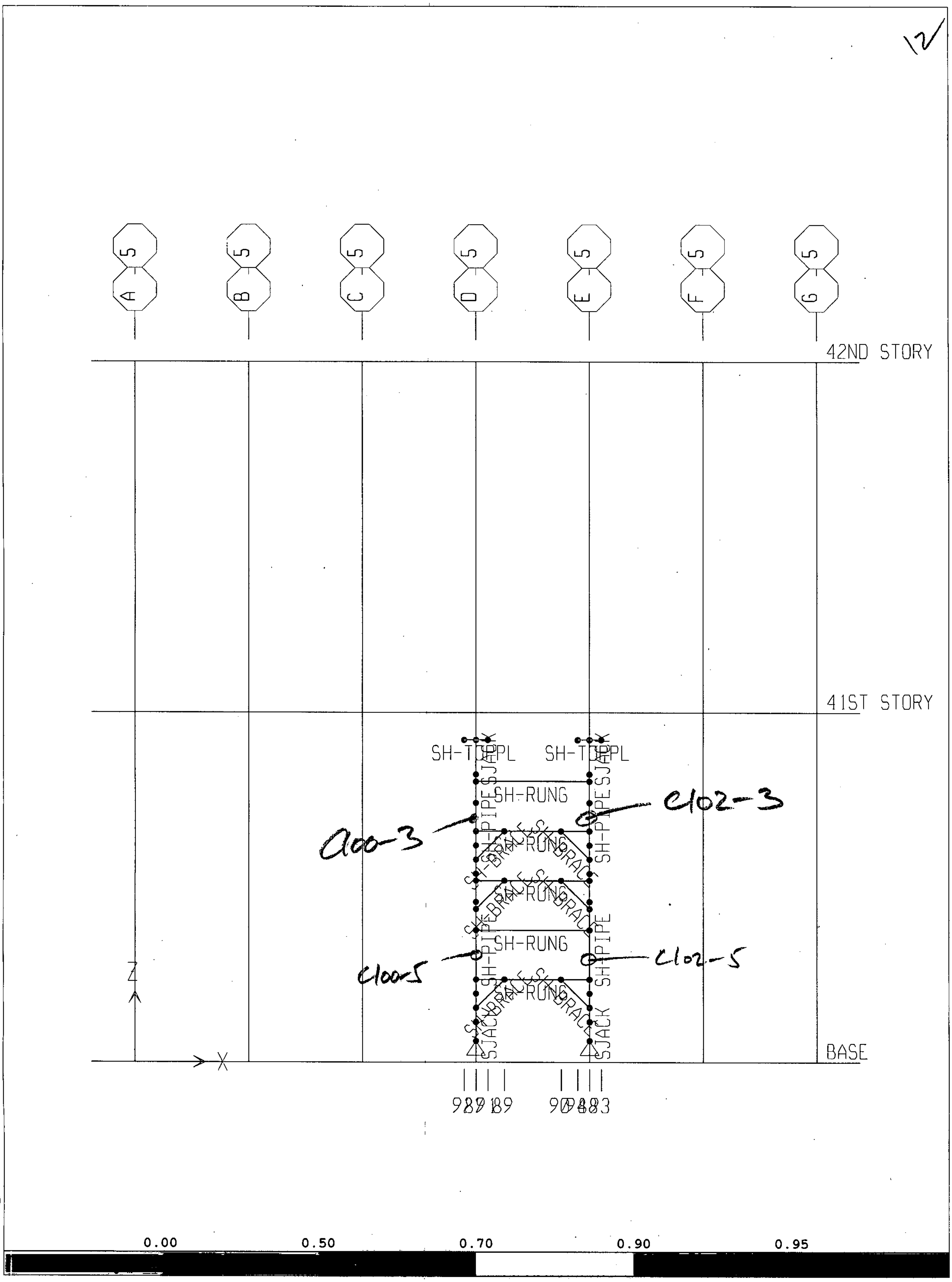
ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C23-6  Station Loc: 18.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.086 = 0.276 + 0.691 + 0.422

STRESS CHECK FORCES & MOMENTS
                              P        M33        M22         V2         V3
    Combo  DSTLS2        -7.773    -14.560     -8.890     -0.600     -0.923

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                             fa         Fa         Ft
                         Stress  Allowable  Allowable
    Axial                 5.788     19.050     21.000

                             fb         Fb         Fe       Cm        K        L       Cb
                         Stress  Allowable  Allowable   Factor   Factor   Factor   Factor
    Major Bending        15.973     23.100    120.071    0.850    1.000    0.500    1.000
    Minor Bending         9.752     23.100    163.430    0.850    1.000    0.429

SHEAR DESIGN
                             fv         FV     Stress
                         Stress  Allowable      Ratio
    Major Shear           0.827     14.000      0.059
    Minor Shear           1.273     14.000      0.091
```

POINT DISPLACEMENTS

STORY	POINT	LOAD	UX	UY	UZ	RX	RY	RZ
41ST STORY	175-23	DEAD	-0.0003	-0.0022	-1.9307	-0.00097	0.01481	0.00000


12
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-RUNG
C100-3
C102-3
C100-5
C102-5
0.00
0.50
0.70
0.90
0.95

19

ETABS Steel Design

Engineer________
Project________
Subject________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-5  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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  1.779 = 0.503 + 0.038 + 1.276

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2   -11.156       -0.874      24.061      -1.468      -1.290

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             8.307      16.510      21.000

                         fb          Fb          Fe        Cm         K         L        Cb
                     Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending     0.959      23.100     120.071     0.850     1.000     0.333     1.000
    Minor Bending    26.394      23.100      34.813     0.850     1.000     0.619

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       2.024      14.000       0.145
    Minor Shear       1.778      14.000       0.127
```

12

ETABS Steel Design

Engineer ________
Project ________
Subject ________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C100-5  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   1.273 = 0.141 + 0.047 + 1.131

STRESS CHECK FORCES & MOMENTS
                              P         M33         M22           V2          V3
    Combo  DSTLS2        -3.989      -0.992     -23.810       -1.438      -1.101

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-2)
                             fa          Fa          Ft
                         Stress   Allowable   Allowable
    Axial                 2.970      16.510      21.000

                             fb          Fb          Fe        Cm        K        L       Cb
                         Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending         1.089      23.100     120.071     0.850    1.000    0.333    1.000
    Minor Bending        26.119      23.100      34.813     0.850    1.000    0.619

SHEAR DESIGN
                             fv          FV      Stress
                         Stress   Allowable       Ratio
    Major Shear           1.983      14.000       0.142
    Minor Shear           1.519      14.000       0.108
```

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-3  Station Loc: 18.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.087 = 0.276 + 0.692 + 0.423

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      -7.773      14.578       8.903       0.599       0.922

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-2)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             5.788      19.050      21.000

                         fb          Fb          Fe        Cm        K        L       Cb
                     Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending    15.992      23.100     120.071     0.850    1.000    0.500    1.000
    Minor Bending     9.766      23.100     163.430     0.850    1.000    0.429

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.826      14.000       0.059
    Minor Shear       1.271      14.000       0.091
```

16

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C100-3  Station Loc: 18.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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   1.051 = 0.227 + 0.706 + 0.425

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2    -6.399      14.874      -8.942       0.659      -0.906

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-2)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             4.764      19.050      21.000

                         fb          Fb          Fe        Cm       K       L      Cb
                     Stress   Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending    16.317      23.100     120.071     0.850   1.000   0.500   1.000
    Minor Bending     9.809      23.100     163.430     0.850   1.000   0.429

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.909      14.000       0.065
    Minor Shear       1.249      14.000       0.089
```

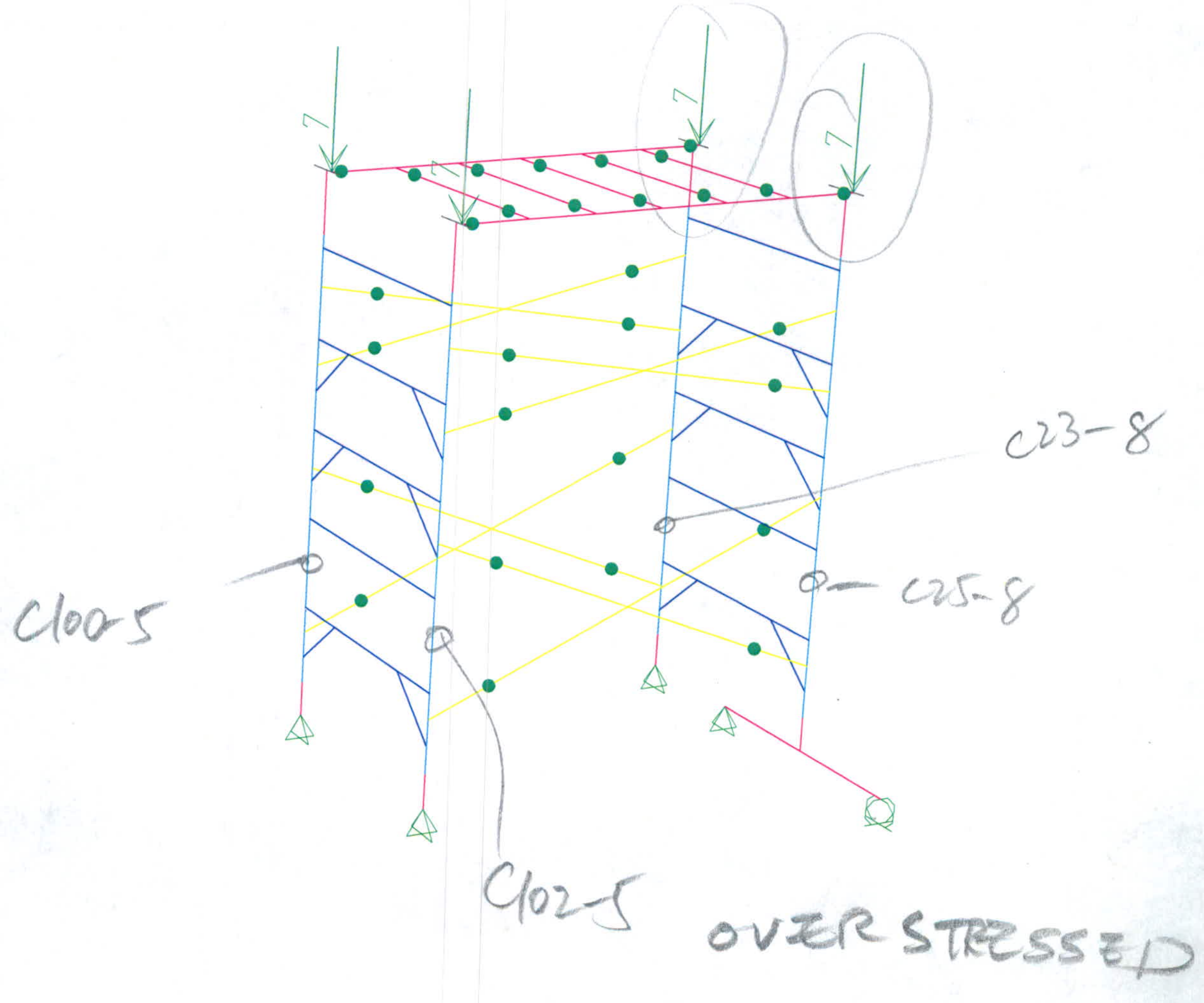
+ 157

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

- The model file is "246SpringST_2009_05_08_singleTowerDL_2inchECC".

Result: shore failure

- Screw jacks do not fail.
- Shoring leg across from the flexible support leg may or may not fail in combination of axial compression and bending. Other 3 legs do not fail.
- Horizontal members fail in bending, and connections from horizontal members to shoring leg fail in weldment.
- Knee brace does not fail.
- The downward deflection at the flexible support is 2.092".

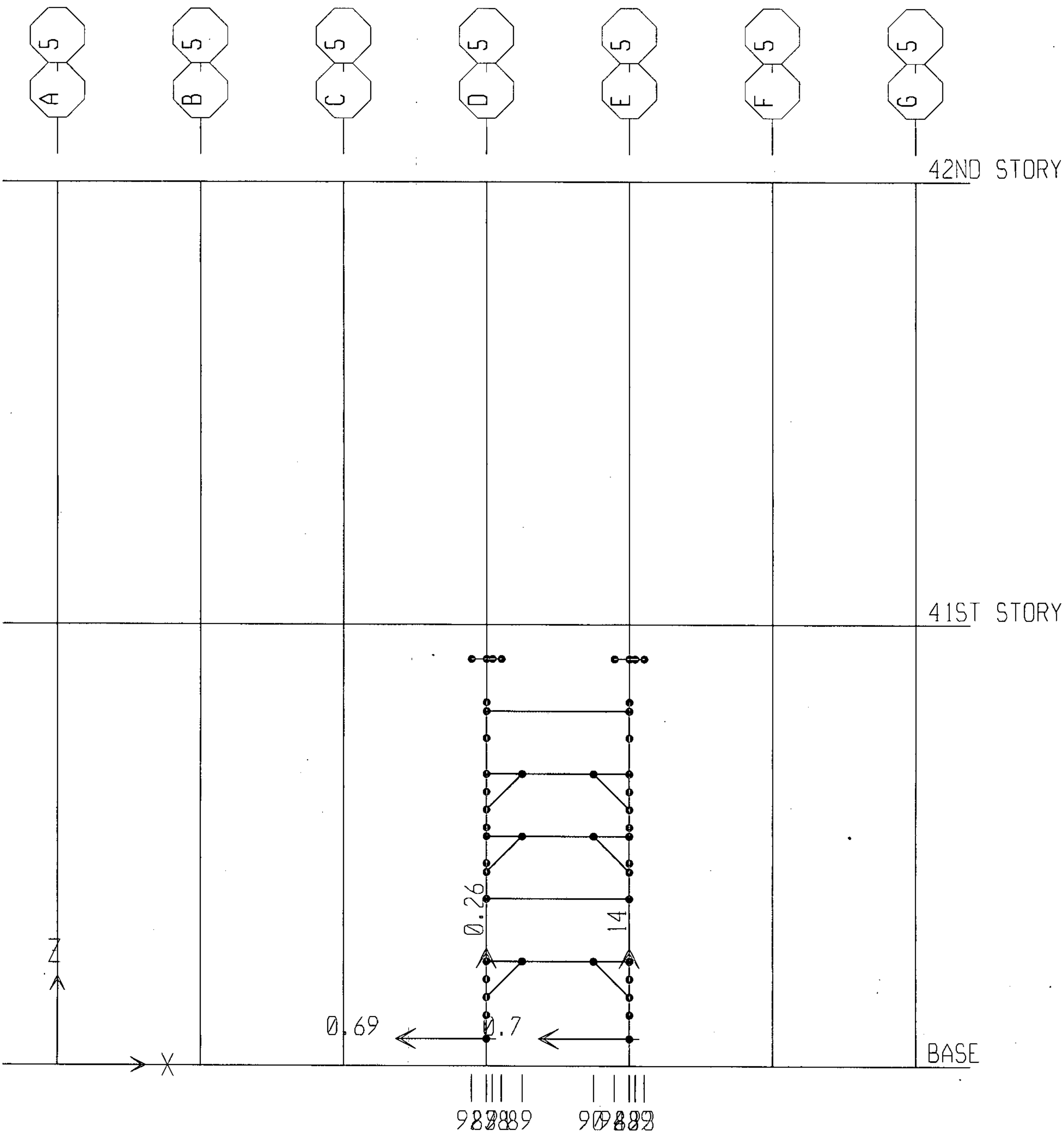

7
7
7
7
C23-8
C100-5
C25-8
C102-5
OVERSTRESSED

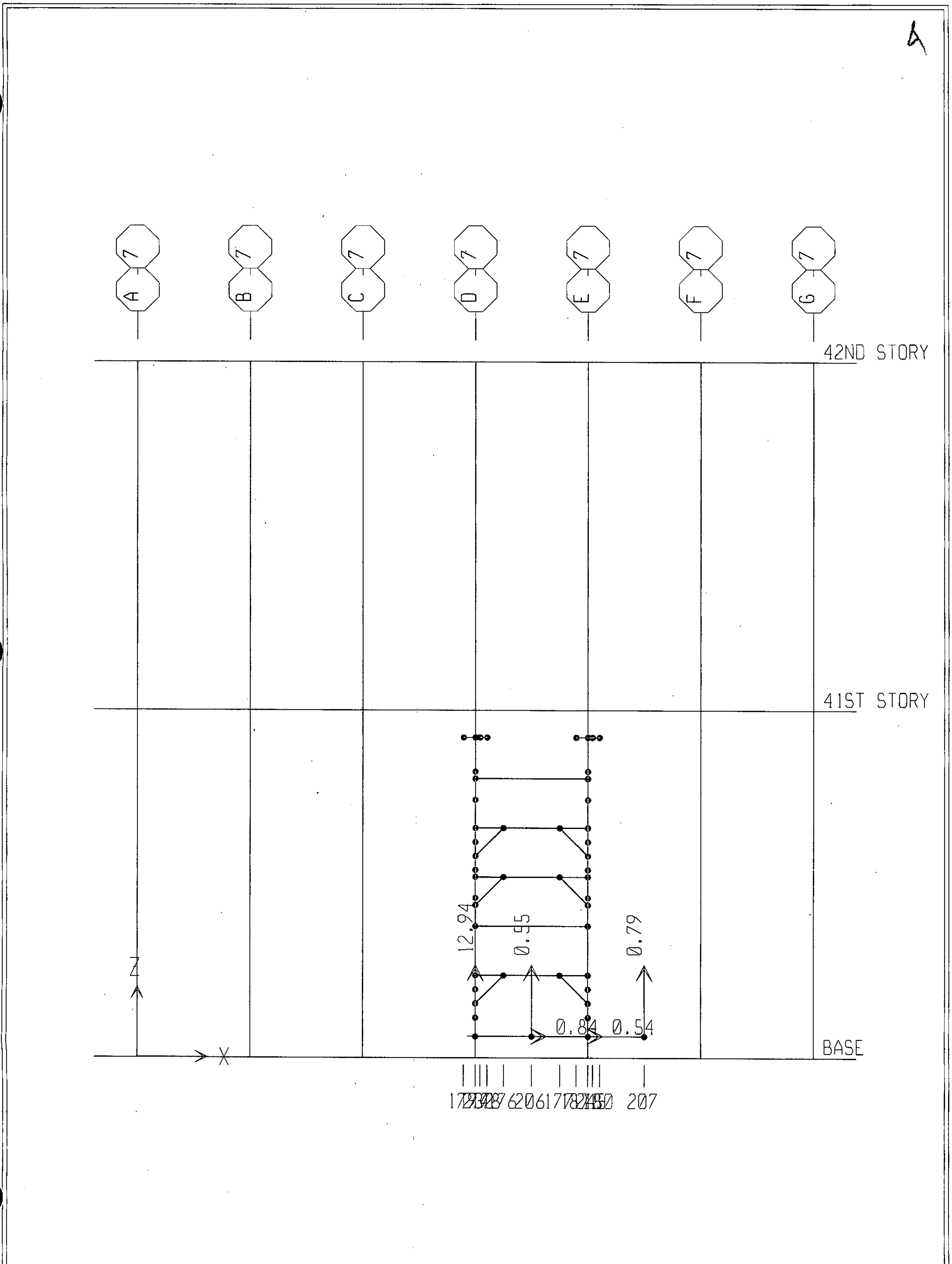
2✓

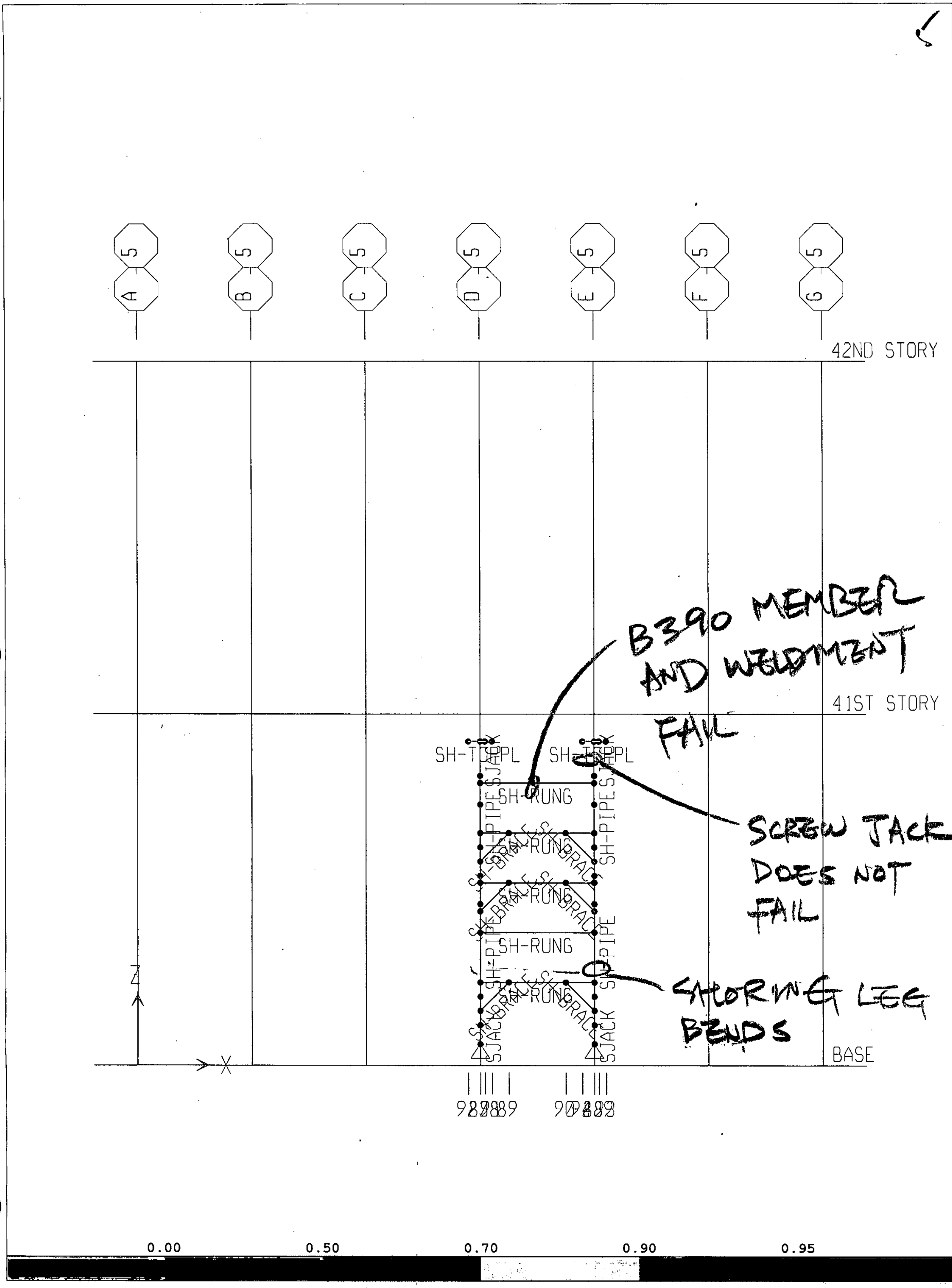
POINT DISPLACEMENTS

STORY	POINT	LOAD	UX	UY	UZ	RX	RY	RZ
41ST STORY	175-23	DEAD	-0.0003	-0.0024	-2.0914	-0.00105	0.01617	0.00000

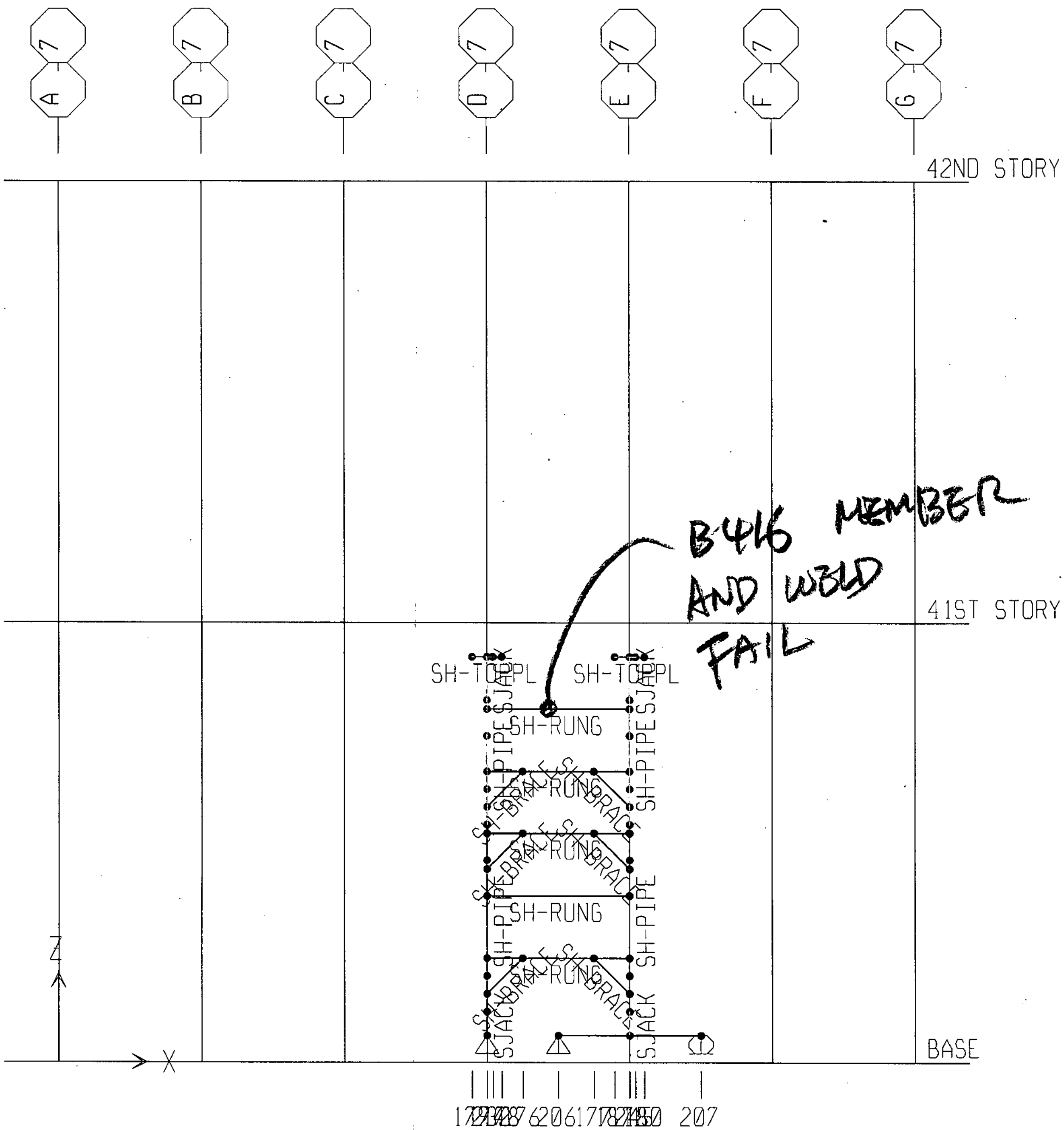
3




A-7
B-7
C-7
D-7
E-7
F-7
G-7
42ND STORY
41ST STORY
BASE
12.94
0.55
0.79
0.84
0.54
Z
X
207


42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-RUNG
SH-PIPE
SH-BRACE
SJACK
B390 MEMBER AND WELDMENT FAIL
SCREW JACK DOES NOT FAIL
SHORING LEG BENDS
0.00
0.50
0.70
0.90
0.95

6



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: 41ST STORY  Element: B390  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  3.999 = 0.006 + 3.015 + 0.978

STRESS CHECK FORCES & MOMENTS
                           P        M33        M22         V2        V3
    Combo  DSTLS2      0.062    -18.998      4.382      0.791    -0.183

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H2-1)
                          fa         Fa         Ft
                      Stress  Allowable  Allowable
    Axial              0.123      5.987     21.000

                          fb         Fb         Fe       Cm       K       L      Cb
                      Stress  Allowable  Allowable   Factor  Factor  Factor  Factor
    Major Bending     63.307     21.000     15.165    1.000   1.000   1.000   2.300
    Minor Bending     20.546     21.000      5.987    1.000   1.000   1.000

SHEAR DESIGN
                          fv         FV     Stress
                      Stress  Allowable      Ratio
    Major Shear        2.344     14.000      0.167
    Minor Shear        0.974     14.000      0.070
```

MEMBER FAILURE STRENGTH

Multimate = 10.4 k-in < M_{33} = 19 k-in

(FAIL)

WELD FAILURE STRENGTH

Multimate = 3.6 k-in < M_{33} = 19 k-in

(FAIL)

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK   Units: Kip-in   (Summary for Combo and Station)
Level: 41ST STORY   Element: B416   Station Loc: 0.000   Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.984 = 0.006 + 0.982 + 0.995

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2     0.064      -6.190      -4.458      -0.257      -0.186

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.127       5.987      21.000

                         fb          Fb          Fe      Cm       K       L      Cb
                     Stress   Allowable   Allowable  Factor  Factor  Factor  Factor
    Major Bending    20.626      21.000      15.165   1.000   1.000   1.000   2.300
    Minor Bending    20.902      21.000       5.987   1.000   1.000   1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.763      14.000       0.054
    Minor Shear       0.991      14.000       0.071
```

MEMBER FAILURE STRENGTH

Multimate = 104 k-in (FAIL IN BIAXIAL BENDING)

WELD FAILURE STRENGTH

Multimate = 3.6 k-in < M_{33} = 6.19 k-in (FAIL)

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C92-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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  1.617 = 0.168 + 1.450 + 0.007

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      -7.075       26.801      -0.132       0.702      -0.007

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                         fa           Fa           Ft
                     Stress    Allowable    Allowable
    Axial             5.532       30.637       33.000

                         fb           Fb           Fe        Cm        K        L        Cb
                     Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending    52.623       36.300      282.842     0.850    1.000    1.000    1.214
    Minor Bending     0.259       36.300      282.842     0.850    1.000    1.000

SHEAR DESIGN
                         fv           FV       Stress
                     Stress    Allowable        Ratio
    Major Shear       0.927       22.000        0.042
    Minor Shear       0.010       22.000        0.000
```

10

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-8  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   1.386 = 0.156 + 0.129 + 1.224

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2    -4.389        2.710      25.772       1.881       1.193

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                         fa           Fa           Ft
                     Stress    Allowable    Allowable
    Axial             3.268       16.510       21.000

                         fb           Fb           Fe        Cm        K        L       Cb
                     Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending     2.973       23.100      120.071     0.850    1.000    0.333    1.000
    Minor Bending    28.272       23.100       34.813     0.850    1.000    0.619

SHEAR DESIGN
                         fv           FV       Stress
                     Stress    Allowable        Ratio
    Major Shear       2.594       14.000        0.185
    Minor Shear       1.645       14.000        0.118
```

ETABS Steel Design

Engineer______
Project______
Subject______

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C23-8  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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  1.857 = 0.487 + 0.102 + 1.366

STRESS CHECK FORCES & MOMENTS
                            P          M33          M22          V2          V3
    Combo  DSTLS2     -10.806        2.362      -26.016       1.673       1.395

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1)
                           fa           Fa           Ft
                       Stress    Allowable    Allowable
    Axial               8.046       16.510       21.000

                           fb           Fb           Fe        Cm        K        L       Cb
                       Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending       2.592       23.100      120.071     0.850    1.000    0.333    1.000
    Minor Bending      28.539       23.100       34.813     0.850    1.000    0.619

SHEAR DESIGN
                           fv           Fv       Stress
                       Stress    Allowable        Ratio
    Major Shear         2.307       14.000        0.165
    Minor Shear         1.923       14.000        0.137
```

ETABS Steel Design

Engineer____________________
Project____________________
Subject____________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-5  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   1.962 = 0.544 + 0.037 + 1.418

STRESS CHECK FORCES & MOMENTS
                        P          M33         M22          V2          V3
    Combo  DSTLS2    -12.062     -0.849      26.057      -1.605      -1.397

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             8.981      16.510      21.000

                         fb          Fb          Fe        Cm        K        L       Cb
                     Stress   Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending     0.931      23.100     120.071     0.850    1.000    0.333    1.000
    Minor Bending    28.585      23.100      34.813     0.850    1.000    0.619

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       2.213      14.000       0.158
    Minor Shear       1.926      14.000       0.138
```

BENDING

19

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C100-5  Station Loc: 12.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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   1.369 = 0.143 + 0.046 + 1.226

STRESS CHECK FORCES & MOMENTS
                         P         M33        M22          V2          V3
    Combo  DSTLS2     -3.164      -0.967    -25.814      -1.575      -1.194

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                        fa          Fa          Ft
                    Stress   Allowable   Allowable
    Axial            2.356      16.510      21.000

                        fb          Fb          Fe       Cm       K       L      Cb
                    Stress   Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending    1.061      23.100     120.071    0.850   1.000   0.333   1.000
    Minor Bending   28.317      23.100      34.813    0.850   1.000   0.619

SHEAR DESIGN
                        fv          FV      Stress
                    Stress   Allowable       Ratio
    Major Shear      2.172      14.000       0.155
    Minor Shear      1.646      14.000       0.118
```

L.E

.7

.8 hr

.7 x .8 = .56

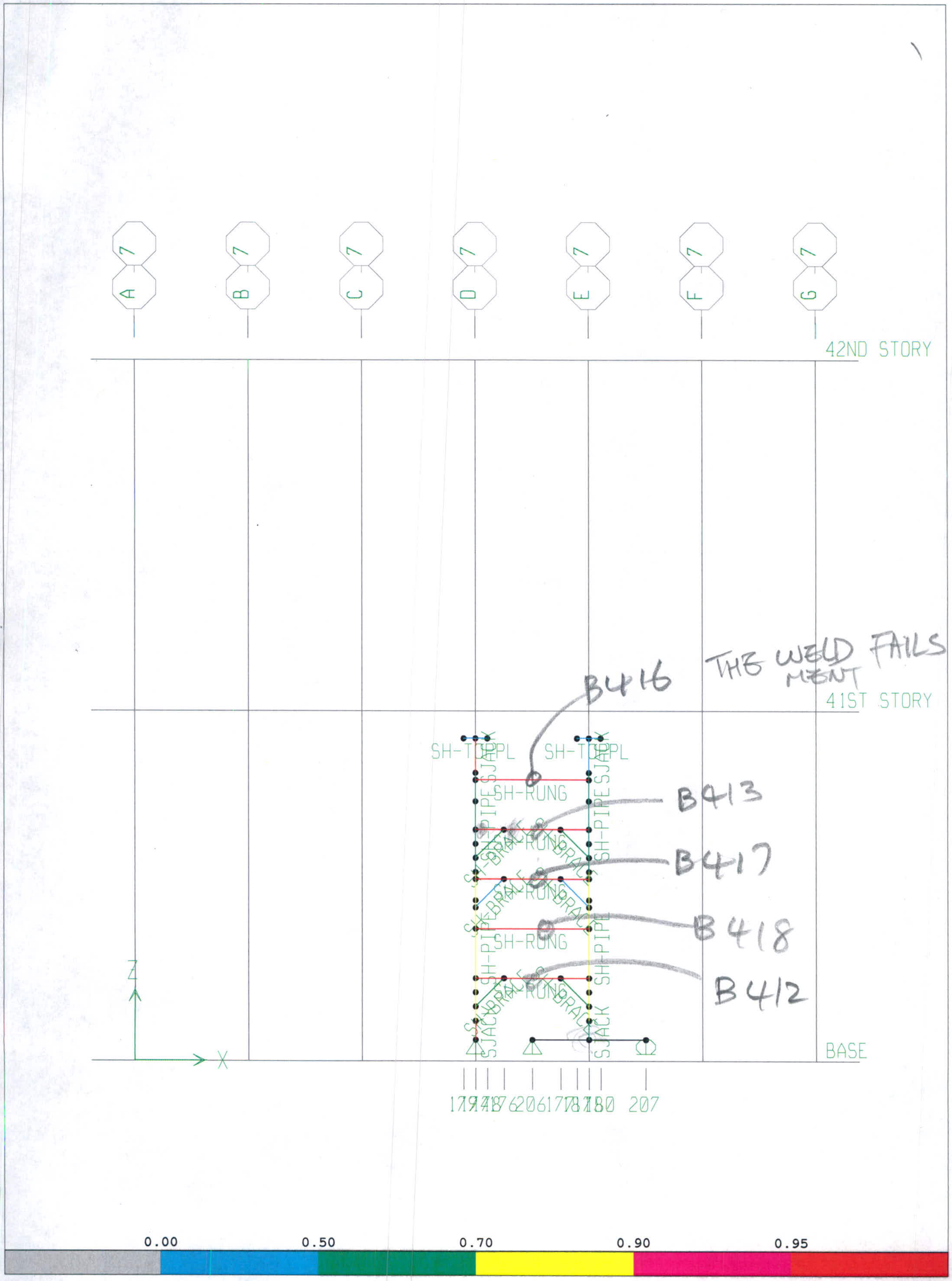
+10885

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

- The model file is “246SpringST_2009_05_08_singleTowerDL_NoECCdeflectionLimit”.

Result: shore failure

- Screw jacks do not fail.
- All 4 Shoring legs do not fail in buckle or bending.
- The connections from the top horizontal members to the leg fail in weldment. The bottom horizontal members fail in bending, and the connections from the bottom horizontal members to the leg fail in weldment. The other horizontal members and connections are still ok.
- Knee brace does not fail in compression, tension or bending.


A-7
B-7
C-7
D-7
E-7
F-7
G-7
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-TOPPL
SH-RUNG
SH-RUNG
SH-RUNG
SH-RUNG
SH-PIPE
SH-PIPE
SJACK
BRACE
B416
B413
B417
B418
B412
THE WELD FAILS
MENT
Z
X
207
0.00
0.50
0.70
0.90
0.95

2

ETABS Steel Design

Engineer___________________
Project___________________
Subject___________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B416  Station Loc: 0.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.129 = 0.002 + 0.746 + 0.381

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2     0.025       -4.702      -1.706      -0.196      -0.071

AXIAL FORCE & BIAXIAL MOMENT DESIGN     (H2-1)
                         fa           Fa          Ft
                     Stress    Allowable   Allowable
    Axial             0.049        5.987      21.000

                         fb           Fb          Fe       Cm       K       L      Cb
                     Stress    Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending    15.669       21.000      15.165    1.000   1.000   1.000   2.300
    Minor Bending     7.997       21.000       5.987    1.000   1.000   1.000

SHEAR DESIGN
                         fv           FV      Stress
                     Stress    Allowable       Ratio
    Major Shear       0.582       14.000       0.042
    Minor Shear       0.379       14.000       0.027
```

MEMBER FAILURE STRENGTH

Multimate = 10.4 k-in > M_{33} = 4.7 k-in (OK)

WELD FAILURE STRENGTH

Multimate = 3.6 k-in < M_{33} = 4.7 k-in (FAIL)

7

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B413  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.525 = 0.006 + 0.754 + 0.766

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2     -0.017      -4.752      -3.429      -0.392      -0.286

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-3)
                         fa           Fa          Ft
                     Stress    Allowable   Allowable
    Axial             0.034        5.987      21.000

                         fb           Fb          Fe        Cm         K         L        Cb
                     Stress    Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending    15.837       21.000      60.662     0.850     1.000     0.500     1.000
    Minor Bending    16.079       21.000       5.987     0.850     1.000     1.000

SHEAR DESIGN
                         fv           FV      Stress
                     Stress    Allowable       Ratio
    Major Shear       1.163       14.000       0.083
    Minor Shear       1.524       14.000       0.109
```

4

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B417  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.195 = 0.003 + 0.636 + 0.557

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      -0.009      -4.006      -2.494      -0.330      -0.208

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.017       5.987      21.000

                         fb          Fb          Fe        Cm       K       L      Cb
                     Stress   Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending    13.349      21.000      60.662     0.850   1.000   0.500   1.000
    Minor Bending    11.694      21.000       5.987     0.850   1.000   1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.977      14.000       0.070
    Minor Shear       1.108      14.000       0.079
```

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B418  Station Loc: 0.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  0.498 = 0.005 + 0.420 + 0.073

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2     0.058      -2.648      -0.326      -0.111      -0.014

AXIAL FORCE & BIAXIAL MOMENT DESIGN     (H2-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.115       5.987      21.000

                         fb          Fb          Fe       Cm        K        L       Cb
                     Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending     8.823      21.000      15.165    1.000    1.000    1.000    2.300
    Minor Bending     1.528      21.000       5.987    1.000    1.000    1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.330      14.000       0.024
    Minor Shear       0.072      14.000       0.005
```

OK

ETABS Steel Design

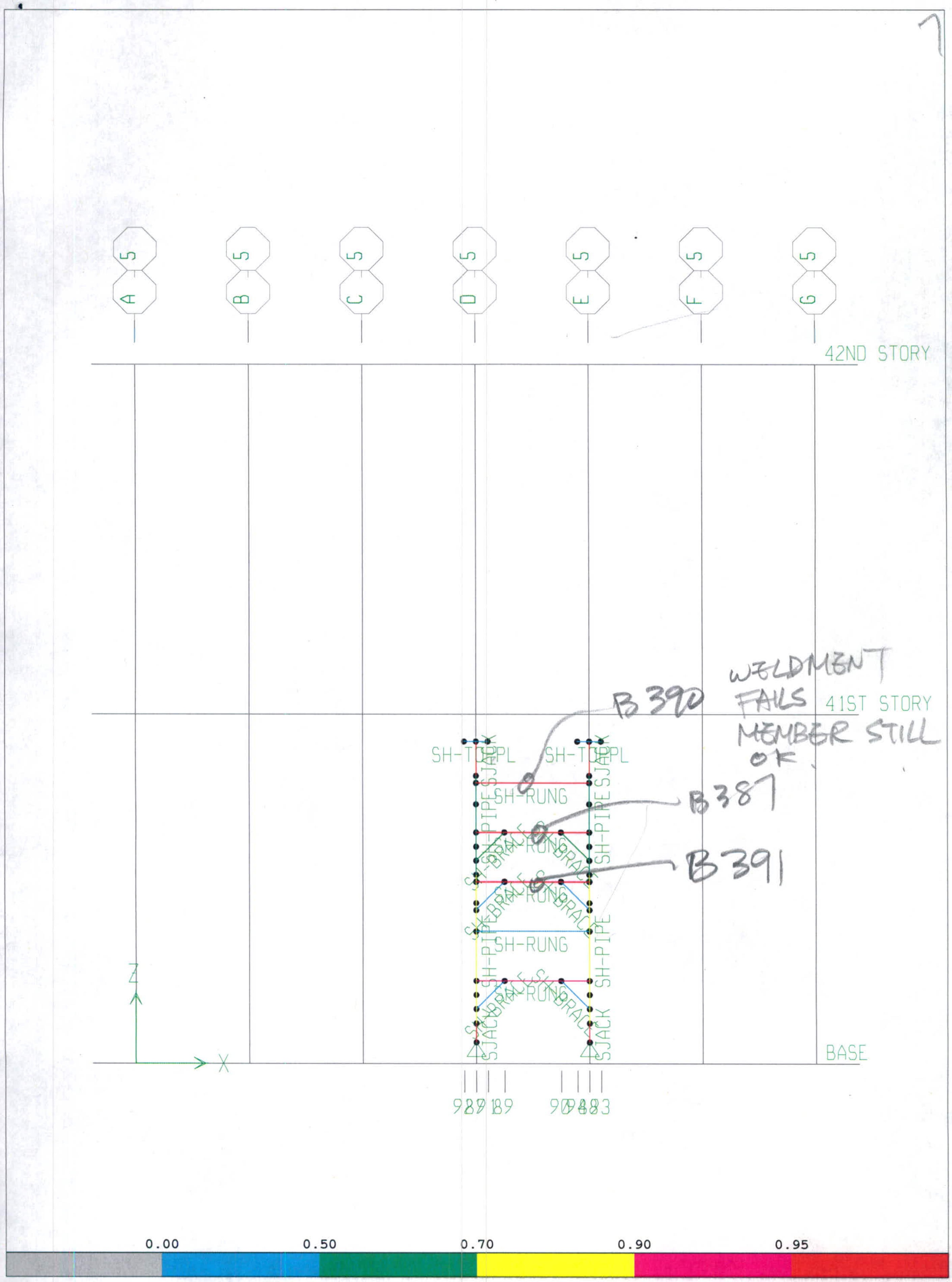
Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B412  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   1.245 = 0.069 + 0.977 + 0.199

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2      -0.208      -6.156       0.890      -0.498       0.074

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.413       5.987      21.000

                         fb          Fb          Fe       Cm       K       L      Cb
                     Stress   Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending    20.514      21.000      60.662    0.850   1.000   0.500   1.000
    Minor Bending     4.174      21.000       5.987    0.850   1.000   1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       1.477      14.000       0.105
    Minor Shear       0.396      14.000       0.028
```

7


A5
B5
C5
D5
E5
F5
G5
42ND STORY
41ST STORY
BASE
SH-TOPPL
SH-RUNG
SH-PIPE
SJACK
BRACE
Z
X
B390
WELDMENT FAILS
MEMBER STILL OK
B387
B391
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: 41ST STORY Element: B390 Station Loc: 48.000 Section ID: SH-RUNG
Element Type: Moment Resisting Frame Classification: Compact

L=48.000
A=0.503 i22=0.133 i33=0.338
s22=0.213 s33=0.300 r22=0.515 r33=0.820
E=10100.000 fy=35.000
RLLF=1.000

WELDMENT FAILS

P-M33-M22 Demand/Capacity Ratio is   1.121 = 0.002 + 0.745 + 0.374

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS2	0.024	-4.692	1.676	0.196	-0.070

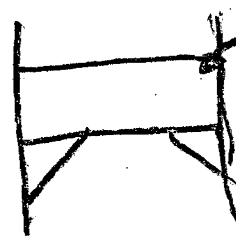
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H2-1)

	fa Stress	Fa Allowable	Ft Allowable
Axial	0.048	5.987	21.000

	fb Stress	Fb Allowable	Fe Allowable	Cm Factor	K Factor	L Factor	Cb Factor
Major Bending	15.635	21.000	15.165	1.000	1.000	1.000	2.300
Minor Bending	7.860	21.000	5.987	1.000	1.000	1.000	

SHEAR DESIGN

	fv Stress	FV Allowable	Stress Ratio
Major Shear	0.580	14.000	0.041
Minor Shear	0.373	14.000	0.027



WELD FAIL STRENGTH

$M_{ultimate} = 3.6$ k-in $< M_{33} = 4.7$ k-in
(FAIL)

a

ETABS Steel Design

Engineer

Project

Subject

AISC-ASD89 STEEL SECTION CHECK Units: Kip-in (Summary for Combo and Station)
Level: 41ST STORY Element: B387 Station Loc: 36.000 Section ID: SH-RUNG
Element Type: Moment Resisting Frame Classification: Compact

L=48.000
A=0.503 i22=0.133 i33=0.338
s22=0.213 s33=0.300 r22=0.515 r33=0.820
E=10100.000 fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is 1.519 = 0.006 + 0.748 + 0.766

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS2	-0.018	-4.711	3.428	0.389	-0.286

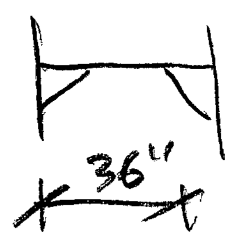
AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-3)

	fa Stress	Fa Allowable	Ft Allowable
Axial	0.035	5.987	21.000

	fb Stress	Fb Allowable	Fe Allowable	Cm Factor	K Factor	L Factor	Cb Factor
Major Bending	15.698	21.000	60.662	0.850	1.000	0.500	1.000
Minor Bending	16.076	21.000	5.987	0.850	1.000	1.000	

SHEAR DESIGN

	fv Stress	FV Allowable	Stress Ratio
Major Shear	1.152	14.000	0.082
Minor Shear	1.524	14.000	0.109



10

ETABS Steel Design

Engineer
Project
Subject

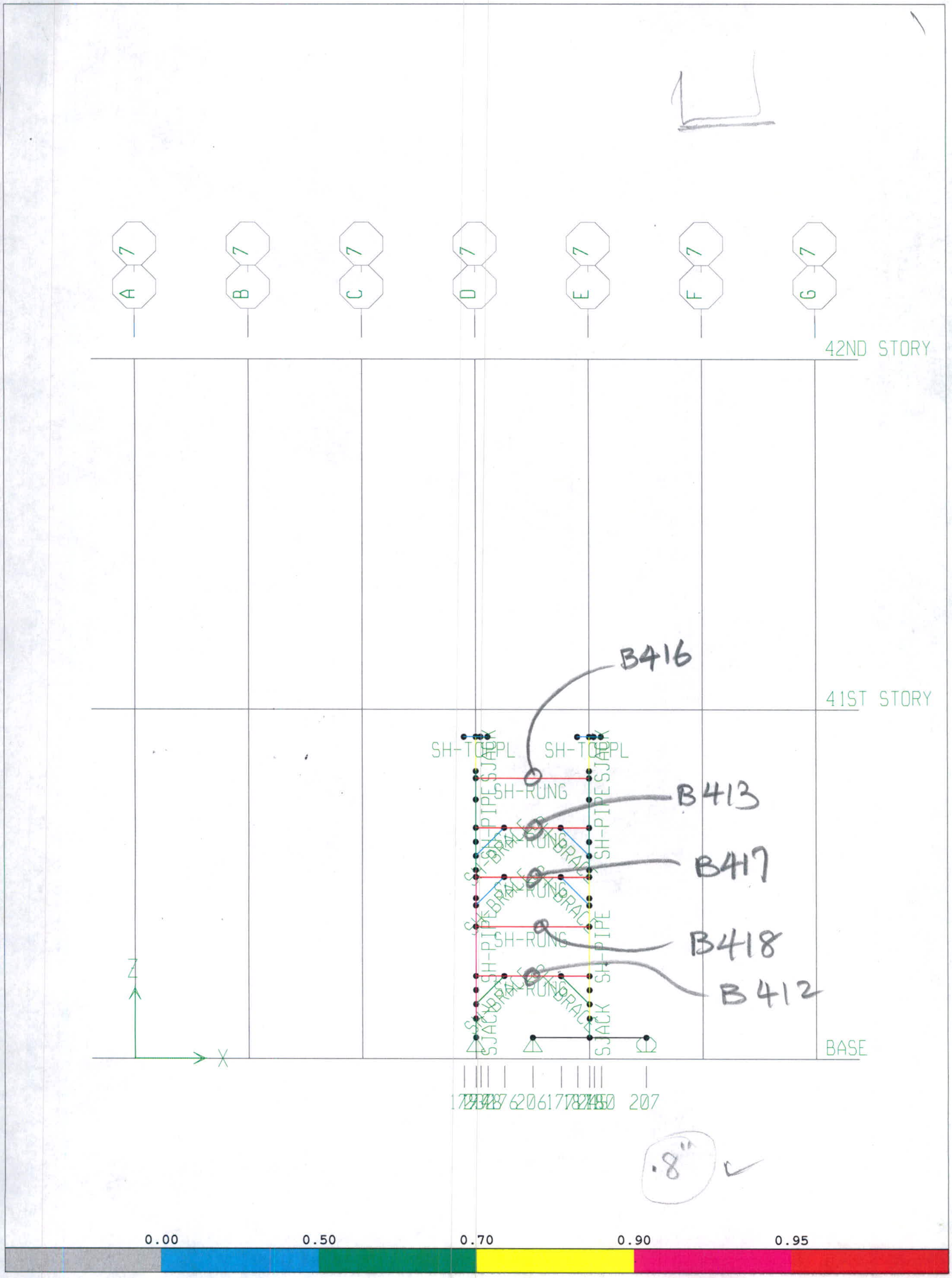
```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B391  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   1.171 = 0.001 + 0.613 + 0.557

STRESS CHECK FORCES & MOMENTS
                           P          M33         M22          V2          V3
    Combo  DSTLS2     -0.004       -3.862       2.492       0.318      -0.208

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                          fa           Fa          Ft
                      Stress    Allowable   Allowable
    Axial              0.007        5.987      21.000

                          fb           Fb          Fe        Cm         K         L        Cb
                      Stress    Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending     12.871       21.000      60.662     0.850     1.000     0.500     1.000
    Minor Bending     11.687       21.000       5.987     0.850     1.000     1.000

SHEAR DESIGN
                          fv           FV      Stress
                      Stress    Allowable       Ratio
    Major Shear        0.942       14.000       0.067
    Minor Shear        1.108       14.000       0.079
```

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

- The model file is "246SpringST_2009_05_08_singleTowerDL_2inchECCdeflectionLimit".

Result: shore failure

- Screw jacks do not fail.
- All 4 Shoring legs do not fail.
- The horizontal members fail in bending, connections from the top horizontal members to the leg fail in weldment.
- Knee brace does not fail



ETABS v9.0.7 - File: 246SpringST_2009_05_08_singleTowerDL_2inchECCdeflectionLimit - May 29,2009 19:12
Elevation View - 7 Steel Design Sections (AISC-ASD89) - Kip-in Units

2

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B416  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  0.633 = 0.003 + 0.218 + 0.412

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22         V2          V3
    Combo  DSTLS2       0.026       -1.374       1.846       0.058      -0.077

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                           fa          Fa          Ft
                       Stress   Allowable   Allowable
    Axial               0.053       5.987      21.000

                           fb          Fb          Fe       Cm        K        L       Cb
                       Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending       4.578      21.000      15.165    1.000    1.000    1.000    2.300
    Minor Bending       8.654      21.000       5.987    1.000    1.000    1.000

SHEAR DESIGN
                           fv          FV      Stress
                       Stress   Allowable       Ratio
    Major Shear         0.171      14.000       0.012
    Minor Shear         0.410      14.000       0.029
```

3

ETABS Steel Design

Engineer___
Project___
Subject___

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B413  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.194 = 0.006 + 0.359 + 0.829

STRESS CHECK FORCES & MOMENTS
                          P         M33         M22          V2          V3
    Combo  DSTLS2    -0.018      -2.262      -3.711      -0.185      -0.309

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.036       5.987      21.000

                         fb          Fb          Fe         Cm       K        L       Cb
                     Stress   Allowable   Allowable     Factor  Factor   Factor  Factor
    Major Bending     7.538      21.000      60.662      0.850   1.000    0.500   1.000
    Minor Bending    17.403      21.000       5.987      0.850   1.000    1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.548      14.000       0.039
    Minor Shear       1.650      14.000       0.118
```

2

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B417  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame  Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   1.170 = 0.003 + 0.564 + 0.603

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2     -0.009       -3.553      -2.702      -0.292      -0.225

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-3)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             0.018       5.987      21.000

                         fb          Fb          Fe       Cm       K       L       Cb
                     Stress   Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending    11.839      21.000      60.662    0.850   1.000   0.500   1.000
    Minor Bending    12.669      21.000       5.987    0.850   1.000   1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.865      14.000       0.062
    Minor Shear       1.201      14.000       0.086
```

ETABS Steel Design

Engineer______________________________
Project______________________________
Subject______________________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B418  Station Loc: 0.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   0.508 = 0.006 + 0.423 + 0.079

STRESS CHECK FORCES & MOMENTS
                             P           M33         M22          V2          V3
    Combo  DSTLS2        0.062        -2.668      -0.354      -0.112      -0.015

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H2-1)
                            fa          Fa          Ft
                        Stress   Allowable   Allowable
    Axial                0.124       5.987      21.000

                            fb          Fb          Fe       Cm        K        L       Cb
                        Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending        8.890      21.000      15.165    1.000    1.000    1.000    2.300
    Minor Bending        1.660      21.000       5.987    1.000    1.000    1.000

SHEAR DESIGN
                            fv          FV      Stress
                        Stress   Allowable       Ratio
    Major Shear          0.333      14.000       0.024
    Minor Shear          0.079      14.000       0.006
```

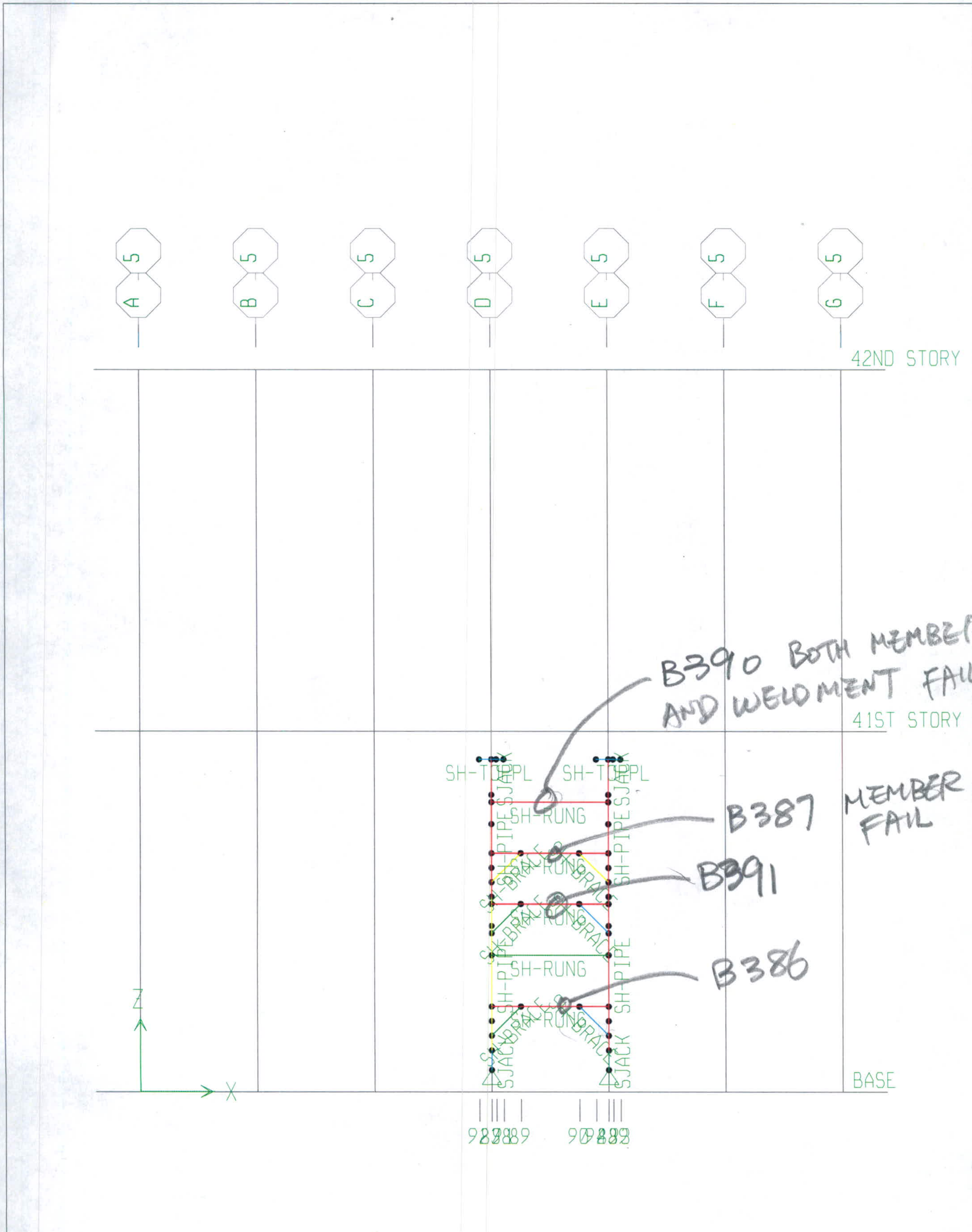
1075

6

ETABS Steel Design

Engineer________________
Project________________
Subject________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B412  Station Loc: 12.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.329 = 0.076 + 1.040 + 0.214

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22          V2          V3
    Combo  DSTLS2      -0.227       -6.552       0.960      -0.530       0.080

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-3)
                           fa          Fa          Ft
                       Stress   Allowable   Allowable
    Axial               0.452       5.987      21.000

                           fb          Fb          Fe        Cm         K         L        Cb
                       Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending      21.833      21.000      60.662     0.850     1.000     0.500     1.000
    Minor Bending       4.500      21.000       5.987     0.850     1.000     1.000

SHEAR DESIGN
                           fv          FV      Stress
                       Stress   Allowable       Ratio
    Major Shear         1.571      14.000       0.112
    Minor Shear         0.426      14.000       0.030
```



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: 41ST STORY  Element: B390  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.228 = 0.002 + 1.820 + 0.405

STRESS CHECK FORCES & MOMENTS
                            P        M33        M22         V2          V3
    Combo  DSTLS2       0.026    -11.470      1.814      0.478      -0.076

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H2-1)
                           fa          Fa          Ft
                       Stress   Allowable   Allowable
    Axial               0.051       5.987      21.000

                           fb          Fb          Fe        Cm        K         L        Cb
                       Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending      38.221      21.000      15.165     1.000     1.000     1.000     2.300
    Minor Bending       8.507      21.000       5.987     1.000     1.000     1.000

SHEAR DESIGN
                           fv          FV      Stress
                       Stress   Allowable       Ratio
    Major Shear         1.417      14.000       0.101
    Minor Shear         0.403      14.000       0.029
```

a

ETABS Steel Design

Engineer

Project

Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B387  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  2.095 = 0.006 + 1.260 + 0.828

STRESS CHECK FORCES & MOMENTS
                               P          M33          M22          V2          V3
    Combo  DSTLS2         -0.019       -7.942        3.710       0.658      -0.309

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                              fa           Fa           Ft
                          Stress    Allowable    Allowable
    Axial                  0.037        5.987       21.000

                              fb           Fb           Fe        Cm        K        L       Cb
                          Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
    Major Bending         26.465       21.000       60.662     0.850    1.000    0.500    1.000
    Minor Bending         17.395       21.000        5.987     0.850    1.000    1.000

SHEAR DESIGN
                              fv           FV       Stress
                          Stress    Allowable        Ratio
    Major Shear            1.950       14.000        0.139
    Minor Shear            1.649       14.000        0.118
```

10

ETABS Steel Design

Engineer
Project
Subject

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B391  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.385 = 0.001 + 0.783 + 0.602

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2     -0.003      -4.932       2.694       0.407      -0.225

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                         fa           Fa          Ft
                     Stress    Allowable   Allowable
    Axial             0.007        5.987      21.000

                         fb           Fb          Fe         Cm        K        L       Cb
                     Stress    Allowable   Allowable    Factor   Factor   Factor   Factor
    Major Bending    16.436       21.000      60.662     0.850    1.000    0.500    1.000
    Minor Bending    12.635       21.000       5.987     0.850    1.000    1.000

SHEAR DESIGN
                         fv           Fv      Stress
                     Stress    Allowable       Ratio
    Major Shear       1.206       14.000       0.086
    Minor Shear       1.198       14.000       0.086
```

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: B386  Station Loc: 36.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.044 = 0.003 + 0.826 + 0.216

STRESS CHECK FORCES & MOMENTS
                            P          M33         M22          V2          V3
    Combo  DSTLS2      -0.009       -5.204      -0.966       0.430       0.081

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-3)
                           fa           Fa          Ft
                       Stress    Allowable   Allowable
    Axial               0.018        5.987      21.000

                           fb           Fb          Fe       Cm       K       L      Cb
                       Stress    Allowable   Allowable   Factor  Factor  Factor  Factor
    Major Bending      17.340       21.000      60.662    0.850   1.000   0.500   1.000
    Minor Bending       4.530       21.000       5.987    0.850   1.000   1.000

SHEAR DESIGN
                           fv           FV      Stress
                       Stress    Allowable       Ratio
    Major Shear         1.274       14.000       0.091
    Minor Shear         0.429       14.000       0.031
```

x3886

3.4 Wood joists ultimate condition

Wood joists ultimate failure check-ultimate design values are based on Final Report from "Wood Advisory Services, Inc.".

- The 3" X 4" joists under 42" beam formwork spanned 4'-0" and and spaced at 8" o.c.
- Mean value ultimate Strength: Modulus of rupture Fb is 3132 psi, and shear strength Fv is 262 psi.
- The bending stress level is about 50 percent of the modulus of rupture strength, and the shear stress is about 46 percent of the ultimate shear strength.

Project: 246 SPRING STREET Date: 7/2/09 Page: 1 of 1

Subject: WOOD JOIST CHECK FOR RECONSTRUCTION OF FORMWORK Engr: GW Checked By: ______ Date: ______

14.2.5

— ALLOWABLE STRESS DESIGN — UNDER 42" BEAM FORMWORK

JOIST SPAN = 4'-0", JOIST SPACING = 8".

$$W = \frac{42''}{12} \times \frac{8''}{12} \times 150\,pcf + .50 \times \frac{8''}{12} = .383\ klf$$

(CONSTRUCTION LIVE LOAD)

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

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

- $F_b = 1700 \times C_D = 2125\ psi$

  $C_D = 1.25$ FOR 7 DAYS DURATION OF WORK

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

  $S_x = 5.1\ in^3$

- $F_v = C_D \times 135 = 169\ psi$

  $f_v = \frac{3 \times V_{max}}{2 A_v} = \frac{3 \times .77}{2 \times 8.75} = 132\ psi < F_v$ (ok)

  $A_v = 8.75\ in^2$

— ULTIMATE FAILURE CHECK

- $F_b = 3132\ psi$; $F_v = 262\ psi$

  $w = 42/12 \times 8/12 \times 150\,pcf = .35$

  $f_b = .35/.383 \times 1812 = 1656\ psi < F_b = 3132\ psi$

  $f_v = .35/.383 \times 132 = 121\ psi < F_v = 262\ psi$

$\therefore$ WOOD JOIST IN COLLAPSED AREA IS ADEQUATE FOR DESIGN LOAD.

assigned based on that GCD. If no GCD was visible, then no visual grade was assigned to that piece and the piece was classified as no GCD, or "NGCD." Of the 396 fragmented pieces of lumber inspected, 184 were classified without a grade controlling defect. One could argue that these unclassified pieces should have been classified as select structural. However, it was our intent to determine the approximate distribution of actual grade characteristics and assign grades based on the worst characteristic observed. Therefore, arbitrarily assigning these pieces a grade of select structural would have no benefit to the objectives of this project.

Following the completion of this project, WAS, Inc. was provided Patent Construction Systems Drawing Number 4607K070 which appears to be the lumber and plywood specifications for the project at 246 Spring Street. A copy of this document is provided in Appendix V. A review of the lumber design values in this document and published allowable stresses for structural lumber indicated that the dimension lumber (3x4 and 4x4) should have been at a minimum No. 1 & Better (BTR) grade of a species grouping such as Douglas fir-larch, or a No. 2 dense southern pine. The current allowable stresses for these species groupings are published in the Supplement to the National Design Specification for Wood Construction. The published size adjusted base values are provided below. The inclusion of the spruce-pine-fir species grouping here will become evident subsequently in this report. In summary, the dimension lumber used on this project was spruce-pine-fir mill run quality. However, even select structural spruce-pine-fir will not meet the specifications.

Property	Drawing No. 4607K070 Required Design Values	Douglas fir - Larch No.1 & BTR	Southern Pine No. 2 Dense	Spruce - Pine - Fir Select Structural
F_b	1,640 psi	1,800 psi	1,700 psi	1,875 psi
F_v	180 psi	180 psi	175 psi	135 psi*
$C_\perp$	625 psi	625 psi	660 psi	425 psi*
$C_\parallel$	1,350 psi	1,783 psi	1,850 psi	1,610 psi
MOE	1,600,000 psi	1,800,00 psi	1,700,00 psi	1,500,00 psi*

* Values more than 5% below specified allowable property

convert allowable design stresses to ultimate stresses, the general adjustment further must be removed from the allowable design stress. This was done to provide the data in the following summary. For stress calculations, we recommend using the L5% for both strength properties (F_b and F_v) since these values represent the samples which would fail first in a collapse.

Visual Grade	Removal of General Adjustment Factor = L5% or $\bar{x}^2$ (psi)				Estimated Allowable Mean Ultimate Stresses = $\bar{x}$ (psi)			
	F_b L5%	E$\bar{x}$	F_vL5%	$Fc_\perp \bar{x}$	$F_b \bar{x}$	E	$F_v \bar{x}$	$Fc_\perp \bar{x}$
SS	2,757	0.98x10⁶	202	603	4,683	0.98x10⁶	262	603
No. 1	1,930	0.92x10⁶	202	603	3,278	0.92x10⁶	262	603
No. 2	1,930	0.92x10⁶	202	603	3,278	0.92x10⁶	262	603
No. 3	1,103	0.79x10⁶	202	603	1,873	0.79x10⁶	262	603
CONST	1,470	0.86x10⁶	202	603	2,496	0.86x10⁶	262	603
STD	809	0.79x10⁶	202	603	1,374	0.79x10⁶	262	603
ECON²	405	----	----	----	689	----	----	----

Although the L5% exclusion value is technically the most important and most appropriate value to use for estimates of ultimate strength for the determination of failure, WAS, Inc. was also requested to provide an estimated mean strength value for the lumber used at 246 Spring Street. This was accomplished by computing a mean value weighted by the lumber grades observed during our inspection. The resulting values are 3132 psi for F_b, 262 psi for F_v, and 603 for $Fc\perp$, and 916,000 psi for E.

x 13875

3.5 Snap Tie Verification

The maximum load impose on the snaptie at typical spacing of 2'-6" is 1.28 kips which is about 57 percent of safe working load of 2.25 kips (Safe working load is based on safety factor of 2) for industry standard snaptie. In worse case scenario when one tie is loose (effective tie spaced at 5'-0"), the tie load (2 x 1.28 x 2 = 2.56 kips) is still less than the ultimate capacity (2 x 2.25 kips = 4.5 kips), and joists spanning 5'-0" is adequate to maintain the beam formwork.

Project: 246 SPRING STREET Date: 7/7/09 Page: 1 of 2

Subject: ACTUAL LOAD ON BEAM FORM TIE Engr: ______ Checked By: ______ Date: ______

SPACING OF BEAM FORM TIE =

USING 1/8 ARCH SCALE

SPACING = 24 UNIT

JOIST WIDTH = 1.75 UNIT = 2.5" ±

$$SPACING = \frac{24}{1.75} \times 2.5'' = 34'' \pm$$

∴ IT IS CONCLUDED THAT TIE SPACING IS AT THE RANGE OF 2'-6" TO 3'-0".

- TYPICAL LOAD AT SNAPTIES.

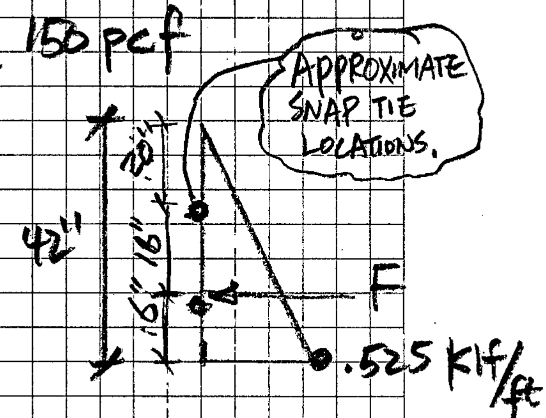
BEAM DEPTH = 42"

$$LATERAL\ PRESSURE = \frac{42''}{12} \times 150\ pcf$$

2'-6" SPACING

$$F = .525 \times 2.5' \times \frac{42''}{2 \times 12}$$

$$= 2.3\ kips$$



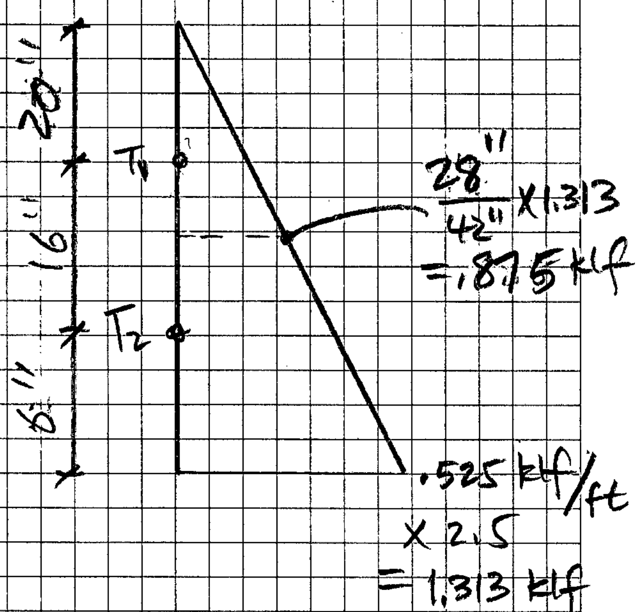
Project: 246 SPRING STREET Date: 7/7/09 Page: 2 of 2

Subject: SNAPTIE CAPACITY CHECK Engr: ____ Checked By: ____ Date: ____

LOADS AT TIES : TYPICAL :

$T_1 = .875 \times \frac{28''}{12 \times 2}$

$= 1.02$ kips



$T_2 = (.875 + 1.313) \times \frac{14''}{2 \times 12}$

$= 1.28$ kips

SNAPTIE STANDARD :

SAFE WORKING LOAD = 2.25 kips

$T_2 <$ 2.25 kips (OK)

∴ THE SNAPTIE FOR BEAM FORMWORK DESIGN IS ADEQUATE.

- LOAD AT TIES (WORST CASE SCENARIO) WHEN ONE TIE IS LOOSE AND RESULTED SPACING OF 5'-0".

$T_1 = 2 \times 1.02 = 2.04$ kips

$T_2 = 1.28 \times 2 = 2.56$ kips

SAFETY FACTOR IS 2.

$T_{ULTIMATE} = 2 \times 2.25 = 4.5 \text{ kip} > T_2$ (OK)

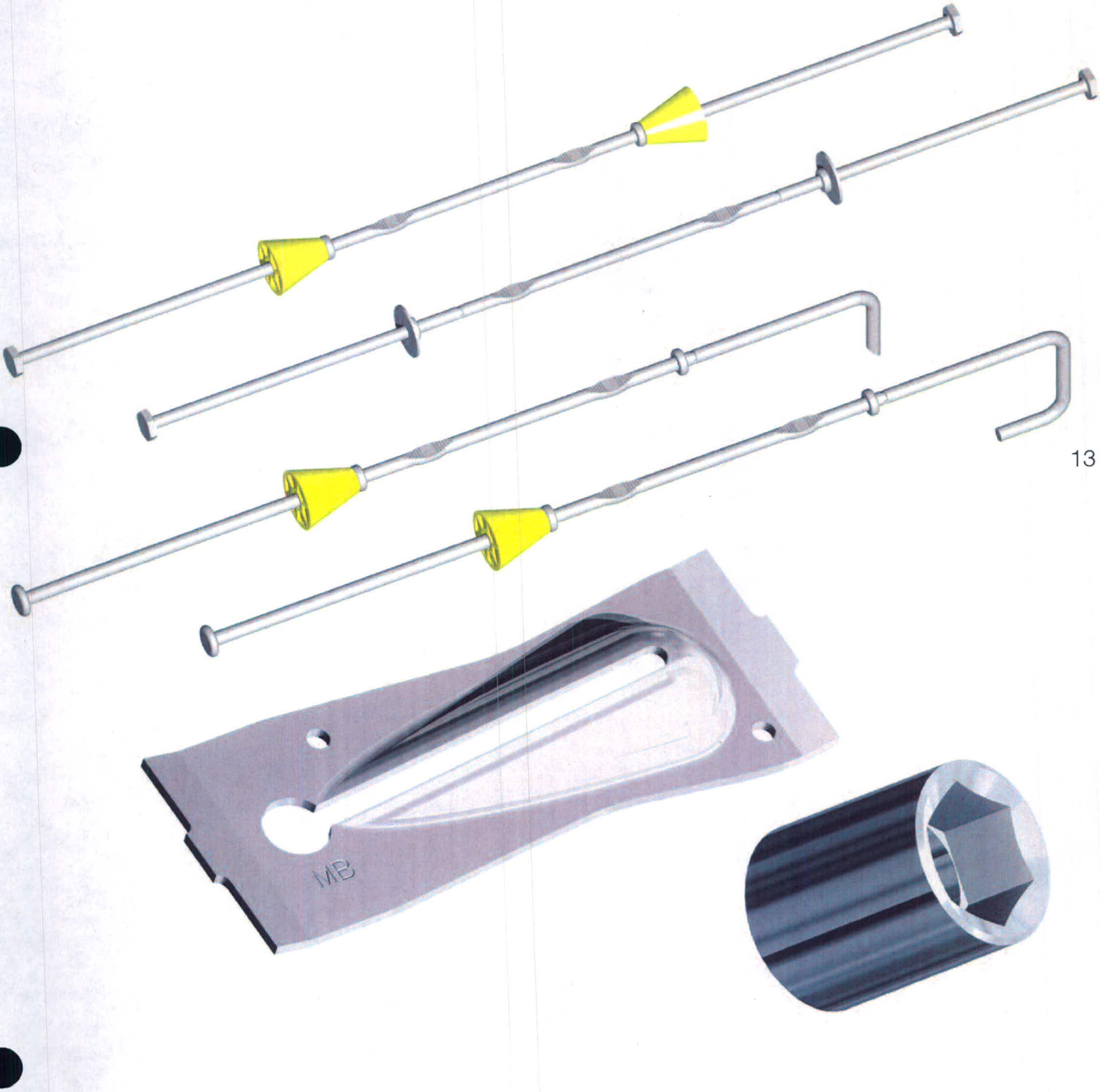
∴ TIES @ 5'-0" O.C. IS ADEQUATE FOR ULTIMATE FAILURE CHECK.


Feldman
LUMBER
SINCE 1912

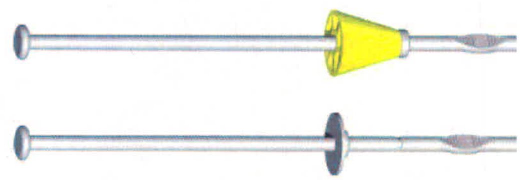

Feldman LUMBER
Feldman LUMBER
Feldman LUMBER
Feldman LUMBER

Meadow Burke Snapties

G



(1001) ST-1 SNAPTIE - STANDARD



The ST-1 Standard Snaptie has round heads, anti-turn deformations and 1" breakback. A 1/2" breakback is available on special order. The Snaptie is available equipped with either plastic spreader cones or loose metal washers. For additional rust and corrosion resistance, the Standard Snaptie is available in stainless steel as an ST-8.

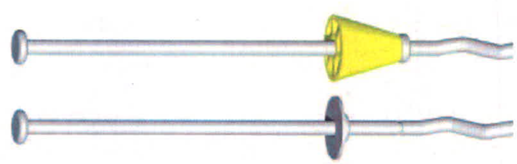
NOTE: The plastic cones, furnished from high impact polystyrene, are available in the sizes 1x1, 1x1-1/2 and 1x2. Cones are preferred over a loose washer tie since it covers the break back portion of the tie. Such guarantee of break back is not available with the loose washer tie. When removed the plastic cone also provides a better cavity for grouting purposes. Attempting to breakback any tie, before the concrete has been allowed to properly set, may result in the entire tie turning freely in the wall, making the normal breakback procedure no longer possible. Washer style snapties should be removed before 24 hours of concrete pour.

SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-1	2,250

Safe working load is based on an approximate 2:1safety factor.

To Order, Specify: quantity, type, L&W, wall thickness, plastic cone or metal washer and breakback.

(1003) ST-3 SNAPTIE - HEAVY



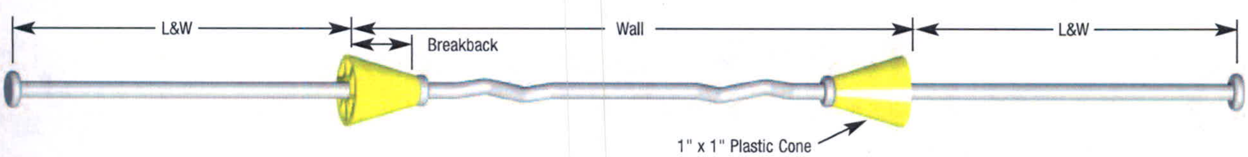
The ST-3 Heavy Snaptie incorporates all of the same design features of the standard snaptie but is fabricated from high carbon steel to produce a higher safe working load. It is available with plastic cones or loose metal washers.

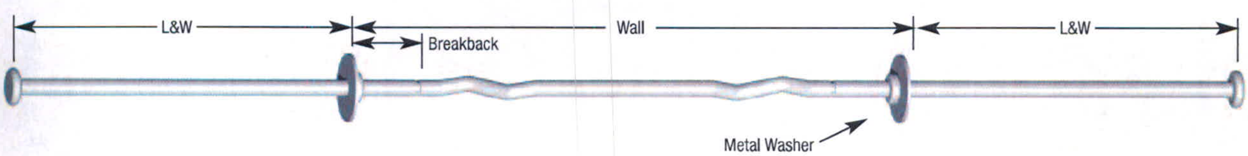
SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-3	3,250

Safe working load is based on an approximate 2:1safety factor.

To Order, Specify: quantity, type, L&W, wall thickness, plastic cone or metal washer and breakback.

(L&W = Lumber & Wedge)





2

SNAPTIE DON'TS

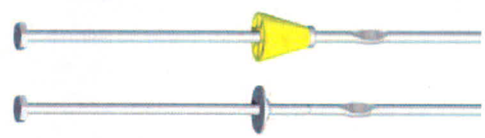
- Do not climb on Snapties in the form.
- Do not over-tighten the tie wedges. This can cause severe pre-loading and premature failure.
- Do not place concrete in just one area of the form and allow it to exceed the design pour rate.
- Do not attempt to move the concrete laterally in the form with a vibrator.
- Do not drop the wet concrete more than 30" when placing into the form. This will result in aggregate segregation and unnecessary dangerous impact loading.
- Do not install bent or damaged ties.
- Do not allow Snaptie ends to remain in the wall beyond 24 hours. Remove the breakback portion of the tie as soon as reasonably possible.
- Do not skip or omit any studs or wales. This will likely cause a premature form failure.
- Do not weld Snapties to any object.

SNAPTIE WATERSEAL

All Meadow Burke Snapties are available with a neoprene washer to aid in preventing moisture seepage along the tie wire. Specify this feature when ordering a snaptie product.

(1005) ST-4 SNAPTIE – HEX HEAD – 6 SIDED

The ST-4 Hex Head Snaptie, (previously known as Wrench Head) provides an effective way to breakback snapties before the formwork has been stripped. The head of the snaptie is grasped by the Hex Head Socket (on Page 18) and with a simple turning motion, breaks off the end of the snaptie. Removing the snaptie ends in this manner increases the ease and speed of the form stripping operation.



Hex Head Snapties are available with 1" x 1" plastic spreader cones and 1" breakback.

Metal washers available on special order only.

SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-4	2,250

Safe working load is based on an approximate 2:1safety factor.

(1008) ST-5 SNAPTIE – THREADED ONE END

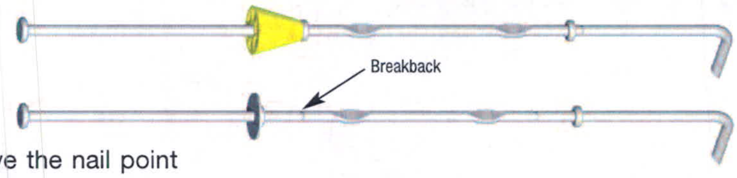
The ST-5 Threaded One End Snaptie is manufactured with 1/4"-20 threads x 2" length on one end and a standard hot forged head on the opposite end. This tie has a metal washer and is used when walls have a variable thickness. A small channel can be installed on either end and then used as a welding tie.

SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-5	250

Safe working load is based on an approximate 2:1safety factor.

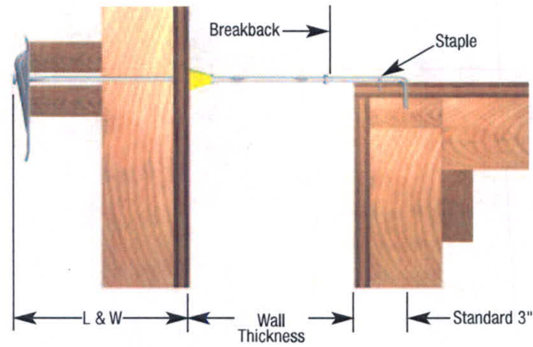
Meadow Burke Snapties

a



(1010) ST-6 SNAPTIE – NAIL POINT

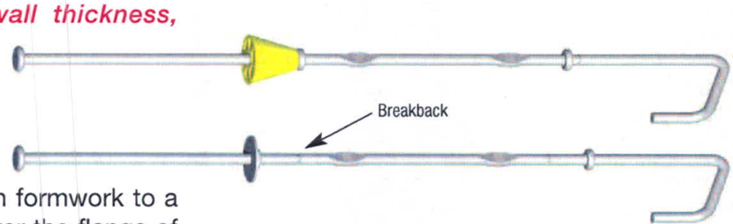
The ST-6 Nail Point Snaptie is designed to have the nail point driven into the formwork and secured with a fence staple. The tie is available with either a plastic spreader cone or a loose metal washer. The plastic cone snaptie is furnished with a standard 1" breakback and the loose metal washer application has a 1/2" breakback.



SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-6	250

Safe working load is based on an approximate 2:1safety factor.

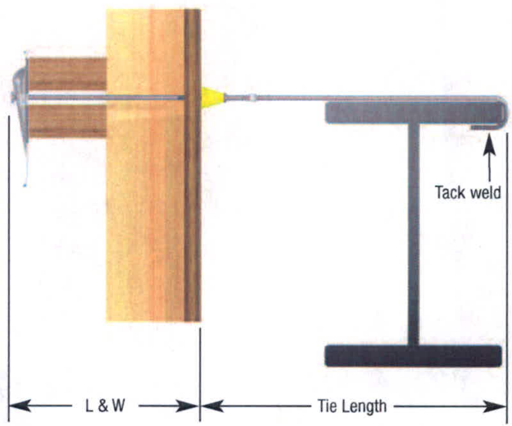
To Order, Specify: quantity, type, L&W, wall thickness, plastic cone or metal washer.



(1012) ST-7 SNAPTIE – HOOKED

16

The ST-7 Hooked Snaptie is designed to attach formwork to a structural beam. The hook end of the tie fits over the flange of the beam and should be tack-welded on the underside of the beam flange for added security. Hooked snapties are available with plastic cone or loose metal washer.



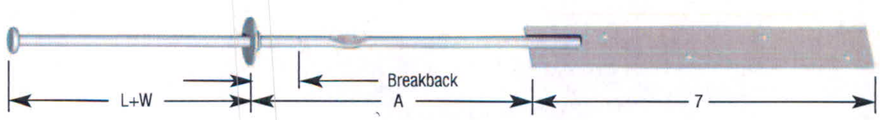
SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-16	250

Safe working load is based on an approximate 2:1safety factor.

To Order, Specify: quantity, type, plastic cone or metal washer, length, flange thickness and form thickness L&W.

(1014) ST-9 SNAPTIE – SPANDREL PLATE

The ST-9 Spandrel Plate snaptie is manufactured with a 16 gauge steel plate with four 1/8" nail holes for nailing direct to formwork. This tie used similarly as the ST-6 nail-point tie. Available with metal washers or plastic cones.

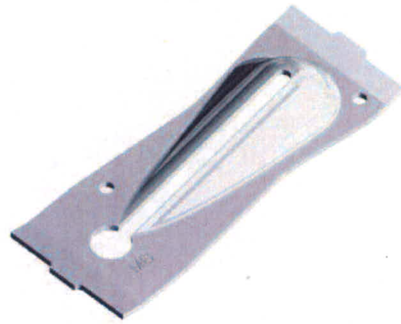


SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-9	250

Safe working load is based on an approximate 2:1safety factor.

To Order, Specify: quantity and type, L+W, and A.

(1020) ST-15 STEEL WEDGE

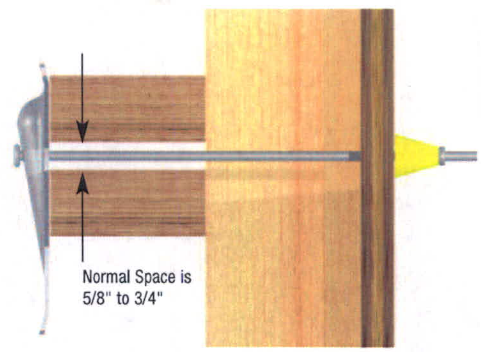


The ST-15 Steel Wedge accommodates either standard or heavy snapties and is designed with sufficient strength to distribute the form loads to the wales.

SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-15	3,250

Safe working load is based on an approximate 2:1safety factor.

To Order, Specify: quantity and type.



Caution: The safe working load of the Steel Wedge can be affected by the position of the wedge on the tie end. Reference Steel Wedge Assembly Precautions below.

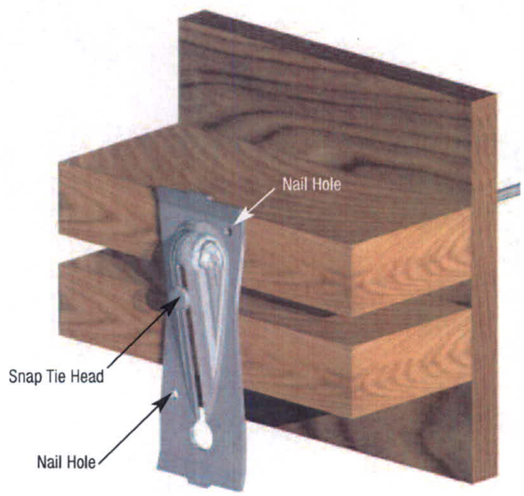
Steel Wedge Assembly Precautions:

Excessive spacing between the walers may cause the steel wedge to bend and result in the cone or washer on the snaptie to become embedded in the concrete. Breakback of the snaptie would be made difficult to accomplish.

Over-tightening the wedge may damage the head of the snaptie, the wedge slot and/or the plastic cone and result in a premature failure.

The Steel Wedge is designed to carry the load at the upper 2/3 of the wedge slot. Load applied too low on the wedge slot may cause the wedge to deform or break.

Nail holes are provided to allow the wedge to be firmly secured to the wales to prevent loosening during vibration.



(1030) ST-21 SNAPTIE WRENCH

The ST-21 Snaptie Wrench is used to quickly and easily breakback snapties. After the forming has been stripped the Snaptie Wrench captures the snaptie end. The tie is bent down to a position nearly parallel to the face of the concrete and a subsequent clockwise rotation of the wrench breaks the tie at the breakback point.

To Order, Specify: quantity and type.

(1015) ST-16 HEX HEAD SOCKET – 6-SIDED

The 3/8" drive ST-16 Hex Head Socket - 6 Sided fits securely over the head of the Hex Head Snaptie. Used primarily on the Single Waler System's short end snap-ties, a small turn of the socket snaps the tie end at the breakback point before the formwork is removed.

To Order, Specify: quantity and type.

(1022) ST-17 RESIDENTIAL SNAPTIE – PLASTIC CONES
(1023) ST-18 RESIDENTIAL SNAPTIE – METAL WASHERS

The ST-17 and ST-18 Residential Snapties are designed to be used on stem-type footing walls, foundation walls, and basement walls.

The ST-17 and ST-18 Residential Snapties are available with fixed 1-1/4" washers for a flush break back. This breakback routinely results in the wire being flush with the surface of the concrete or slightly protruding outside the surface. When a finished wall is desired, this tie is also available with in a 1" cone and a 1" breakback.

18

With an end dimension of 1-5/8", the Meadow Burke Residential Snapties can be used with ST-15 Steel Wedges. Frequently used with our Quick Cleat (see page 32).

SAFE WORKING LOAD	
TYPE	SWL (lbs)
ST-17 / 18	2,250

Safe working load is based on an approximate 2:1 safety factor.

To Order, Specify: quantity, type, L&W, wall thickness, plastic cones or metal washer and breakback.

12

FIELD MEASUREMENT
FOR SNAPTIE

GuoZhan Wu

From: Dan Eschenasy

Sent: Wednesday, July 22, 2009 12:59 PM

To: GuoZhan Wu

Subject: ties

30 inch on center
16 oc vertical
6" from ply edge (would say bottom edge)

Dan Eschenasy, PE
Chief Structural Engineer
NYC Buildings
280 Broadway, NY, NY 10007
(212) 566 3845
deschenasy@buildings.nyc.gov

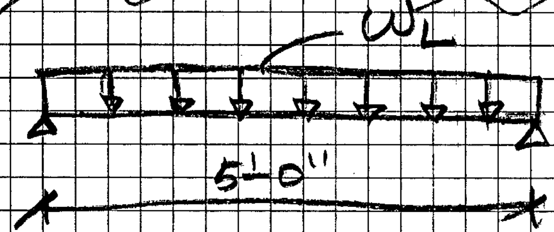
Project: 246 Spring Street Date: 7/22/09 Page: ___ of ___

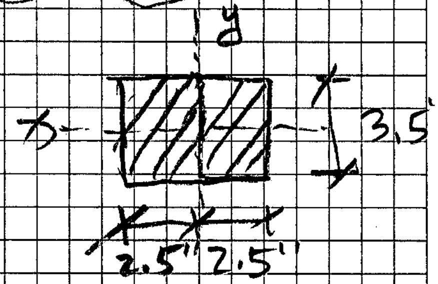
Subject: BEAM FORMWORK Engr: ___ Checked By: ___ Date: ___

JOIST CHECK

- JOIST SPAN = 5'-0"

(ASSUMING SNAP TIE @ 5'-0")

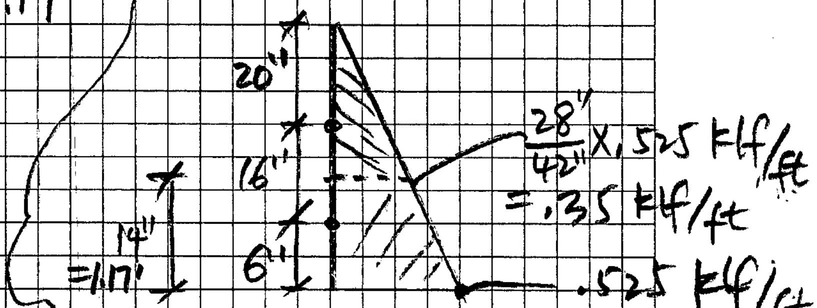




CROSS-SECTION

$W_L = \left(\frac{.35 + .525}{2}\right) \times 1.17'$

$= .52$ klf



$M = \frac{W_L \times 5^2}{8}$

$= 1.625$ k-ft

$= 19.5$ k-in

$S_x = (2.5 \times 2) \times 3.5^2/6 = 10.2\ in^3$

$f_b = \frac{M}{S_x} = \frac{19.5 \text{ k-in}}{10.2} = 1.91$ ksi

ULTIMATE STRENTH $F_b = 3132$ psi $= 3.13$ ksi

$f_b < F_b$ (ok)

7\12\94

4 Validity of Computer Analysis

the following calculations compare the valuers obtained by computer analysis with test results.

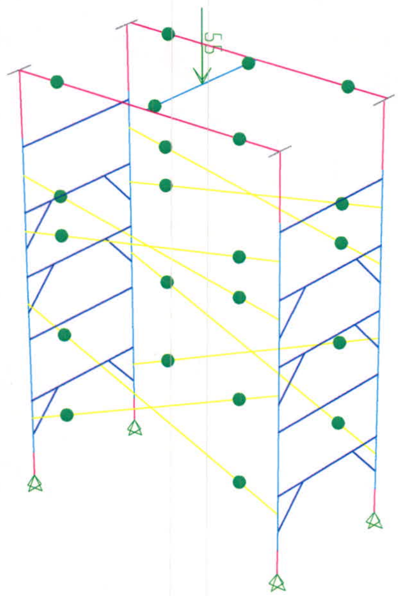
It considers dead weight only. The overall frame is made of 5'-3" bottom frame and 3'-6" top frame, and the frame is 8'-0" apart. Two loading conditions have been considered and studied.

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.

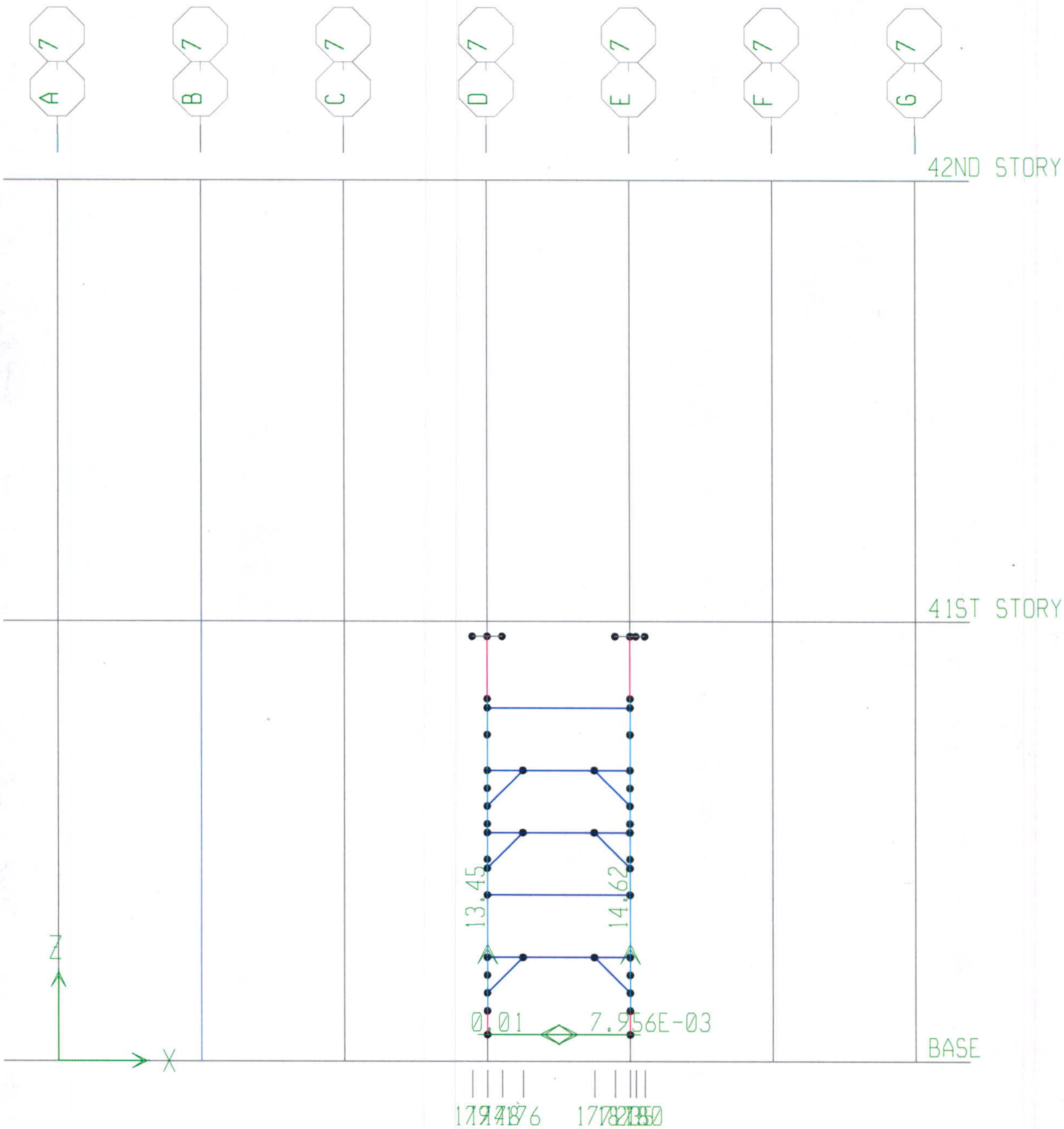
- The model file is "246SpringST_2009_05_08_singleTowerDL_1sideECC55KipsTestLoadedinMiddle"

Result: shore failure

- Screw jacks on the 2" loaded sides buckles and has large deflection.
- All 4 Shoring legs do not fail in buckle or bending.
- **The connection from the top horizontal members to the legs fail in weldment.**
- Knee brace does not fail in compression, tension or bending.
- **The horizontal top deflection is in the order of .8", and the deflection from Lehigh Testing Lab for the similar condition is in order of .6". The results are comparable.**

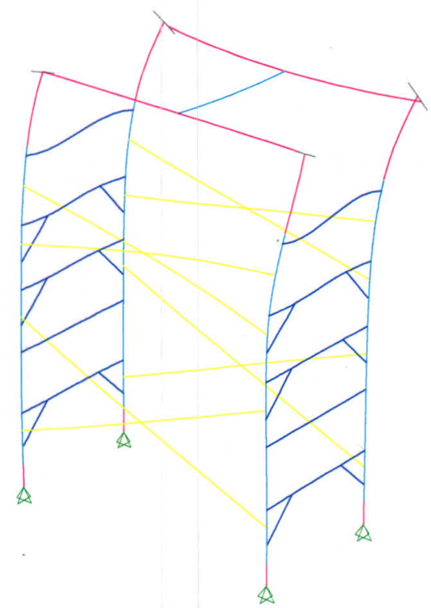

5.5

2

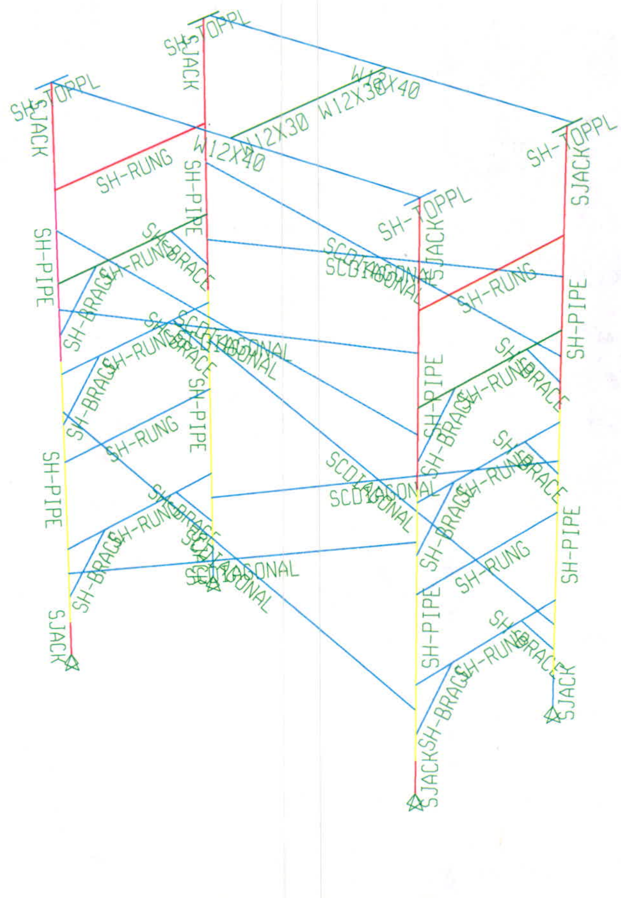


ETABS v9.0.7 - File: 246SpringST_2009_05_08_singleTowerDL_1sideECC55KipsTestLoadedInMiddle - June 10,2009 19:07
Elevation View - 7 Restraint Reactions (DEAD) - Kip-in Units

3



4



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: 41ST STORY  Element: B390  Station Loc: 48.000  Section ID: SH-RUNG
Element Type:  Moment Resisting Frame   Classification:  Compact

L=48.000
A=0.503  i22=0.133  i33=0.338
s22=0.213  s33=0.300  r22=0.515  r33=0.820
E=10100.000  fy=35.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is  1.205 = 0.230 + 0.947 + 0.028

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22         V2          V3
    Combo  DSTLS2     -0.692      -6.386       0.113       0.267      -0.005

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1)
                         fa          Fa          Ft
                     Stress   Allowable   Allowable
    Axial             1.377       5.987      21.000

                         fb          Fb          Fe       Cm        K        L       Cb
                     Stress   Allowable   Allowable   Factor   Factor   Factor   Factor
    Major Bending    21.279      21.000      15.165    0.850    1.000    1.000    2.300
    Minor Bending     0.528      21.000       5.987    0.850    1.000    1.000

SHEAR DESIGN
                         fv          FV      Stress
                     Stress   Allowable       Ratio
    Major Shear       0.790      14.000       0.056
    Minor Shear       0.025      14.000       0.002
```

6

ETABS Steel Design

Engineer ______
Project ______
Subject ______

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C102-3  Station Loc: 42.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=42.000
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  1.300 = 0.495 + 0.805 + 0.002

STRESS CHECK FORCES & MOMENTS
                          P          M33         M22          V2          V3
    Combo  DSTLS2   -13.970       16.950      -0.043      -0.746      -0.003

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-2)
                         fa           Fa          Ft
                     Stress    Allowable   Allowable
    Axial            10.402       19.050      21.000

                         fb           Fb          Fe        Cm       K       L      Cb
                     Stress    Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending    18.594       23.100     120.071     0.850   1.000   0.500   1.315
    Minor Bending     0.047       23.100     163.430     0.850   1.000   0.429

SHEAR DESIGN
                         fv           FV      Stress
                     Stress    Allowable       Ratio
    Major Shear       1.029       14.000       0.074
    Minor Shear       0.004       14.000       0.000
```

ETABS Steel Design

Engineer______________________
Project______________________
Subject______________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C19-1  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  2.095 = 0.331 + 1.764 + 0.000

STRESS CHECK FORCES & MOMENTS
                           P         M33         M22          V2          V3
    Combo  DSTLS2    -13.963      32.621       0.017      -0.746      -0.003

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-2)
                          fa          Fa          Ft
                      Stress   Allowable   Allowable
    Axial             10.917      29.179      33.000

                          fb          Fb          Fe        Cm       K       L      Cb
                      Stress   Allowable   Allowable    Factor  Factor  Factor  Factor
    Major Bending     64.051      36.300     134.847     0.850   1.000   1.000   1.285
    Minor Bending      0.033      36.300     134.847     0.850   1.000   1.000

SHEAR DESIGN
                          fv          FV      Stress
                      Stress   Allowable       Ratio
    Major Shear        0.986      22.000       0.045
    Minor Shear        0.004      22.000       0.000
```

ETABS v9.0.7 File:246SPRINGST_2009_05_08_SINGLETOWERDL_1SIDEECC55KIPSTESTLOADEDINMIDDLE Units:Kip-in June 10, 2(

8

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	87-5	DEAD	0.8087	0.0937	-0.0623	-0.01574	0.02223	0.00000
41ST STORY	174-1	DEAD	0.8046	-0.0937	-0.0623	0.01574	0.02165	0.00000
41ST STORY	233-1	DEAD	1.0561	-0.0001	-0.1756	0.00001	0.05538	0.00000
41ST STORY	235-1	DEAD	1.0529	0.0001	-0.1744	-0.00001	0.05478	0.00000

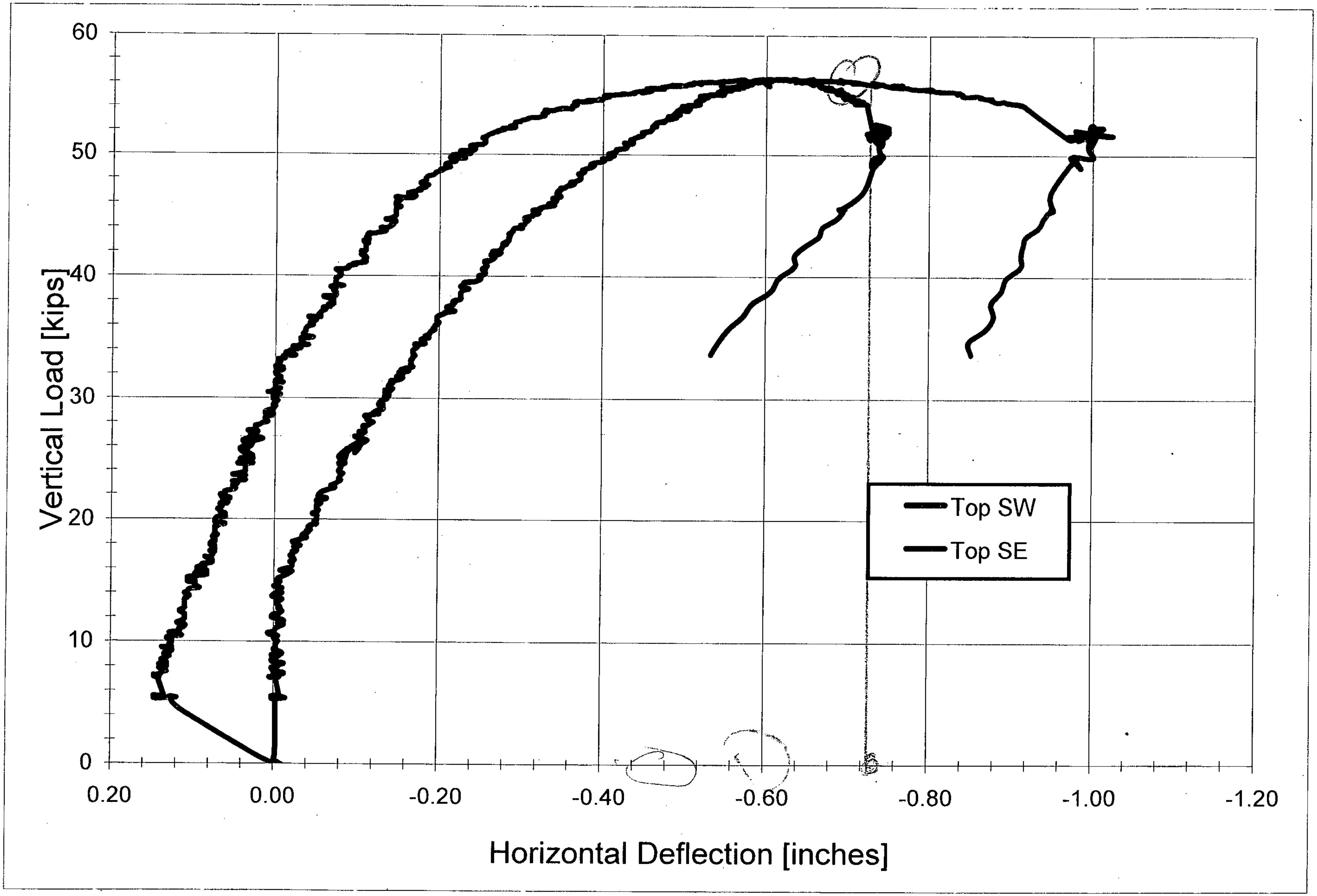
Test	Test Type	Bottom Screw Jack [in]				Top Screw Jack [in]				Total Height [in]				Max Load [lbs]	Failure mode
		NW	SW	NE	SE	NW	SW	NE	SE	NW	SW	NE	SE		
1	Concentric Load Tower "A"	12	12	12 1/8	12 1/16	17 13/16	18	17 7/8	17 15/16	135 3/4	135 3/4	135 7/8	135 7/8	159,000	Racking
2	Concentric Load Tower "C"	11 5/8	11 13/16	11 5/8	11 3/4	18 3/16	18	18 1/8	17 15/16	135 5/8	135 3/4	135 3/4	135 3/4	154,500	Racking
3	Concentric Load Tower "B"	-	-	-	-	18	18	18	18	129 7/16	129 7/16	129 5/16	129 1/4	152,100	Racking
4	Eccentric Load (Various Components)	12	12 1/8	12 1/16	12 1/16	18	18	18 1/8	18	135 7/8	135 15/16	135 7/8	135 7/8	61,300	Screw jack buckling
5	Eccentric Load (Various Components)	-	-	-	-	21	21	21 1/16	21	132 7/16	132 7/16	132 7/16	132 7/16	52,100	Screw jack buckling
6	Eccentric Load (Various Components)	12 3/16	12	12	12	21 1/8	21 1/8	21 1/8	21 1/4	139	139	138 7/8	139	56,400	Top plate fractured

All specimens loaded at a rate of 6-8 kips/min
Load was held for 1 minute at 30-60-90-120 kips
All specimens were plumb to 1/8" in 3" except for Test 2

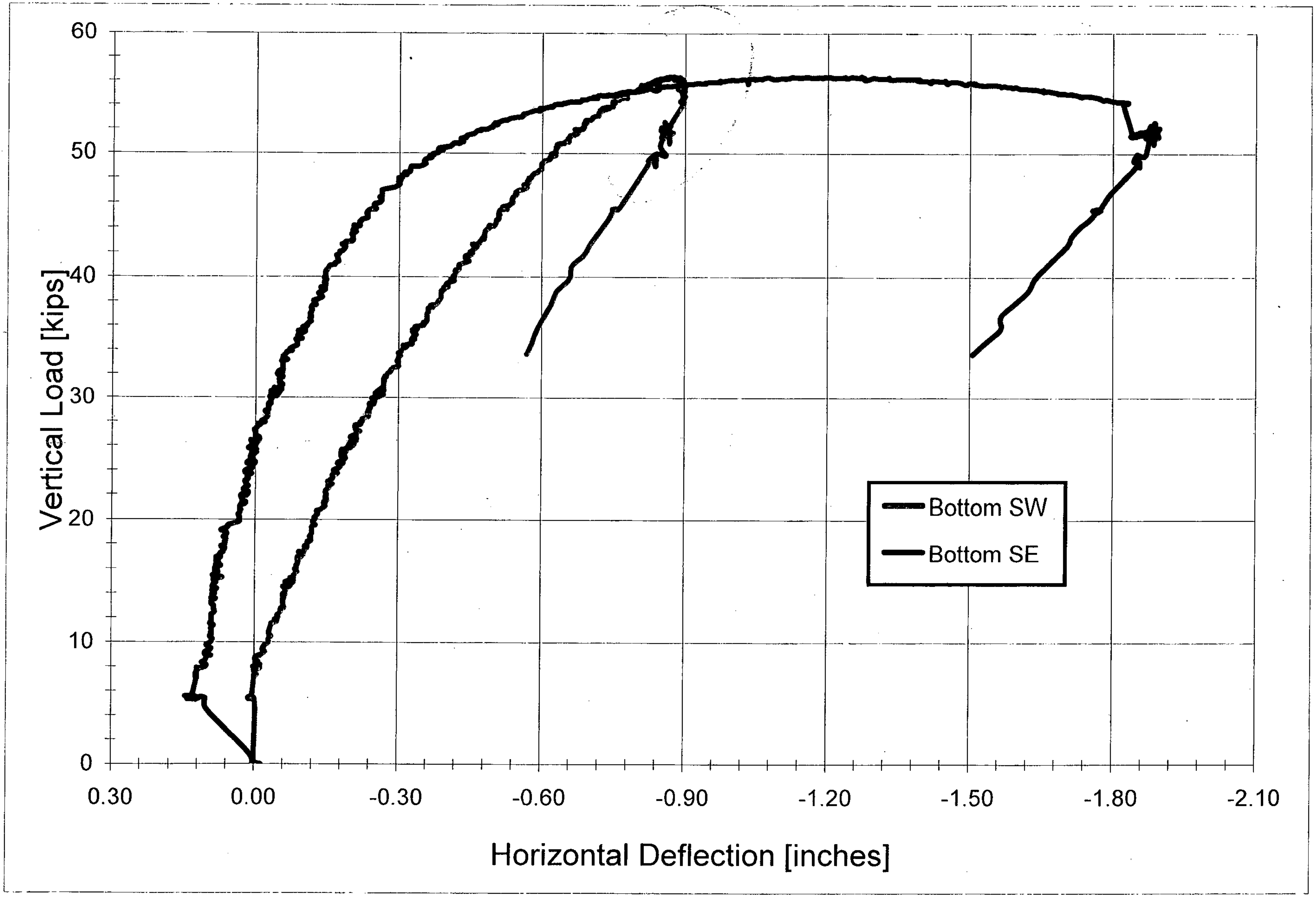
10

Test Name = Ecc Test 3
Test Date = 14:55:31 May 05 2009

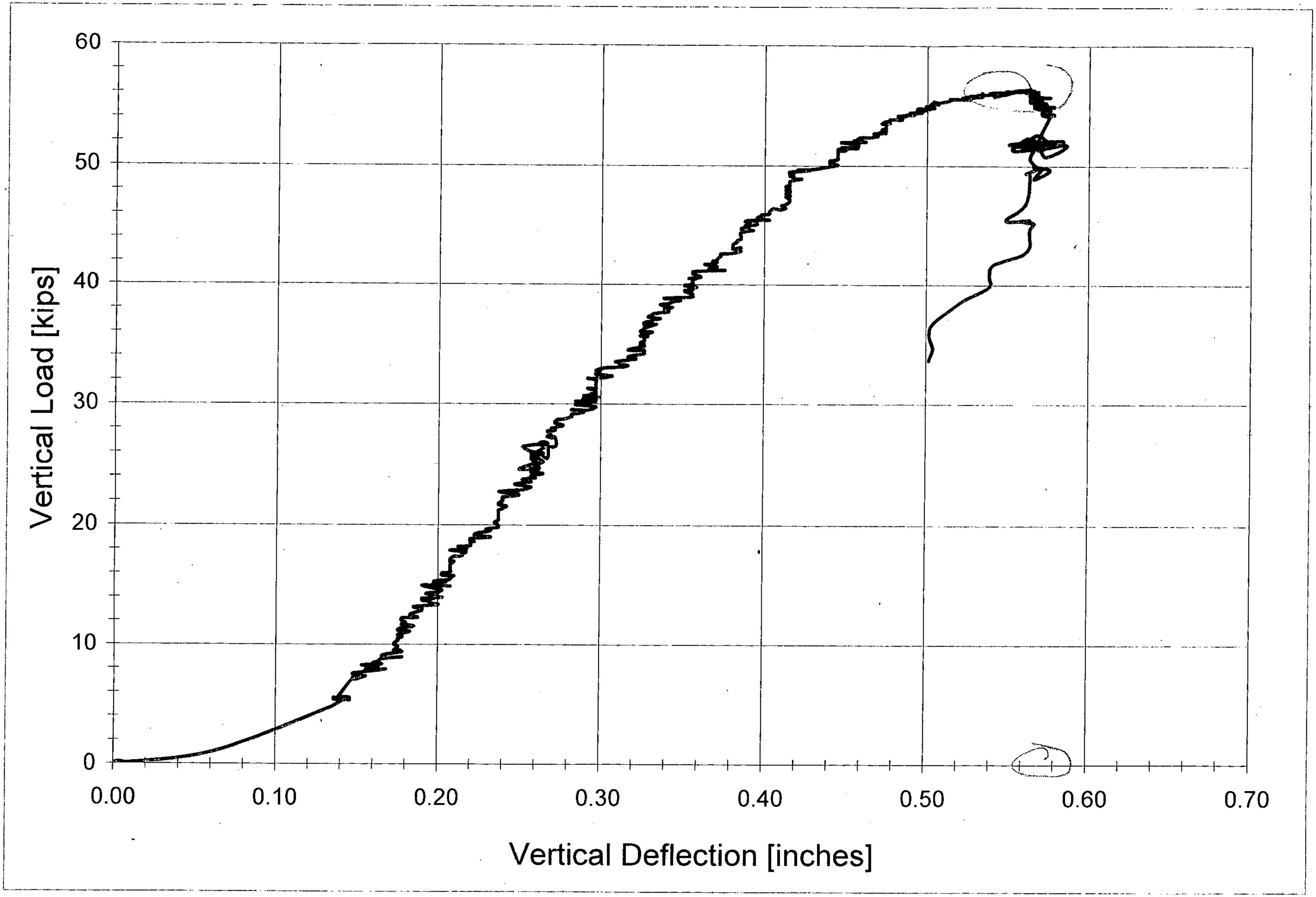
	CH #'s	CH-01	CH-02	CH-03	CH-04	CH-05	CH-00
	NAME	SW Top	SW Bot	SE Top	SE Bot	Zero Vert	Load
	UNITS	Inches	Inches	Inches	Inches	Inches	Kips
10	15:43:55	0.00	0.00	0.00	0.00	0.01	-0.04
11	15:43:56	0.00	0.00	0.00	0.00	0.00	-0.03
12	15:43:57	0.00	-0.01	-0.01	0.00	0.00	0.02
13	15:43:58	0.00	0.00	0.00	0.00	0.00	0.04
14	15:43:59	0.00	0.00	0.00	0.00	0.00	-0.16
15	15:44:00	0.00	0.00	0.00	0.00	0.00	0.13
16	15:44:01	0.00	0.00	0.00	0.00	0.01	0.02
17	15:44:02	0.00	0.00	0.00	0.00	0.00	0.00
18	15:44:03	0.00	-0.01	0.00	0.00	0.00	0.02
19	15:44:04	0.00	0.00	-0.01	0.00	0.00	-0.05
20	15:44:05	0.00	0.00	0.00	0.00	0.01	0.03
21	15:44:06	0.00	0.00	0.00	0.00	0.00	-0.09
22	15:44:07	0.00	0.00	0.00	0.00	0.00	-0.12
23	15:44:08	0.00	0.00	0.00	0.00	0.01	-0.08
24	15:44:09	0.00	0.00	0.00	0.00	0.00	0.00
25	15:44:10	0.00	0.00	0.00	0.00	0.00	-0.02
26	15:44:11	0.00	-0.01	-0.01	0.00	0.01	0.03
27	15:44:12	0.00	0.00	0.00	0.00	0.00	-0.02
28	15:44:13	0.00	0.00	0.00	0.00	0.00	0.07
29	15:44:14	0.00	0.00	0.00	0.00	0.00	-0.02
30	15:44:15	0.00	0.00	0.00	0.00	0.00	-0.02
31	15:44:16	0.00	-0.01	-0.01	0.00	0.01	0.04
32	15:44:17	0.00	0.00	0.00	0.00	0.00	0.03
33	15:44:18	0.00	0.00	0.00	0.00	0.00	-0.05
34	15:44:19	0.00	0.00	0.00	0.00	0.01	-0.06
35	15:44:20	0.00	0.00	0.00	0.00	0.00	-0.04
36	15:44:21	0.00	0.00	0.00	0.00	0.00	0.00
37	15:44:22	0.00	0.00	0.00	0.00	0.00	-0.05
38	15:44:23	0.00	0.00	0.00	0.00	0.00	-0.01
39	15:44:24	0.00	0.00	0.00	-0.01	0.00	-0.06
40	15:44:25	0.00	0.00	-0.01	0.00	0.00	-0.08
41	15:44:26	0.00	0.00	-0.01	-0.01	0.00	-0.01
42	15:44:27	0.00	0.00	0.00	-0.01	0.01	-0.02
43	15:44:28	0.00	-0.01	0.00	0.00	0.00	-0.06
44	15:44:29	0.00	0.00	0.00	0.00	0.00	0.03
45	15:44:30	0.00	0.00	0.00	0.00	0.00	0.05
46	15:44:31	0.00	0.00	-0.01	0.00	0.00	-0.13
47	15:44:32	0.00	0.00	0.00	0.00	0.00	0.12
48	15:44:33	0.00	0.00	0.00	0.00	0.00	-0.09
49	15:44:34	0.00	0.00	0.00	-0.01	0.00	-0.02
50	15:44:35	0.00	0.00	0.00	0.00	0.01	-0.01
51	15:44:36	0.00	0.00	-0.01	0.00	0.00	-0.04
52	15:44:37	0.00	0.00	0.00	0.00	0.00	-0.02
53	15:44:38	0.00	0.00	0.00	0.00	0.00	0.00
54	15:44:39	0.00	0.00	0.00	0.00	0.00	-0.03
55	15:44:40	0.00	0.00	0.00	0.00	0.00	0.00
56	15:44:41	0.03	0.00	0.00	0.01	0.06	0.98
57	15:44:42	0.12	0.00	0.00	0.10	0.13	4.52
58	15:44:43	0.13	0.00	0.00	0.10	0.14	5.42



60
50
40
30
20
10
0
Vertical Load [kips]
0.20
0.00
-0.20
-0.40
-0.60
-0.80
-1.00
-1.20
Horizontal Deflection [inches]
Top SW
Top SE


Vertical Load [kips]
60
50
40
30
20
10
0
0.30
0.00
-0.30
-0.60
-0.90
-1.20
-1.50
-1.80
-2.10
Horizontal Deflection [inches]
Bottom SW
Bottom SE

12

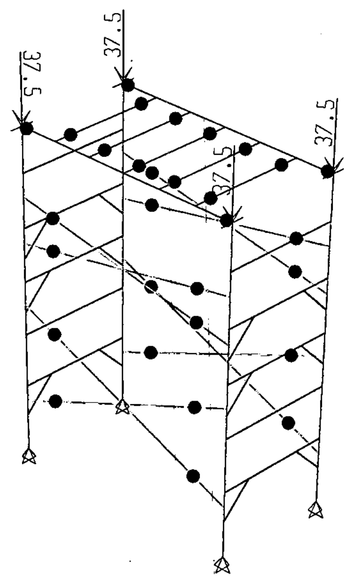

60
50
40
30
20
10
0
Vertical Load [kips]
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
Vertical Deflection [inches]

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).

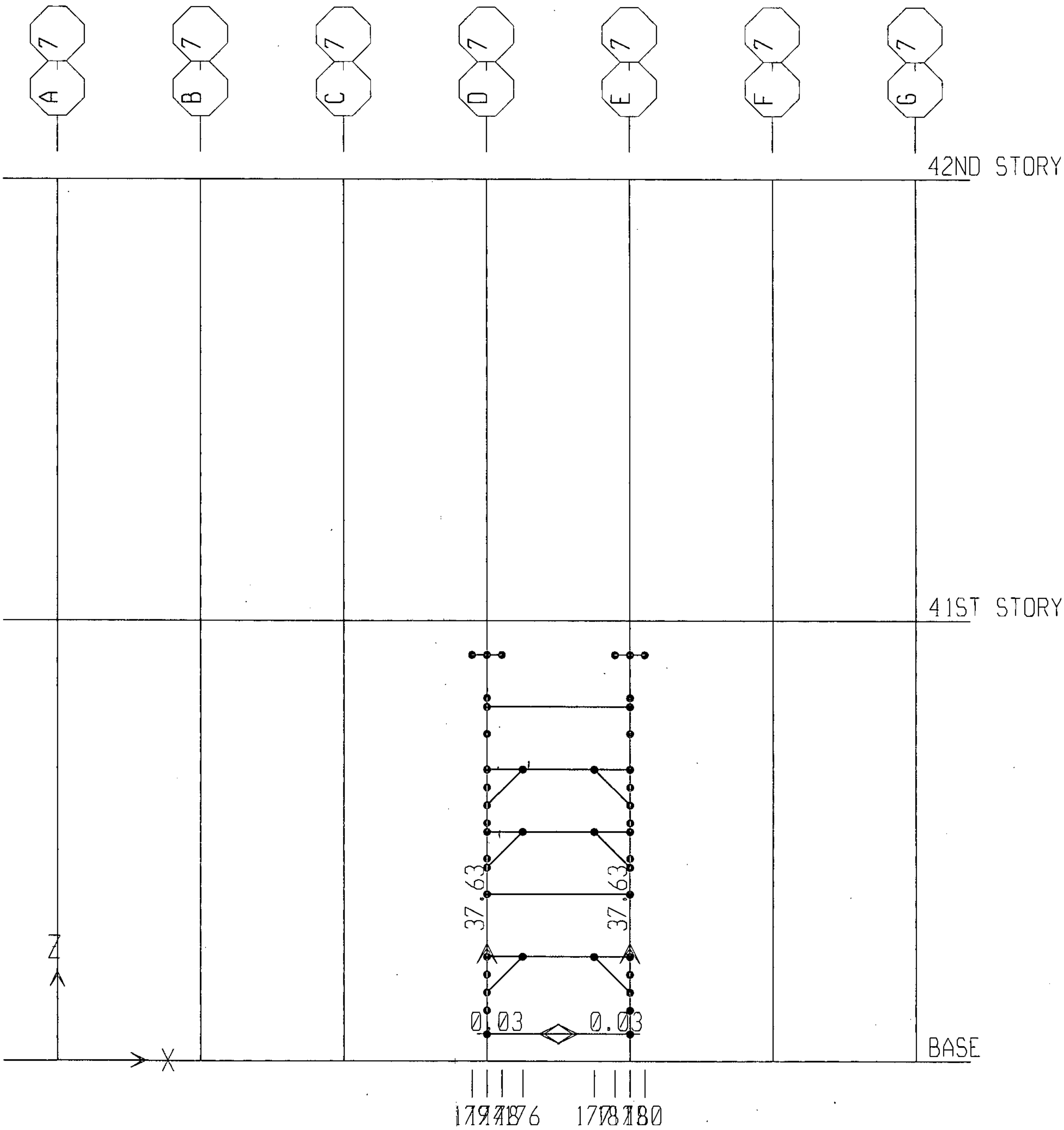
- The model file is "246SpringST_2009_05_08_singleTowerDL_NoECC150KipsTestLoad".

Result: shore failure

- Screw jacks do not fail in combination of compression and bending.
- **All 4 Shoring legs fail in combination of compression and bending.**
- All horizontal members and connections do not fail.
- Knee brace does not fail in compression, tension or bending.
- Test and computer analysis are comparable


37.5
37.5
37.5
37.5

2


A 7
B 7
C 7
D 7
E 7
F 7
G 7
42ND STORY
41ST STORY
BASE
37.63
37.63
0.03
0.03
Z
X

3

ETABS Steel Design

Engineer__________
Project__________
Subject__________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C25-8  Station Loc: 18.000  Section ID: SH-PIPE
Element Type:  Moment Resisting Frame   Classification:  Compact

L=63.000
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  1.788 = 1.696 + 0.020 + 0.090

STRESS CHECK FORCES & MOMENTS
                            P          M33          M22           V2           V3
   Combo  DSTLS2      -37.605       -0.372       -0.436       -0.022       -0.003

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1)
                           fa           Fa           Ft
                       Stress    Allowable    Allowable
   Axial               28.000       16.510       21.000

                           fb           Fb           Fe        Cm        K        L       Cb
                       Stress    Allowable    Allowable    Factor   Factor   Factor   Factor
   Major Bending        0.408       23.100      120.071     0.850    1.000    0.333    1.000
   Minor Bending        0.478       23.100       34.813     0.850    1.000    0.619

SHEAR DESIGN
                           fv           FV       Stress
                       Stress    Allowable        Ratio
   Major Shear          0.030       14.000        0.002
   Minor Shear          0.005       14.000        0.000
```

4

ETABS Steel Design

Engineer__________________________

Project__________________________

Subject__________________________

```
AISC-ASD89 STEEL SECTION CHECK    Units: Kip-in  (Summary for Combo and Station)
Level: 41ST STORY  Element: C92-11  Station Loc: 0.000  Section ID: SJACK
Element Type:  Moment Resisting Frame   Classification:  Compact

L=14.500
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   0.926 = 0.917 + 0.000 + 0.009

STRESS CHECK FORCES & MOMENTS
                             P         M33         M22          V2          V3
    Combo  DSTLS2      -37.575       0.017      -0.189       0.001      -0.013

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1)
                            fa          Fa          Ft
                        Stress   Allowable   Allowable
    Axial               29.379      32.025      33.000

                            fb          Fb          Fe        Cm         K         L        Cb
                        Stress   Allowable   Allowable    Factor    Factor    Factor    Factor
    Major Bending        0.033      36.300    1213.622     0.850     1.000     0.483     1.281
    Minor Bending        0.371      36.300    1213.622     0.850     1.000     0.483

SHEAR DESIGN
                            fv          FV      Stress
                        Stress   Allowable       Ratio
    Major Shear          0.002      22.000   6.934E-05
    Minor Shear          0.017      22.000       0.001
```

+16495

Appendix:

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

LEHIGH UNIVERSITY

Structural Testing Laboratories
Fritz Engineering Laboratory
13 East Packer Avenue
Bethlehem, PA 18015-4729
(610) 758-5498 Fax (610) 758-5902

June 15, 2009
FL2009.1208.1

Dan Eschenasy
Department Chief Structural Engineer
NYC Buildings
280 Broadway, 7th Floor
New York, NY 10007

Subject: Testing of Shoring Towers for NYC Buildings

Dear Mr. Eschenasy,

On May 4th and 5th, 2009, six shoring towers were tested in the Fritz Lab Baldwin-Lima-Hamilton 5,000K testing machine. The 5,000K machine was calibrated on April 8, 2009. Three towers had concentric axial force applied, and three towers had eccentric axial force applied. Five string pot type displacement transducers were used to measure deflections for the eccentric load tests. The test types and results are summarized in Table 1. Before and after photos of the six test specimens are shown in Figures 1-12.

Load was applied to the towers using an H frame arrangement. A spherical bearing block was affixed to the bottom of the sensitive crosshead which loaded on a spreader beam which in turn loaded on two load beams. The concentric test specimens were loaded directly through the four columns using spacer blocks on top of the screw jack feet. The three eccentric load tests were performed by moving the south load beam so that the center of the load was 2" outboard of the centerline of the screw jack. The North load beam was centered over the screw jacks. Stringer beams were also placed on top of the screw jack feet for the eccentric tests. The bottom screw jack extensions were 12" for all tests except Tests 3 and 5, which had no bottom screw jacks. The top screw jack extensions were either 18" or 21".

Table 1: Summary of Test Results

Test	Test Type	Btm SJ [in]	Top SJ [in]	Total Height [in]	Max Load [lbs]	Failure mode
1	Concentric Load Tower "A"	12	18	136	159,000	Racking
2	Concentric Load Tower "C"	12	18	136	154,500	Racking
3	Concentric Load Tower "B"	None	18	130	152,100	Racking
4	Eccentric Load (Various Components)	12	18	136	61,300	Screw jack buckling
5	Eccentric Load (Various Components)	None	21	133	52,100	Screw jack buckling
6	Eccentric Load (Various Components)	12	21	139	56,400	Top plate fractured

2

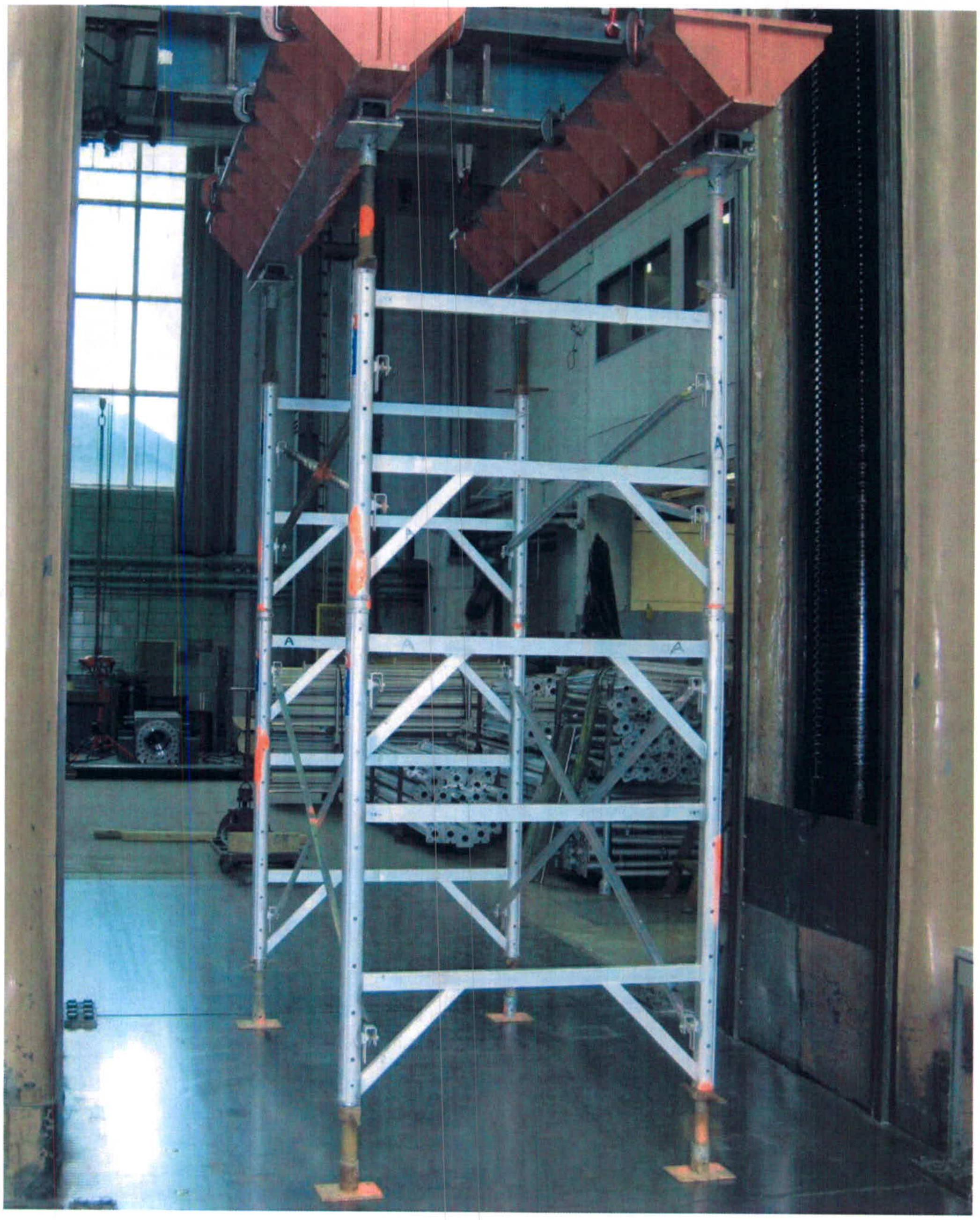


Figure 1: Tower 1 Pretest

3

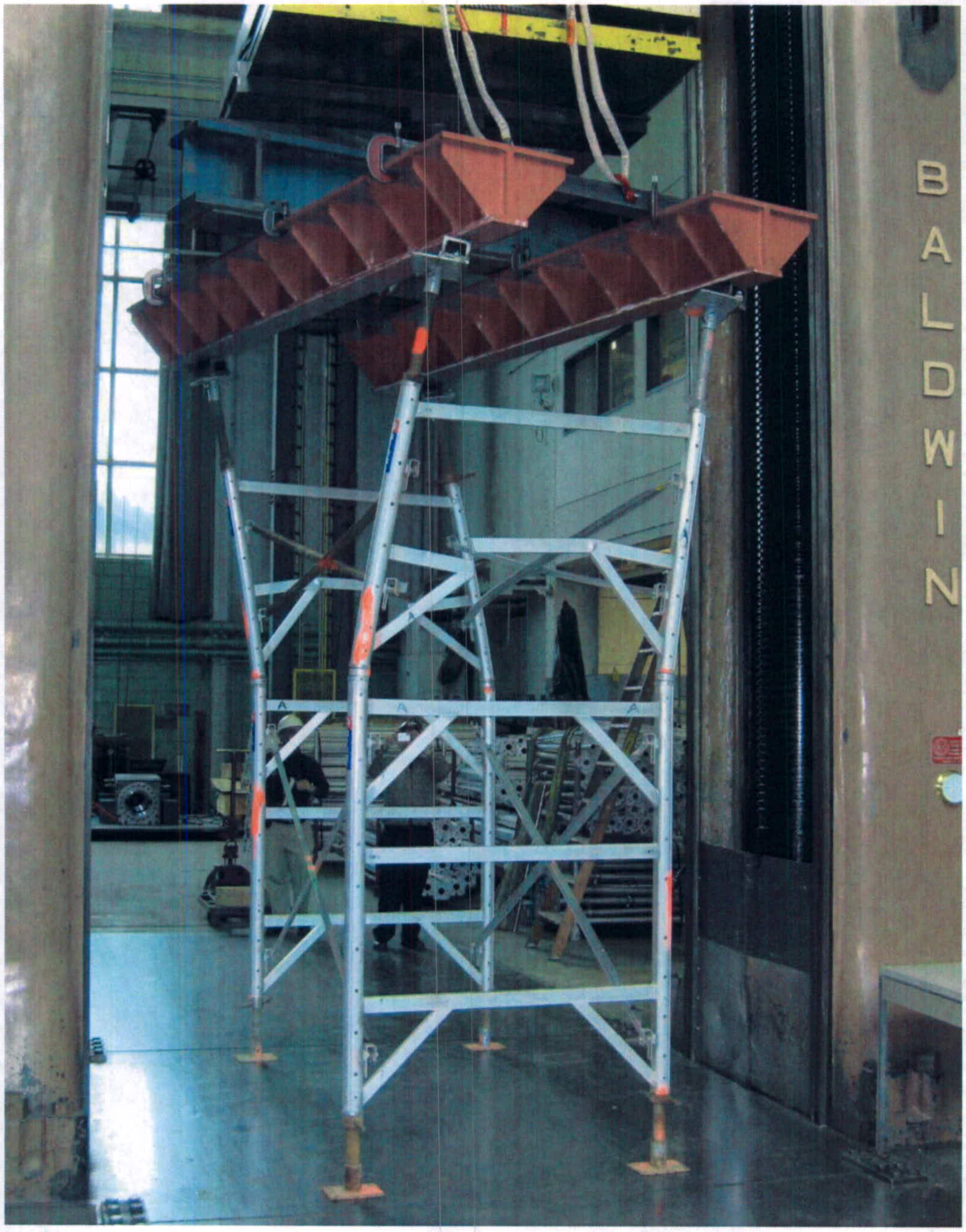


Figure 2: Tower 1 Post Test

4

Figure 3: Tower 2 Pretest

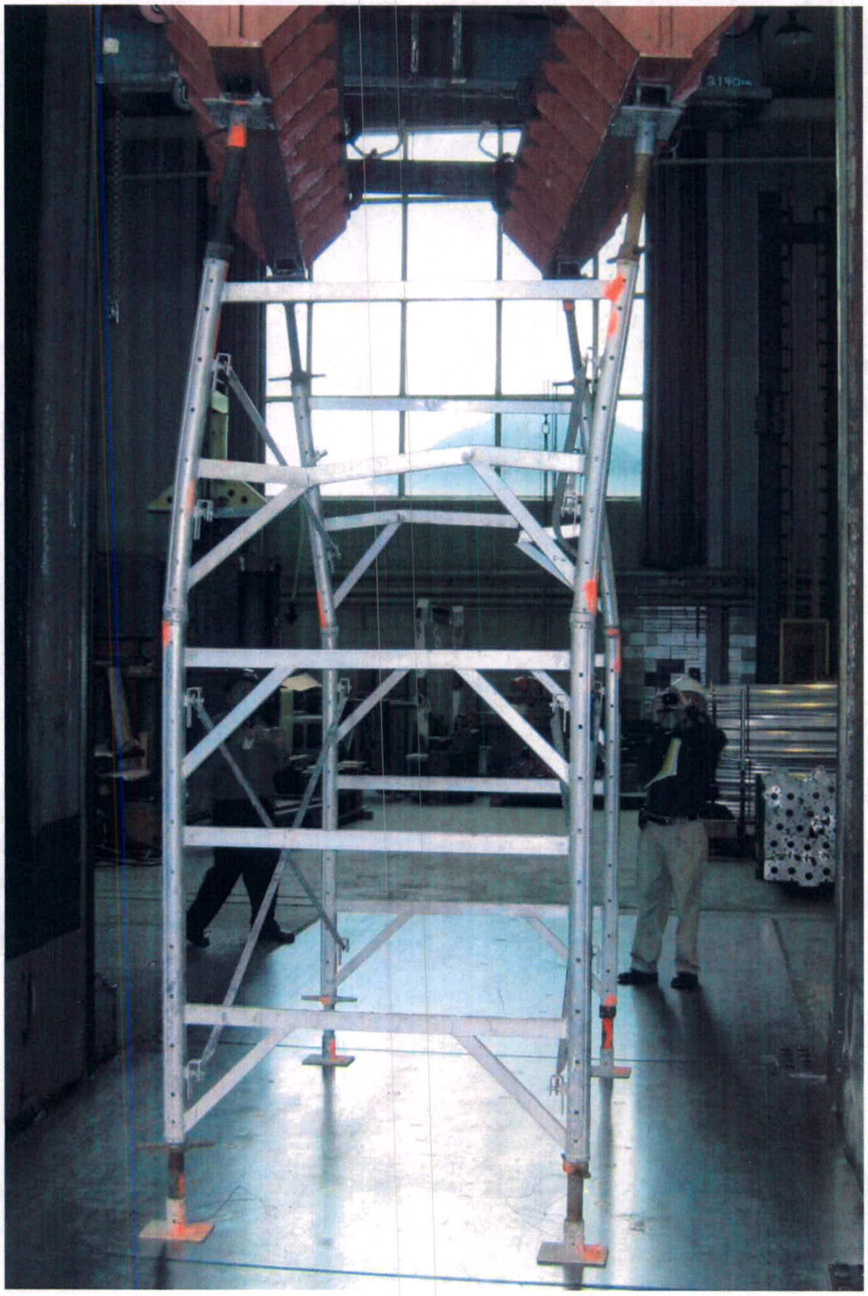


Figure 4: Tower 2 Post Test

Figure 5: Tower 3 Pretest

Figure 7: Tower 4 Pretest

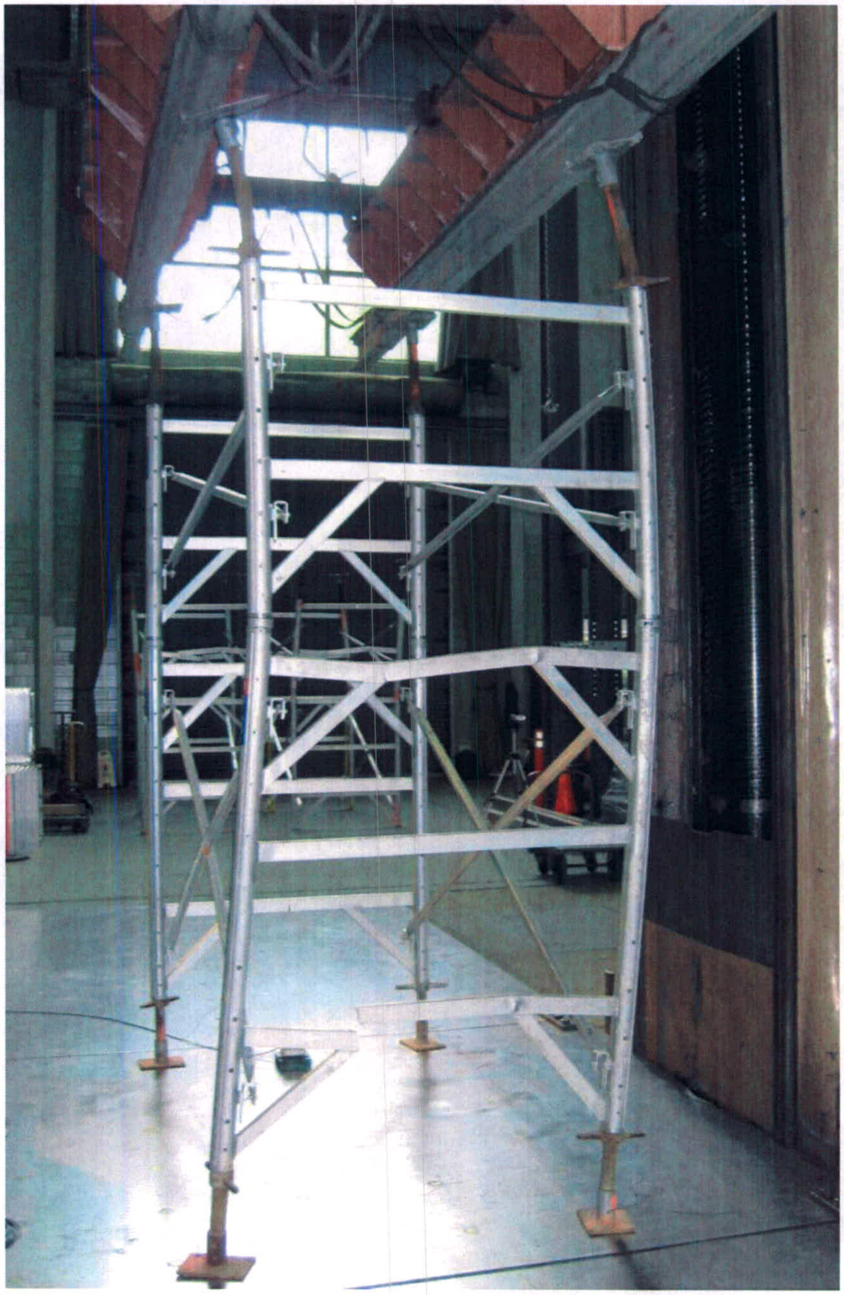


Figure 8: Tower 4 Post Test

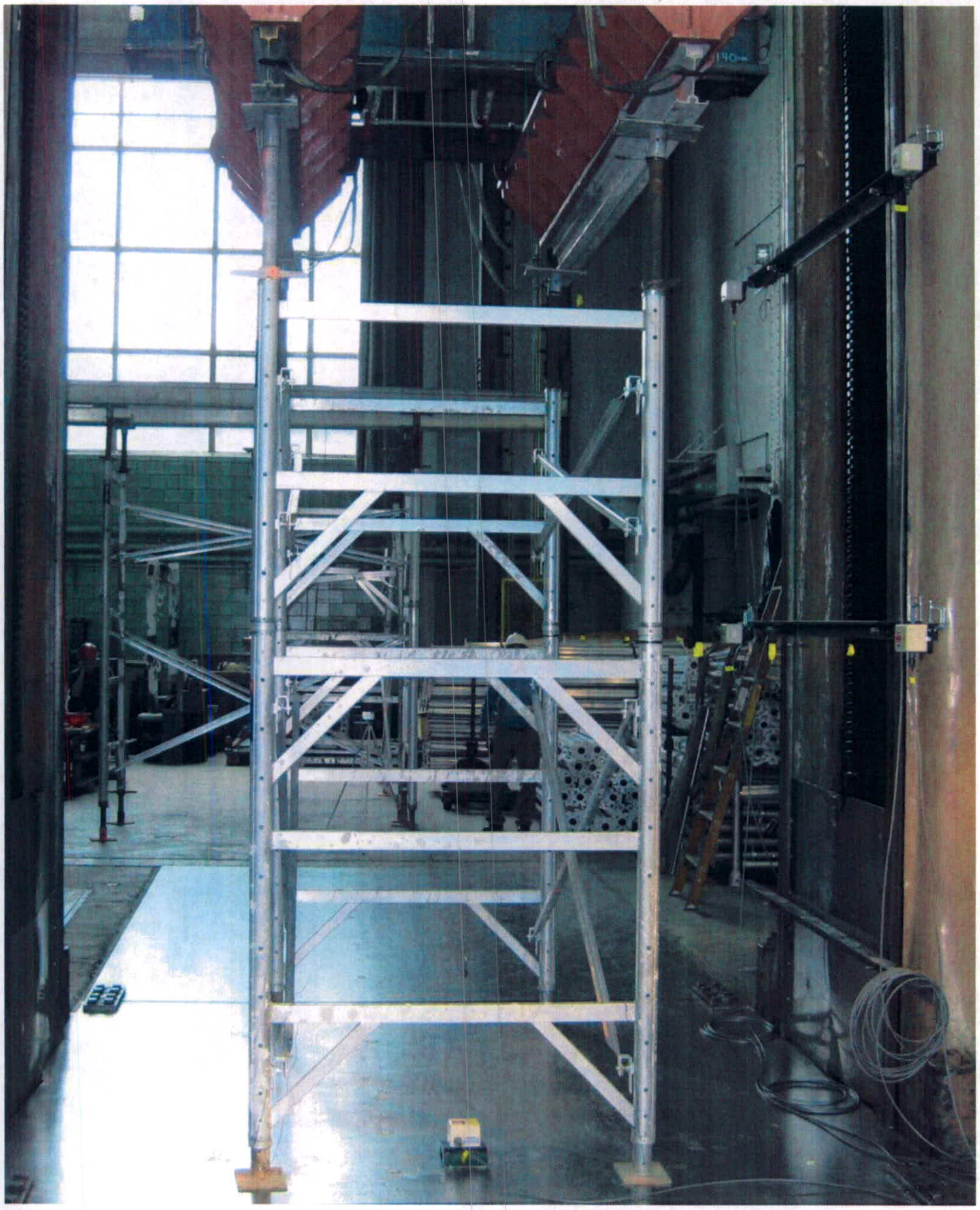


Figure 9: Tower 5 Pretest

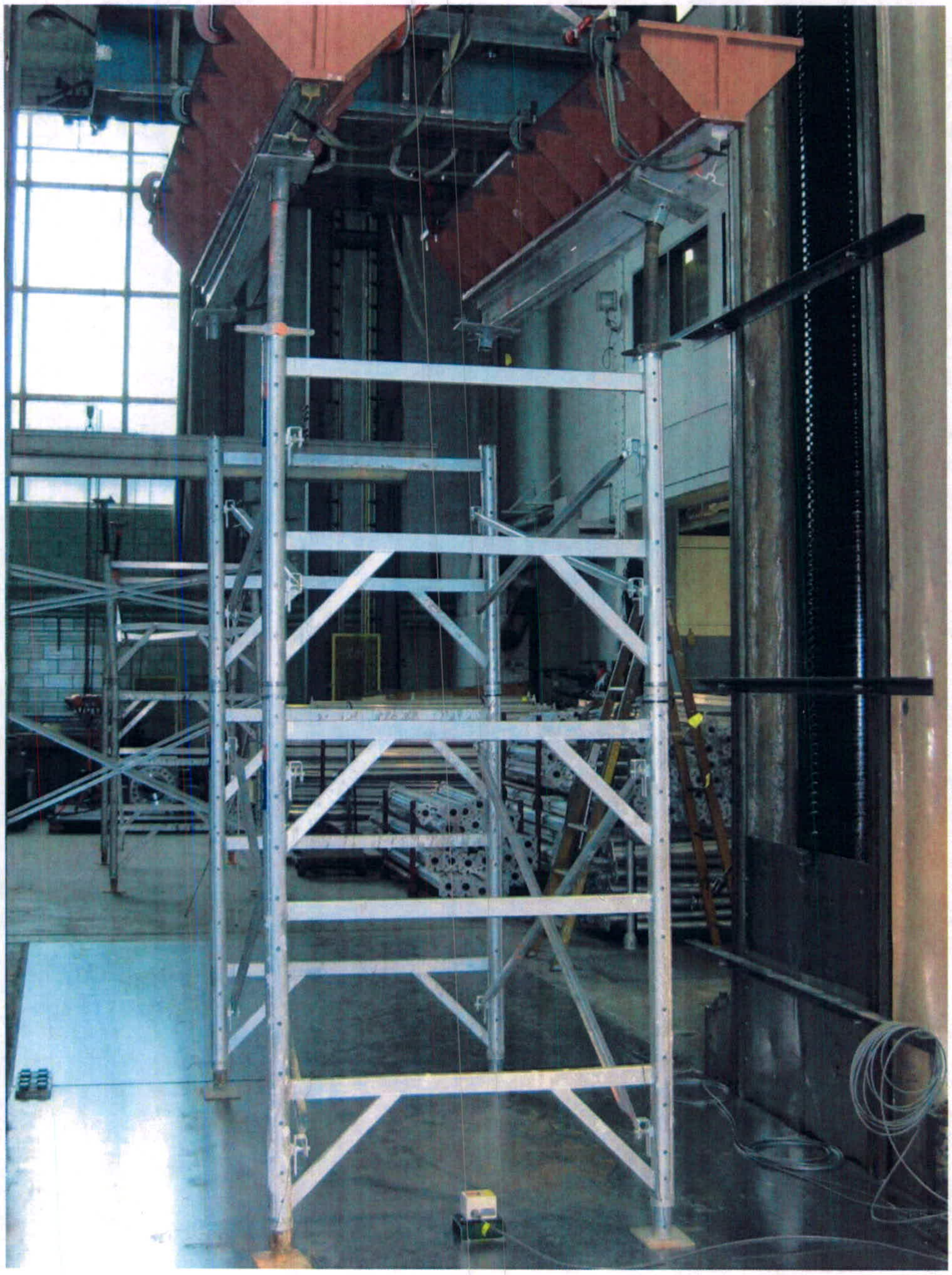


Figure 10: Tower 5 Post Test

12

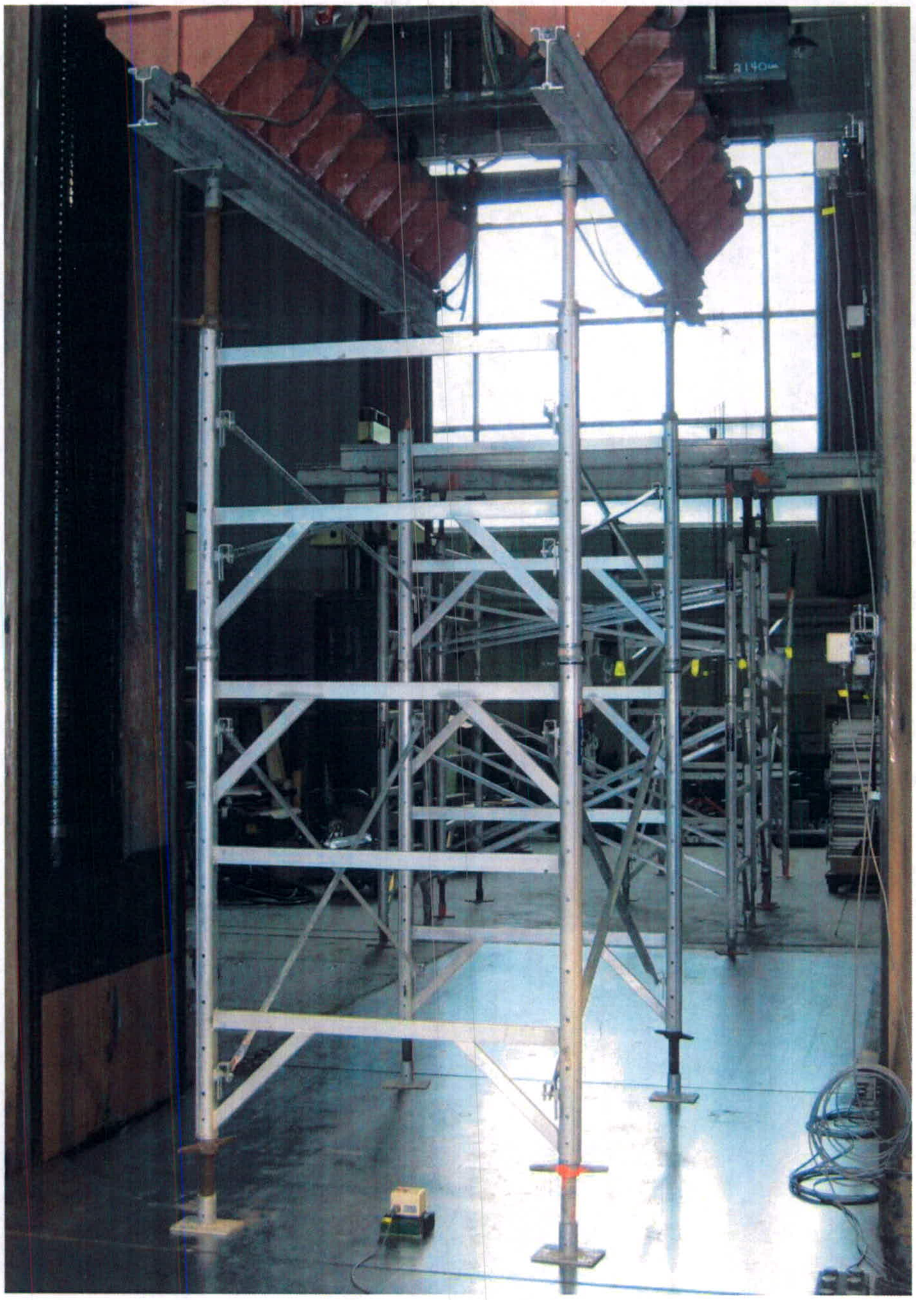


Figure 11: Tower 6 Pretest

13

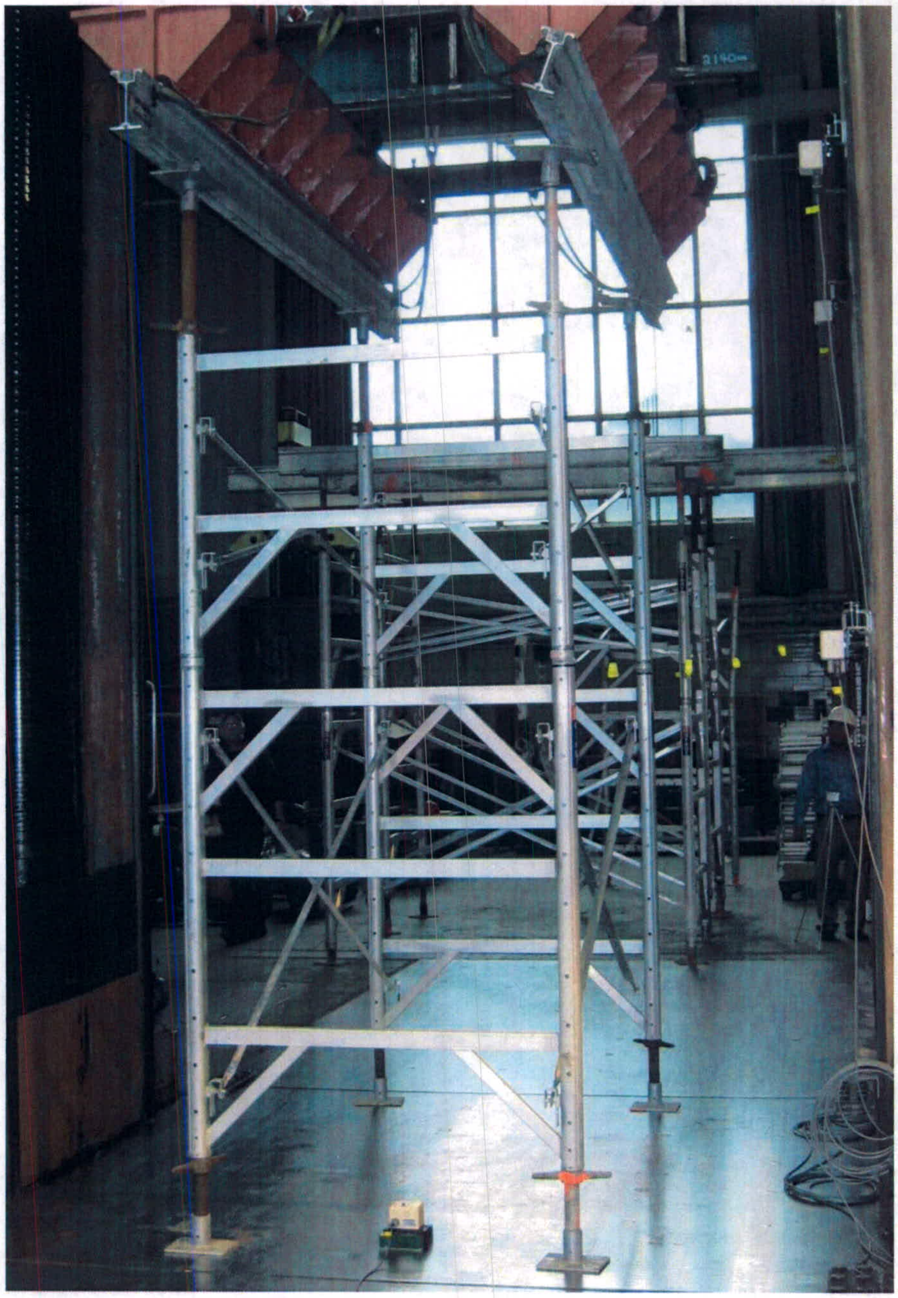


Figure 12: Tower 6 Post Test

14

The locations of the sensors used for the testing are given using the cardinal directions. Figure 13 Shows the North and South directions relative to the 5,000K testing machine. Figure 14 shows the five string pots and their designations.

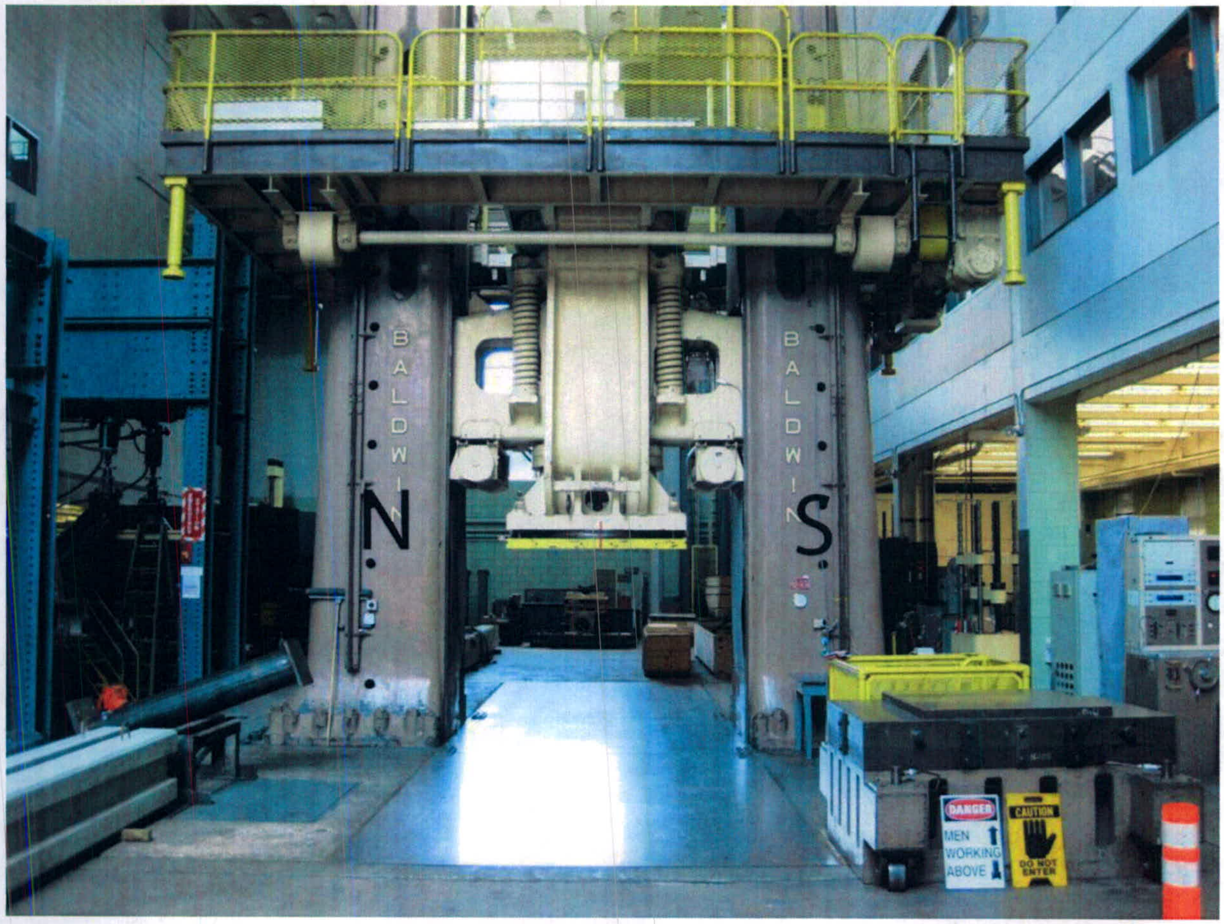


Figure 13: 5,000K Machine with Columns Labeled N for North and S for South West is in front of the machine, and East is in the back

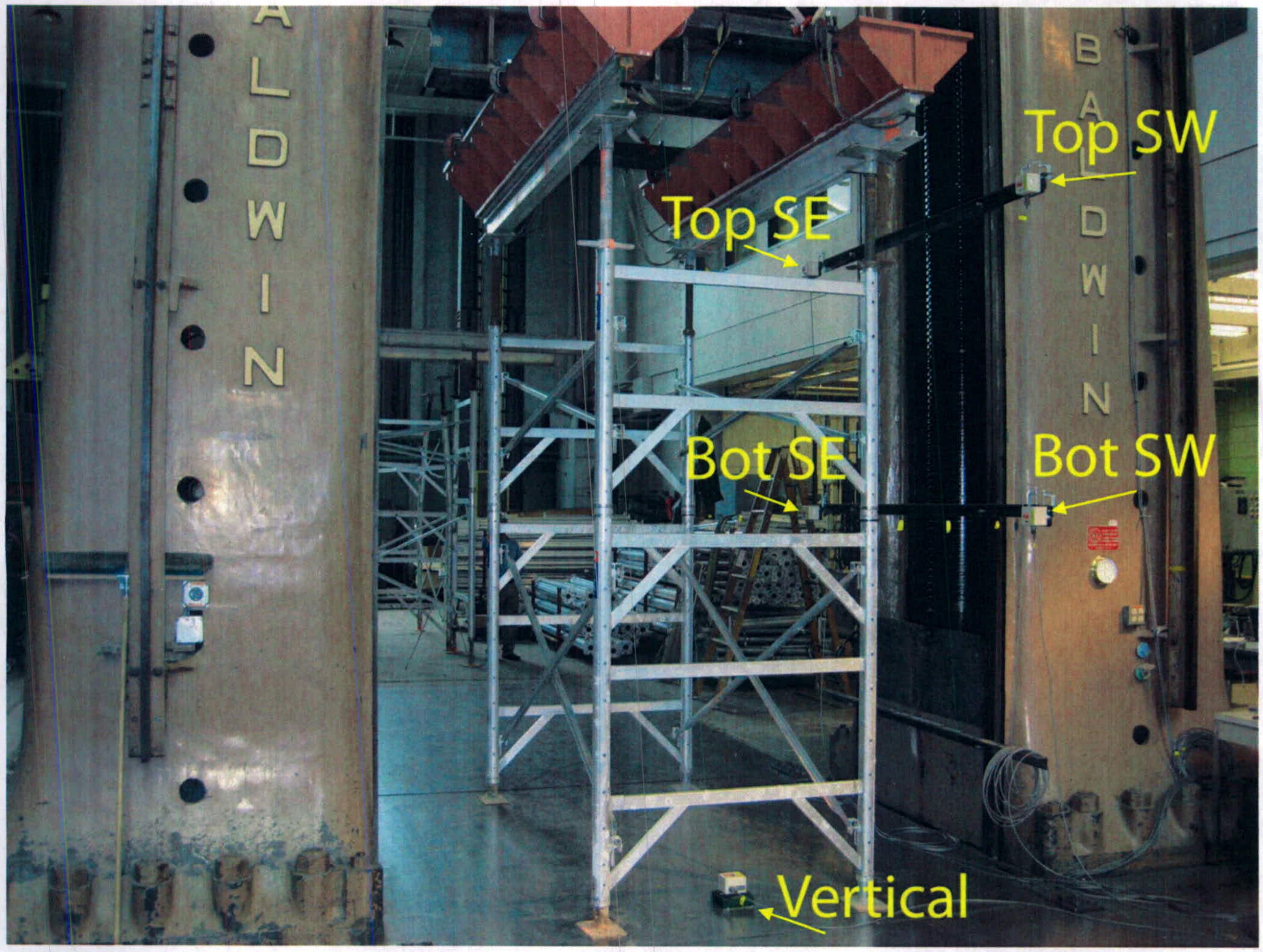


Figure 14: Location of String Pots

Included with this report is a CD containing the 5,000K testing machine calibrations certificate, additional test pictures, load deflection plots for the three eccentric load tests and an electronic copy of this report.

Sincerely,

Robin J. Hendricks

Cc: Frank E. Stokes – ATLSS

Frank E Stokes

The following disclaimer belongs on the back of the first page:

The results of the project presented in this report are provided on an "AS IS" basis. University makes no warranties of any kind, express or implied, as to any matter whatsoever, including, without limitation, warranties with respect to the merchantability or fitness for a particular purpose of the project or any deliverables. University makes no warranty of any kind with respect to freedom from patent, trademark, copyright or trade secret infringement arising from the use of the results of the project, deliverables, services, intellectual property or other materials provided hereunder. University shall not be liable for any direct, indirect, consequential, punitive, or other damages suffered by Sponsor or any other person resulting from the project or use of any deliverables. Sponsor agrees that it shall not make any warranty on behalf of University, express or implied, to any person containing the application of the results or any deliverables of this project.

APPENDIX "B"
Wood Formwork Testing and Report
Wood Advisory Services

ces, Inc.

Submitted to:

Mr. Dan Eschenasy, PE

Chief Engineer

NYC Department of Buildings

280 Broadway • 7th Floor

New York, NY 10007

Evaluation of Concrete Formwork Associated with the 42nd Floor Collapse at 246 Spring Street • New York, NY

August 12, 2009

WOOD ADVISORY SERVICES, INC.

PO Box 1322

3700 Route 44

Millbrook, NY 12545

08.125.01

TABLE OF CONTENTS

1. Scope

Evaluate the lumber and plywood associated with the 42nd floor concrete formwork collapse at 246 Spring Street, New York, NY. Inspect and determine the visual grades of the lumber, document grade stamps observed, perform microbiological analyses to verify the presence or absence of wood decay, and perform mechanical testing to determine residual property values. Provide current allowable working stresses for the lumber and the estimated ultimate strength of the lumber at the time of the collapse.

Additionally, inspect and document any grade stamps on the plywood formwork and perform appropriate mechanical testing on the plywood based on the visual inspections. Provide summaries and conclusions of our evaluation of the plywood formwork.

2. Introduction

Wood Advisory Services, Inc. (WAS, Inc.) was retained by the New York City Department of Buildings (NYCDOB) to evaluate the lumber and plywood concrete formwork associated with the 42nd floor collapse at 246 Spring Street, New York, NY. On-site inspections were performed by WAS, Inc. under the supervision of the NYCDOB on three different days. Other parties present during the WAS, Inc. inspections included representatives from Thornton Tomasetti and Bovis Lend Lease. The WAS, Inc. inspection dates and personnel present were as follows:

Inspection Date:	Representative Present:
May 28, 2008	Mr. Matthew Anderson
June 2, 2008	Mr. Matthew Anderson, Dr. Albert L. De Bonis
June 4, 2008	Mr. Matthew Anderson, Dr. Albert L. De Bonis

3. Inspections

During three separate on-site inspections, representatives from WAS, Inc. inspected all of the lumber and plywood which was being stored on the 40th floor, as well as, all of the lumber and plywood stored in two separate shipping containers at street level. The visual characteristics of all the plywood and lumber stored were documented. Any grade stamps observed were documented and as much of the lumber was visually graded as possible. Photographs were taken at the time of inspection and those photographs referenced in this report are provided in Appendix I.

3.1 Dimension Lumber

The lumber was visually graded according to the National Grade Rule for Dimension Lumber, also referred to as the NGR. All of the accredited lumber agencies in the United States and Canada, visually grade dimension lumber according to the NGR, regardless of species. A majority of the lumber inspected at Spring Street were nominal 3x4s with actual dimensions of approximately 2-1/2" x 3-1/2". Occasionally, a nominal 4x4 (i.e. 3-1/2" x 3-1/2"), or a scaffold plank, 2 x10 (i.e., 1-1/2" x 9-3/4"), was observed. In total, during the inspection, nearly 400 fragmented pieces of broken lumber were inspected.

In order to determine the visual grade of a single piece of dimension lumber, all four sides of the lumber, the full length, and at least one end must be visible. This is the only way, following the NGR, that a piece of lumber can be assigned a visual grade. However, most of the material inspected at Spring Street was in fragmented, broken pieces, essentially making the visual grading process impossible by industry standards. Therefore, whenever WAS, Inc. representatives inspected a piece of broken material, the grade controlling defect (such as knots or slope-of-grain), also referred to as the GCD, of that piece was documented and the grade of that fragmented piece was assigned based on that GCD. If no GCD was visible, then no visual grade was assigned to that piece and the piece was classified as no GCD, or "NGCD." Of the 396 fragmented pieces of lumber inspected, 184 were classified without a grade controlling defect. One could argue that these unclassified pieces should have been classified as select structural. However, it was our intent to determine the approximate distribution of actual grade characteristics and assign grades based on the worst characteristic observed. Therefore, arbitrarily assigning these pieces a grade of select structural would have no benefit to the objectives of this project.

Following the completion of this project, WAS, Inc. was provided Patent Construction Systems Drawing Number 4607K070 which appears to be the lumber and plywood specifications for the project at 246 Spring Street. A copy of this document is provided in Appendix V. A review of the lumber design values in this document and published allowable stresses for structural lumber indicated that the dimension lumber (3x4 and 4x4) should have been at a minimum No. 1 & Better (BTR) grade of a species grouping such as Douglas fir-larch, or a No. 2 dense southern pine. The current allowable stresses for these species groupings are published in the Supplement to the National Design Specification for Wood Construction. The published size adjusted base values are provided below. The inclusion of the spruce-pine-fir species grouping here will become evident subsequently in this report. In summary. the dimension lumber used on this project was spruce-pine-fir mill run quality. However, even select structural spruce-pine-fir will not meet the specifications.

Property	Drawing No. 4607K070 Required Design Values	Douglas fir - Larch No.1 & BTR	Southern Pine No. 2 Dense	Spruce - Pine - Fir Select Structural
F_b	1,640 psi	1,800 psi	1,700 psi	1,875 psi
F_v	180 psi	180 psi	175 psi	135 psi*
$C_\perp$	625 psi	625 psi	660 psi	425 psi*
$C_{/\!/}$	1,350 psi	1,783 psi	1,850 psi	1,610 psi
MOE	1,600,000 psi	1,800,00 psi	1,700,00 psi	1,500,00 psi*

* Values more than 5% below specified allowable property

3.2 Lumber • 3x4

A detailed list of the lumber visual grades observed during our inspection is provided in Appendix II. The 3x4 lumber was graded as "Structural Light Framing" and also as "Light Framing." These two visual grade classifications have slightly different allowable characteristics and size adjustments for design calculations. A summary of the visual grading results are provided below, however, this data is not directly applicable for use in design calculations due to the fact that wood decay was observed in many fragmented pieces and due to the levels of the resulting strength reductions found during mechanical testing. Both of these issues are discussed in sections 5.2 and 6.1 of this report.

- Total number of lumber pieces inspected = 396
- Visual grade summary of 3x4s - Structural Light Framing (2"- 4" thick, 2"- 4" wide)

Visual Grade	Pieces	Percentage
Select Structural	59	28%
No. 1	50	23%
No. 2	41	19%
No. 3	48	23%
Economy	14	7%
TOTAL	212	100%

- Total number of pieces inspected = 396
- Visual grade summary of 3x4s - Light Framing (2"- 4" thick, up to 4" wide).

Visual Grade	Pieces	Percentage
Construction	159	75%
Standard	39	18%
Utility	----	----
Economy	14	7%
TOTAL	212	100%

In addition to determining the visual grade of each piece of lumber, the failure characteristics of all the pieces, if present, were documented. Additionally, we documented factory cuts, field cuts, and holes when present even though they are not failure characteristics. The characteristics were defined as follows:

B	= Brash; consistent with decay or low specific gravity,
BK	= Brash (w/saw kerf),
T	= Tension, or typical bending failure,
BT	= Brash/tension; combination of B and T,
F	= Factory cut end (not an actual failure),
FC	= Field cut end (not an actual failure),
C	= Compression failure, and
H	= Hole cut into the lumber (not an actual failure).

Two summaries are provided below. The first summary represents all of the documented characteristics and the second summary represents only the failure characteristics (i.e., B, BK, T, and BT).

Summary of All 3x4 Lumber Characteristics Observed

Characteristic	Count	Percentage
B	86	11.5%
BK	3	0.5%
BT	182	25%
F	188	25.5%
FC	96	13%
T	168	23%
C	1	0%
H	10	1.5%
TOTAL	734	100%

Summary of all the 3x4 Failure Modes Only

Characteristic	Count	Percentage
B	86	20%
BT	182	42%
T	168	38%
TOTAL	436	100%

When the results from the previous 3x4 data were examined, two observations became apparent. First is the high percentage of low grade material (i.e., numerous economy pieces, No. 3, and Standard) and second is the high percentage of brashness present in the material examined (i.e., B and BT classifications). Brashness in wood is characterized by a fracture which progresses more or less across the grain (see photos 1 - 3) as opposed to failures that follow the grain. The presence of a brash failure is often associated with wood decay and/or pieces of wood with low specific gravity values. Brash failures are not typical in normal wood.

Two different visual grade classifications were provided in the summary tables for the 3x4 lumber. Visual grades were provided for Structural Light Framing, which includes Select Structural (SS), No. 1, No. 2, and No. 3, and apply to lumber that is 2 to 4 inches thick and 2 inches and wider. The use of visual grade design values for these grades requires a size adjustment be applied to the basic values which can be found in Table 4A of the National Design Specifications (NDS) for Wood Construction. The factors listed in Table 4A would be used to adjust the published characteristic value for 12" wide lumber to the 4" nominal width of the 3x4 lumber. The other visual grade classification provided in this report is Light Framing, which includes of Construction, Standard, and Utility grades for dimension lumber 2 to 4 inches thick and up to 4 inches wide. Use of these visual grades does not require an adjustment for size.

Dimension lumber which does not meet the minium requirements for either No. 3 (Structural Light Framing) or Utility (Light Framing) were classified as Economy grade, and have no published design values. One of the most distinct grade controlling characteristics documented during our inspections was slope-of-grain (SOG). We observed severe SOG in numerous pieces (photos 4 - 6) which resulted in down grading the lumber to either a No. 3 or Economy grade.

During our inspection, we did not observe any grade stamps on any of the 3x4 lumber examined. However, the name "LAUZON" (photo7) was observed on one single piece. This stamp appears to be associated with Marcel Lauzon, Inc., a Canadian company which produces lumber, including 3x4 lumber, just north of the Vermont border in Quebec. Additionally, numerous pieces of the 3x4 lumber were examined microscopically and two distinct species were identified, spruce and eastern-fir. Therefore, the lumber inspected may be classified as SPF since it appears to be manufactured in Canada. If the material were manufactured in the US, it would be classified as SPF-s which is assigned different design stresses then SPF. Additionally, since no grade stamps were observed on any of the fragmented pieces of lumber and the grading characteristics were so variable, the material was most likely "mill-run" lumber.

Even after classifying the visual grades for the 3x4 lumber population, the published design values would not apply to this material due to the presence of wood decay. Evidence of wood decay was visually observed in the lumber by the presence of brash failures and surface mycelium (photos 8, and 9). Verification of the presence of wood decay is provided and discussed in section 5.1 of this report, and the reductions in strength properties, as a result of the wood decay, are discussed in section 5.2. The overall allowable design stresses and the estimated ultimate stresses in the 3x4 lumber at the time of the collapse are also provided and discussed in section 6.1.

3.3 Lumber • 4x4s

Twenty pieces of nominal 4x4 lumber were inspected. Two grade stamps were observed on two different pieces. The stamps were as follows:

1. STAND&BTR, D-FIR, S-GRN, PLIB (photo 10), and
2. NO. 2, S-GRN, WWPA, D-FIR, MILL 266 (photo 11).

For 13 of the 4x4s, we were able to determine the visual grades. The results were as follows:

Visual Grade	Pieces	Percentage
Select Structural	1	7%
No. 1	3	23%
No. 2	8	63%
No. 3	1	7%
TOTAL	13	100%

All of the characteristics observed would have qualified for the STAND&BTR grade, including the sample graded as a No. 3. The No. 3 had a SOG of 1:7, which also qualifies as STAND&BTR.

Similar to the 3x4 lumber, WAS, Inc. personnel documented the characteristics of each 4x4. Nearly 60% of the material had factory cut ends or field cut ends. The actual failure modes included 3 brash, 8 combination brash/tension, and 4 tension only. The results are similar to the 3x4 lumber, with nearly 75% of the failures exhibiting brashness or a combination of brash and tension. These numbers are based on a significantly smaller population than the 3x4s, but never the less, indicate there may be decay present in some of the 4x4s, although we did not visually observe any mycelium on the surface of any of these members when we performed our inspection.

Summary of all the 4x4 Lumber Characteristics Observed

Characteristic	Count	Percentage
B	3	8.3%
BK	----	----
BT	8	22.2%
F	18	50%
FC	3	8.3%
T	4	11.1%
C	----	----
H	----	----
TOTAL	36	100%

Summary of all the 4x4 Failure Modes Only

Characteristic	Count	Percentage
B	3	20%
BT	8	53%
T	4	27%
TOTAL	15	100%

3.4 Other Lumber

There were two nominal 2x10 planks and two nominal 1x4s inspected as well. One of the 2x10 planks qualified as a No. 2, and the second plank was an on-grade scaffold plank; SPIB, DI-65, MEETS SCAFFOLD KD 19 OSHA 1910.28, #350. Neither of the 1x4s were visually graded since they do not qualify as dimension lumber. No evidence of wood decay was observed on either 2x10 or either 1x4.

3.5 Moisture Content of Lumber

The NYCDOB had requested that WAS, Inc. personnel provide an opinion regarding the storage conditions of the lumber prior to our inspection, and if that storage had any impact on our findings. Moisture content (MC) levels during our on-site inspection were consistently below 20%, which is generally considered the minimum threshold required to facilitate the growth of wood decay. During our three day inspection, we recorded nearly 60 MC values and only eight were above 20%, with the highest value recorded at 24.5%. The lumber we inspected was stored in clean shipping containers from February through May. The months of February and March consistently have temperatures below 60°F, whereas April and May would have temperatures more favorable to wood decay growth. However, most of the lumber testes exhibited MC values below the 20% threshold. In addition to adequate moisture levels, wood decay also requires an optimal temperature of approximately 60°F or greater. Below 60°F growth slows significantly. Wood decay can occur between 32°F and 60°F, but growth is limited particularly at the lower temperature levels. Storage in the containers would have significantly restricted drying of the lumber since the containers were enclosed with little ventilation or air circulation. Therefore, we wouldn't anticipate a significant difference in MC levels between February, when the lumber was initially stored, and May, when we inspected the lumber. Relative humidity values of 59% and 62% were recorded in both containers during our inspection which is equivalent to a MC level of approximately 12% at 70°F.

One final and critical piece of evidence we observed was the presence of wood decay on 3x4 lumber ribs which were still in use on the job site during our inspection. These ribs had not been kept in storage for four months (photos 12 and 13), and yet contained some mycelium growth on the surface similar to some of our observations of stored material.

Based on all the evidence discussed above, it is our professional opinion that the storage of the material between February and May had no effect on the dimension lumber with respect to promoting wood decay. The evidence gathered during our inspection and scientific literature do not warrant any reason to suggest otherwise.

3.6 Plywood

During our three day inspection, we inspected over 100 broken pieces of plywood of varying sizes. All of the concrete formwork was 5-ply plywood with melamine overlays on both faces which were marked either "Feldman Lumber" (photo 14), or "Mid-South Lumber Company" (photo 15). No visible grade stamps could be documented for this material since both faces were covered with a melamine overlay. One grade stamp was documented on several panels which did not have melamine overlays. The stamp read; Futter, Mill 332, Uni-Form, B-B Class (photo 16).
Several of the broken formwork panels had consistent linear failures which appeared to be punch-through failures (photos 17 - 19). Based on this consistent pattern, it appeared that these failures were caused by the steel column base plates (6" x 8") which had punched through the plywood panels. As a result of this observation, the NYCDOB requested that a concentrated load test be performed on the plywood to examine the load carrying capacity of the plywood formwork using a 6" x 8" steel column base plate retained from the site, and 3x4 framing at 16" on-center spacing.

Following the completion of this project, WAS, Inc. was provided Patent Construction Systems Drawing Number 4607K070 which appears to be the lumber and plywood specifications for the project at 246 Spring Street. A copy of this document is provided in Appendix V. A review of this specification indicates that the plywood, in order to conform to the specification, needed to meet the requirements of the APA, The Engineered Wood Association specifications for plyform class I, with B-B grade face & back veneers, and exterior adhesive and meet the requirements of PS1-95.

4. Laboratory Procedures

During our on-site inspection, we randomly selected some samples and systematically selected other representative samples for mechanical testing, microbiological analyses, and wood species identifications. At the request of the NYCDOB some of the 3x4 pieces were randomly selected for analysis using a random number generator. Other samples were specifically selected by WAS, Inc. personnel based on the presence of brashness and evidence of wood decay. Two groups of test specimens were selected for the laboratory analyses; the 3x4 lumber ribs and plywood. The plywood selected was neither randomly selected nor systematically selected. They were simply the only non-damaged full size sheets available for testing. All the samples retained were marked by representatives from the NYCDOB and Thornton Tomasetti. Each sample was returned to our laboratory by representatives form WAS, Inc. and were marked as follows:

3x4 Lumber Ribs	**Plywood**
TP1-A	TP8-A
TP2-A	TP9-A
TP3-A	TP10-A
TP4-A	PW4055
TP5-A (randomly selected)	
TP6-A (randomly selected)	
TP7-A (randomly selected)	
TB40215	
TB40146	
TB40244	
TB40189	
TB40143	

4.1 Microbiological Analyses

At WAS, Inc., microbiological activity, including evaluation of the presence of wood decay fungi, evidence of bacterial attack and the presence of sap stain fungi, and the condition of the wood cellular structure are conducted using a light microscope and the deterioration is documented using two simple classification systems.

Wood decay (i.e., the levels of hyphae observed microscopically) is classified as one or a combination of the following six categories: "none", "occasional", "light", "moderate" , "heavy", or "very heavy."

Category	Definition of Classification
None	Describes the lack of wood decay hyphae or evidence of wood decay in any view. Hyphae are root-like structures of wood decay fungi. The category "none" does not mean that no hyphae or evidence was present in the entire sample. The category "none" simply means that none were observed in the limited sections viewed microscopically.
Occasional	Describes the observation of hyphae in limited regions of the wood cells, but they were not observed in every view.
Light	Describes the observation of a limited network of hyphal growth or evidence of deterioration, but was not observed in every view.
Moderate	Describes an organized network of hyphae or evidence which was observed in most views but is limited to certain cells.
Heavy	Describes an organized network of hyphae or advanced evidence which was observed in every view and in practically all cells.
Very Heavy	Describes an extremely advanced network of hyphae or evidence which was observed in every view and in all cells.

Bacterial attack is difficult to analyze with a light microscope because of the extremely minute size of bacteria and their effects. However, some limited evidence of their attack can sometimes be observed and the evidence is classified as either present or absent. Sap stain fungi do not cause cellular degradation and they are classified as either present or absent.

A second simple classification system, used at WAS, Inc., describes the wood cellular structure of the microbiological samples. Four classes are used: "good", "fair", "poor", and "very poor."

Class	**Definition of Classification**
Good	Represents typical wood cellular structure.
Fair	Represents a condition where the structure of some wood cells has been compromised.
Poor	Represents the condition where the structure of most wood cells has been compromised.
Very Poor	Represents the most compromised level of cellular structure in which many types of wood cells are unrecognizable.

There are times when sectioning difficulties can occur during the preparation of microbiological samples and results in a "fair" classification of cellular structure, which may not indicate actual cellular deterioration. However, sectioning difficulties can also be indicative of the presence of wood decay. All analyses were conducted using a compound light microscope at magnifications of 100 to 400x.

All results from the microbiological analyses will be presented in Section 5.1 of this report.

4.2 Mechanical Testing

4.2.1 Lumber • 3x4

The mechanical testing methodology for evaluating the 3x4 lumber was performed in accordance with ASTM D 143, "Standard Test Methods for Small Clear Specimens of Timber." The mechanical properties evaluated, as directed by the NYCDOB and Thornton Tomasetti, were:

1. Modulus of Rupture (MOR) or bending strength,
2. Modulus of Elasticity (MOE),
3. Shear Parallel-to-Grain, (F_v) and
4. Compression perpendicular-to-grain ($C_\perp$).

Determination of MOR and MOE was performed in accordance with Section 8, "Static Bending." Shear parallel-to-grain was performed in accordance with Section 14, "Shear Parallel-to-Grain," and the compression tests were performed in accordance with Section 12, "Compression perpendicular-to-grain." All the tests were conducted on a Tinius Olsen, 10,000 lb. capacity universal bench top test machine which is calibrated annually and has not required any load cell adjustment in over 10 years (all certifications are available at our offices upon request).

During the week of September 15, 2008, numerous parties were present to witness the fabrication and testing of all the small clear test specimens. Microbiological analyses and wood species identifications were performed on September 18 and 19, 2008. The parties present and the dates they were present are as follows:

Name	Dates Present	Company
Mr. Matthew Anderson	09/15, 09/16, 09/17, 09/18, 09/19	Wood Advisory Services, Inc.
Dr. Albert De Bonis	09/15, 09/16, 09/17, 09/18, 09/19	Wood Advisory Services, Inc.
Ms. Caitlin Kevins	09/15, 09/16, 09/18, 09/19	Thornton Tomasetti
Mr. Perides Stivaros	09/15, 09/16	Feld, Kaminetzky & Cohen
Mr. Luis Valderruten	09/15, 09/17	Thornton Tomasetti
Mr. Guo Zhan (John) Wu	09/15, 09/16, 09/17	NYCDOB
Mr. Carl Schoenberger	09/17, 09/18, 09/19	Feld, Kaminetzky & Cohen
Mr. Walter Karon	09/18	R.L. Grunos
Mr. Ari Bauer	09/18	Dillinger, Miller & Torallo

All the results for the small clear tests were compared to historical small clear data published for spruce and eastern-fir grown in Canada as published in ASTM D 2555, "Standard Practice for Establishing Clear Wood Strength Values." Residual properties were determined by comparing the mean value of the test results to the mean value of the published data for that particular species. The test data obtained using the procedures described in this section will be discussed in Section 5.2 of this report

4.2.2 Plywood Concentrated Load Testing

Four 4' x 4' plywood samples were selected during our inspections to evaluate the capacity of the material when a load was applied to the plywood using one of the 6" x 8" steel column base plates. The panels were selected because they were the only large size panels we inspected which exhibited no apparent visible damage. A consistent and unique failure pattern was observed in several of the broken plywood panels inspected which was consistent with a punch through failure of the steel base plates (photos 17 - 19). Therefore, we elected to perform a modified ASTM test procedure to apply a concentrated static loading specific to the observed failure characteristics of the concrete formwork associated with the collapse at 246 Spring Street. This test method was designed to be used in conjunction with the APA - The Engineered Wood Association (APA-EWA) concentrated load requirements which require the application of a static concentrated load through a 3" diameter disk to minimum failure loads of 400 - 550 lbs. for roof and floor sheathing, not for concrete formwork panels. Therefore, our in-house plywood test assembly was modified to simulate the particular concentrated loads applied to the plywood concrete formwork assembly constructed at 246 Spring Street. Additionally, using the APA-EWA test standard, deflections are measured up to a 200 lb. concentrated load level. The NYC DOB requested deflection be measured up to and including punch through failure. Therefore, our in-house equipment also needed to be additionally modified to accomplish deflection measurement throughout the duration of each test.

The load was applied manually using a hand pump. The hydraulic capacity of the ram was rated to 10,000 lbs. maximum capacity. A test frame was constructed to imitate the 3x4 concrete formwork framing used at 246 Spring Street. Nominal 4x6 lumber was bandsawed at WAS, Inc. to a width of 2-1/2" which was the actual width of the 3x4 lumber ribs used at 246 Spring Street. Two types of loading conditions were evaluated as follows:

1. Base plate located at center of 16" span, interior edge of plate 11" from panel edge.
2. Base plate located at center of 16" span, interior edge ~24-26" from panel edge.

To calibrate the test assembly, a 10,000 lb. proving ring was used which was calibrated prior to the testing. Using the proving ring, we were able to develop an accurate linear correlation between the digital meter output display and the proving ring increments. Therefore, we were able to convert the digital meter outputs to actual failure loads.

Deflections were measured by projecting a laser beam on an engineering scale which was attached to the concentrated load base plate. Deflections were recorded at various load levels throughout the test until punch through occurred.

The concentrated load testing was performed at the laboratories of WAS, Inc. on January 21, 2009, and the following parties were present:

Name	Company
Mr. Matthew Anderson	Wood Advisory Services, Inc.
Dr. Albert De Bonis	Wood Advisory Services, Inc.
Mr. Luis Valderruton	Thornton Tomasetti
Mr. Carl Schoenberger	Feld, Kaminetzky & Cohen
Mr. Walter Karon	R.L. Grunos
Mr. Guo Zhan (John) Wu	NYCDOB

The test data obtained using the procedures described here will be subsequently discussed in Section 5 of this report and specifically in Section 5.3.

5. Results

The results of the microbiological analyses, wood species identifications, and the mechanical testing, are discussed in the following three sections (5.1, 5.2, and 5.3). Summaries of the laboratory analyses are provided in Tables 1, 2, 3a - 3f, and 4a - 4b, and the detailed results are provided in Appendices III and IV.

5.1 Microbiological Analyses & Wood Species Identifications

A total of twelve, 3x4 lumber cross sections were microscopically evaluated for wood species and the presence of wood decay. Each sample was evaluated using a cross sectional grid with 11 different locations. This provided the opportunity to evaluate the entire cross section. A schematic of the cross sectional grid and the 11 locations examined for each sample is provided in Appendix III. A summary of the microbiological analyses and wood species identifications is provided in Table 1.

Locations 1, 2, 3, and 4 (Schematic 1, Appendix III) were first evaluated during each microbiological analysis. If wood decay was observed at any of those locations, then we proceeded to the next depth of ½", or locations 5, 6, 7, and 8. If wood decay was present at any of these locations, we proceeded to locations 9 and 10, or a depth of 1". If applicable, we proceeded to the center, location 11. If no wood decay was documented at any surface location, then we did not evaluate the sample at the next depth. For example, if wood decay was observed at location 1, then location 5 was also examined, and so on. If no wood decay was observed at location 2, then a classification of "none" was provided for location 2 in Table 1, and locations 6 and 9 were marked "N/A" (i.e., not applicable or not analyzed).

A total of twelve cross sections were microscopically evaluated for wood species and microbiological wood decay. Seven of the samples were identified as spruce and five were identified as eastern-fir. There is a distinct microscopic characteristic present in western fir which can be used to differentiate these species from eastern-fir. Dark brown crystal-like extractives are visible in specific cells of western-fir but not eastern-fir. That distinct characteristic was not observed in any of the five samples identified as fir, therefore, each sample was further classified as eastern-fir. There are no distinct characteristics which can be used to differentiate the eastern and western spruce species. As was previously discussed in section 3.2, a "LAUZON" stamp was observed on one of the 3x4 lumber ribs. Research that we conducted found that Lauzon is actually a lumber company in Quebec, Canada and the full name is Marcel Lauzon, Inc. Based on our laboratory analyses and research, all evidence indicates that the 3x4 lumber ribs would be classified as SPF (Canadian design values) and not SPF-s (US design values).

Table 1
Results of the Microbiological Analyses & Wood Species Identifications of Twelve, 3x4 Lumber Cross Sections from the Concrete Formwork Collapse at 246 Spring Street, New York, NY.

Sample	Location	Wood Decay	Cellular Structure	Species
TP1-A	1	None	Good	Spruce
	2	None	Good	
	3	Light	Good	
	4	Moderate-Light	Good	
	5	N/A	----	
	6	N/A	----	
	7	Light	Good	
	8	Occasional	Good	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	
TP2-A	1	Occasional-Light	Good	Spruce
	2	Occasional	Good	
	3	Occasional	Good	
	4	N/A	----	
	5	None	Good	
	6	None	Good	
	7	None	Good	
	8	N/A	----	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	

Table 1 (Continued)
Results of the Microbiological Analyses & Wood Species Identifications of Twelve, 3x4 Lumber Cross Sections from the Concrete Formwork Collapse at 246 Spring Street, New York, NY.

Sample	Location	Wood Decay	Cellular Structure	Species
TP3-A	1	None	Good	Spruce
	2	None	Good	
	3	Occasional-Light	Good	
	4	Occasional-Light	Good	
	5	N/A	----	
	6	N/A	----	
	7	None	Good	
	8	None	Good	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	
TP4-A	1	Moderate	Fair	Spruce
	2	None	Fair	
	3	None	Fair	
	4	Light	Fair	
	5	Light	Good	
	6	Light	Good	
	7	Knot - N/A	----	
	8	Light	Fair	
	9	Light	Fair	
	10	Occasional	Good	
	11	Occasional	Good	
TP5-A	1	None	Good	Spruce
	2	None	Good	
	3	Knot - N/A	----	
	4	Light	Fair	
	5	N/A	----	
	6	N/A	----	
	7	N/A	----	
	8	Occasional-Light	Good-Fair	
	9	N/A	----	
	10	None	Good	
	11	N/A	----	

Table 1 (Continued)
Results of the Microbiological Analyses & Wood Species Identifications of Twelve, 3x4 Lumber Cross Sections from the Concrete Formwork Collapse at 246 Spring Street, New York, NY.

Sample	Location	Wood Decay	Cellular Structure	Species
TP6-A	1	None	Good	Eastern-fir
	2	None	Good	
	3	None	Good	
	4	None	Good	
	5	N/A	----	
	6	N/A	----	
	7	N/A	----	
	8	N/A	----	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	
TP7-A	1	Occasional	Good	Eastern-fir
	2	Light	Good	
	3	Knot - N/A	Good-Fair	
	4	Moderate-Heavy	Fair	
	5	Light	Good	
	6	None	Good	
	7	None	Good	
	8	Light	Fair	
	9	None	Fair	
	10	None	Fair	
	11	N/A	----	
TB40215 (Micro 8)	1	Occasional	Good	Eastern-fir
	2	Occasional	Good	
	3	None	Good	
	4	None	Good	
	5	N/A	----	
	6	N/A	----	
	7	N/A	----	
	8	N/A	----	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	

Table 1 (Continued)
Results of the Microbiological Analyses & Wood Species Identifications of Twelve, 3x4 Lumber Cross Sections from the Concrete Formwork Collapse at 246 Spring Street, New York, NY.

Sample	Location	Wood Decay	Cellular Structure	Species
TB40416 (Micro 9)	1	Occasional	Good	Eastern-fir
	2	Light	Good	
	3	Occasional	Good-Fair	
	4	Light	Good-Fair	
	5	None	Good	
	6	Light	Good	
	7	Occasional	Good	
	8	Occasional	Good	
	9	None	Good	
	10	None	Good	
	11	N/A	----	
TB401244 (Micro 10)	1	Occasional	Good	Spruce
	2	Occasional	Good	
	3	Occasional	Good	
	4	Occasional	Good	
	5	N/A	----	
	6	N/A	----	
	7	N/A	----	
	8	N/A	----	
	9	N/A	----	
	10	N/A	----	
	11	N/A	----	
TB 40189 (Micro 11)	1	Occasional	Good	Spruce
	2	Light	Good	
	3	None	Good	
	4	None	Good	
	5	N/A	----	
	6	Light-Moderate	Good	
	7	Occasional	Good	
	8	N/A	----	
	9	Light-Moderate	Good	
	10	N/A	----	
	11	Occasional	Good	

Table 1 (Continued)
Results of the Microbiological Analyses & Wood Species Identifications of Twelve, 3x4 Lumber Cross Sections from the Concrete Formwork Collapse at 246 Spring Street, New York, NY.

Sample	Location	Wood Decay	Cellular Structure	Species
TB40143 (Micro 12)	1	Light	Good-Fair	Eastern-fir
	2	Light	Good-Fair	
	3	Light	Good-Fair	
	4	Light	Good-Fair	
	5	Occasional	Good-Fair	
	6	Occasional	Good-Fair	
	7	Light	Good-Fair	
	8	Light	Good-Fair	
	9	Occasional	Good-Fair	
	10	Occasional	Good-Fair	
	11	N/A	----	

Evidence of wood decay was documented in eleven of the twelve samples evaluated. Of the 11 samples which exhibited wood decay, five samples contained decay limited to the surface area, and seven samples contained wood decay that ranged from a depth of ½" up to the center, or 1-½" depth.

Sample TP6-A was the only sample which exhibited no evidence of wood decay. Samples TP2-A, TP3-A, TP5-A, TB40215 (micro 8), and TB40244 (micro 10), each exhibited evidence of wood decay which was limited to the surface region (i.e., only locations 1, 2, 3, and 4 exhibited wood decay and no evidence observed at locations 5, 6, 7, and 8). The level of decay documented in the surface region of these five samples was either occasional or light with good cellular structure (see photos 20 and 21).

Seven samples exhibited light to heavy wood decay anywhere from the ½" depth in samples TP1-A, TP4-A, TP7-A, TB40416 (micro 9), and TB40143 (micro 12) (see photos 22 and 23). Evidence of wood decay was observed to depths of 1" and even at the center of samples TP4-A and TB40189 (micro 11).

The presence of wood decay resulted in strength reductions in some of the samples discussed above. Additional discussion of these strength reductions is provided in Section 5.1. Many of the samples chosen by WAS, Inc. personnel were selected because they exhibited evidence of brashness. The presence of wood decay in the samples evaluated, verifies that the brashness observed was due in part to the presence wood decay.

As was previously discussed in Section 3.2, a high percentage of the bending failures documented during our inspection were brash (B) or a combination of brash and typical bending (BT). The laboratory analyses in combination with our inspection observations are important, because the laboratory analyses verified the presence of wood decay in several samples that exhibited brashness. In addition to our inspection of the material in the shipping containers, we also observed brash failures in 3x4 ribs which were currently being used (not a part of the collapse) in the construction formwork system during our on-site inspection of floors 40-42 (see photos 12 and 13). The reduction in strength as a result of the wood decay will be discussed in Section 5.2 and the overall implications associated with the visual grades is provided in Section 6.

5.2 Mechanical Test Results • 3x4 Lumber

Eleven of the 3x4 lumber samples retained by WAS, Inc. were used to fabricate small clear test specimens. The 12th sample (TB40143, or micro 12) was only a short, 12" long section of a failure. Therefore, no test specimens could be fabricated from this sample.

Instead, only microscopic analyses were performed on TB40143 (micro 12) and light levels of wood decay were verified in this sample.

The mechanical properties evaluated for the 3x4 lumber were static bending, MOR or bending strength, MOE, shear parallel-to-grain, and compression perpendicular-to-grain at 0.4" deflection. With each lumber section, we first attempted to fabricate two static bending test specimens (1" x 1" x 16") then, if possible, two shear parallel-to-grain test specimens (2" x 2" x 2-1/2") then, if possible, one compression perpendicular-to-grain test specimen (2" x 2" x 6"). The results of the test specimen fabrication process which includes the numbered and marked test specimens fabricated from each 3x4 lumber sample and for each mechanical properly are provided in Table 2. Missing test specimens are either the result of insufficient material or they were culled due to the presence of strength reducing characteristics.

Table 2
Summary of Fabricated Specimens to be Tested Using
ASTM D 143 Small Clear Procedures & to Undergo Microbiological Analyses.

3x4 Lumber Sample:	Static Bending MOR/MOE	Shear$_{//}$	Compression⊥
TP1-A	1-1 1-3	1-1	
TP2-A	2-2 2-3	2-1	2-1
TP3-A	3-1 3-2	3-1 3-2	3-1
TP4-A	4-1 4-2	4-1 4-2	
TP5-A	5-1	5-1	5-1
TP6-A	6-2	6-1 6-2	6-1

Table 2 (Continued)
Summary of Fabricated Specimens to be Tested Using
ASTM D 143 Small Clear Procedures & to Undergo Microbiological Analyses.

3x4 Lumber Sample:	Static Bending MOR/MOE	Shear$_{//}$	Compression$\perp$
TP7-A	7-1 7-2 7-3	7-1 7-2	7-1
TB40215 (micro 8)	8-1 8-2	8-1	
TB40146 (micro 9)	9-2 9-3		
TB40244 (micro 10)	10-1		
TB40189 (micro 11)	11-1 11-2	11-1 11-2	11-1

A summary of the small clear test results is provided in Tables 3a - 3f. Since two different species were identified in the lumber samples, then two different comparisons were made for each property. The evidence from our inspections, analyses, and research strongly indicates the lumber was manufactured in Canada. Therefore, it was necessary to calculate, for each species, weighted mechanical properties based on the volume of spruce and fir standing timber in Canada published in ASTM D 2555. There are four species of spruce harvested for lumber in Canada and two species of fir. Based on the volume of standing timber for each species group (i.e., spruce and fir), we calculated weighted means for MOR, MOE, shear$_{//}$, and compression perpendicular-to-grain at 0.4". Once our tests were completed, we compared our small clear test results to the historical weighted values published for spruce and fir grown in Canada at an adjusted moisture content (MC) level of 12%. Table 3a and 3b, 3c and 3d, and 3e and 3f are the summarized results for MOR and MOE, shear parallel-to-grain, and compression perpendicular-to-grain at 0.4" deflection for spruce and fir respectively.

As was previously discussed in this report, seven samples were identified as spruce and five were identified as eastern-fir. The results discussed herein are for comparison purposes only. The actual tested strength values listed in each summary table <u>are not</u> to be used as design stresses, working stresses, or ultimate loads for design purposes. Those values will be discussed in Section 6.1 of this report.

The mean MOR and MOE values for the spruce test specimens were 9,623 psi, and 1.46 x 10^6 psi respectively (Table 3a). The mean residual values were 0.88 for MOR and 0.92 for MOE. The range of residual values for MOR was 0.71 to 1.00, and 0.70 to 1.00 for MOE. The mean MOR and MOE values for the fir test specimens were 8,640 psi and 1.47 x 10^6 psi respectively (Table 3b). The mean residual values were 0.98 and 0.96. The range of residual values for MOR was 0.93 to 1.00 and 0.88 to 1.00 for MOE.

The mean shear parallel-to-grain for the spruce test specimens was found to be 1,205 psi with a mean residual value of 0.94. The residual values ranges from 0.75 to 1.00. The mean shear strength for the fir test specimens was found to be 1,126 psi, with a mean residual value of 0.99 and a range of residual values from 0.97 to 1.00.

The compression perpendicular-to-grain results varied considerably for the spruce and fir test specimens. The mean compression at 0.4" deflection was 773 psi with a mean residual value of 0.81. The range of residual values varied from 0.69 to 1.00. The mean compression at 0.4" deflection result for the fir test specimens was 495 psi. Only two compression test specimens could be fabricated from the 3x4 fir lumber samples. The residual values were 0.56 and 0.58 which resulted in a mean value of 0.57.

Table 3
Summary of the ASTM D 143 Small Clear Tests Performed on the 3x4 Lumber Samples from 246 Spring Street (all values reported at a MC of 12%).

3a. Static Bending (Spruce)

Sample	MOR (psi)	MOE (x10^6 psi)	Residual MOR	Residual MOE	SG
1-1	7,842	1.36	0.74	0.90	0.36
1-3	9,447	1.39	0.89	0.92	0.36
2-2	8,110	1.28	0.76	0.85	0.32
2-3	8,053	1.06	0.76	0.70	0.33
3-1	12,303	1.55	1.00	1.00	0.48
3-2	11,768	1.69	1.00	1.00	0.45
4-1	7,550	1.43	0.71	0.95	0.40
4-2	8,530	1.14	0.80	0.75	0.37
5-1	10,561	1.51	0.99	1.00	0.41
10-1	9,693	1.68	0.91	1.00	0.42
11-1	10,555	1.79	0.99	1.00	0.47
11-2	11,070	1.68	1.00	1.00	0.42
Mean	**9,623**	**1.46**	**0.88**	**0.92**	**0.40**

3b. Static Bending (Eastern-fir)

Sample	MOR (psi)	MOE (x10^6 psi)	Residual MOR	Residual MOE	SG
6-2	9,800	1.58	1.00	1.00	0.38
7-1	7,823	1.26	0.94	0.88	0.35
7-2	10,033	1.82	1.00	1.00	0.40
7-3	10,846	1.68	1.00	1.00	0.39
8-1	8,823	1.49	1.00	1.00	0.36
8-2	8,180	1.17	0.98	0.82	0.34
9-2	8,840	1.42	1.00	0.99	0.35
9-3	7,779	1.46	0.93	1.00	0.38
Mean	**8,640**	**1.47**	**0.98**	**0.96**	**0.37**

3c. Shear, (Spruce).

Sample	Shear Strength (psi)	Residual Value	SG
1-1	1,109	0.94	0.33
2-1	891	0.75	0.31
3-1	1,573	1.00	0.42
3-2	1,543	1.00	0.42
4-1	1,178	1.00	0.35
4-2	1,162	0.98	0.37
5-1	1,286	1.00	0.36
11-1	1,000	0.85	0.37
11-2	1,165	0.99	0.38
Mean	**1,205**	**0.94**	**0.37**

3d. Shear, (Eastern-fir).

Sample	Shear Strength (psi)	Residual Value	SG
6-1	1,251	1.00	0.36
6-2	1,244	1.00	0.36
7-1	1,088	1.00	0.36
7-2	1,156	1.00	0.37
8-1	889	0.97	0.33
Mean	**1.126**	**0.99**	**0.36**

3e. Compression $_\perp$ @ 0.4" (Spruce).

Sample	Compression Strength (psi)	Residual Value	SG
2-1	688	0.69	0.34
3-1	1,110	1.00	0.42
5-1	798	0.79	0.39
11-1	762	0.76	0.42
Mean	**773**	**0.81**	**0.39**

3f. Compression $_\perp$ @ 0.4" (Eastern-fir).

Sample	Compression Strength (psi)	Residual Value	SG
6-1	486	0.56	0.36
7-1	504	0.58	0.34
Mean	**495**	**0.57**	**0.35**

A detailed list of all the test results is provided in Appendix III. The results are color coded with various shades of green which represent the levels of wood decay observed in each sample. Bending strength reductions up to 30%, or 0.70 residual values, were observed in three of the spruce test specimens (TP1-A, TP2-A, TP4-A). All of these lumber samples were selected by WAS, Inc. personnel during our first inspection of the material stored on the 40th floor. These samples were specifically selected because they exhibited evidence of wood decay. Wood decay was also observed in TP3-A, however, no reductions were found when compared to the historical data. This result may be somewhat misleading for the following reason. Our comparisons are based on mean values, therefore, it is very possible that the original strength of TP3-A was higher than the published mean (i.e., 50% chance) for spruce, and that the effects of the decay were masked.

Samples TP5-A, TP6-A, and TP7-A were randomly selected from material on the 40th floor by the NYCDOB. Wood decay was verified in two of the three test specimens and strength reductions were found in TP7-A.

The remaining samples were selected by WAS, Inc. personnel during our inspection of the material in both shipping containers. Wood decay and slight strength reductions were found in three of the four samples. The 12th sample was not mechanically evaluated, but wood decay was observed to the center of that sample (TB40189, Table 1).

The results provided within this section and their ramifications will be discussed in section 6.1. Current working design stresses and approximate actual strengths at the time of the collapse will be provided.

5.3 Concentrated Load Testing • Plywood

During our on-site inspection, as was previously discussed in section 4.3.2, we observed a consistent and unique failure pattern which appeared to be a punch through failure of the steel column support base plates (photos 17 - 19). Therefore, it was recommended to perform a modified test procedure following ASTM E 661-88. That ASTM standard was specifically designed for concentrated load testing of floor and roof sheathing using a 3" diameter steel disk. The base plates associated with the concrete formwork at 246 Spring Street were 6" x 8". During our testing, we simply followed the ASTM E 661 test procedures except we replaced the 3" diameter disk with an actual 6"x8" base plate we retained from 246 Spring Street during our inspections. Additionally, the tests were continued until punch through occurred.

One additional request from the NYCDOB for our testing protocol was recording deflection measurements to failure. To measure deflection, we attached an engineers scale to the top of the base plate and set up a laser level to measure the base plate displacement during our load application. Therefore, we were able to provide load/deflection data where the deflection represented base plate displacement and not a direct deflection measurement of the plywood. A summary of the test results is provided in Table 4a and 4b and all the load/deflection data and charts, along with the maximum loads, is provided in Appendix IV.

The test procedure used for the testing and a list of witnesses present were provided previously in section 4.2.2. On the day of the testing, we performed the static concentrated loading on two types of test specimens which was also discussed in section 4.2.2.

Panels TP8, TP9, and TP10 were 5-ply formwork panels with melamine overlays on both faces of each panel, resulting in an average thickness of 0.65 inches. Therefore, the panels are likely 5/8" panels. Sample PW4055 was a 4-ply "UNI-FORM" panel and had a grade stamp that indicated the panel was B-B, Class-1, manufactured by Futter, Mill 332. Test panel PW4055 had no legible thickness on the grade stamp, but the thickness was measured to be 0.60-0.61. Therefore, the manufactured thickness is either 19/32", or 5/8".

For loading condition #1, with the base plate located at the center of the 16" span, and the interior edge of the base plate 11" from the edge of the panel, test specimens TP8-A - #1, TP8-A - #2, and TP9-A - #2 demonstrated maximum failure loads of 4,988 lbs, 5,151 lbs, and 4,857 lbs, respectively. Panel test specimen PW4055 - #2 resulted in a maximum failure load of 3,586 lbs. The failure modes of each panel were punch through failures (photos 24 - 26)and were consistent with the panel failures we observed on several panels during our on-site inspection (photos 17 - 19).

For loading condition #2, with the base plate located at the center of the 16" span and the interior edge of the base plate approximately 20" to 24" from the panel edge, test specimen TP10-A and TP9-A demonstrated maximum failure loads of 8,967 lbs and 6,765 lbs, respectively. The maximum failure load of panel PW4055 - #1 was 3,716 lbs.

Table 4

Summary of the Static Concentrated Load Test Results for Four Panels from 246 Spring Street.

4a. Loading Condition #1 • 2'x4' Test Specimen with Base Plate at Center of 16" Span & Interior Edge of Base Plate 11" from Panel Edge.

Sample	Thickness (in.)	Maximum Load (lbs.) (Punch Through)	Deflection at Maximum Load (in) (Punch Through)
TP8-A #1	0.65	4,988	0.73
TP8-A #2	0.64	5,151	1.70
TP9-A #2	0.65	4,857	1.70
PW4055 #2	0.61	3,586	0.63
Mean	----	4,646	1.19

4b. Loading Condition #2 • 2'x4' Test Specimen with Base Plate at Center of 16" Span & Interior Edge of Base Plate 20"-24" from Panel Edge.

Sample	Thickness (in.)	Maximum Load (lbs.) (Punch Through)	Deflection at Maximum Load (in) (Punch Through)
TP10-A	0.65	8,697	1.00
TP9-A	0.65	6,765	0.90
PW4055 #1	0.61	3,753	0.60
Mean	----	6,393	0.83

4c. Loading Condition #1 and #2 Combined.

Sample	Maximum Load (lbs.) (Punch Through)	Deflection at Maximum Load (in) (Punch Through)
Overall Mean	**5,394**	**1.04**

6. Discussion

6.1 Lumber • 3x4
Allowable Working Stresses & Approximate Load Carrying Capacity at Failure

The NYCDOB had requested that WAS, Inc. provide the allowable working stresses of the 3x4 lumber at the time of failure, and the approximate ultimate load carrying capacity of the lumber at failure.

The allowable design values published in the NDS Supplement for SPF apply to the 3x4 lumber which is the subject of this investigation. For design purposes, a standard requirement of using published design stresses is that the lumber to which these design stresses are applied is free of wood decay. This is due to the fact that strength reductions in lumber caused by decay with varying amounts of decay in that lumber are unknown and the brash behavior of decayed lumber is unpredictable. This project, however, differs from a normal design-build project because attempts are being made here to establish contributing factors to the failure of a structural system rather than designing a new system or building. Thus, we have developed an adjustment for the presence of and degree of decay observed during the microbiological analysis, and the strength reductions found during the mechanical testing performed on the 3x4 samples. Additionally, since we are investigating a structural collapse, we recommend the use of the lowest residual values found for each property evaluated. Therefore, residual factors of 0.70 for F_b, 0.70 for MOE, or E, and 0.75 for F_v should be applied to the 3x4 dimension lumber used in the concrete formwork system that collapsed at 246 Spring Street.

The residual values for the compression perpendicular-to-grain test results appear to have been significantly affected by grain angle in two of the eastern fir test specimens and one spruce test specimen. Reductions of 40% in compression perpendicular-to-grain strength have been found to be associated with a growth ring angle of 45°. Every attempt was made during our testing to fabricate the compression test specimens with growth ring angles at 90°, however, this was not always possible since we were limited by using lumber in commercial size. Test specimens 6-1 and 7-1 had growth ring angles near 45° and test specimen 2-1 had some distorted grain. Therefore, test specimens 6-1, 7-1, 2-1 were removed from the analysis. After accounting for this ring angle anomaly, we recommend using a residual factor of 0.85 for compression perpendicular grain (mean residual values of 1.00, 0.79, and 0.76 for the remaining compression test specimens). Compression perpendicular grain is always reported as a mean value.

A total of 7% of both visual grade populations for the 3x4 lumber were found to only qualify for Economy grade, primarily as a result of excessive SOG (greater than 1:4 as allowed in No. 3 and Standard grades). Slope-of-grain has a significant effect on bending strength. The relationship between SOG and strength is exponential. Significant reductions occur especially at SOG's steeper than 1:8. The strength ratio published for a SOG of 1:8 in ASTM D 245 is 53% of the strength of straight grained lumber for bending strength. A strength ratio is the ratio between the strength of wood containing particular strength reducing characteristics, such as slope-of-grain, and the strength of wood without those strength reducing characteristics. At a SOG of 1:6, the bending strength drops to 40% when compared to straight grain lumber. If this data were extrapolated to 1:4, the ratio would be approximately 25%. During our inspection, we observed SOG's of 1:3 and 1:2 on several pieces we classified as economy which would correspond to strength ratios of approximately 10% - 15% of straight grained lumber. As a result, we can only provide estimates rather then actual computed design stresses and ultimate stresses for the Economy grade lumber. Furthermore, the only data provided in ASTM D 245 on the effects of SOG on strength are for bending strength and compression parallel-to-grain. Therefore, we can only provide data for MOR and F_b in this report for Economy grade material.

Table 5 provides a list of the applicable working design stresses of all the 3x4 lumber graded during our inspection and Table 6 provides an approximate estimate of the ultimate stresses at the time of the collapse. The values provided are F_b, E, F_v, and $Fc_\perp$ for SS, No. 1, No. 2, No. 3, CONST, and STAND visual grades. There are no published values for Economy material. However, 14 pieces of lumber were classified as "ECON" because of SOG's greater than 1:4. To provide estimates of design stresses and approximate ultimate stresses for these pieces, we reviewed ASTM D 245, "Standard Practice for Establishing Structural Grades & Related Allowable Properties for Visually Graded Lumber," Table 1, "Strength Ratios Corresponding to Various Slopes of Grain." Additionally, we reviewed the underlying data that was used to develop Table 1 in ASTM D 245, as well as other industry data on the effects of SOG on strength properties. The only strength properties based on SOG provided in Table 1 of ASTM D 245 are applied to bending strength and compression parallel-to-grain. Therefore, we have only provided estimates for F_b in the ECON grade in Tables 5 and 6 of this report.

The working design values listed in Table 5 are provided for Structural Light Framing (i.e., SS, No. 1, No. 2, and No. 3 with a size adjustment to 3x4 lumber) and Light Framing (i.e., CONST and STAND with no size adjustment necessary). The published values in the NDS for SPF are provided in the first set of columns under the heading "Value Published in NDS." The second set of columns under the heading "Adjustments for Decay/Size," lists the adjustment factors for the effects of wood decay (i.e., 0.70 for bending strength and modulus of elasticity, 0.75 for shear, and 0.85 for compression perpendicular-to-grain) and for size (i.e., 1.5 for SS, No. 1, No. 2, and No. 3) since the values published in the NDS for structural joists and planks are at a characteristic width of 12". After applying these adjustments, the allowable design stresses for F_b', E', F_v', and $Fc_\perp'$ are provided in the last four columns of Table 5 for each visual grade. These values are applicable for design purposes based on our laboratory analyses. If this data were to be used to design this system, we would recommend using a visual grade class of ECON & Better. However, the only property that could be provided for the Economy grade would be F_b. The effect of slope-of-grain is only reported for bending strength and compression parallel-to-grain in ASTM D 245. A more reasonable procedure would be to eliminate Economy grade material from the lumber population and use either Standard and Better or No. 3 and Better material.

Table 5
Summary of the Allowable Working Design Stresses for the 3x4 SPF Lumber Associated with the Concrete Formwork Collapse at 246 Spring Street.

Visual Grade	Value Published in NDS (psi)				Adjustments for Decay/Size[1]				Current Allowable Stresses (psi)			
	F_b	E	F_v	$Fc_\perp$	F_b	E	F_v	$Fc_\perp$	F_b'	E'	F_v'	$Fc_\perp'$
SS	1,250	1.5x10^6	135	425	0.70x1.5	0.70	0.75	0.85	1,300	1.1x10^6	100	350
No. 1	875	1.4x10^6	135	425	0.70x1.5	0.70	0.75	0.85	925	1.0x10^6	100	350
No. 2	875	1.4x10^6	135	425	0.70x1.5	0.70	0.75	0.85	925	1.0x10^6	100	350
No. 3	500	1.2x10^6	135	425	0.70x1.5	0.70	0.75	0.85	525	0.8x10^6	100	350
CONST	1,000	1.3x10^6	135	425	0.70	0.70	0.75	0.85	700	0.9x10^6	100	350
STAND	550	1.2x10^6	135	425	0.70	0.70	0.75	0.85	375	0.8x10^6	100	350
ECON	275	----	----	----	0.70	----	----	----	200	----	----	----

[1] Adjustments for decay: F_b x 0.70, E x 0.70, F_v x 0.75, $Fc_\perp$ x 0.85 — Adjustment for size: F_b x 1.5

Adjustment for SOG - Economy = STD x 0.5

The values provided in Table 6 are values resulting from calculations which were performed to estimate the ultimate stresses in the 3x4 lumber at the time of the collapse. The first four columns, "Unrounded Stress," represent the unrounded values from the columns under the heading "Current Allowable Stresses" in Table 5. The second four columns in Table 6, "Removal of GAFs," list values for F_b L5%, F_v L5%, E$\bar{x}$, and $Fc_{\perp}\bar{x}'$. The L5% notation is referred to as the lower 5th percentile limit. When calculating wood design stresses for strength, the L5% value is used as the basic strength of that property, rather than the mean ($\bar{x}$). The mean values are only applicable to serviceability properties, such as E and $C_{\perp}$. For simplification and safety, selection of this near minimum value (L5%) insures that approximately 95% of the material is stronger than that level of strength and represents the weakest link in the population of strength values.

Table 6
Summary of the Approximate Ultimate Stresses for the 3x4 Lumber Associated with the Concrete Formwork Collapse at 246 Spring Street[1] (all values reported in psi units).

Visual Grade	Unrouded Stress (from table 5) (psi)				Removal of GAFs Ultimate Stress = L5% or $\bar{x}$ (psi)				Mean Ultimate Stress = $\bar{x}$ (psi)			
	F_b'	E'	F_v'	$Fc_{\perp}'$	F_b L5%	E$\bar{x}$	F_vL5%	$Fc_{\perp}\bar{x}$	$F_b\bar{x}$	E	$F_v\bar{x}$	$Fc_{\perp}\bar{x}$
SS	1,313	$1.05x10^6$	101	361	2,757	$0.98x10^6$	202	603	4,683	$0.98x10^6$	262	603
No. 1	919	$0.98x10^6$	101	361	1,930	$0.92x10^6$	202	603	3,278	$0.92x10^6$	262	603
No. 2	919	$0.98x10^6$	101	361	1,930	$0.92x10^6$	202	603	3,278	$0.92x10^6$	262	603
No. 3	525	$0.84x10^6$	101	361	1,103	$0.79x10^6$	202	603	1,873	$0.79x10^6$	262	603
CONST	700	$0.91x10^6$	101	361	1,470	$0.86x10^6$	202	603	2,496	$0.86x10^6$	262	603
STD	385	$0.84x10^6$	101	361	809	$0.79x10^6$	202	603	1,374	$0.79x10^6$	262	603
ECON[2]	193	----	----	----	405	----	----	----	689	----	----	----

[1] Notations for stresses in chart are as follows:

F_b', E', F_v', and $Fc_{\perp}'$ are each the unrounded values from Table 5.
F_bL5% = is the lower 5th percentile values after removal of the general adjustment factor
F_vL5% = is the lower 5th percentile values after removal of the general adjustment factor
F_b $\bar{x}$ = is the approximate mean based on coefficient of variation = 0.25
F_v $\bar{x}$ = is the approximate mean based on coefficient of variation = 0.14
E$\bar{x}$ and $Fc_{\perp}\bar{x}$ = is the approximate mean after removal of the general adjustment factor (0.94 for E and 1.67). This property is always reported as a mean value, not based on L5% calculations.

[2] The only SOG data allowable for adjustment of economy lumber is for bending strength.

In order to estimate ultimate properties from published allowable properties in the NDS one must remove certain adjustment factors for items such as duration of load and a true safety factor for unknown job site conditions. These are called the general adjustment factors (GAFs). In the case of bending stresses, the general adjustment factor for F_b, is equal to 2.1. Therefore, to derive an ultimate stress from an allowable design stress in bending, one must remove this GAF by multiplying the published design value by 2.1. For purposes of this report, we multiplied the value in column 1 of Table 6, F_b', by the 2.1 general adjustment factor. Therefore, the value provided in column 5 of Table 6, F_bL5%, is the unrounded ultimate lower 5th percentile stress in bending after adjusting the NDS design value for size and stress the effects of wood decay found during the testing for this project. Similar adjustments for different issues are made to the other properties of interest listed in Table 6. A complete discussion of allowable property development for wood is beyond the scope of this report, thus, rather than going into detail, the second set of 4 columns in Table 6 reflect removal of each of the GAFs from each allowable property resulting in ultimate stresses for each grade of lumber. An additional calculation was then made to estimate the mean ultimate stresses from the 5th percentile stresses for F_b and F_v and are results provided in the last four columns (9 - 12) under the heading "Allowable Mean Ultimate Stresses." The final calculation was performed assuming a coefficient of variation of 25% for bending strength, 14% for shear, and 16% for compression perpendicular-to-grain and applying the standard statistical conversion for equating mean values to standard deviations and coefficients of variation.

For purposes of ultimate stress calculations the weakest link in the group is the L5% value for the ECON grade listed as 405 psi. Again, the L5% values provided in the second set of columns in Table 6, "Removal of GAFs," represent the approximate ultimate stress levels at the lower 5th percentile level of the population. This is the "weakest link" value for all practical purposes, for the Economy grade representing the lowest strength levels of all the material. Mean values are provided in the table, but <u>do not</u> represent the population strength. The population strength for wood is governed by the L5% values.

Although the L5% exclusion value is technically the most important and most appropriate value to use for estimates of ultimate strength for the determination of failure, WAS, Inc. was also requested to provide an estimated mean strength value for the lumber used at 246 Spring Street. As previously described, the estimated mean values are provided in columns 9-12 in Table 6 for each property and grade. Since the lumber supplied was mill run material, an overall mean value can only be estimated by developing a weighted value based on the number of lumber pieces within each visual grade as we observed during the inspection. Therefore, by combining the number of pieces of each grade observed with grade controlling defects (212 in total) from section 3.2 of this report, with the estimated mean ultimate strengths in Table 6, which were adjusted downward based on the mechanical testing performed in our laboratory, an estimated mean ultimate strength of 3,132 psi was computed for F_b, 262 psi for F_v, and 603 for $Fc_{\perp}$ and 916,000 psi for E.

6.2 Plywood - Ultimate Loads & Nailing Pattern

As reported in section 5.3, the plywood failure loads ranged from 3,586 lbs. to 4,988 lbs. with a mean of 4,646 lbs. for loading condition #1, and 3,716 lbs to 8,697 lbs. with a mean of 6,393 lbs. for loading condition #2. The overall combined mean for both loading conditions was 5,394 lbs. In addition to maximum load (punch through), deflections at punch through were also measured. For loading condition #1, the mean deflection at punch through was 6.49". For loading condition #2, it was 6.23" with a combined mean of 6.38". Loading condition #1 consisted of aligning the base plate at the center span between two joists spaced 16" on-center, and the interior edge of the base plate 11" from the panel edge. Loading condition #2 also aligned the base plate at the center span, however, the interior edge of the base plate was located 20" to 24" from the panel edge. The ultimate load results were significantly different for the concrete formwork plywood panels TP8, TP9, and TP10 than for panel PW4055. Many of the concrete formwork panels assessed during our inspection exhibited linear failure patterns consistent with base plate punch through failures (photos 17-19). The testing failure patterns were similar to those observed during our inspection (photos 24-26).

Therefore, it is our professional opinion that the results of our modified concentrated load testing simulates the actual loads to the plywood forms as occurred in service and, thus, the ultimate load carrying capacity of the concrete formwork plywood panels observed during testing are representative of the actual loads to the plywood panels that occurred during the collapse at 246 Spring Street.

The nailing pattern used for the plywood testing was at 12" intervals around the entire panel during our testing, as well as the interior of the panel. The nailing pattern observed during our inspection was not exactly at 12" increments, however, for all practical purposes, this is the pattern selected for the tests to insure that all tests were consistent with each other. The spacing documented during our inspection ranged from 3" to 20" around the panel edges with an average spacing of 9-1/4". Nailing was primarily observed around the edges of the panels and not within the field of the panels.

7. Summary & Opinions

Based on our inspection, testing using consensus based standard procedures, and laboratory analyses performed on the 3x4 ribs and plywood associated with the concrete formwork collapse at 246 Spring Street, New York, NY, our knowledge, education, and experience, as well as documented scientific literature and industry publications, we offer the following summary and opinions:

1. Nearly 400 fragmented pieces of dimension lumber and over 100 fragmented pieces of plywood associated with the collapse at 246 Spring Street were physically inspected. Visual grades of the dimension lumber were documented whenever possible. Failure characteristics of the dimension lumber and plywood were also documented. Twelve, 3x4 lumber ribs were evaluated following ASTM standard test

procedures, and four, 4'x8' sheets of plywood were evaluated following a modified concentrated load ASTM standard test procedure. Additionally, for each 3x4 lumber rib that was mechanically tested, the rib was also microscopically evaluated to document levels of wood decay and to determine the wood species.

2. The visual grades for the 3x4 dimension lumber were found to have a high percentage of low grade material. In fact, numerous pieces did not even qualify as structural material and were, therefore, classified as "Economy" which has no associated published design values. The 3x4 dimension lumber was visually graded using two different classifications; Structural Light Framing and Light Framing. Since no grade stamps were observed in the 3x4 lumber, they were graded using both classifications. Only slight differences exist between these classifications and those differences project through to the published allowable design properties for the grades within these two systems. A summary of the results are as follows, however, the published design values do not apply to these grades without first adjusting for decay which is discussed later in this summary:

- Total number of lumber pieces inspected = 396
- Visual grade summary of 3x4s - Structural Light Framing (2"-4" thick, 2"-4" wide)

Visual Grade	**Pieces**	**Percentage**
Select Structural	59	28%
No. 1	50	23%
No. 2	41	19%
No. 3	48	23%
Economy	14	7%
TOTAL	212	100%

- Total number of pieces inspected = 396
- Visual grade summary of 3x4s - Light Framing (2"-4" thick, up to 4" wide).

Visual Grade	Pieces	Percentage
Construction	159	75%
Standard	39	18%
Utility	----	----
Economy	14	7%
TOTAL	212	100%

3. The failure characteristics of the 3x4 dimension lumber were found to have a high percentage of brashness which is associated with wood decay and/or low specific gravity values; both of which are not typical in normal wood. The following is a summary of 436 failure modes observed in the 3x4 dimension lumber (B = brash failure, BT = combination of brash and typical tension failure, T = typical tension failure):

Characteristic	Count	Percentage
B	86	20%
BT	182	42%
T	168	38%
TOTAL	436	100%

4. No grade stamps were observed on any of the 3x4 dimension lumber samples inspected except for the name "LAUZON," which was observed on one piece. Further research was conducted and revealed that this stamp is likely associated with a lumber mill located in Quebec, Canada, just north of the Vermont border. Based on all the characteristics of the lumber inspected, the 3x4 lumber ribs appear to be "mill run" material.

5. The species of the lumber inspected was a combination of spruce and eastern fir. Therefore, the lumber would be classified as S-P-F (spruce-pine-fir) which is manufactured in Canada.

6. The 3x4 lumber does not meet the specification requirements present in Patent Construction Systems Drawing No. 4607K070.

7. Two different grade stamps were documented for the 4x4 dimension lumber; STAND & BTR and No. 2. All the 4x4 dimension lumber examined qualified for both grades. The failure modes of the 4x4 dimension lumber were similar to the 3x4 dimension lumber and are listed below:

Characteristic	Count	Percentage
B	3	20%
BT	8	53%
T	4	27%
TOTAL	15	100%

8. The 4x4 lumber does not meet the specification requirements present in Patent Construction Systems Drawing No. 4607K070.

9. Based on all the evidence collected during our inspection and documented scientific literature, it is our opinion that the storage of the material inside the shipping containers between February and May had no effect on the lumber with respect to wood decay at the time we performed our inspection.

10. Over 100 broken pieces of plywood were inspected. Several failures appeared to be consistent with a punch through of the 6" x 8" steel column base plates. Therefore,

full plywood sheets were retained to perform concentrated static load tests. No grade stamps were observed on most of the concrete formwork panels since they had melamine overlays on both faces. The only markings observed were "Feldman Lumber" and "Mid-South Lumber Company." One grade stamp was observed on several other panels which read: Futter, Mill 332, Uni-form, B-B Class.

11. Only some plywood panels contained grade stamps which could be used to determine conformance to the Patent Construction Systems Drawing No. 4607K070. The plywood panels with Feldman Lumber Company overlays or Mid South Lumber Company overlays could not be verified to be either in compliance or out of compliance with the specifications. Those panels with a Futter Lumber Company stamp were in compliance with the specifications since they were stamped with a Pittsburgh Testing Laboratory stamp illustrating B-B grade veneer, Class 1 with an exterior adhesive.

12. Twelve 3x4 dimension lumber samples were retained for mechanical testing and microbiological analyses. Evidence of wood decay was documented in 11 of the 12 samples. Five of the 11 samples had decay limited to the surface region and seven had wood decay ranging from a depth of ½" to a depth of 1-1/2".

13. Reductions in mechanical properties resulted from the wood decay in the dimension lumber samples. Small clear tests were performed on several test specimens and reductions in MOR, MOE, Shear parallel-to-grain, and C perpendicular-to-grain were found. As a result of the testing, we recommended the following residual value factors for the published design values of the 3x4 dimension lumber:

Property	Residual Factor
F_b	0.70
E	070
F_v	0.75
$Fc_{\perp}$	0.85

14. The table below is a summary of the allowable properties of the various visual grades for the 3x4 dimension lumber at the time of the collapse based on our laboratory results. The allowable bending strength provided for the ECON grade was calculated based documented scientific literature cited in ASTM D 245 regarding the effect of slope-of-grain on bending strength. No other allowable stresses could be calculated for the economy grade material.

Visual Grade	**Current Allowable Stresses (psi)**			
	F_b'	**E**	F_v'	$Fc_{\perp}'$
SS	1,300	$1.1\text{x}10^6$	100	350
No. 1	925	$1.0\text{x}10^6$	100	350
No. 2	925	$1.0\text{x}10^6$	100	350
No. 3	525	$0.8\text{x}10^6$	100	350
CONST	700	$0.9\text{x}10^6$	100	350
STAND	375	$0.8\text{x}10^6$	100	350
ECON	200	----	----	----

15. A summary of the approximate mean ultimate stresses for each grade of the 3x4 lumber are provided below. For the two strength properties (F_b and F_v), the L5% (lower 5th percentile) value was determined where as only mean ($\bar{x}$) values were reported for E and $F_{c\perp}$ which are considered primarily serviceability properties. Mean values were also reported for F_b and F_v using published coefficients of variation. To

convert allowable design stresses to ultimate stresses, the general adjustment further must be removed from the allowable design stress. This was done to provide the data in the following summary. For stress calculations, we recommend using the L5% for both strength properties (F_b and F_v) since these values represent the samples which would fail first in a collapse.

Visual Grade	**Removal of General Adjustment Factor = L5% or $\bar{x}^2$ (psi)**				**Estimated Allowable Mean Ultimate Stresses = $\bar{x}$ (psi)**			
	F_b L5%	E$\bar{x}$	F_vL5%	$Fc_\perp\bar{x}$	$F_b\bar{x}$	E	$F_v\bar{x}$	$Fc_\perp\bar{x}$
SS	2,757	0.98x10^6	202	603	4,683	0.98x10^6	262	603
No. 1	1,930	0.92x10^6	202	603	3,278	0.92x10^6	262	603
No. 2	1,930	0.92x10^6	202	603	3,278	0.92x10^6	262	603
No. 3	1,103	0.79x10^6	202	603	1,873	0.79x10^6	262	603
CONST	1,470	0.86x10^6	202	603	2,496	0.86x10^6	262	603
STD	809	0.79x10^6	202	603	1,374	0.79x10^6	262	603
ECON[2]	405	----	----	----	689	----	----	----

Although the L5% exclusion value is technically the most important and most appropriate value to use for estimates of ultimate strength for the determination of failure, WAS, Inc. was also requested to provide an estimated mean strength value for the lumber used at 246 Spring Street. This was accomplished by computing a mean value weighted by the lumber grades observed during our inspection. The resulting values are 3132 psi for F_b, 262 psi for F_v, and 603 for $Fc\perp$, and 916,000 psi for E.

16. Two loading conditions were used to examine the ultimate concentrated load levels in four 4"x8" concrete formwork plywood panels. The load conditions were as follows:

    1. Base plate located at center of 16" span, interior edge of plate 11" from panel edge.
    2. Base plate located at center of 16" span, interior edge ~24-26" from panel edge.

The resulting failure patterns at the time of testing were consistent with those punch through failures observed during our inspection. The results of the concentrated load testing for each loading condition are provided in both tables below:

Loading Condition #1 • 2'x4' Test Specimen with Base Plate at Center of 16" Span & Interior Edge of Base Plate 11" from Panel Edge.

Sample	**Thickness (in.)**	**Maximum Load (lbs.) (Punch Through)**	**Deflection at Maximum Load (in) (Punch Through)**
TP8-A #1	0.65	4,988	0.73
TP8-A #2	0.64	5,151	1.70
TP9-A #2	0.65	4,857	1.70
PW4055 #2	0.61	3,586	0.63
Mean	----	4,646	1.19

Loading Condition #2 • 2'x4' Test Specimen with Base Plate at Center of 16" Span & Interior Edge of Base Plate 20"-24" from Panel Edge.

Sample	Thickness (in.)	Maximum Load (lbs.) (Punch Through)	Deflection at Maximum Load (in) (Punch Through)
TP10-A	0.65	8,697	1.00
TP9-A	0.65	6,765	0.90
PW4055 #1	0.61	3,753	0.60
Mean	----	6,393	0.83

In addition to maximum load (punch through), deflections at punch through were also measured. For loading condition #1, the mean deflection at punch through was 1.19". For loading condition #2, it was 0.83" with a combined mean of 1.04".

17. The nailing pattern spacing observed during our inspection ranged from 3" to 20" with an average of 9-1/4". Nailing was primarily observed around the panel edges and not within the field of the panels.

Respectfully Submitted,
Wood Advisory Services, Inc.

M.E. Anderson, M.S.
Wood Scientist
MEA:krEschenasy2.DRAFT.R08125.01.wpd

Wood Advisory Services, Inc.

A.L. De Bonis, Ph.D.
President/Principal Wood Scientist

APPENDIX I
Photographs

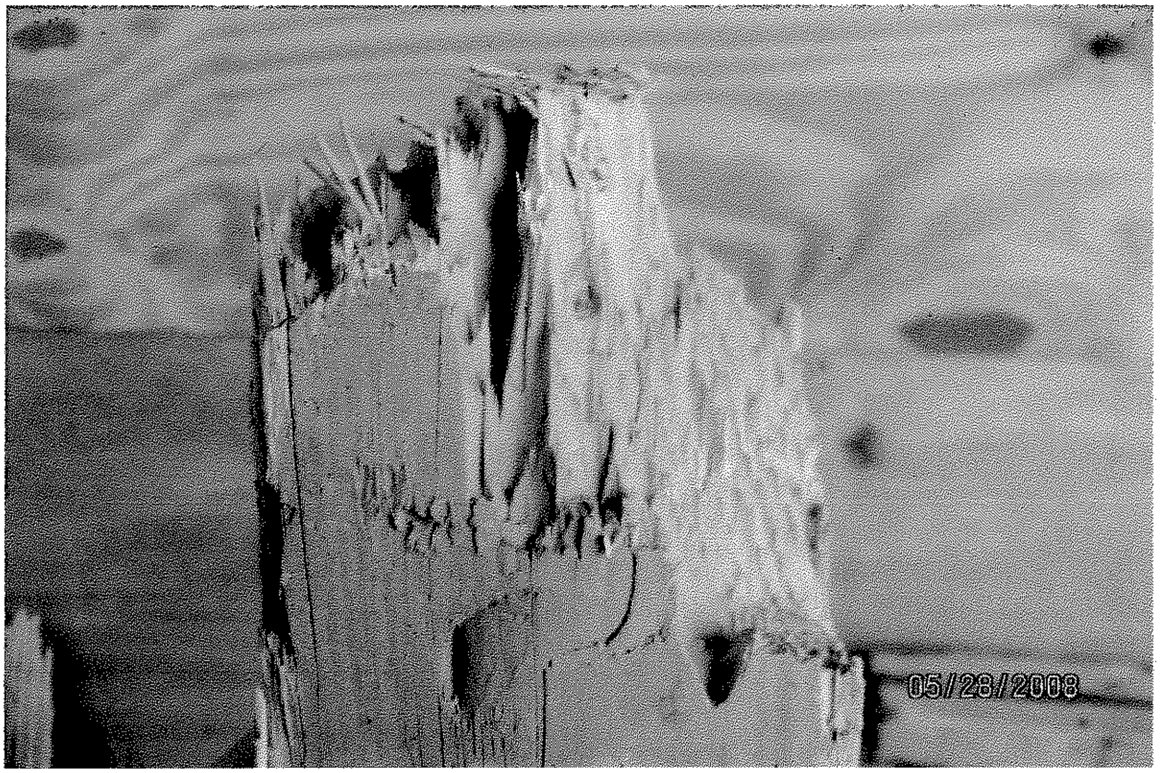


Photo 1 - Brash failure with partial tension finger in 3x4 stored on 40th floor.

Photo 2 - Brash failure along bottom edge of sample with DOB #TB40070.

Photo 3 - Brash failure in sample with DOB #TB40087.

Photo 4 - Slope-of-grain of 1:3 in 3x4 stored on 40th floor.

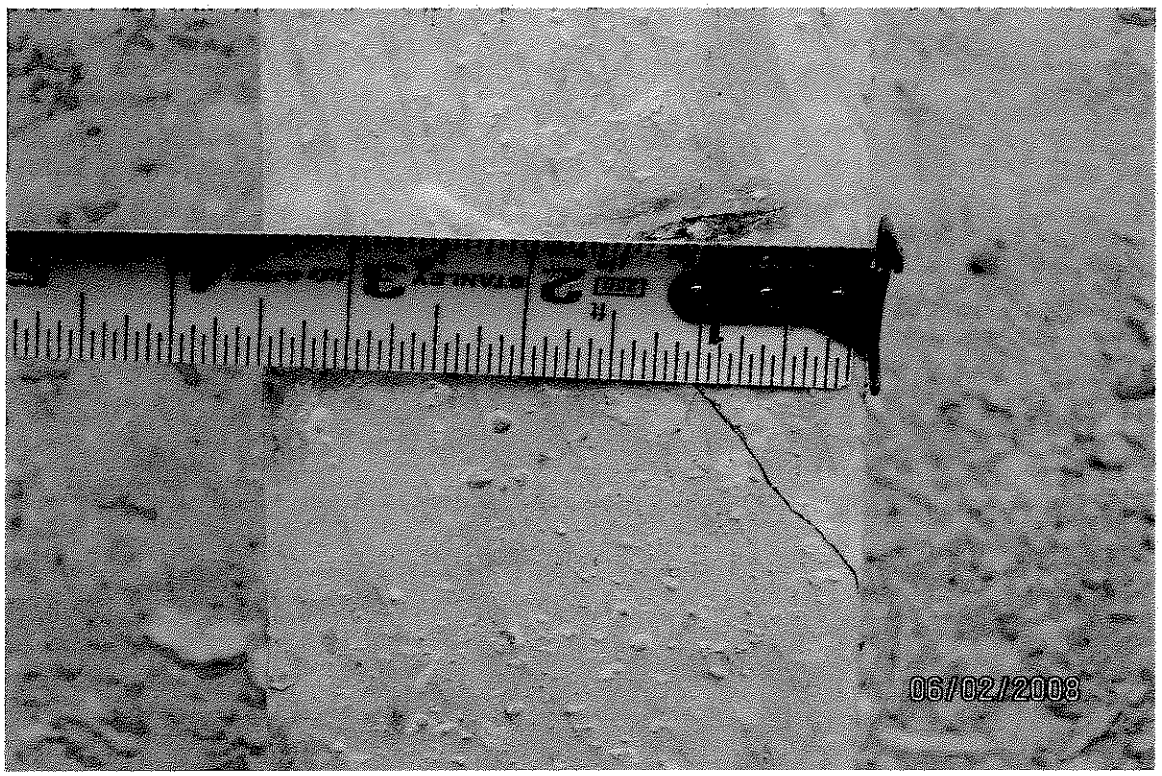


Photo 5 - Slope-of-grain of 1:1 in sample with DOB #TB40316.

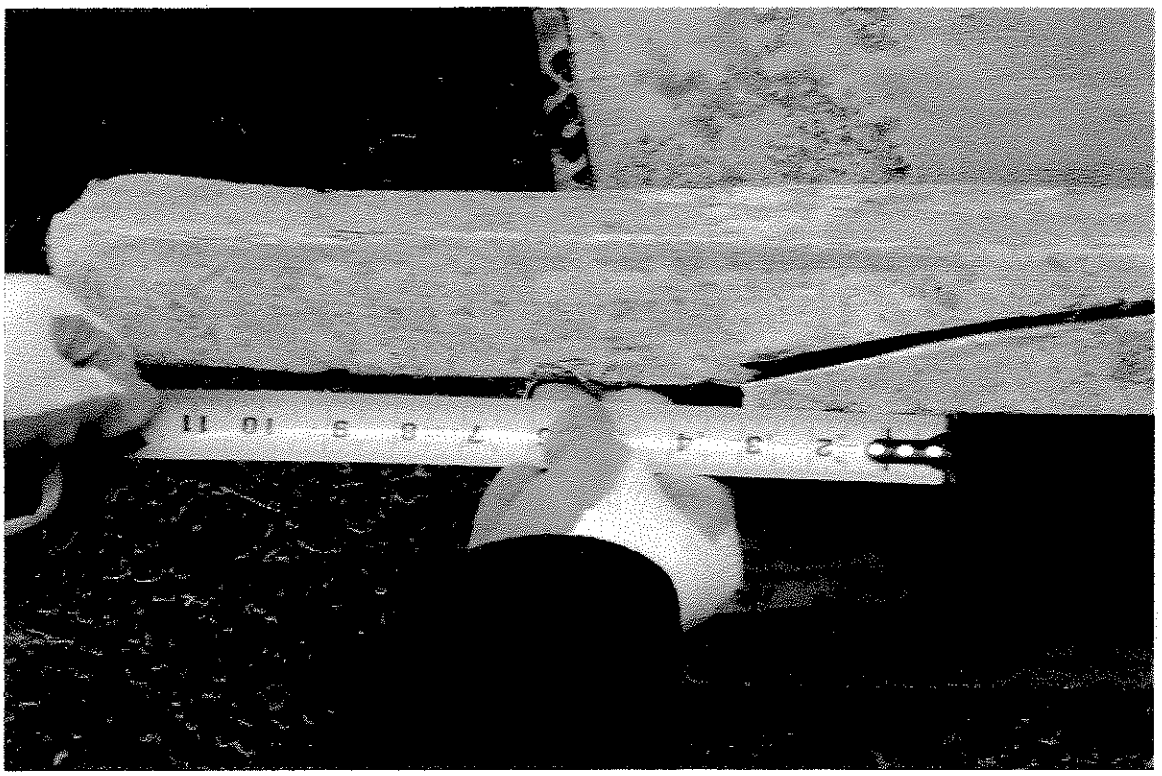


Photo 6 - Slope-of-grain of 1:3 on lumber attached to sample with DOB #PW40007.

Photo 7 - Stamp and 3x4 lumber, "LAUZON."

Photo 8 - Visible wood decay growth on 3x4 lumber specimen marked DOB #TB40291.

Photo 9 - Visible wood decay growth on 3x4 lumber specimen marked DOB #TB40101.

Photo 10 - Standard and Better (STAND&BTR) lumber stamp on 4x4, sample with DOB #41035C.

Photo 11 - No. 2 grade stamp on 4x4, sample with DOB #40021.

Photo 12 - Two 3x4 lumber ribs exhibiting brash failures at the ends.

Photo 13 - A 3x4 lumber rib exhibiting a brash failure being used in formwork during our inspection.

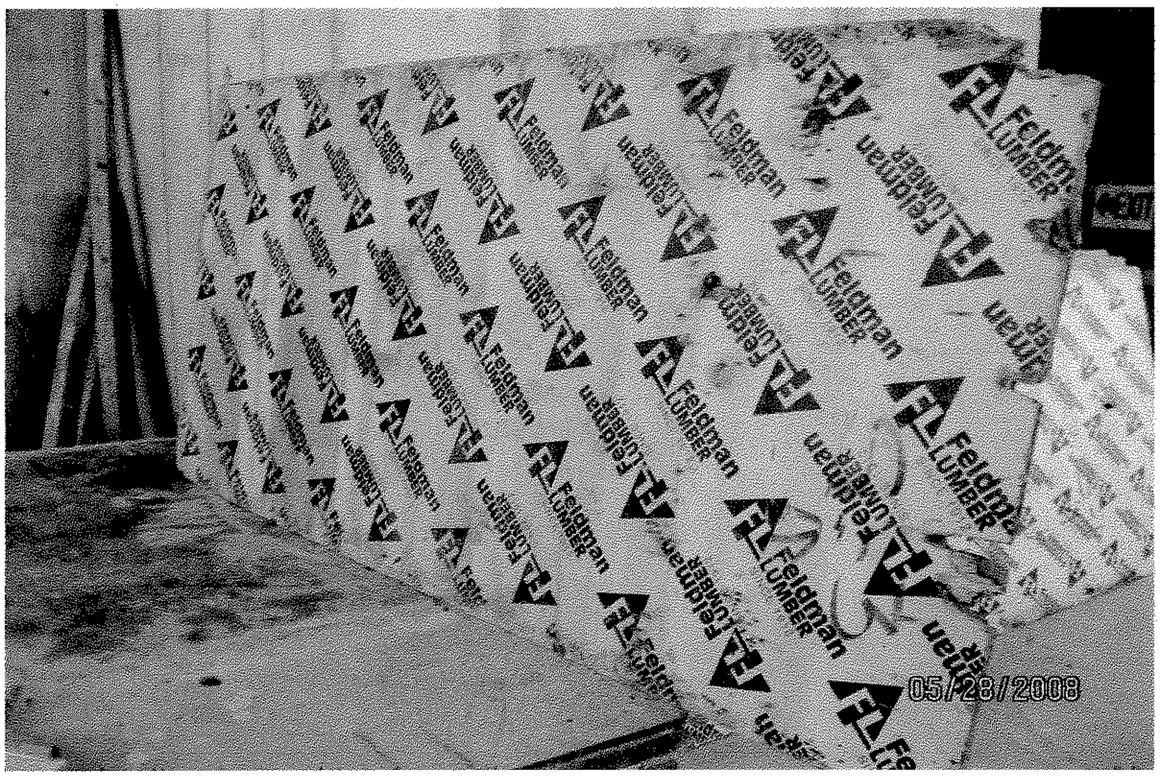


Photo 14 - 4'x8' plywood sheet with "Feldman Lumber" melamine overlay.

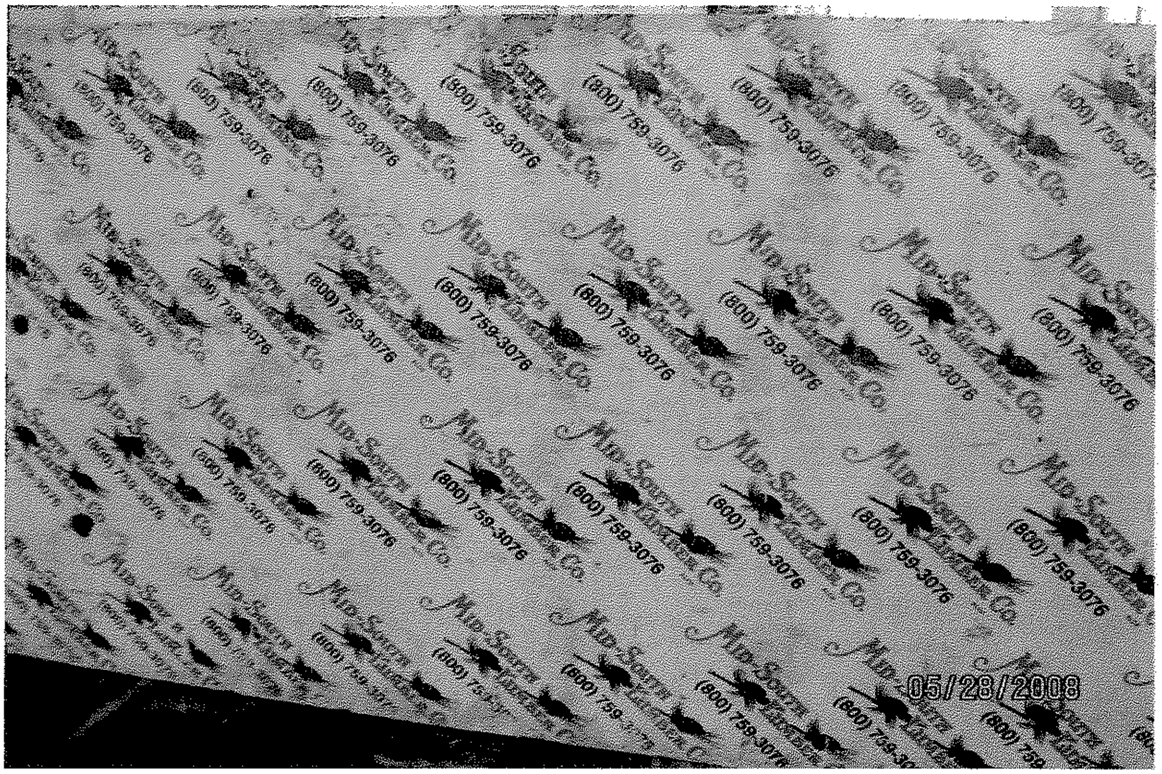


Photo 15 - 4'x8' plywood sheet with "Mid-South Lumber Company" melamine overlay.

Photo 16 - 4'x8' plywood sheet with a Futter Lumber Company grade stamp.

Photo 17 - Failure pattern consistent with a base plate punch through failure in panel DOB #PW41019.

Photo 18 - Failure pattern consistent with a base plate punch through failure in panel DOB #PW41007A.

Photo 19 - Failure pattern consistent with a base plate punch through failure in panel DOB #PW41003.

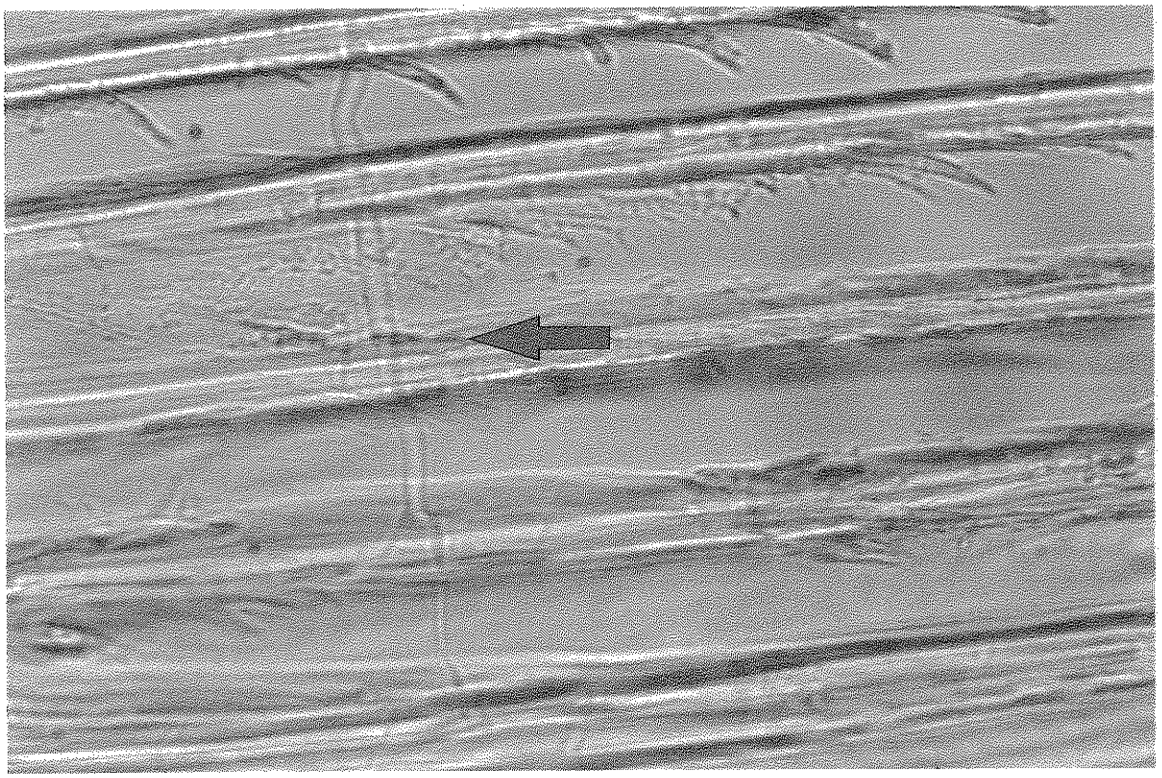


Photo 20 - Occasional wood decay hyphae observed near the surface of TP3-A.

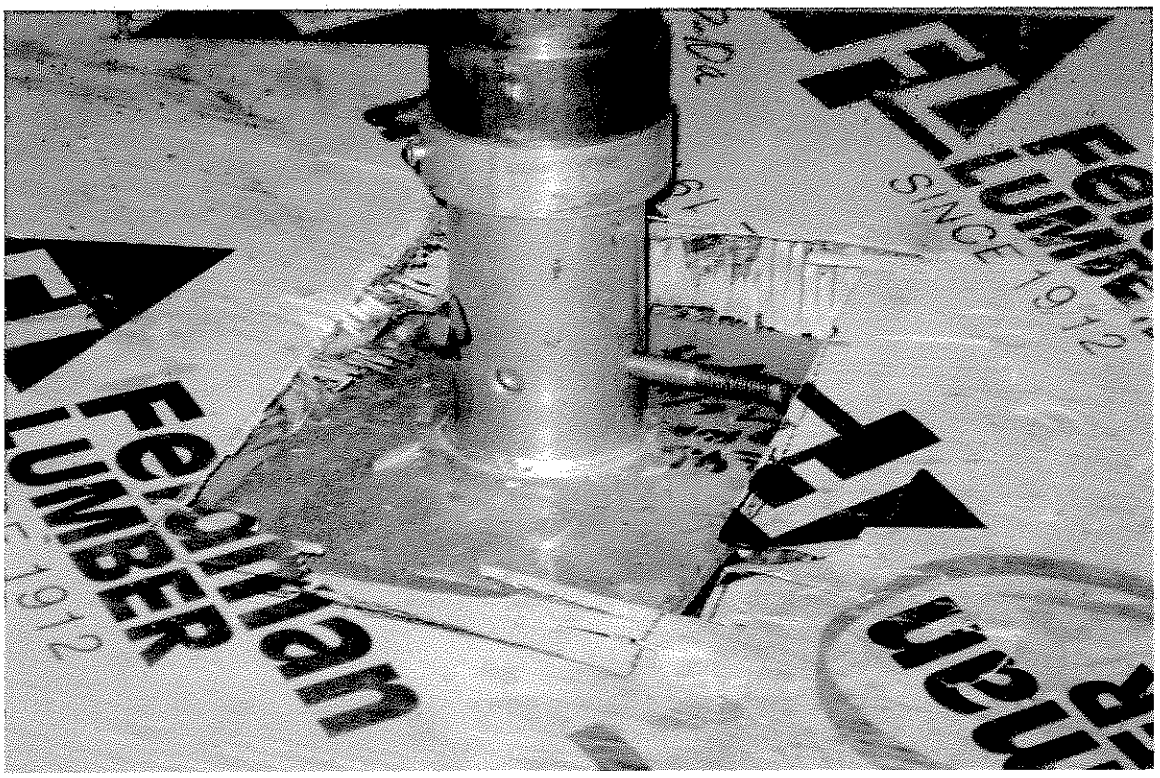


Photo 25 - Punch through failure of test specimen TP10-A.

Photo 26 - Punch through failure of test specimen TP9-A.

APPENDIX II
Summary of the Visual Grade Results & Inspection of Lumber

Client: Dan Eschenasy
Project Spring Street
Job No. 08.125
Test Visual Grading and Inspection of Dimension Lumber

Failure Modes:
T = Typical Bending Failure
B = Brash Bending Failure
BK = Brash bending failure with saw kerf
BT = Brash/tension combination
F = Factory Cut
FC = Field Cut
HOLE = Hole cut into lumber
C = Compression

Other:
NGCD = No Grade Contrlling Defect
VD = Visible Decay
PC = Pith Center

Results of Field Inspection and Visual Grading of Dimension Lumber at Spring Street

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
PW40013	3X4	T	B			47	No 3	CONST	SOG 1:7
PW41014 B		F	FC			32			NGCD PHOTO 177
PW41014 C		F	FC			31			NGCD PHOTO 177
TB 40310	757	B	B		15.8	25	No. 3	STAND	SOG 1:6
TB 40322	680 B	B	T			20 1/2			
TB 40325	687	B	T			20 1/2			
TB04232	183	BT	F			87			NGCD
TB40001	ZONE 40K	F	BT			32	No 3	STAND	1-5/8 EDGE
TB40002	113 ZONE 40K	FC	FC			113	No 3	STAND	SOG 1:6 LAUZON STAMPE PHOTOS 144-145
TB40003	ZONE 40K	F	T			76	No 3	STAND	SOG 1:6
TB40004	ZONE 40K	BT	F	4X4		25	SS		3/4" & 1/2" EDGE COMBO
TB40005	ZONE 40J	F	T			74	No 3	CONST	1" TWEENER, SOG 1:6
TB40006	ZONE 40F	FC	FC			145			VISIBLE HYPHAE ON SURFACE 25%
TB40007	ZONE 40F	F	BT			40	SS	CONST	3/4" EDGE
TB40008	ZONE 40F	F	HOLE			75	SS	CONST	4" BREAK + 20% KNOT
TB40009	ZONE 40F	F	F	2X4		96	No 1		30% COMBO
TB40010	40 F/245 A&B	F	T			64,64	SS	CONST	TOGETHER; A-NGCD B-3/4" EDGE
TB40011	240 F	BT	BT			17			NGCD
TB40012	ZONE 40F	F	T			57.5	SS	CONST	25%DISP
TB40014	ZONE 40F	F	T			81			NGCD
TB40015	40F	F	FC			76			NGCD
TB40016	40 F	FC	F			58			NGCD
TB40017	40 F	BT	BT			64			NGCD
TB40018	ZONE 40F	FC	F			48	No 2	CONST	3/4" NARROW FACE HOLE
TB40019	ZONE 40F	F	FC		14.2	24	SS	CONST	7/16" NARROW FACE

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40020	ZONE 40F	F	HOLE			49	No 3	STAND	HOLE 1-3/4
TB40021	ZONE 40K	F	BT	4X4		58	No 2		S-GRN, 266, S-GRN NO. 2 40%
TB40021	ZONE 40K	BT	F	4X4		58			STRAPPED TOGETHER PHOTOS 153-154
TB40023	128	FC	T			80			NGCD
TB40024	126	F	BT		15.3	63			NGCD
TB40025	175	FC	BT			70	No 2	STAND	3/4" NARROW FACE, LOCAL GRAIN
TB40026	274	F	T	4X4		65			COVERED CONCRETE
TB40027	239	F	T			55			NGCD
TB40028	307	F	BT			56	No 3	STAND	SOG 1:4
TB40029	309	BT	T			29	No 3	CONST	SOG 1:6
TB40030	306	T				38	No 3	CONST	TENSION FINGER SOG 1:7
TB40031	303	F	B			30			NGCD
TB40032	296	BT	F			60	SS	CONST	5/8 C KNOT
TB40033	308	T	BT			59	No 1	CONST	1" EDGE KNOT
TB40034	295	F	T		17.3	48	SS	CONST	25% DISP
TB40035	304	F	T			22.5	No 3	CONST	SOG 1:6
TB40036	294	F	BT			21			NGCD
TB40037	305	T	T			33	No 3	CONST	5/16 KNOT SOG 1:6
TB40038	226	BT	BT			52			NGCD
TB40039	206	FC	BT			61			NGCD
TB40040	243	F	B			82	SS	CONST	3/4" EDGE KNOT AT FAILURE
TB40041	230	FC	BT			63	SS	CONST	1/2" C KNOT
TB40042 A	294 A	F	T	F		82			NGCD PHOTO 168-172
TB40043	207	FC	BT			42			NGCD
TB40043 B	294B, 28								
TB40044	249 B	F	BT	4X4		113	No 2		KNOT & LOCAL GRAIN 50%
TB40045	204	HOLE	HOLE		14.6	71	No 2	CONST	1-1/4" HOLES
TB40046	208	FC	BT			44			NGCD
TB40047	224	F	T			93	No 3	STAND	SOG 1:5
TB40048	272	T	F			98			NGCD
TB40049	172	F	T		15.2	66			NGCD
TB40050	173	F	BT		15.9	63	SS	CONST	3/4" EDGE KNOT
TB40051	283	F	BT		17.9	108	No 2	CONST	SOG 1:8
TB40052	282	T	F			120			NGCD
TB40053	279	F	T			123	No 2	CONST	SOG 1:8
TB40054	108	BK	T	B		118	No 2	CONST	KERF ON ONE END (1-1/4" AT FAILURE HOLE NARROW FACE)
TB40055	184	F	BT			79			NGCD
TB40056	334	F	T			79			NGCD
TB40057	330	B	BT			37	SS	CONST	1/2" EDGE
TB40058	331	F	T			31	No 3	CONST	SOG 1:6
TB40059	332	T				26			TENSION FINGER

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40061	618 B	FC	BT			40	No 1	STAND	5/8, 5/8, 1/2, & 1"
TB40061	287	F	B			54.5			NGCD
TB40061 A	280 A	F	T			51			NGCD
TB40061 B	280 B	BT	T			46	No 2	CONST	SOG 1:9
TB40062	620	BT	BT	BT	18.3	33	SS	CONST	1/4" KNOTS 2
TB40062	292	BT	BT			37.5			NGCD
TB40062 B	291 B	BT	F			26.5	SS	CONST	20% DISP
TB40063	233 C	FC	B			32	No 1	CONST	1" C
TB40063	253 B	F	BT		14	47	No 2	CONST	SOG 1:8
TB40063 A	291 A	BT	T			23	SS	CONST	25% DISP
TB40064	606 C	B	BT			18	No 1	CONST	1-1/8 NARROW FACE
TB40064	293	F	BT			49			NGCD
TB40065	631	F	BT		19.8	18	No 3	STAND	SOG 1:4
TB40065	298	F	T			50	ECON	ECON	1:1.5 LOCAL GRAIN AT FAILURE
TB40066	632	F	B	4x4		36	No 3		SOG 1:7
TB40066	290	FC	BT		21.5	56			NGCD
TB40067	335	FC	F		14.7	47	No 2	CONST	1-1/4" EDGE KNOT
TB40068	508	F	FC			21.5	No 1	STAND	2 KNOTS 5/8, 1-1/8
TB40068	286	F	BK			68			NGCD
TB40069	386	FC	BT			30			NGCD
TB40069	297	F	T			47	No 3	CONST	SOG 1:6
TB40070	616	F	BT	FC		48	SS	CONST	3/4" EDGE KNOT
TB40070	300 & 284	T	T			71	No 3	STAND	SOG 1:4
TB40071	591 B	F	T			75			NGCD
TB40071	591 B	T	F	4x4		77			PLIB STD + BTR (PHOTO 34,35)
TB40071	301	F	T			30	No 1	CONST	T AT 1" EDGE KNOT
TB40072	639	BT	BT			54			NGCD
TB40072	302	F	B		18.1	64			NGCD
TB40073	285	F	BT			91	SS	CONST	1/4" NARROW FACE KNOT
TB40074	278	F	B			76			NGCD
TB40075	101	F	T			101	SS	CONST	LOCAL DEVIATION - 20%
TB40076	289	B	F			105			NGCD
TB40077	303	T	T			50	No 1	CONST	40% COMBO
TB40078	362	F	T			53	No 3	STAND	30% DISTORTED GRAIN
TB40079	407	T	B			45	No 3	STAND	SOG 1:5
TB40080	430	HOLE	T			94	No 2	STAND	1-1/4" HOLE SOME LIMITED SURFACE HYPHAE
TB40081	129	F	BT			90			NGCD
TB40082	434					38			2X10 POSS SCAFFOLD 1"C
TB40083	393	T	FC			97			NGCD
TB40084	479	BT	BT			27	No 1	CONST	1-1/4" C KNOT
TB40085	431 B	BT	F			66	No 3	STAND	1-1/2 HOLE

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40086	433	FC	BT			41	SS	CONST	KERF 1/2" DEEP
TB40087	488	FC	B			26			PHOTO 106
TB40087 C	623	F	BT			17			NGCD
TB40088	484					36			3 SECTIONS OF PLYWOOD PHOTOS 101,102
TB40089	503	T	T			72	No 3	STAND	SOG 1:5
TB40090	299	F	B			40	SS	STAND	25% DISP COMBO
TB40091	500	B	B			36	No 2	STAND	1-1/4" HOLE DRILLED
TB40092	466	B	B			26			NGCD
TB40093	389	BT	F			74	No 1	CONST	1" EDGE KNOT
TB40094	544	FC	FC			43			NGCD
TB40095	421	FC	T			30	No 3	STAND	SOG 1:5
TB40096	567	BT				9			NGCD
TB40097	575			1X4		48			1"X4" PHOTO 104,105 (EDGE BENDING)
TB40098	194	BT	F	4X4		62	No 2		7/8" NARROW FACE
TB40098	195	F	BT	4X4		64	No 2		1" NARROW FACE
TB40099	402	B	BT			39			NGCD
TB40100	510	FC	B			67	No 1	CONST	FAILED AT 1" HOLE
TB40101	573	FC	T		18.2	36	No 2	CONST	SOG 1:8 PHOTO 108-109 SURFACE HYPHAE
TB40103	561	B	FC		16.3	27			NGCD
TB40104	435	FC				35	No 3	STAND	1-1/2 HOLE
TB40105	446	FC	T			22	No 1	CONST	3/4 NARROW FACE
TB40106	581	BT	F			19			NGCD
TB40107	449	FC	T			38.5			NGCD
TB40108	461	B	BT			72	No 1	CONST	30% DISP
TB40109	584 B	BT	BT			19			NGCD
TB40109	560	F	BT			39	No 1	CONST	SOG 1:10
TB40110	450	FC	T			41	SS	CONST	3/8" C KNOT
TB40111	471	BT	BT	BT		69			NGCD
TB40113	534	F	B			32			NGCD
TB40114	499	BT	BT			20			NGCD
TB40115	506	FC	T			54	No 3	CONST	SOG 1:6
TB40116	552	FC	BT			42			NGCD
TB40117	469	FC	BT			19.5			NGCD
TB40118	437	FC	T			52			SHEAR FAILURE SHAKE (PHOTO 107) FULL LENGTH
TB40120	541	F	FC	4X4		23			NGCD
TB40121	528	F	FC	4X4		37.5			4X4 NGCD
TB40122	584	F	BT			22.5	No 1	CONST	3/4" SAW KERF FULL 3" WIDTH
TB40122	436	FC	BT			40			NGCD
TB40123	438	F	T			30			NGCD
TB40124	375	B	T			49	No 1	CONST	30% DISP
TB40125	511	BT	BT			24			NGCD

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40126	585	FC	BT			28			NGCD
TB40127	562	BT	BT			29			NGCD
TB40128	588	F	BT			29	SS	CONST	1/2" NARROW FACE
TB40129	564	FC	T			40	No 3	CONST	SOG 1:6
TB40130	390	BT	F			84			NGCD
TB40131	394	F	T			90	No 1	CONST	40% DISPLACE
TB40131	218	BT	BT			52	No 3	CONST	SOG 1:6
TB40132	779	T	BT			54	SS	CONST	1/2" C
TB40132	329	F	T			94	SS	CONST	25% DISP
TB40133	257	T	T			37.5	SS	CONST	5/8" EDGE KNOT
TB40134	231 B	B	BT			104			NGCD
TB40135	387	BT	F			85	No 1	CONST	40% DISP
TB40136	607	FC	T			20	No 1	CONST	30% DISP
TB40137	606	FC	T			43	No 2	CONST	SOG 1:8 3/4" NARROW FACE
TB40138	575	FC	T			24			NGCD
TB40139	603	FC	T			18	ECON	ECON	2"C & SEVERE LOCAL GRAIN
TB40140	440	FC	T		14.3	32	No 1	CONST	1/2", 3/4, 7/8"
TB40141	132	F	BT			42.5			NGCD
TB40143	573 B	FC	FC			31			NGCD SURFACE HYPHAE (PHOTO 110-111)
TB40144	392	F	T			90	No 3	CONST	SOG 1:6
TB40145	587	FC	F			25.5	No 2	STAND	45% CROSS SECTION
TB40146	396	FC	B			55			NGCD
TB40147	447	FC	F	4X4		40			NGCD
TB40148	406	FC	B		17.9	34	No 3	CONST	SOG 1:6
TB40149	592			1x4		27.5			3-1X4 ZIPTIED TOGETHER (PHOTO 103)
TB40152	610	B	T			31	No 3	CONST	CONCRETE POSS. 1:6 SOG
TB40153	609	FC	BT			51	No 3	STAND	SOG 1:5
TB40154	374	F				33			NGCD
TB40155	494	FC	F			36	SS	CONST	1/2" NARROW FACE - COVERED IN CONCRETE
TB40156	483	B	BT			24			NGCD
TB40157	428	T				34			COVERED IN CONCRETE
TB40158	426	F	T	1X4		67			1X4
TB40159	400	T	BT			64	No 3	CONST	SOG 1:7
TB40160	324	FC	BT			61	SS	CONST	20% DISP
TB40167	638	F	B			15.5			NGCD
TB40167	521	FC	T			60			NGCD
TB40168	690	BT	T			36	No 3	CONST	SOG 1:6
TB40169	702 B	T				27.5			NGCD
TB40169	702 A					35			
TB40170	795	B	T			36			NGCD
TB40171	866 B	T				19			NGCD

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40171 A	806 A					20			1 TENSION FINGER
TB40173	774	FC	BT			45	No 1	STAND	2" (4 KNOTS) PC COMBO
TB40174?	685	B	BT			28	SS	CONST	3/4" TWEENER (PHOTO 47)
TB40175	758	B	BT			26			NGCD
TB40176	439 C	FC	T			50	No 2	STAND	1", 1", 1 1/2"
TB40177	707					36	No 1	CONST	SOG 1:10
TB40178	446	T	FC		17.5	65	SS	CONST	2 KNOTS 2-1/2" EDGE KNOTS
TB40179	182	F	BT			100	No 1	CONST	SOG 1:10
TB40180	791	B	B			39	SS	CONST	3/4" EDGE KNOT
TB40181	366	F	B		15.2	36			NGCD
TB40182	458	B	T			51	No 2	CONST	SOG 1:8
TB40183	131	FC	T			63			NGCD
TB40184	363	T	BT			30	SS	CONST	3/4" NARROW FACE
TB40185	686	T	BT			39			NGCD
TB40186	595	BT	FC			30			NGCD
TB40187	617	BT	B			51			NGCD
TB40187 B	617 B	B	T			27	ECON	ECON	SOG 1"-1" EXTREME LOCAL DEVIATION
TB40188	313	F	BT			118	No 1	CONST	
TB40189	311	F	B			108	SS	CONST	1/4" NARROW FACE KNOTS
TB40190	310	BT	F			117	No 1	CONST	1-1/8 NARROW FACE KNOT
TB40191	318	BT	F			136	No 2	CONST	SOG 1:8
TB40192	314	BT	F			117	No 2	STAND	1 1/8", 1", and 5/8"
TB40193	315	F	B			120	SS	CONST	1/2" C
TB40194	782	T	T			98	SS	CONST	SOG 1:12
TB40195	327	F	T			121	No 1	CONST	5/8" EDGE KNOT
TB40199	439	FC	T		15.1	24	No 1	CONST	5/8 NARROW FACE
TB40200	245	F	BT			53	No 2	CONST	1-1/4" PHOTOS 16,17,18, GENERAL ON CDBOARD
TB40201	201	PIPE	FC			66	No 1	CONST	1" HOLE PIPE
TB40202	624	F	T		21.1	59			NGCD
TB40203	247	F	T			58			NGCD
TB40204	246	F	T			54			NGCD
TB40205	608	F	PIPE			48	No 1	CONST	PIPE 1"
TB40206	258	F	B			51	No 1	CONST	1-3/8 C KNOT
TB40207	238	F	T			54	ECON	ECON	3" WIDE FACE TO 0 WITH 1:2 SOG
TB40208	214	F	BT			73			NGCD
TB40209	176	F	T		21.1	62	No 1	CONST	SOG 1:10
TB40210	244	T	T	T		67.5			NGCD (SOME TRIM/KNOTS) (PHOTOS 13,14)
TB40211	178	F	T			60	No 1	CONST	SOG 1:10
TB40212	360	BT	T			57			
TB40213	373	BT	T	T		59	No 2	CONST	SOG 1:8
TB40214	621	BT	BT			39			NGCD

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40215	625	B	B	F		64			NGCD
TB40216	174	T	BT			76	No 2	CONST	1-1/4 - 2 IN.
TB40217	254	T	FC			76	No1	CONST	3 KNOTS 1/2" EACH
TB40218	196	BT	F			89	No 2	CONST	SOG 1:9
TB40219	372	F	BT			30			TOO MUCH CONCRETE
TB40220	659	BT	BT			33			NGCD
TB40222	651 B	T	BT			25			NGCD
TB40223	312	B	F			112.5	No 1	CONST	7/8", 1/2", 5/8"
TB40224	319	T	F			124	No 1	CONST	1" KNOT
TB40225	125	F	BT			69			
TB40226	317	BT	F			123	No 2	CONST	SOG 1:9
TB40227	351	BT	F			101	No 1	CONST	7/8", 1/2", 1"
TB40228	133	F	T			50	ECON	ECON	SOG 1:3 (PHOTO #15)
TB40229	294 A 4X4	F	BT			82	No 3	CONST	SOG 1:7
TB40230	374	BT	B			111	SS	CONST	1/2" KNOT
TB40231	133 B	F	BT			71			NGCD
TB40233	215	F	T			60	No 2	CONST	1-1/4" EDGE KNOT
TB40234	232	F	B			112	No 2	CONST	1", 1", 1 1/2"
TB40236	171	T	T			46	ECON	ECON	SOG 1:3 (PHOTO #14)
TB40237	354	T	T	BT		50	No 3	STAND	SOG 1:4 @ MIDDLE FAILURE
TB40238	378	BT	BT			37	No 1	CONST	SOG 1:10
TB40239	401	B	BT			44			NGCD
TB40240	213	F	T			36			NGCD
TB40241	369	F	BT			28			NGCD
TB40242	130	F	T		17.3		No 2	CONST	1" EDGE OR SOG 1:8
TB40243	654	BT	BT			27			NGCD
TB40244	361	B	B			51	SS	CONST	3/4" EDGE KNOT
TB40245	657	F	BT			40			NGCD
TB40246	494 B	FC	T			12	No 3	STAND	1:4 SOG
TB40246	359			2X10		66			SPIB DI-65 MEETS SCAFFOLD KD 19 OSHA 1910,28 #350 (PHOTOS 20,21,22)
TB40247	418 D	BT	B		24.8	28			NGCD (1/2 WIDTH OF PIECE)
TB40249	235 B	FC	T			61			NGCD
TB40250	376	F	BT			60			NGCD
TB40251	512 B	FC	T			43			NGCD
TB40252	242	F	BT			45.5	SS	CONST	3/4" EDGE KNOT
TB40253	131 B	T	BT			37			NGCD
TB40254	368	F	BT			38.5	SS	CONST	1/2" SPIKE ON 1 SURFACE
TB40255	259	F	T			40			NGCD
TB40256	676	T	T			34	No 1	CONST	2-3/14" (2KNOTS) COMBO PC
TB40257	241	T	BT			34	No 2	CONST	FULL LENGTH FAILURE SOG 1:8
TB40259	367	BT	B			26			NGCD

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40260	575	FC	PIPEHOLE			41	No 2	CONST	P-HOLE 1" DIAMETER NARROWFACE
TB40261	365	FC	B			45	SS	CONST	3/4" EDGE KNOT
TB40262	208 B	T	T		15.2	52	SS	CONST	3/4" C ONE FACE LOTS OF CONCRETE ATTACHED
TB40263	673	BT	BT			48	SS	CONST	5/8C
TB40264	658 (598)	F	T			48			NGCD
TB40265	671	BT	BT			28	No 2	CONST	SLOPE 1:7
TB40266	651	T	BT			35	No 1	CONST	2-1/4 TOTAL (3 KNOTS)
TB40267	607 B	FC	T			22.25	No 1	CONST	2-1/4 (2KNOTS) COMBO PC
TB40268	212	F	T	T		120			NGCD (FAILED AT KNOT)
TB40269	481	BT	BT			54			NGCD
TB40270	487	T	BT		14.8	49	No3	STAND	SOG 1:5, 6 TYPEND
TB40271	628	F	BT			52	No 2	STAND	2" TOTAL 4 KNOTS (PHOTO 33)
TB40272	803	F	FC			31.5	No 3	CONST	SOG 1:6
TB40273	840	BT	T	BT		56	ECON	ECON	SOG 1:3 AT CENTER FAILURE
TB40274	737	BT	BT			45	SS	CONST	5/8" C
TB40275	582	F	BT			42	No 1	CONST	7/8, 1-1/2 PITH ON EDGE 2 KNOTS COMBO
TB40276	118	BT	BT			38			NGCD
TB40277	824	B	BT			17			NGCD
TB40278	480	B	T			31	No 3	STAND	T @ 1:4
TB40279	823	F	T			30	No 1	CONST	1-1/2 TOTAL (3 KNOTS COMBO)
TB40280	698	F	C			22			NGCD
TB40281	773	B	T			27			NGCD
TB40282	523 B	FC	BT			31.5	SS	CONST	3/8" EDGE KNOT
TB40283	739	FC	B			25	SS	CONST	1/4" EDGE KNOT
TB40284	787	BT	T			28	No 2	CONST	SOG 1:8
TB40285	502	FC	BT		23.2	17.5			NGCD
TB40286	557	T	BT			28	SS	CONST	1-1/2" NARROW FACE KNOT
TB40287	228	BT	BT			56			BOTH BROKE @ KNOTS 1/2" EDGE KNOT
TB40288	330	T	B	BT		49	No 2	STAND	2-1/2" KNOTS
TB40289	708	BT	B			34	ECON	ECON	TYP @ PIPE HOLE @ B/2"-3C PC COMBO
TB40290	756	F	B			12	SS	CONST	1-1/2" TOTAL COMBO 2 KNOTS
TB40291	706	FC	T			36	SS	CONST	VD (PHOTO #12) 1/4" EDGE
TB40292	228	FC	BT			57	No 1	CONST	2 KNOTS 1" EACH NARROW FACE
TB40293	520	FC	BT			53	ECON	ECON	SOG LOCALIZE DISTORTED GRAIN
TB40294	635	F	T			59	No 1	CONST	5/8" EDGE KNOT
TB40295	517	F	T			83	No 2	CONST	COVERED IN CEMENT - EST. #2
TB40296	518	FC	BT			69	No 3	CONST	SOG 1:6
TB40297	726	F	B		15.7	30.5	No 3	CONST	SOG 1:6
TB40298	736	F	B			39			NGCD
TB40299	777	B	T		18.9	41	ECON	ECON	SOG 1:3 (PHOTO #11)
TB40300	852	B	T			36	ECON	ECON	T @ 1:3 (PHOTO #10)

White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB40301	817	BT	B			33	SS	CONST	1-3/4" TOTAL (3 KNOTS)
TB40302	770	B	F			42	ECON	ECON	SOG 1:3 (PHOTO #9)
TB40303	764	BT	BT			34			NGCD
TB40304	683 C	BT	BT	4x4		26	No 1		2" (2 KNOTS) PC COMBO
TB40306	559	B	T			35	ECON	ECON	SOG 1:3 TYPEND
TB40307	482					32	No 1	CONST	2" (2KNOTS) COMBO
TB40308	177	BT	F		13.6	48	SS	CONST	SOG 1:12
TB40309	683 B	FC	BT	4x4		29	No 2		3" TOTAL PITH CTR COMBO 4 KNOTS
TB40311	735	F	B	4x4		32	No 1		1" EDGE KNOT
TB40312	543 B	FC	BT	F		68			NGCD
TB40313	819	T	F			76			NGCD
TB40314	516	F	BT			86			NGCD
TB40315	261	F	B	2x10	16.8	121	No 2		SOG 1:8 FLATWISE FAILURE AT END
TB40316	136	F	BT		13.6	110			REJECT (PHOTOS 53-56) BUT DID NOT BREAK AT DEFECT
TB40317	515	T	T			36	ECON	ECON	SOG 1:2
TB40318	812	T	B		15.4	31	No 1	CONST	SOG 1:10
TB40319	837	FC	B			11			NGCD
TB40320	843	T	T			29	No 3	STAND	SOG 1:4 (PHOTO #8)
TB40323	314 B	B	B			22	SS	CONST	1/2" C KNOT
TB40324	792	B	BT			22	No 1	CONST	1" NF KNOT
TB40325	260	F	B			33.5			NGCD
TB40326	714	F	B			19.5			NGCD
TB40327	512	FC	BT			21	No 1	CONST	BT @ 1-3/16 NFKNOT
TB40328	680	F	T			23.5			NGCD
TB40329	715	F	BT		17.2	32	SS	CONST	5/8 EDGE
TB40330	587	F	BT			30.5	No 2	CONST	3" TOTAL (4 RINGS) COMBO
TB40825	786	B	B		15.1	24			VG, NGCD
TB41005 A	5 ZONE 41A-B	F	FC			89.5			NGCD
TB41005 B	ZONE 41 A-B	FC	F			102			NGCD
TB41006	413	F	F			120			65 DNS IND, K019 MILL 350 PHOTO 155
TB41007 B	ZONE 41G	F	FC			28.5	SS	CONST	20% COMBO
TB41008	ZONE 41B	F	F			168	No 3	STAND	1-1/2" EDGE KNOT
TB41009	ZONE 41B-C	F	T	F		160	SS	CONST	25% AT FAILURE 5/8 C
TB41010	ZONE 41B-C, 22A, 22B	F	BT			118			NGCD
TB41011 A	ZONE 41B-C	FC	T			120			NGCD
TB41011 B	ZONE 41 BC	F	BT			36			NGCD
TB41012	40 ZONE 41B-C	F	T			140	No 2	STAND	11/16 EDGE KNOT & LOCAL DIST GRAIN
TB41013 A	28 B	F	T			95			NGCD PHOTO 173-176
TB41013 B		T	F			29			NGCD PHOTO 173-176
TB41014	ZONE 41B	BT	F			99			NGCD
TB41015	ZONE 41B	F	T	F	17.7	120			32" TO FAIL FROM END N6CD

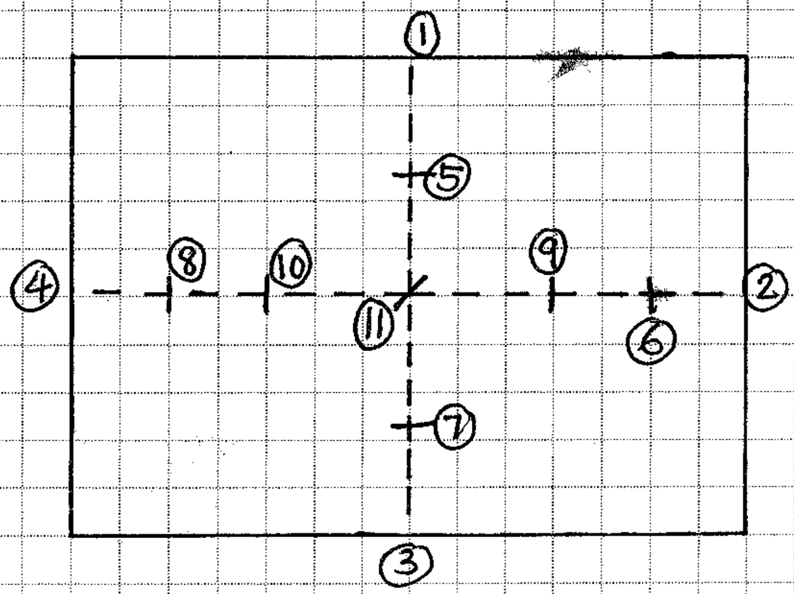
White Tag	Black Mark	Failure Modes			MC (%)	Length (in.)	VisGrd (joists)	VisGrd (C,S,U)	Comments
TB41016	ZONE 41B	F	BT			89	No 3	CONST	BT AT 1/2" KERF SOG IN 1:6
TB41017		F	BT	BK		78	No 1	CONST	1-1/8 C KNOT
TB41019	308 ZONE 41B	T	F			104	No 2	CONST	SOG 1:8
TB41020	ZONE 41B	FC	BT			87			NGCD
TB41021	ZONE 41B	F	F	4X4		91	No 2		50% COMBO
TB41022	10 ZONE 41B	F	T			102	No 3	CONST	SOG 1:7
TB41023	ZONE 41F	F	HOLE			75	No 3	STAND	1-3/4" HOLE AT FAILURE
TB41025	66 ZONE 41G	T	F			50	SS	CONST	20% DISP COMBO
TB41026	ZONE 41G	F	T	4X4			No 1		A&B 128-23" 1"C PHOTO 145
TB41027	72 ZONE 41G	F	BT			59	No 1	CONST	SOG 1:11
TB41028	71 ZONE 41G	BT	F			63	SS	CONST	7/8 C KNOT
TB41029	ZONE 41G	F	T			74.5	SS	CONST	25% DISP COMBO
TB41030	ZONE 41G	F	T			142			NGCD
TB41031	ZONE 41G	B	B			65			DISCOLORED POSSIBLE DECAY NGCD
TB41032	79 ZONE 41G	T	FC			82	SS	CONST	5/8 EDGE
TB41033	ZONE 41G	B	T			75	No 3	CONST	SOG 1:6 AT T FAIL
TB41034	ZONE 41G	F	T		19.9	71	No 3	STAND	HOLE 1-1/2 NARROW FACE + LOCAL GRINA DEVIATION/CONCRETE
TB41035	ZONE 41J	T	T	F		132	No 2	CONST	6' + 5' = 11' 40% DIST PHOTOS 150-152
TB41035 C	ZONE 41C	B	F	4X4		52			PLIB STD & BTR GRAIN D-FIR PHOTOS 112, 113
TB41036	ZONE 41J	F	B			81.5	No 2	STAND	LOCAL GRAIN + SPIKE KNOT = 50
TB41037	ZONE 41J	F	T			133.5			NGCD
TB41038	ZONE 41J	F	T			89			T AT KERF - NGCD
TB41039	50 ZONE 41E	F	BT			151	SS	CONST	25% COMBO
TB41040	81 ZONE 41G	F	T	4X4		83	No 2		LOCAL GRAIN 40% CONCRETE
TB41040 B	ZONE 41G	F	T	4X4		120	No 2		SOG 1:9
TB41041	ZONE 41G	F	FC			28.5			NGCD
TB41041	ZONE 41J	F	BT	BT		101	No 2	CONST	SOG 1:8
TB41042	ZONE 41J	F	B		17	141			NGCD
TB41043	ZONE 41J	F	BT			56			BT WITH CONCRETE NGCD
TB41045	ZONE 41G	FC	FC			28.5			NGCD
TB41046	ZONE 41G	F	FC			28.5	SS	CONST	5/8" EDGE KNOT
TB41047	ZONE41G	F	FC			29	SS	CONST	3/4 C
TB41048	ZONE 41G	F	BT		18.7	84	No 2	CONST	SOG 1:8
TB41048 A	ZONE 41G	F	B			111	No 2	CONST	SOG 1:9
TB41049 A	ZONE 41G	FC	T		14.1	99			NGCD
TB41049 B	ZONE 41G	T	BT	T		60			COVERED IN CONCRETE
TB41050 A	ZONE 41G	T	F			134.5			NGCD
TB41050 B	109B ZONE 41G	F	T			109	No 1	CONST	SOG 1:10 AT FAIL
TB41051	113 ZONE 41G	F	HOLE		17.3	161	No 3	STAND	HOLE 1-1/2"
TB41053 B	ZONE 41G	F	T			51			NGCD

APPENDIX III
Small Clear Mechanical Test Results

WOOD ADVISORY SERVICES, INC.
3700 Route 44 Suite 102
P.O. Box 1322
MILLBROOK, NEW YORK 12545
(845) 677-3091
FAX (845) 677-6547

JOB ____
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE ____
CHECKED BY ____ DATE ____
SCALE ____

TYPICAL CROSS SECTION OF 3X4 LUMBER
(2½" x 3½")



LOCATIONS 1 THROUGH 11 EVALUATED
FOR EACH MICROBIOLOGICAL ANALYSIS
PERFORMED ON EACH 3X4
LUMBER RIB

SCHEMATIC #1

Client: Dan Eschenasy
Project Spring Street
Job No. 08.126
Test ASTM D-143 Static Bending

Spruce (weighted average of red, balck and white):
MOR at 12% = 10,644 psi
MOE at 12% = 1.511 x10^6 psi

Fir (Balsam fir)
MOR at 12% = 8,341 psi
MOE at 12% = 1.432 x10^6 psi

Wood Decay Chart:

- Occasional
- Light
- Moderate
- Heavy

Eastern Spruce

Sample	Span (in.)	Height (in.)	Width (in.)	Load/Defl. (lb./in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	MOR (psi)	MORadj (psi)	MOE (psi)	MOEadj (psi)	Residual MOR	Residual MOE	MC%	SG
1-1	14	0.97	0.98	1737.1	328.0	6.30	5.56	15.32	7470	7842	1.33E+06	1.36E+06	0.74	0.90	13.3	0.36
1-3	14	0.98	0.98	1831.4	398.2	6.41	5.64	15.74	8885	9447	1.36E+06	1.39E+06	0.89	0.92	13.7	0.36
2-2	14	0.98	0.98	1674.9	336.8	5.60	4.91	15.18	7515	8110	1.25E+06	1.28E+06	0.76	0.85	14.1	0.32
2-3	14	0.98	0.98	1404.0	345.4	5.58	4.93	15.08	7707	8053	1.04E+06	1.06E+06	0.78	0.70	13.2	0.33
3-1	14	0.98	0.97	2013.7	509.9	8.23	7.23	15.10	11494	12303	1.51E+06	1.55E+06	1.00	1.00	13.8	0.48
3-2	14	0.97	0.98	2135.4	473.2	7.80	6.82	15.02	10777	11768	1.64E+06	1.69E+06	1.00	1.00	14.4	0.45
4-1	14	0.97	0.98	1847.4	320.5	6.91	6.12	15.36	7299	7550	1.42E+06	1.43E+06	0.71	0.95	12.9	0.40
4-2	14	0.98	0.98	1523.4	375.9	6.23	5.64	14.99	8387	8530	1.13E+06	1.14E+06	0.80	0.75	12.5	0.37
5-1	14	0.97	0.97	1910.3	436.3	7.04	6.21	15.17	10039	10561	1.48E+06	1.51E+06	0.99	1.00	13.4	0.41
10-1	14	1.00	1.00	2373.7	423.0	7.65	6.69	16.03	8883	9693	1.63E+06	1.68E+06	0.91	1.00	14.3	0.42
11-1	14	1.00	1.00	2536.0	461.4	8.23	7.20	15.47	9689	10555	1.74E+06	1.79E+06	0.99	1.00	14.3	0.47
11-2	14	1.00	1.00	2366.9	482.3	7.39	6.46	15.42	10128	11070	1.62E+06	1.68E+06	1.00	1.00	14.4	0.42
Mean									9023	9623	1429761	1462660	0.88	0.92	13.7	0.40

Eastern Fir

Sample	Span (in.)	Height (in.)	Width (in.)	Load/Defl. (lb./in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	MOR (psi)	MORadj (psi)	MOE (psi)	MOEadj (psi)	Residual MOR	Residual MOE	MC%	SG
6-2	14	0.98	0.97	2077.1	412.8	6.60	5.72	15.15	9305	9800	1.56E+06	1.58E+06	1.00	1.00	13.6	0.38
7-1	14	0.98	0.98	1674.3	335.3	5.92	5.22	15.00	7481	7823	1.25E+06	1.26E+06	0.94	0.88	13.4	0.35
7-2	14	0.97	0.98	2354.9	429.6	6.70	5.94	14.88	9784	10033	1.81E+06	1.82E+06	1.00	1.00	12.8	0.40
7-3	14	1.00	1.00	2416.0	497.0	6.94	6.13	15.74	10437	10846	1.66E+06	1.68E+06	1.00	1.00	13.2	0.39
8-1	14	1.00	0.99	2121.1	397.4	6.50	5.73	15.86	8430	8823	1.47E+06	1.49E+06	1.00	1.00	13.4	0.36
8-2	14	1.01	1.01	1743.4	378.4	6.08	5.34	15.91	7713	8180	1.15E+06	1.17E+06	0.98	0.82	13.9	0.34
9-2	14	0.99	1.00	1984.6	395.8	6.47	5.71	16.13	8481	8840	1.40E+06	1.42E+06	1.00	0.99	13.3	0.35
9-3	14	1.01	1.01	2192.6	367.5	6.95	6.14	15.98	7491	7779	1.45E+06	1.46E+06	0.93	1.00	13.2	0.38
Mean									8640	9016	1467142	1485295	0.98	0.96	13.4	0.37

Client:	Dan Eschenasy
Project	Spring Street
Job No.	08.125
Test	ASTM D-143 Shear Parallel to Grain

Spruce (weighted average of red, balck and white):

$Shear_{//}$ at 12% = 1,182 psi

Fir (Balsam fir)

$Shear_{//}$ at 12% = 938 psi

Wood Decay Chart:

Occasional	
Light	
Moderate	
Heavy	

Spruce

Sample	Length (in.)	Width (in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	SHEAR (psi)	SHEARadj (psi)	Residual Shear	MC%	SG	SG adj
1-1	2.00	2.00	4259	18.95	16.73	46.71	1065	1109	0.94	13.3	0.36	0.33
2-1	2.01	2.00	3379	16.72	14.69	42.89	841	891	0.75	13.8	0.34	0.31
3-1	2.00	2.01	5689	24.45	21.43	45.67	1415	1513	1.00	14.1	0.47	0.42
3-2	2.01	2.00	5740	25.77	22.52	47.62	1428	1543	1.00	14.4	0.47	0.42
4-1	2.01	2.01	4658	20.01	17.76	45.63	1153	1178	1.00	12.7	0.39	0.35
4-2	2.03	2.00	4587	25.61	22.69	54.89	1130	1162	0.98	12.9	0.41	0.37
5-1	2.03	2.00	4969	17.86	15.73	40.08	1224	1286	1.00	13.5	0.39	0.36
11-1	2.05	2.01	3913	22.53	19.83	47.69	950	1000	0.85	13.6	0.42	0.37
11-2	2.02	2.01	4479	21.57	18.97	44.95	1103	1165	0.99	13.7	0.42	0.38
Mean							**1145**	**1205**	**0.94**	**13.6**	**0.41**	**0.37**

Fir

Sample	Length (in.)	Width (in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	SHEAR (psi)	SHEARadj (psi)	Residual Shear	MC%	SG	SG adj
6-1	2.05	2.00	5128	20.70	18.27	46.38	1251	1303	1.00	13.3	0.39	0.36
6-2	2.01	2.01	5027	20.73	18.33	46.40	1244	1288	1.00	13.1	0.40	0.36
7-1	2.01	2.00	4375	21.14	18.74	46.79	1088	1116	1.00	12.8	0.40	0.36
7-2	2.03	2.00	4694	21.43	18.98	46.41	1156	1190	1.00	12.9	0.41	0.37
8-1	2.01	2.01	3592	18.67	16.54	45.53	889	914	0.97	12.9	0.36	0.33
Mean							**1126**	**1162**	**0.99**	**13.0**	**0.39**	**0.36**

Client: Dan Eschenasy
Project Spring Street
Job No. 08.125
Test ASTM D-143 Compression Perpendicular to Grain

Spruce (weighted average of red, balck and white):
Comp.Perp at 12% = 1,004 psi

Fir (Balsam fir)
Comp.Perp at 12% = 862 psi

Wood Decay Chart:

Occasional	
Light	
Moderate	
Heavy	

Spruce

Sample	Width (in.)	Height (in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	Comp (psi)	Comp adj (psi)	Residual Comp	MC%	SG	SG adj
2-1	2.01	2.00	2545	25.11	22.07	59.65	633	688	0.69	13.8	0.37	0.34
3-1	2.01	2.02	4024	32.85	28.71	61.40	991	1110	1.00	14.4	0.47	0.42
5-1	2.00	2.00	3075	27.76	24.61	56.41	769	798	0.79	12.8	0.44	0.39
11-1	2.01	1.99	2795	32.79	28.80	60.90	699	762	0.76	13.9	0.47	0.42
Mean							**773**	**840**	**0.81**	**13.7**	**0.44**	**0.39**

Fir

Sample	Width (in.)	Height (in.)	Max Load (lb)	Green Wt (g)	OD WT (g)	OD Vol (g)	Comp (psi)	Comp adj (psi)	Residual	MC%	SG	SG adj
6-1	2.00	2.00	1855	27.84	24.64	62.58	464	486	0.56	13.0	0.39	0.36
7-1	2.00	2.01	1919	27.26	24.10	63.99	477	504	0.58	13.1	0.38	0.34
Mean							**471**	**495**	**0.57**	**13.0**	**0.39**	**0.35**

APPENDIX IV
Concentrated Load Test Results

Client: Dan Eschenasy
Project Spring Street
Job No. 08.125
Test Concentrated Load Test Data

Loading Condition #1 - Base plate with interior edge 11" from panel edge
Loading Condition #2 - Base plate with interior edge 20"-24" from panel edge
Defl. - The deflection read on the engineers ruler during the test
Actual Defl. - The actual deflection after adjusting for the initial offset (first reading for Defl.)
Meter Value - The digital output from the meter
Actual Value - The digital meter output value adjusted using calibration between proving ring and digital meter

TP8-A #1: 2'x4', Loading Condition #1 — **Thickness = 0.647 in.**

	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5.3	0	0	0
	5.4	0.1	500	499
	5.5	0.2	1000	1002
	5.55	0.25	1500	1506
	5.6	0.3	2000	2009
	5.65	0.35	2500	2513
	5.7	0.4	3000	3017
	5.775	0.475	3500	3520
	5.825	0.525	4000	4024
	5.9	0.6	4500	4527
Max.	6.025	0.725	4958	4988

TP8-A #2: 2'x4', Loading Condition #1 — **Thickness = 0.642 in.**

	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5.2	0	0	0
	5.275	0.075	500	499
	5.3	0.1	1000	1002
	5.375	0.175	1500	1506
	5.425	0.225	2000	2009
	5.475	0.275	2500	2513
	5.525	0.325	3000	3017
	5.6	0.4	3500	3520
	5.7	0.5	4000	4024
	5.8	0.6	4500	4527
	5.95	0.75	5000	5031
Max.	6.9	1.7	5119	5151

TP10A: 4'x4', Loading Condition #2 — **Thickness = 0.653 in.**

Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
5.3	0	0	0
5.375	0.075	500	499
5.425	0.125	1000	1002
5.475	0.175	1500	1506
5.5	0.2	2000	2009
5.575	0.275	2500	2513
5.625	0.325	3000	3017
5.675	0.375	3500	3520
5.7	0.4	4000	4024
5.775	0.475	4500	4527
5.8	0.5	5000	5031
5.825	0.525	5500	5534
		5900	5937
5.925	0.625	6000	6038
6.025	0.725	6500	6541
6.075	0.775	7000	7045

	6.125	0.825	7500	7549
	6.2	0.9	8000	8052
	6.3	1	8500	8556
Max.	6.3	1	8640	8697

TP9A: 4'x4', Loading Condition #2 **Thickness = 0.653 in.**

	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5.2	0	0	0
	5.275	0.075	500	499
	5.325	0.125	1000	1002
	5.375	0.175	1500	1506
	5.425	0.225	2000	2009
	5.475	0.275	2500	2513
	5.5	0.3	3000	3017
	5.55	0.35	3500	3520
	5.6	0.4	4000	4024
	5.7	0.5	4500	4527
	5.775	0.575	5000	5031
	5.825	0.625	5500	5534
	5.9	0.7	6000	6038
	5.95	0.75	6500	6541
	6	0.8	6575	6617
Max.	6.1	0.9	6722	6765

TP9A-#2: 2'x4', Loading Condition #1 **Thickness = 0.653 in.**

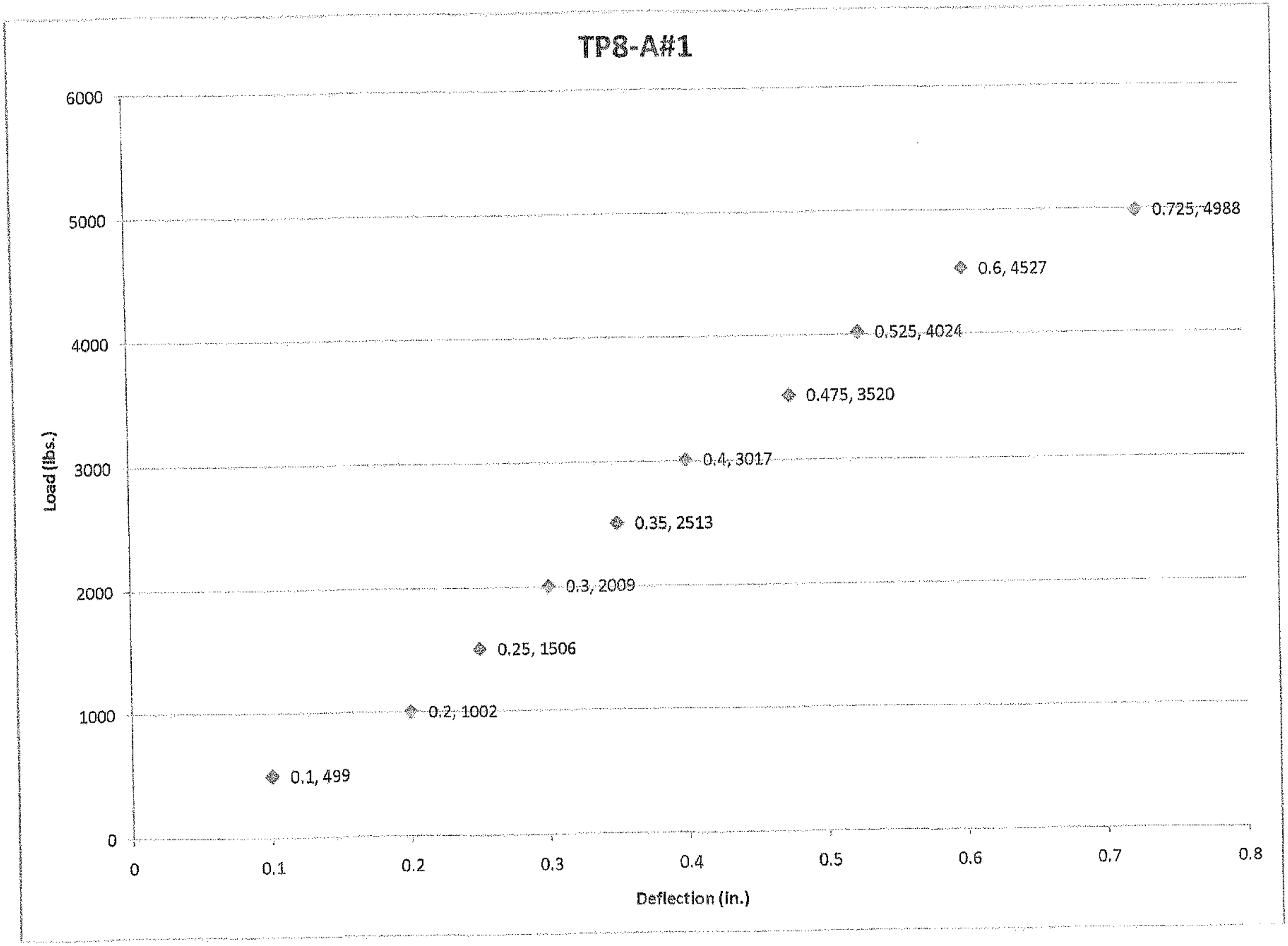
	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5	0	0	0
	5.1	0.1	500	499
	5.15	0.15	1000	1002
	5.2	0.2	1500	1506
	5.25	0.25	2000	2009
	5.3	0.3	2500	2513
	5.35	0.35	3000	3017
	5.425	0.425	3500	3520
	5.6	0.6	4000	4024
	5.625	0.625	4100	4124
	5.65	0.65	4200	4225
	5.7	0.7	4300	4326
	5.75	0.75	4400	4427
	5.825	0.825	4500	4527
	5.9	0.9	4590	4618
	6.2	1.2	4600	4628
	6.3	1.3	4700	4729
	6.425	1.425	4800	4829
Max.	6.7	1.7	4827	4857

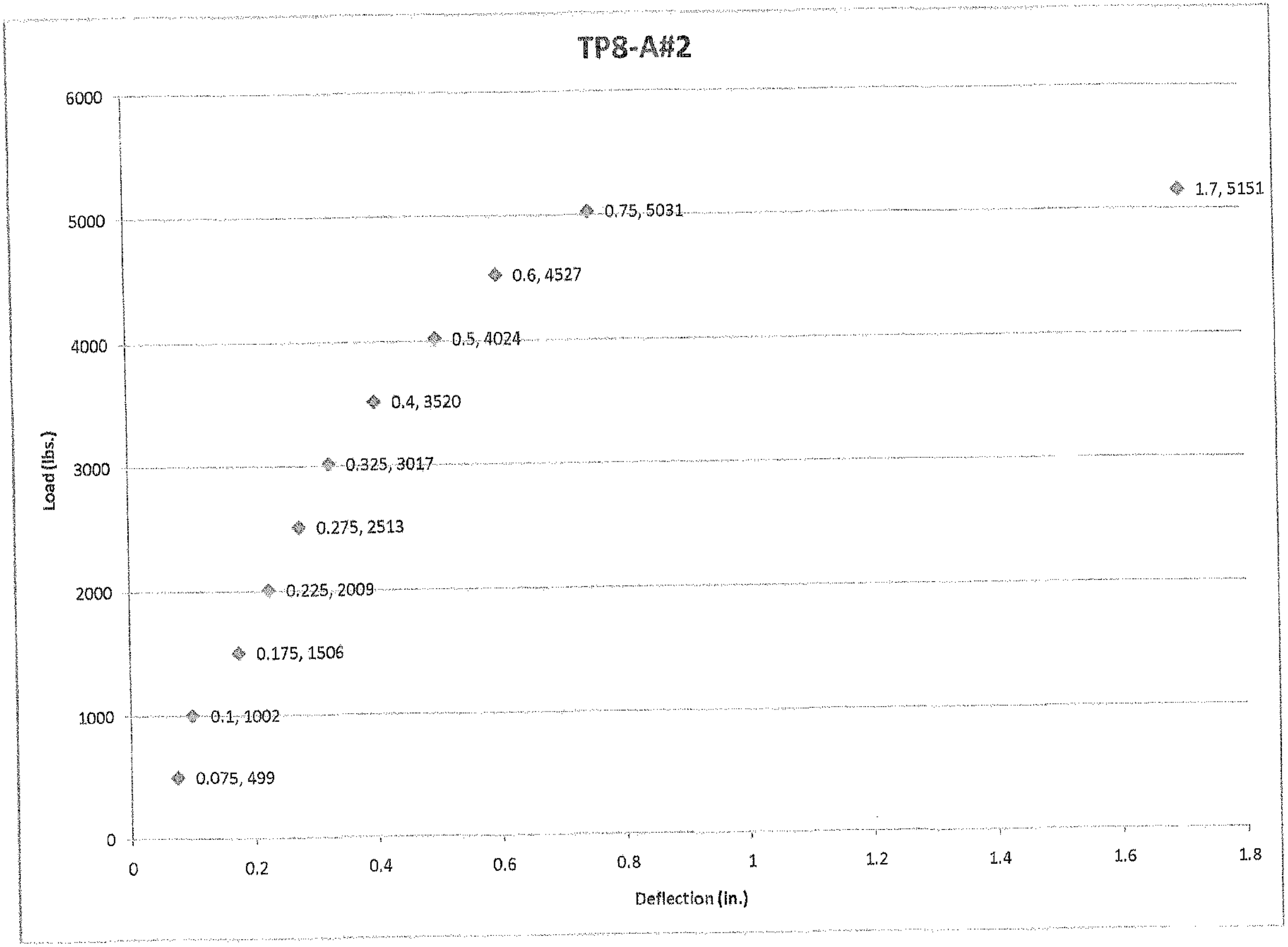
PW4055-#1: 4'x4', Loading Condition #2 **Thickness = 0.606 in.**

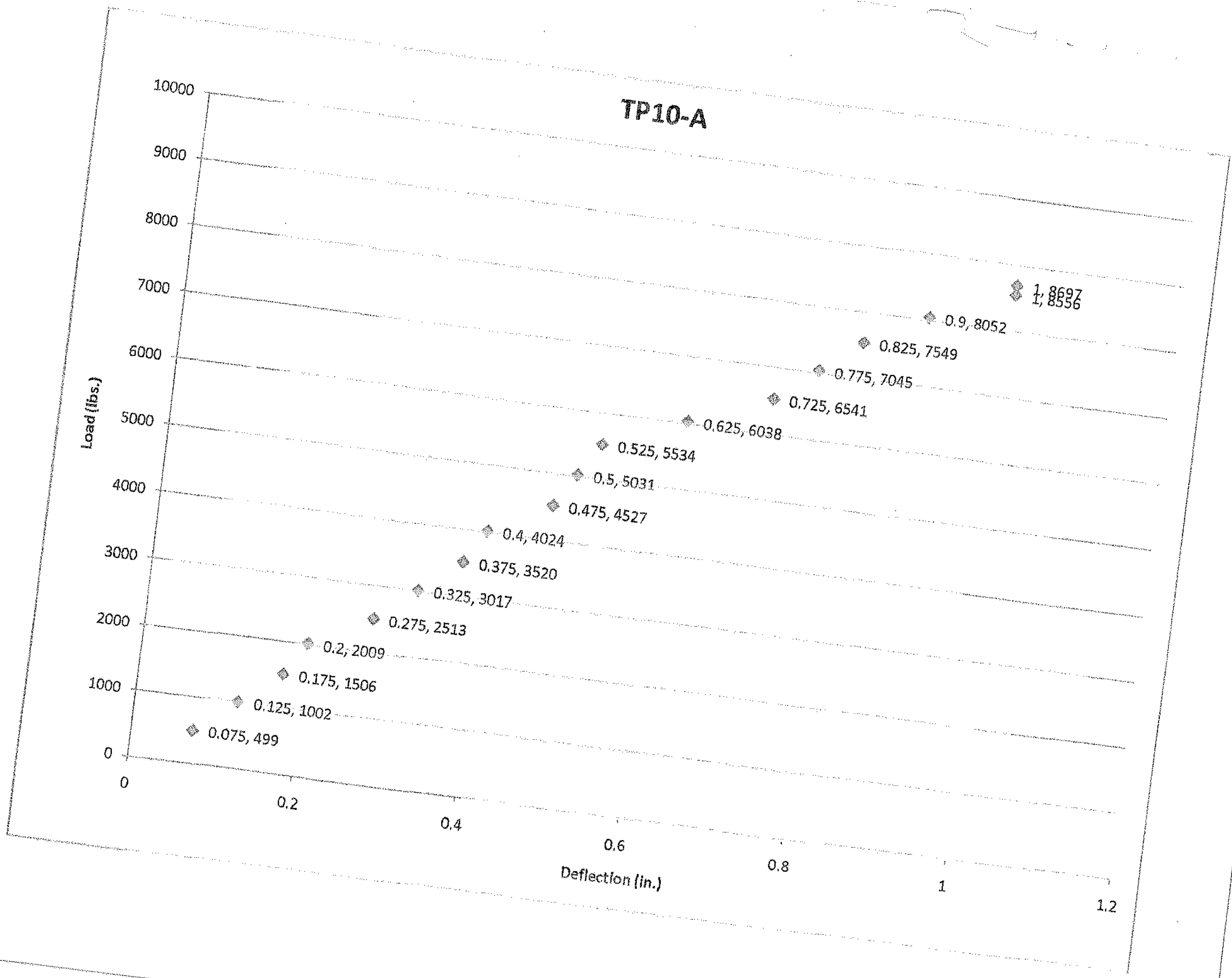
	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5	0	0	0
	5.075	0.075	500	499
	5.15	0.15	1000	1002
	5.2	0.2	1500	1506
	5.275	0.275	2000	2009
	5.325	0.325	2500	2513
	5.4	0.4	2600	2614
	5.475	0.475	3000	3017
	5.5	0.5	3131	3148
	5.525	0.525	3500	3520
Max.	5.6	0.6	3731	3753
	6.2	1.2	3695	3716

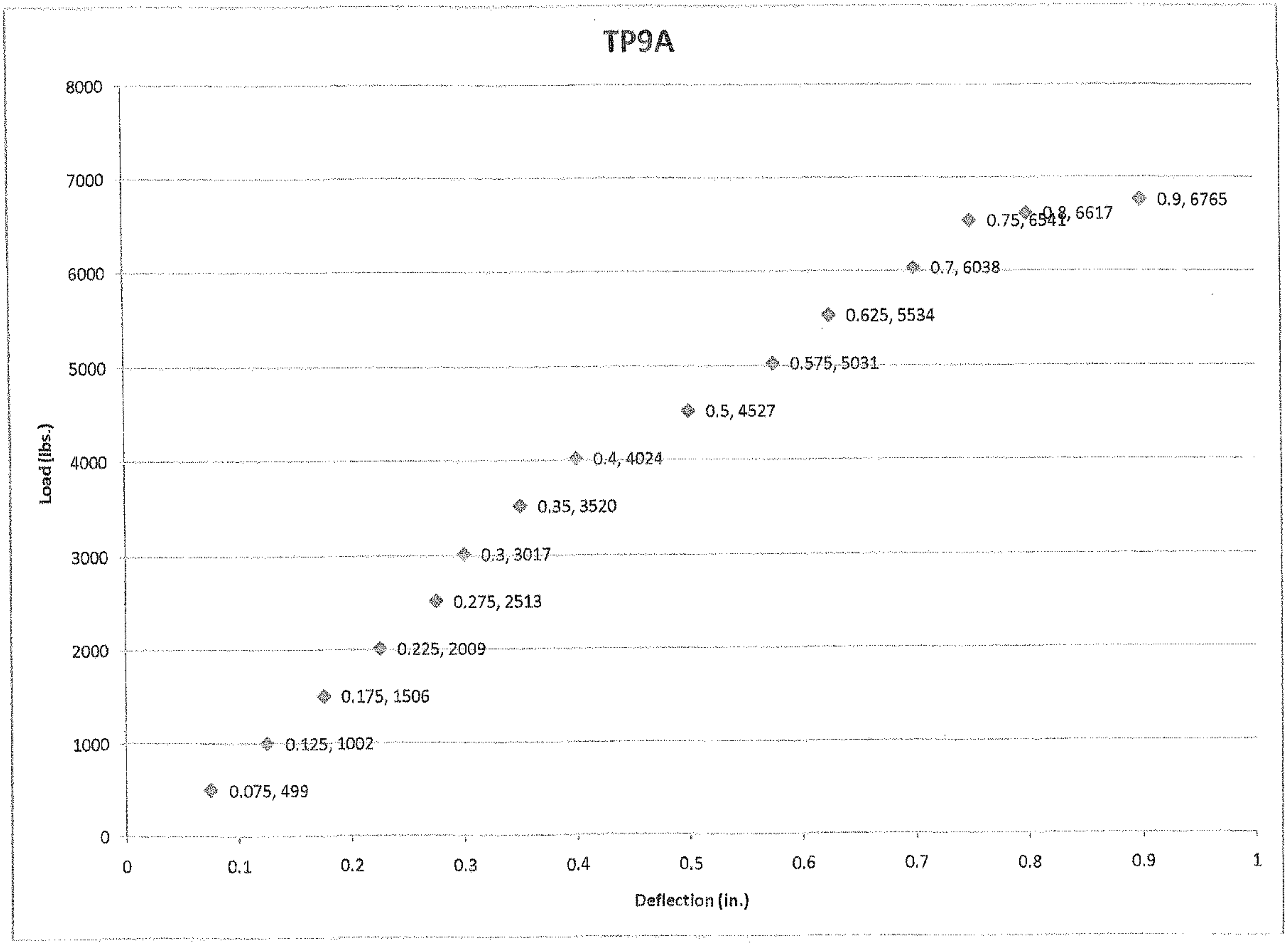
PW4055-#2: 2'x4', Loading Condition #1 **Thickness = 0.606 in.**

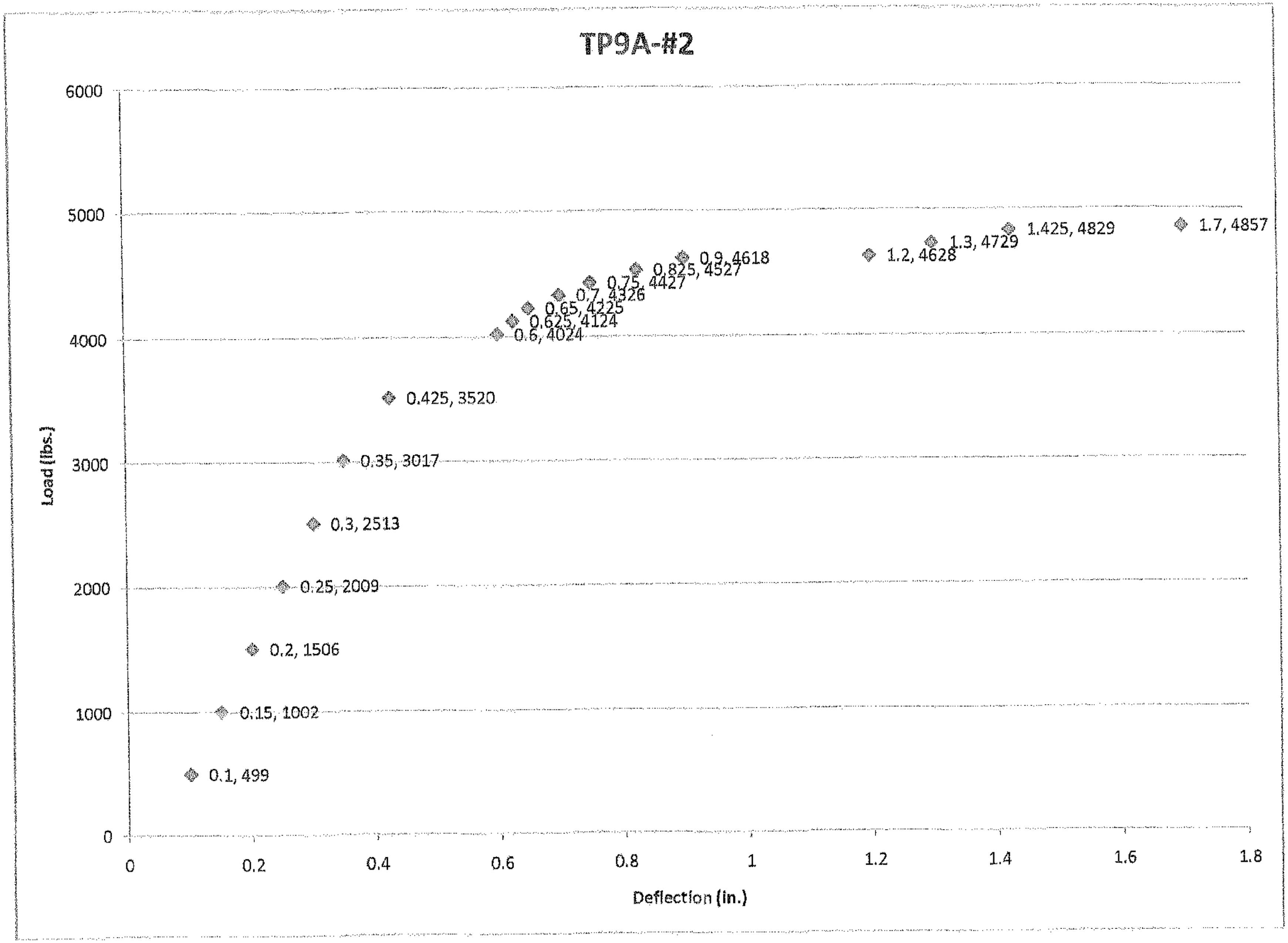
	Defl.	Actual Defl. (in.)	Meter Value (units)	Actual Value (lbs.)
	5	0	0	0
	5.125	0.125	500	499
	5.2	0.2	1000	1002
	5.25	0.25	1500	1506
	5.3	0.3	2000	2009
	5.375	0.375	2500	2513
	5.425	0.425	2600	2614
	5.5	0.5	3000	3017
Max.	5.625	0.625	3566	3586
	5.8	0.8	2923	2939

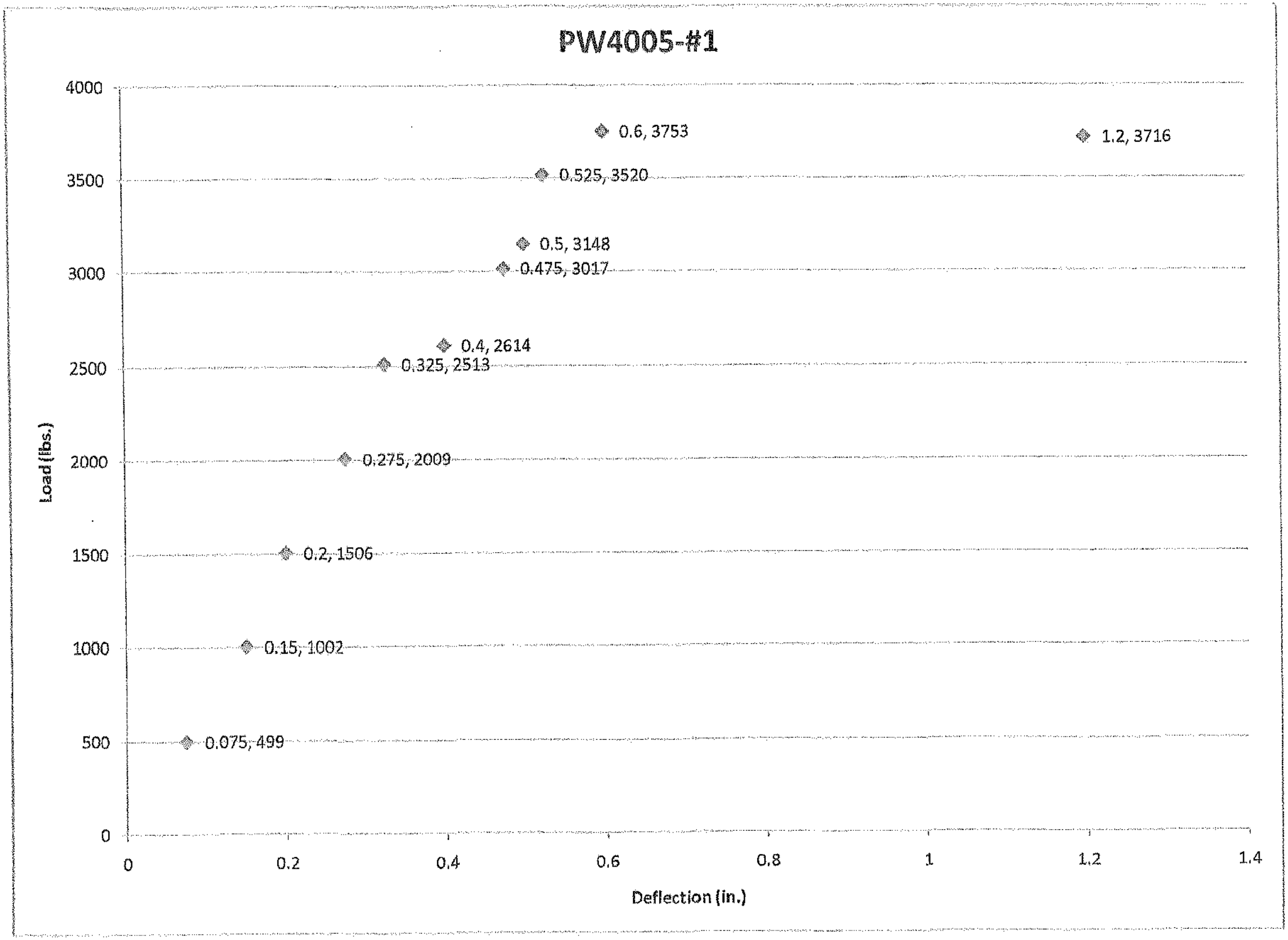

TP8-A#1
Load (lbs.)
Deflection (in.)
6000
5000
4000
3000
2000
1000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.1, 499
0.2, 1002
0.25, 1506
0.3, 2009
0.35, 2513
0.4, 3017
0.475, 3520
0.525, 4024
0.6, 4527
0.725, 4988

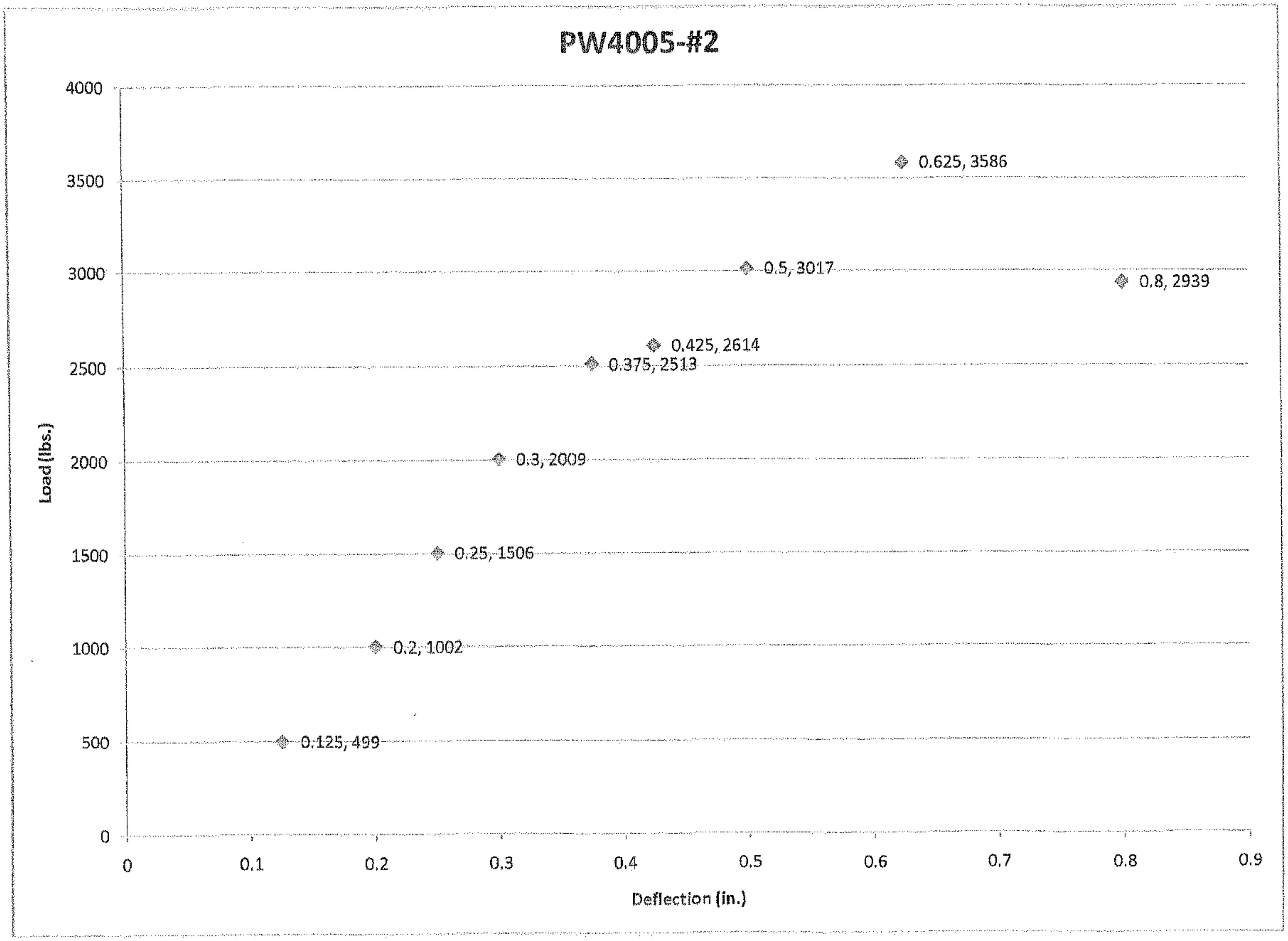

TP8-A#2
6000
5000
4000
3000
2000
1000
0
Load (lbs.)
1.7, 5151
0.75, 5031
0.6, 4527
0.5, 4024
0.4, 3520
0.325, 3017
0.275, 2513
0.225, 2009
0.175, 1506
0.1, 1002
0.075, 499
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
Deflection (in.)


TP10-A
Load (lbs.)
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Deflection (in.)
0
0.2
0.4
0.6
0.8
1
1.2
1, 8697
1, 8556
0.9, 8052
0.825, 7549
0.775, 7045
0.725, 6541
0.625, 6038
0.525, 5534
0.5, 5031
0.475, 4527
0.4, 4024
0.375, 3520
0.325, 3017
0.275, 2513
0.2, 2009
0.175, 1506
0.125, 1002
0.075, 499


TP9A
Load (lbs.)
8000
7000
6000
5000
4000
3000
2000
1000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Deflection (in.)
0.075, 499
0.125, 1002
0.175, 1506
0.225, 2009
0.275, 2513
0.3, 3017
0.35, 3520
0.4, 4024
0.5, 4527
0.575, 5031
0.625, 5534
0.7, 6038
0.75, 6541
0.8, 6617
0.9, 6765


TP9A-#2
Load (lbs.)
Deflection (in.)
6000
5000
4000
3000
2000
1000
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
0.1, 499
0.15, 1002
0.2, 1506
0.25, 2009
0.3, 2513
0.35, 3017
0.425, 3520
0.6, 4024
0.625, 4124
0.65, 4225
0.7, 4326
0.75, 4427
0.825, 4527
0.9, 4618
1.2, 4628
1.3, 4729
1.425, 4829
1.7, 4857


PW4005-#1
Load (lbs.)
Deflection (in.)
4000
3500
3000
2500
2000
1500
1000
500
0
0.2
0.4
0.6
0.8
1
1.2
1.4
0.6, 3753
1.2, 3716
0.525, 3520
0.5, 3148
0.475, 3017
0.4, 2614
0.325, 2513
0.275, 2009
0.2, 1506
0.15, 1002
0.075, 499


PW4005-#2
Load (lbs.)
Deflection (in.)
0.125, 499
0.2, 1002
0.25, 1506
0.3, 2009
0.375, 2513
0.425, 2614
0.5, 3017
0.625, 3586
0.8, 2939
0
500
1000
1500
2000
2500
3000
3500
4000
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9

APPENDIX V

Patent Construction Systems Drawing

No. 4607K070

PATENT CONSTRUCTION SYSTEMS LAYOUT DRAWING NOTES

GENERAL NOTES

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:

FOR VERTICAL SHORING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

12 inches for 18,000

REFER TO SAFETY RULES AND INSTRUCTIONS # 32470

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

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

FOR HORIZONTAL SHORING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

24. Ensure that horizontal shore bearing ears are fully supported. Vertical shores and beam side studs shall be positioned directly under each horizontal shore bearing ear unless otherwise detailed herein.
25. Tighten adjusting bolts with 10" long #4 re-bar to good snug fit. Do not overtighten.
26. Horizontal shore bearing ears should not be nailed in place except for temporary positioning during erection.
27. Never place intermediate shoring under round bars of lattice section of horizontal shore.
28. Horizontal shores adjacent to walls, beams, etc. may be underloaded. Immediately after pour, adjust camber bolt to lower to desired level.

REFER TO SPANALL FIELD ERECTION MANUAL & SAFETY RULES

HORIZONTAL SHORE DESIGN & BEARING EAR DATA					
SPANALL MEMBER ASSEMBLY	MAX. ALLOW. BENDING MOMENT	MAX. ALLOW. END REACTION	MINIMUM LEDGER WIDTH	MINIMUM AREA OF EARS	THICKNESS OF EARS
730 (2 member)	12,200 ft. lb.	3,740 lb.	2 3/8"	6.75 SQ. IN.	1/2" MAX.
730 (3 member)	10,800 ft. lb.	3,740 lb.	2 3/8"	6.75 SQ. IN.	1/2" MAX.
1015	9,100 ft. lb.	3,300 lb.	2 1/4"	6.00 SQ. IN.	1/2" MAX.
814	6,200 ft. lb.	3,300 lb.	2 1/4"	6.00 SQ. IN.	1/2" MAX.
810	4,000 ft. lb.	2,750 lb.	2 1/4"	4.80 SQ. IN.	1/2" MAX.
47	1,450 ft. lb.	1,330 lb.	3"	4.80 SQ. IN.	1/2" MAX.

FOR INTERFORM TRUSS TABLES, COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

FOR FORMING DEFINED AS THE SUPPORTING MEMBERS IN DIRECT CONTACT WITH THE FRESHLY PLACED CONCRETE, COMPLY WITH THE FOLLOWING ADDITIONAL NOTES:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

FOR LOGIC PANELS COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

NOTED DESIGN DATA

Max design pressure ______ PSF with Concrete density of ______ PCF

CONCRETE DESIGN PRESSURES FOR WALL FORMS

MINIMUM SAFETY FACTORS OF FORMWORK ACCESSORIES

FOR ACCESS SCAFFOLDING DEFINED AS A TEMPORARY ELEVATED PLATFORM AND ITS SUPPORTING STRUCTURE OR A SUSPENDED PLATFORM USED TO SUPPORT USERS OR MATERIALS, OR BOTH, COMPLY WITH THE FOLLOWING ADDITIONAL NOTES:

FOR "TROUBLESAVER" SECTIONAL, GES, CUPLOCK, TUBELOX SCAFFOLDING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

FOR SPRINT SCAFFOLDING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

FOR SUSPENSION SCAFFOLDING COMPLY WITH THE FOLLOWING ADDITIONAL ITEMS:

REFER TO SAFETY RULES AND INSTRUCTIONS # ________

WARNING

Serious injury may result if you fail to use safe practice in the erection, dismantling or use of PCS equipment. Erectors, dismantlers, and users must be familiar with and follow current laws and regulations, safe practice and the Safety Rules and Instructions. Individuals using this equipment must be instructed in these requirements. Safety Rules and Instructions pertaining to the products shown herein are provided upon sale or rental of equipment. Additional copies or further information will be provided upon your request.

It is important to note that current OSHA regulations require the use of guardrail systems and/or fall protection devices at all working levels, open sides, and at all other openings on platforms and work areas above certain heights, as specified by OSHA. In all cases, where a user is exposed to a fall hazard in the use of this equipment, guardrail systems, where appropriate or other personal fall protective devices, must be utilized. Means of access must be made available to the customer to all locations where people are expected to work. Materials for the provision of such means of access may be job built by the customer, or, at the customer's option, be obtained through Patent Construction Systems or other suppliers. Patent Construction Systems will, at customer's request, consult on alternate means of access.

DURING USE OF EQUIPMENT ALWAYS FOLLOW SERARATE SAFETY RULES & INSTRUCTIONS AS INDICATED IN EACH SPECIFIC SECTION.

LUMBER DESIGN VALUES

Extreme fiber stress in bending	1,640 PSI
Horizontal shear	180 PSI
Compression perp. to grain	625 PSI
Compression parallel to grain	1,250 PSI
Modulus of elasticity	1,800,000 PSI

TITLE	ZOKA SHORING LAYOUT
DATE	12/14/2007
PROJECT	SOHO HOTEL
LOCATION	246 SPRING STREET NYC
CUSTOMER	DI FAMA CONCRETE
DRAWING NUMBER	4607K070
SHEET NO.	1

patent

Patent Construction Systems
Harsco Corporation

RECEIVED
RECEIVED
RECEIVED

12/14/07

DURING USE OF EQUIPMENT ALWAYS FOLLOW SERARATE SAFETY RULES & INSTRUCTIONS AS INDICATED IN EACH SPECIFIC SECTION.

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.

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
1600 Lower Road
Linden, New Jersey 07036
(732) 396-1269

DRAWING NUMBER: 4607K070

SHEET NO. 1

RECEIVED JAN 16 2008 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT

RECEIVED JA 1 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT

RECEIVED JA 1 2008 DEPARTMENT OF BUILDINGS FORENSIC ENGINEERING UNIT

APPENDIX “C”
Report on Patent Aluminum Shores Tests
ATLSS Lehigh University

LEHIGH
UNIVERSITY

Structural Testing Laboratories
Fritz Engineering Laboratory
13 East Packer Avenue
Bethlehem, PA 18015-4729
(610) 758-5498 Fax (610) 758-5902

June 15, 2009
FL2009.1208.1

Dan Eschenasy
Department Chief Structural Engineer
NYC Buildings
280 Broadway, 7th Floor
New York, NY 10007

Subject: Testing of Shoring Towers for NYC Buildings

Dear Mr. Eschenasy,

On May 4th and 5th, 2009, six shoring towers were tested in the Fritz Lab Baldwin-Lima-Hamilton 5,000K testing machine. The 5,000K machine was calibrated on April 8, 2009. Three towers had concentric axial force applied, and three towers had eccentric axial force applied. Five string pot type displacement transducers were used to measure deflections for the eccentric load tests. The test types and results are summarized in Table 1. Before and after photos of the six test specimens are shown in Figures 1-12.

Load was applied to the towers using an H frame arrangement. A spherical bearing block was affixed to the bottom of the sensitive crosshead which loaded on a spreader beam which in turn loaded on two load beams. The concentric test specimens were loaded directly through the four columns using spacer blocks on top of the screw jack feet. The three eccentric load tests were performed by moving the south load beam so that the center of the load was 2" outboard of the centerline of the screw jack. The North load beam was centered over the screw jacks. Stringer beams were also placed on top of the screw jack feet for the eccentric tests. The bottom screw jack extensions were 12" for all tests except Tests 3 and 5, which had no bottom screw jacks. The top screw jack extensions were either 18" or 21".

Table 1: Summary of Test Results

Test	Test Type	Btm SJ [in]	Top SJ [in]	Total Height [in]	Max Load [lbs]	Failure mode
1	Concentric Load Tower "A"	12	18	136	159,000	Racking
2	Concentric Load Tower "C"	12	18	136	154,500	Racking
3	Concentric Load Tower "B"	None	18	130	152,100	Racking
4	Eccentric Load (Various Components)	12	18	136	61,300	Screw jack buckling
5	Eccentric Load (Various Components)	None	21	133	52,100	Screw jack buckling
6	Eccentric Load (Various Components)	12	21	139	56,400	Top plate fractured

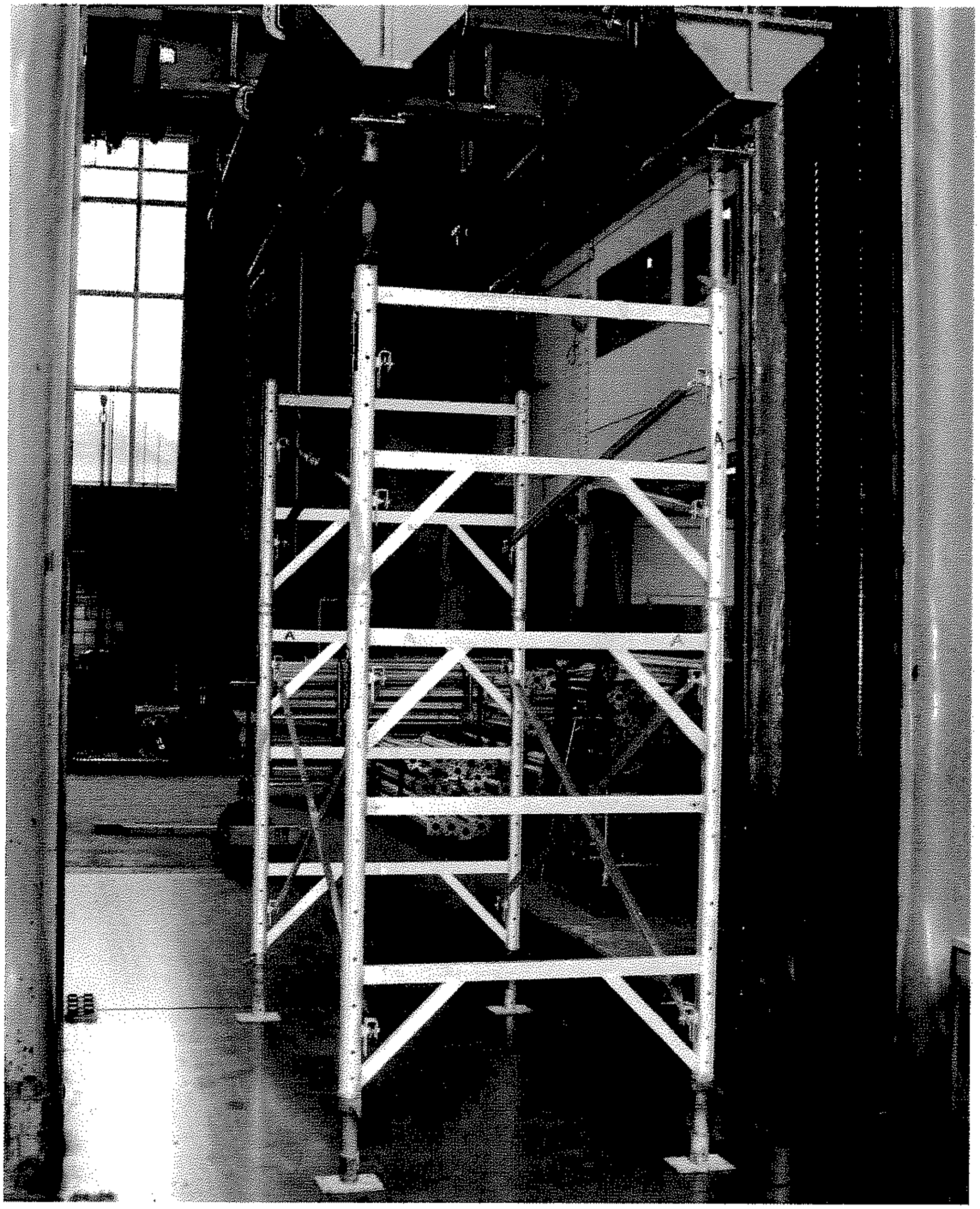


Figure 1: Tower 1 Pretest

Figure 2: Tower 1 Post Test

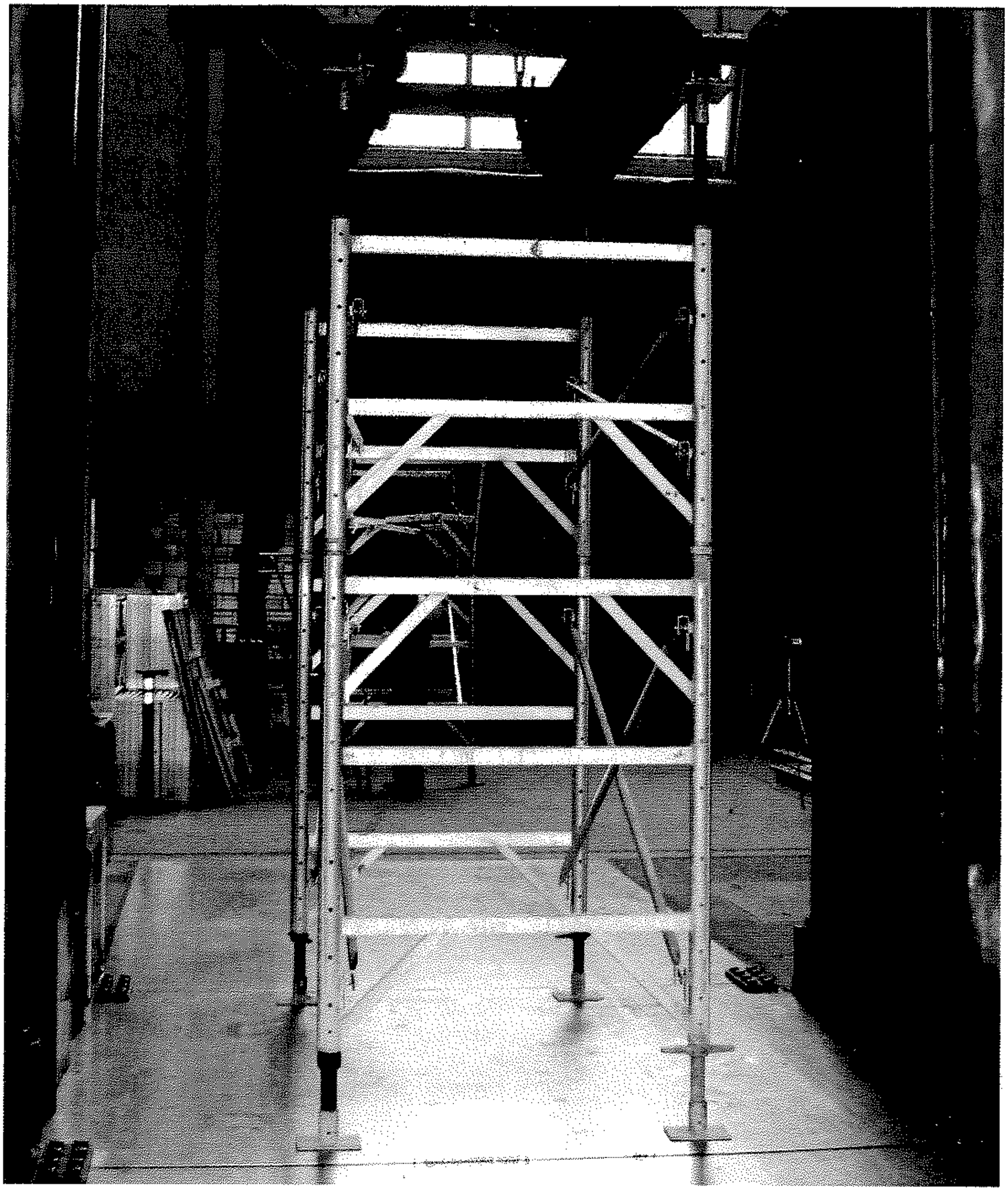


Figure 3: Tower 2 Pretest

Figure 4: Tower 2 Post Test

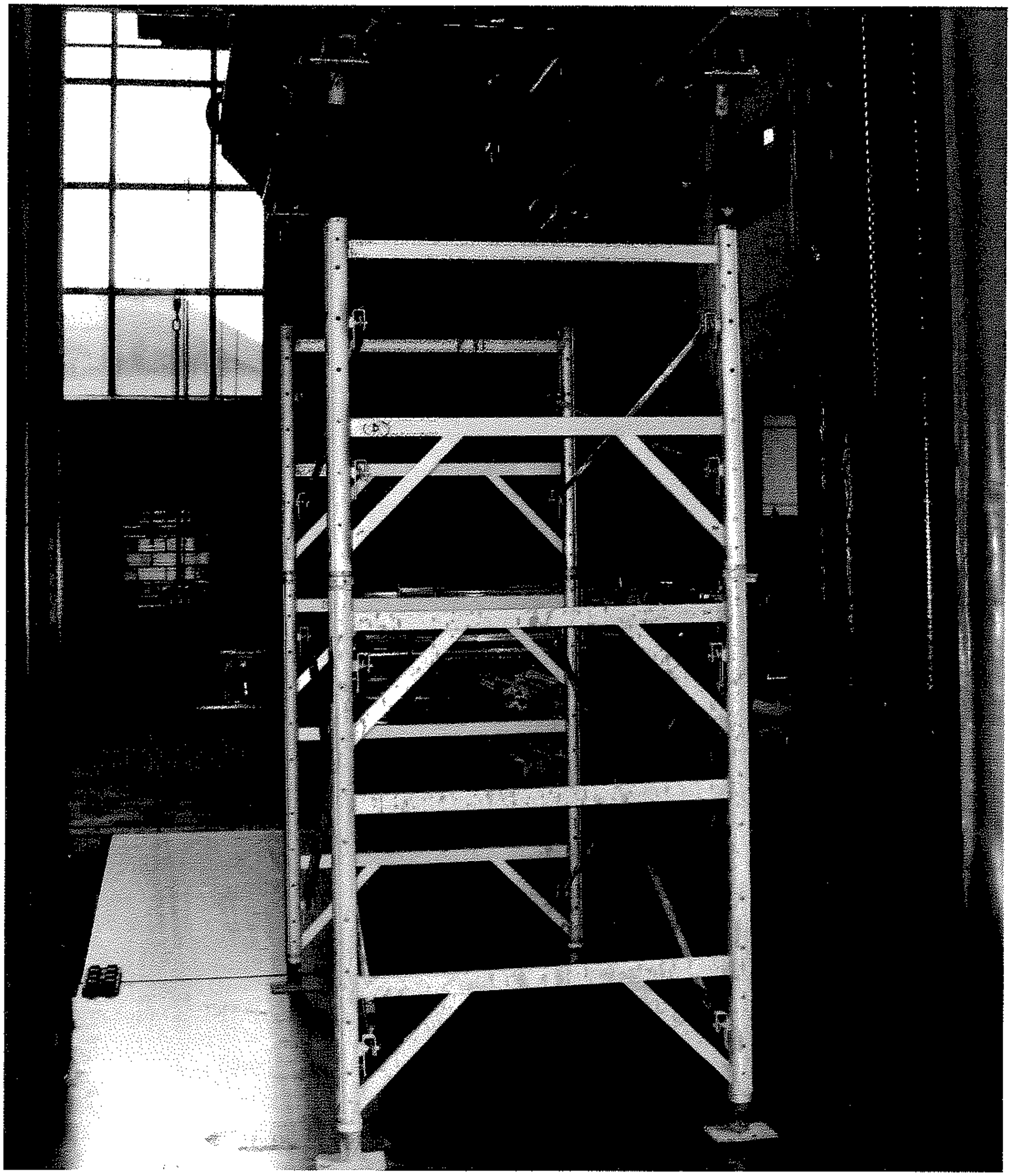


Figure 5: Tower 3 Pretest

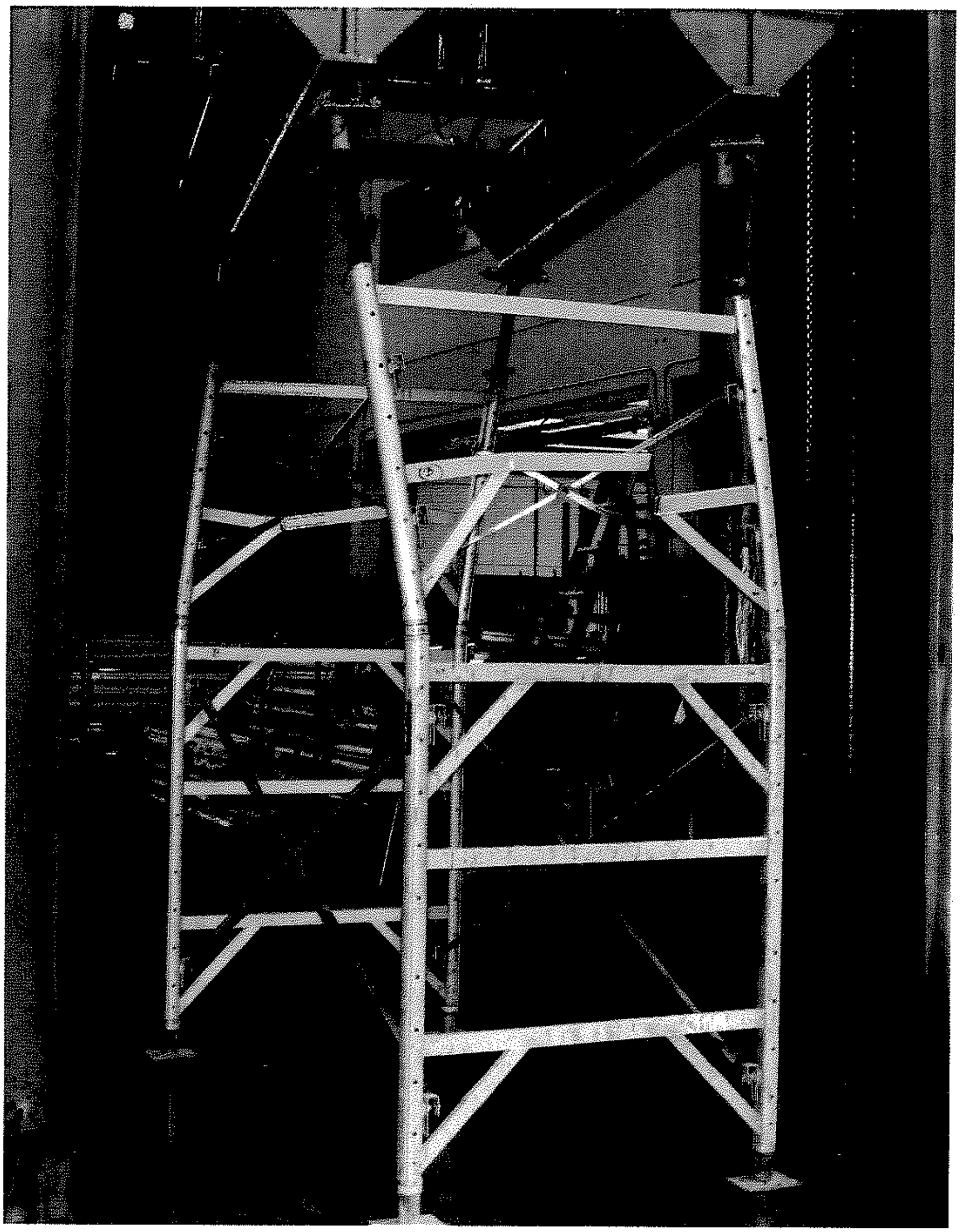


Figure 6: Tower 3 Post Test

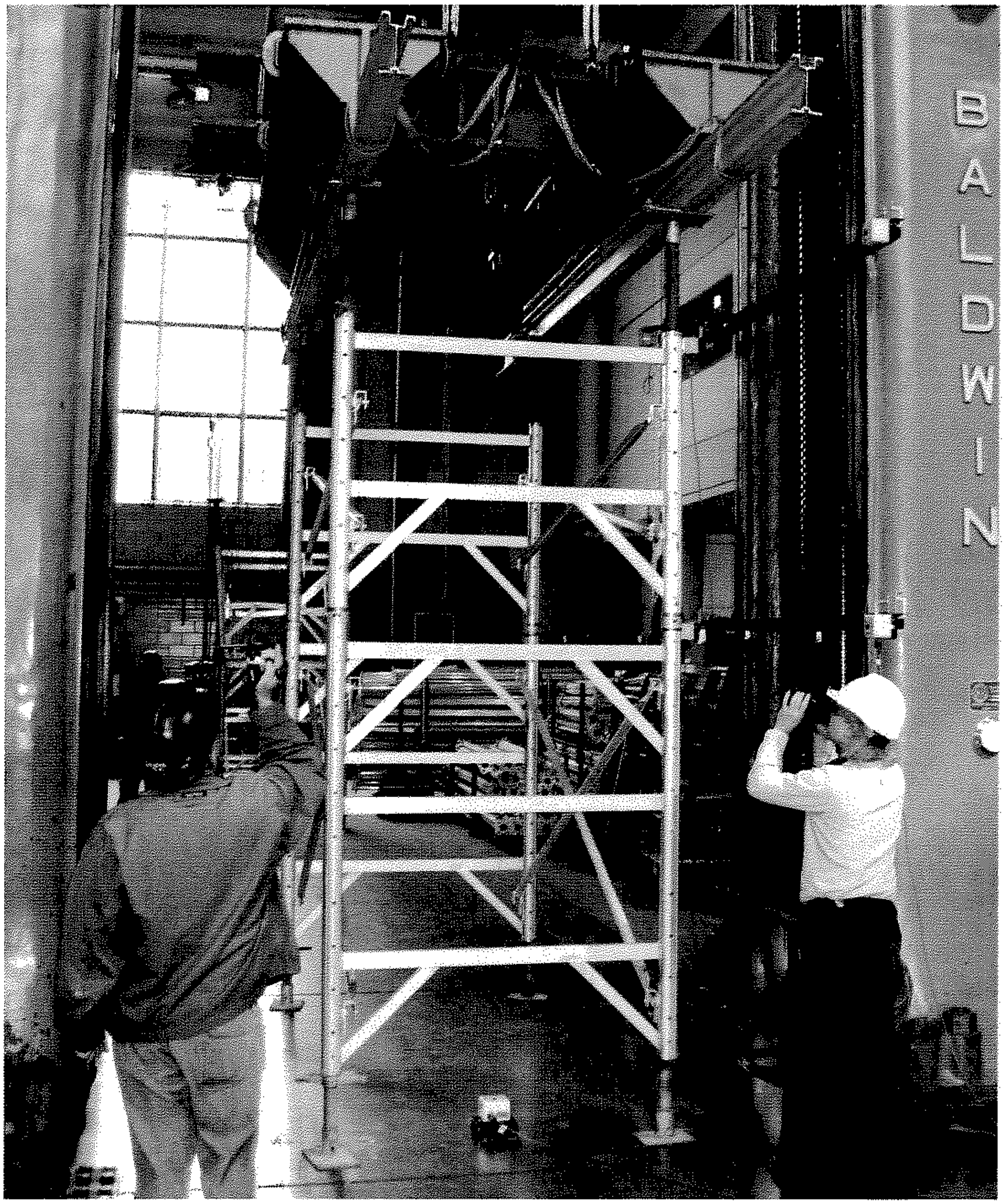


Figure 7: Tower 4 Pretest

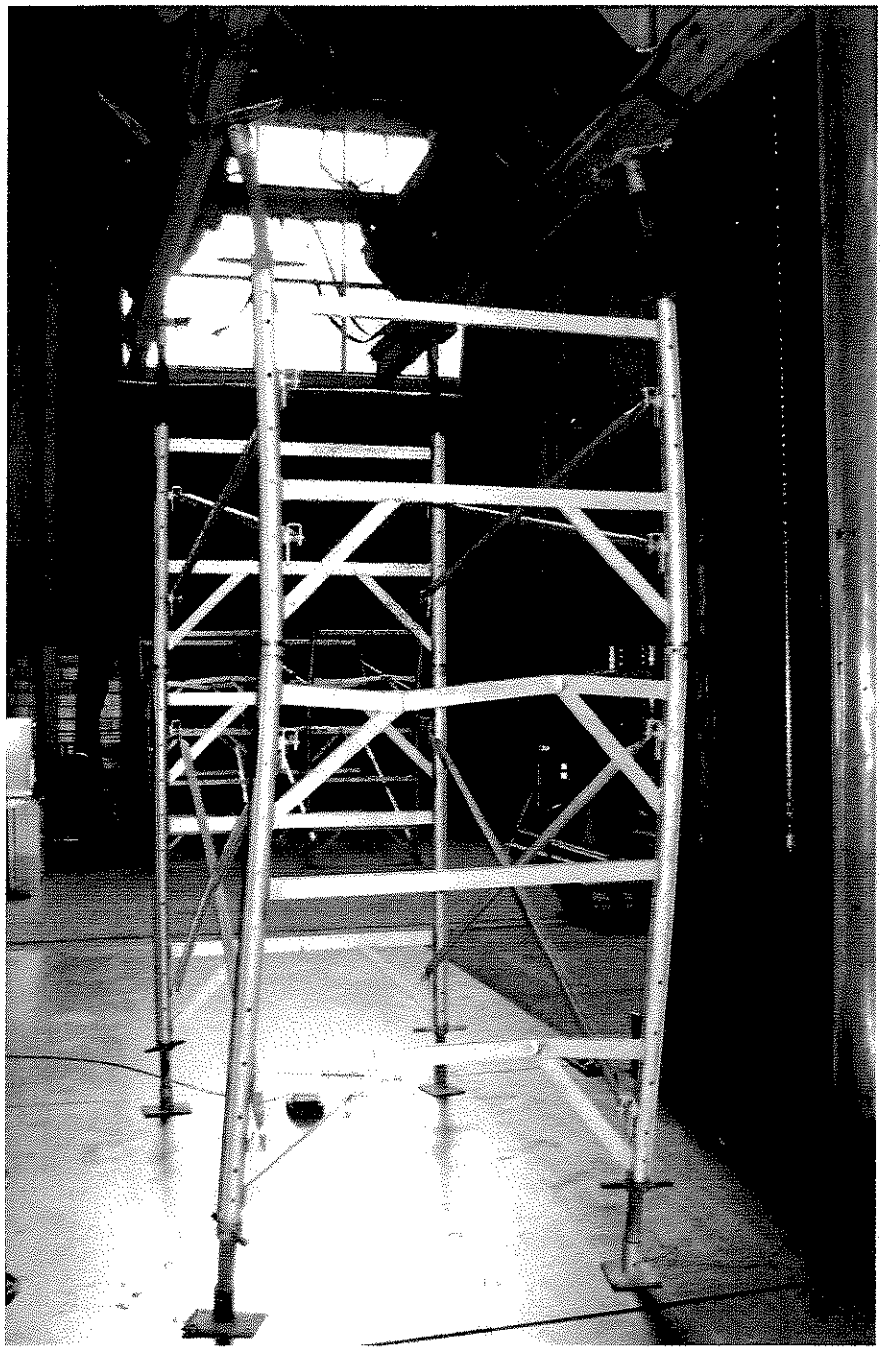


Figure 8: Tower 4 Post Test

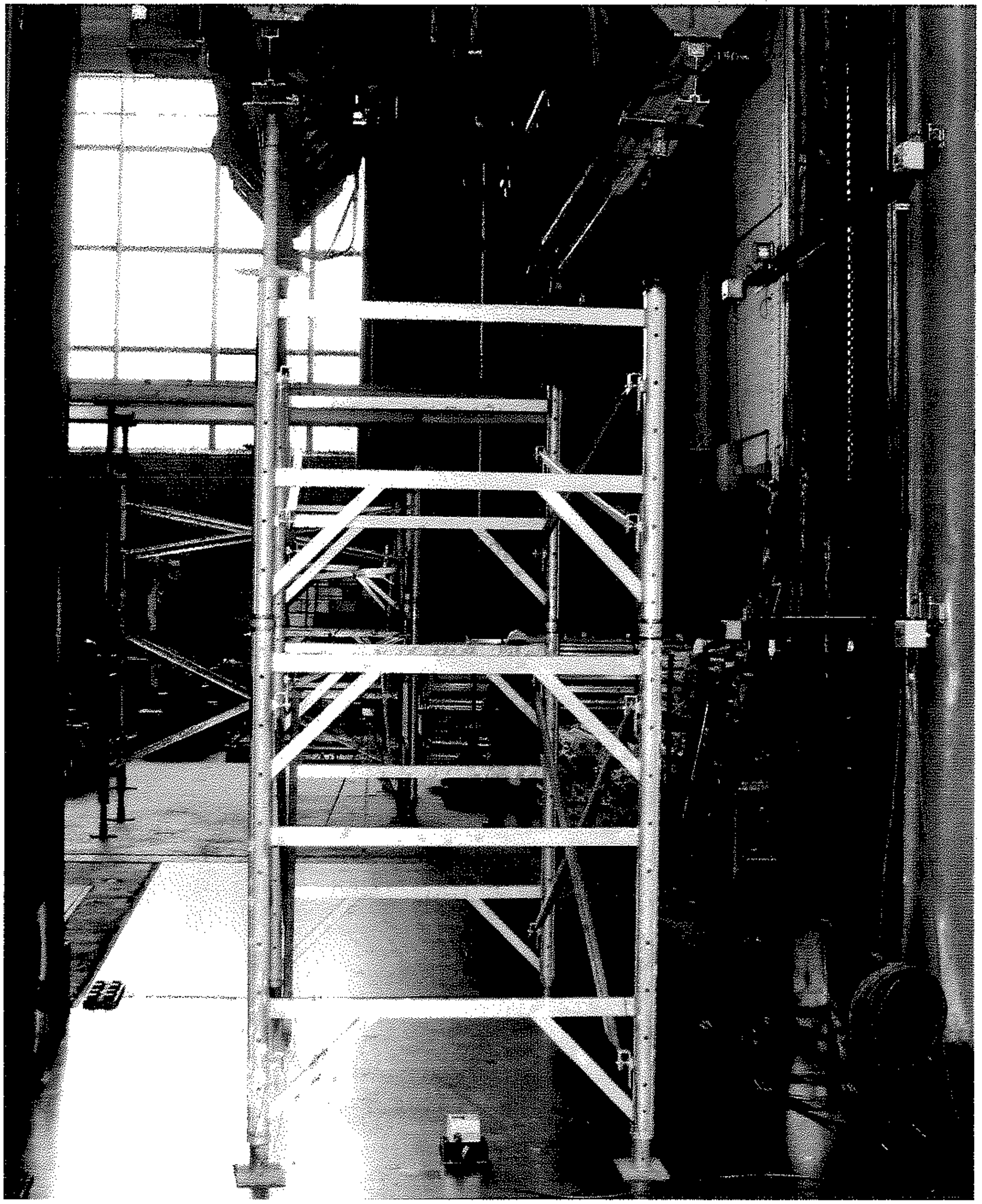


Figure 9: Tower 5 Pretest

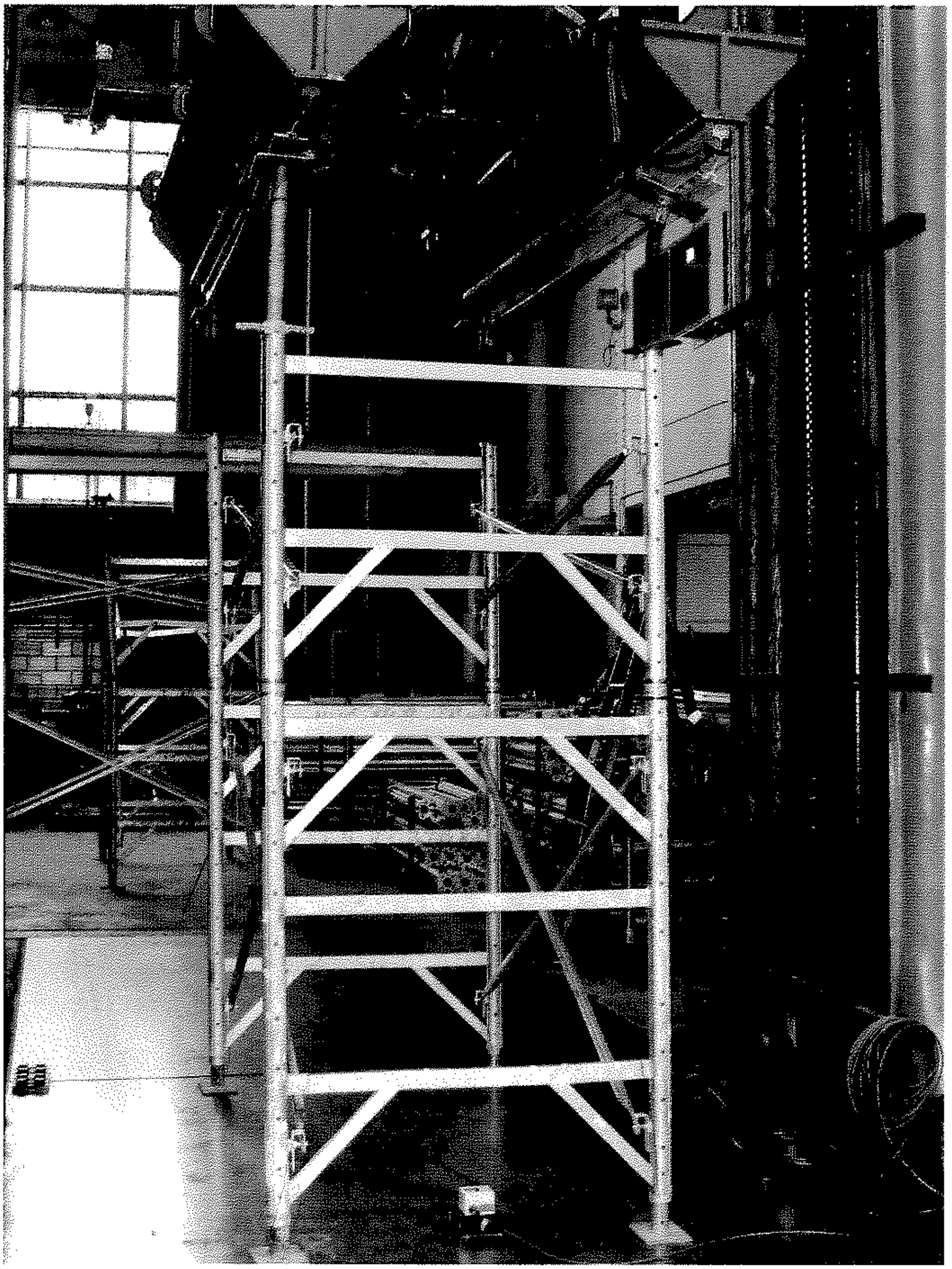


Figure 10: Tower 5 Post Test

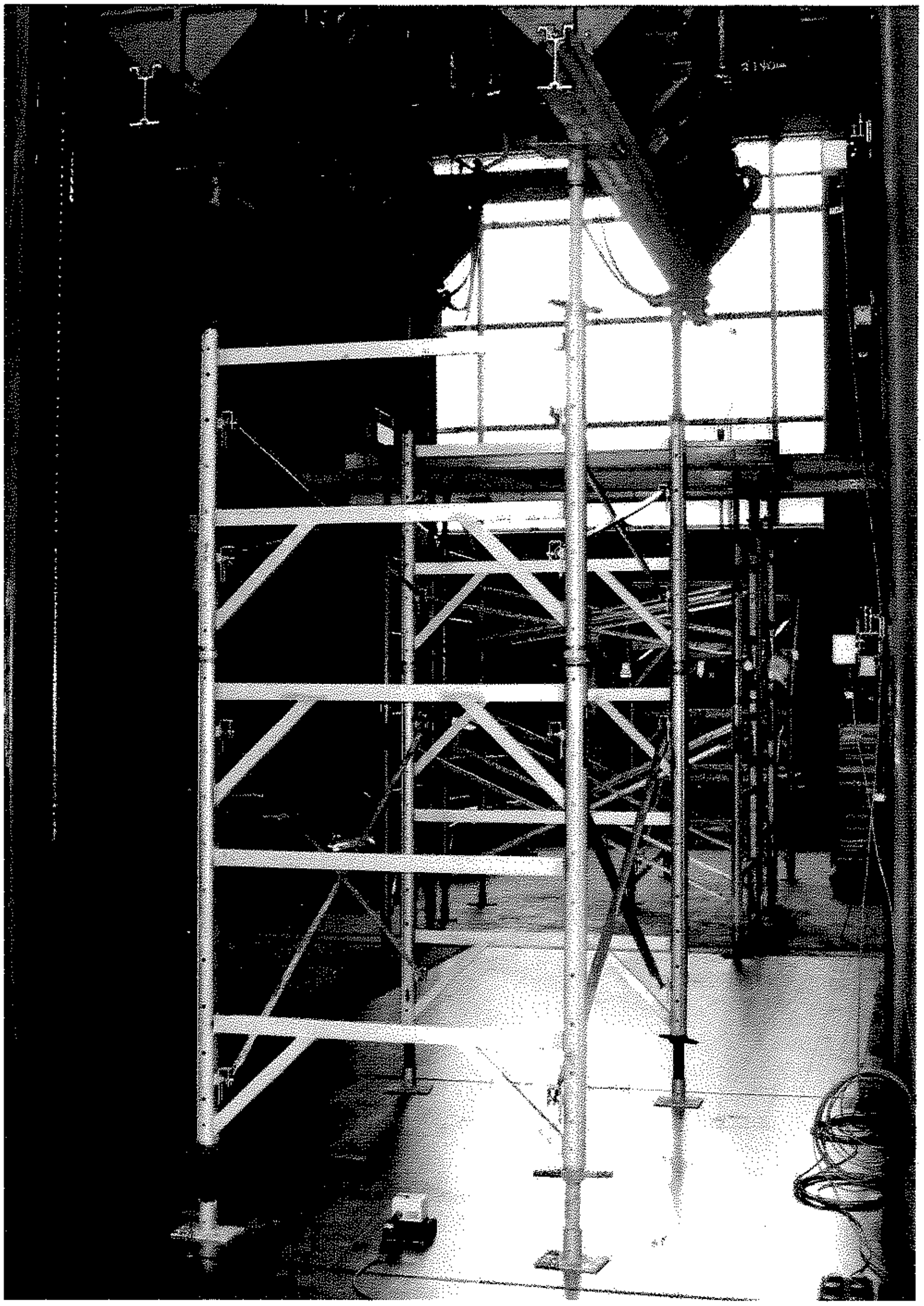


Figure 11: Tower 6 Pretest

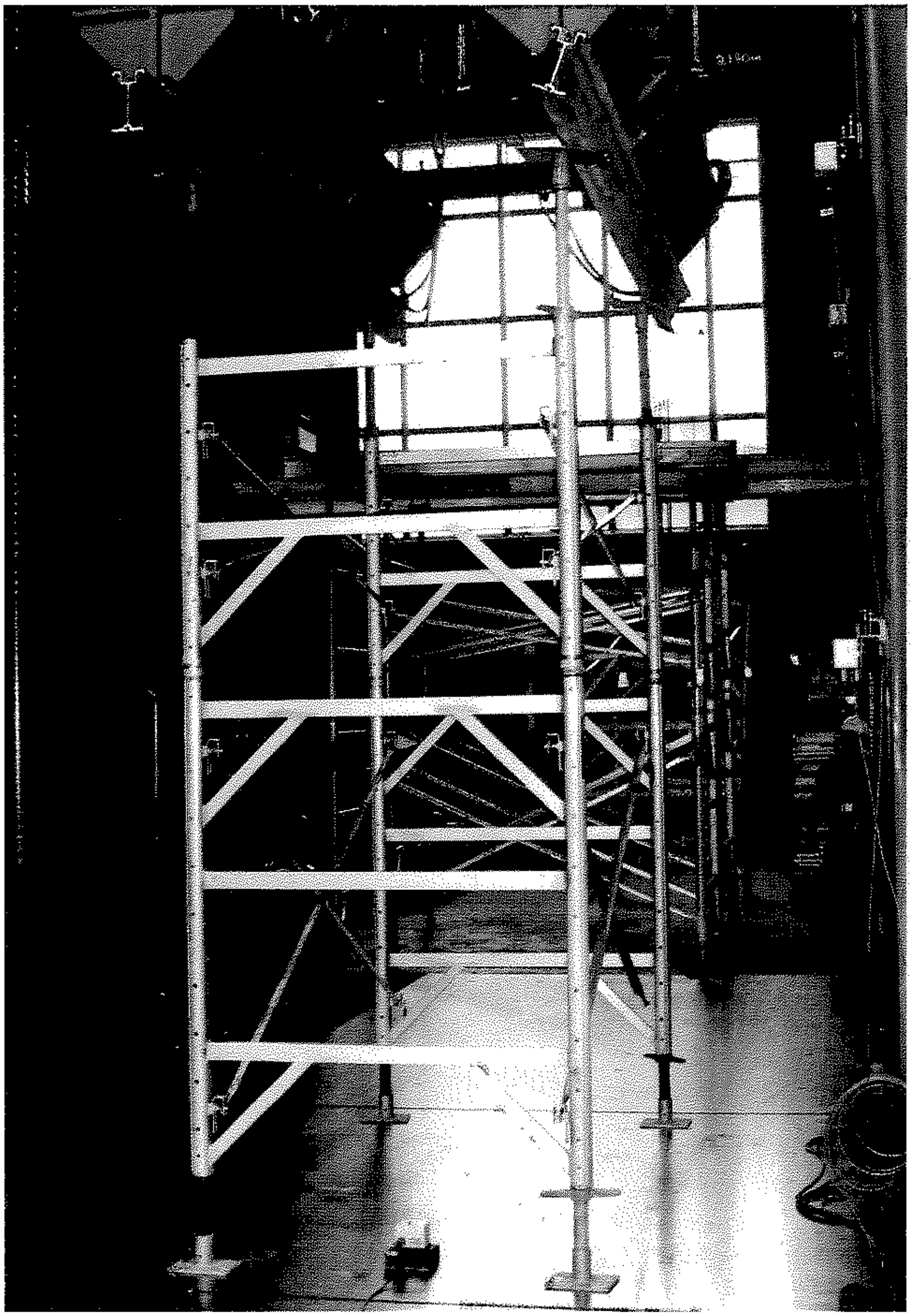


Figure 12: Tower 6 Post Test

The locations of the sensors used for the testing are given using the cardinal directions. Figure 13 Shows the North and South directions relative to the 5,000K testing machine. Figure 14 shows the five string pots and their designations. The vertical string pot measured the total axial compression deflection of the tower. The remaining four string pots measured the lateral deflection of the columns at the top of the screw jacks and at the bottom of the second tower tier.

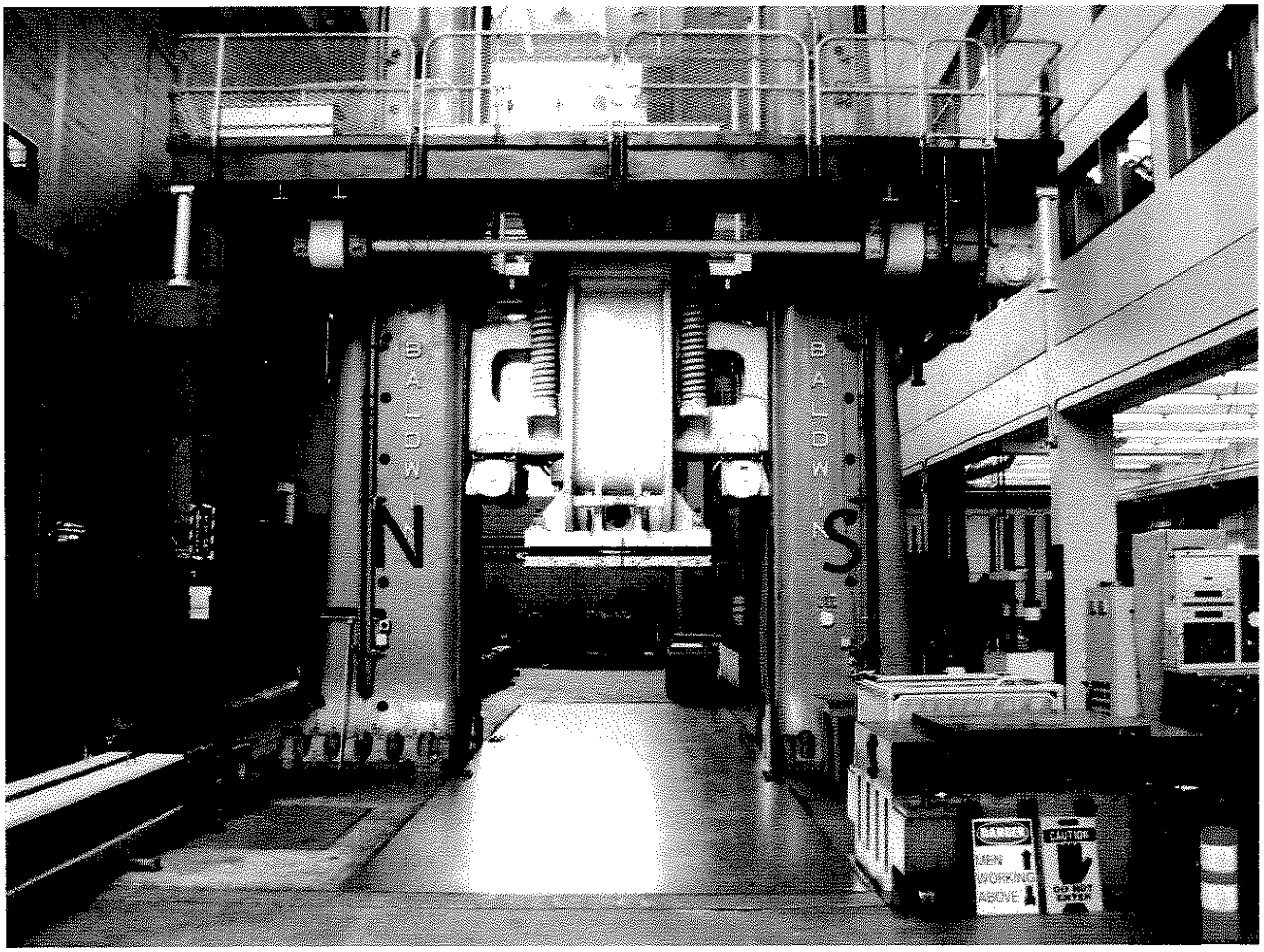


Figure 13: 5,000K Machine with Columns Labeled N for North and S for South West is in front of the machine, and East is in the back

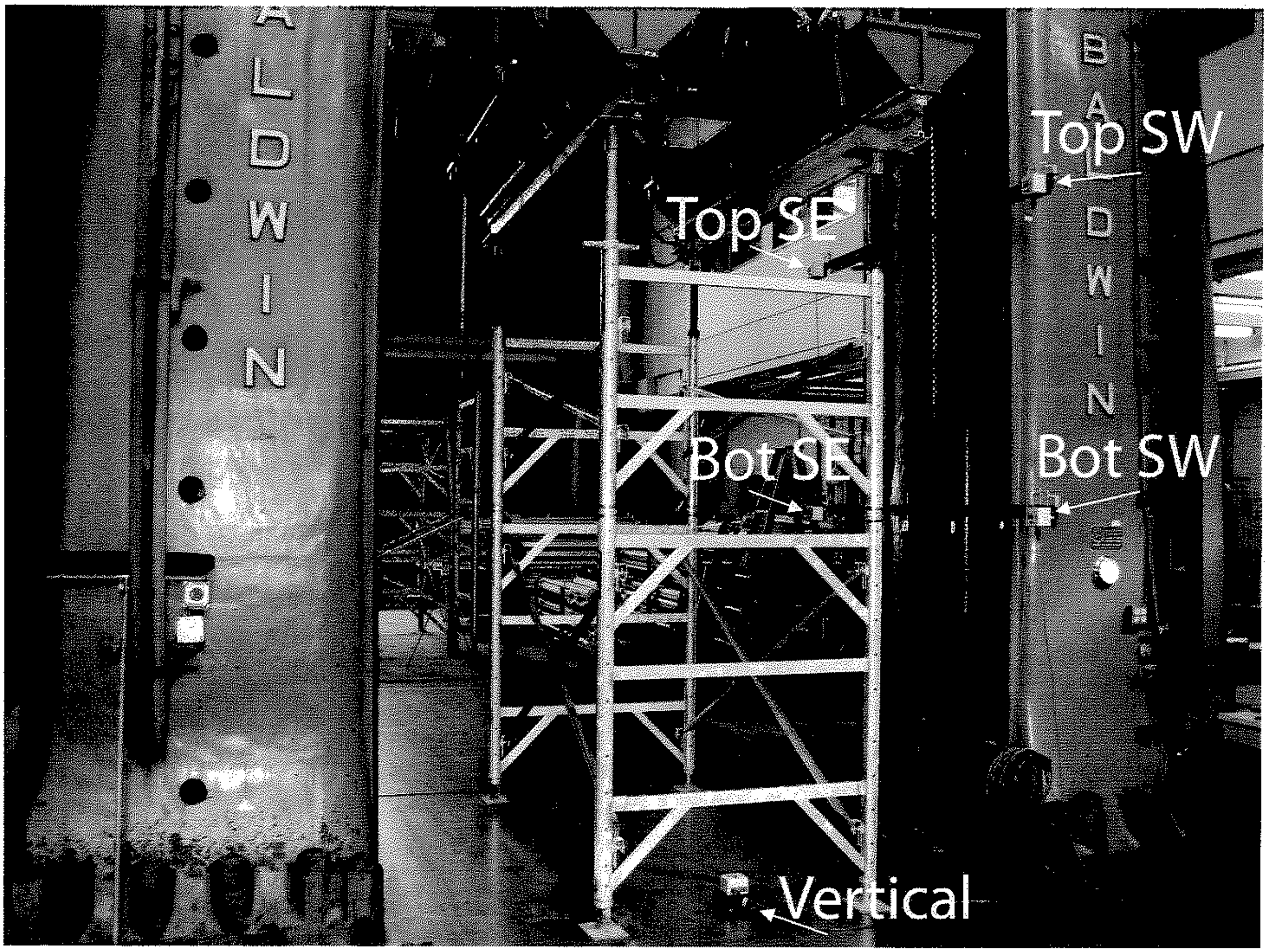


Figure 14: Location of String Pots

Included with this report is a CD containing the 5,000K testing machine calibrations certificate, additional test pictures, load deflection plots for the three eccentric load tests and an electronic copy of this report.

Sincerely,

Robin J. Hendricks

Cc: Frank E. Stokes – ATLSS

APPENDIX “D”
Documentation and Preparatory Documents for Shoring Layout

SH 40113 40G

BOTTOM PL — NOT FULLY IN

SH 40122 40G

FRACTURE

AB 40008 12FT OC
40B

AB 40009 40A 12FT OC

AB 40007 40C 12FT OC

AB 40010 40 A 12FT OC

NO NAIL

AB 40020 40G 12FT OC

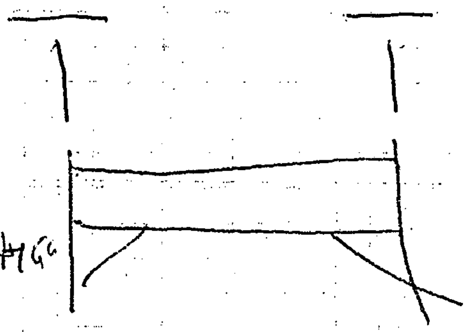
1 NAIL

AB 40014 40G-F

157"

BURN

4H 40009   40B



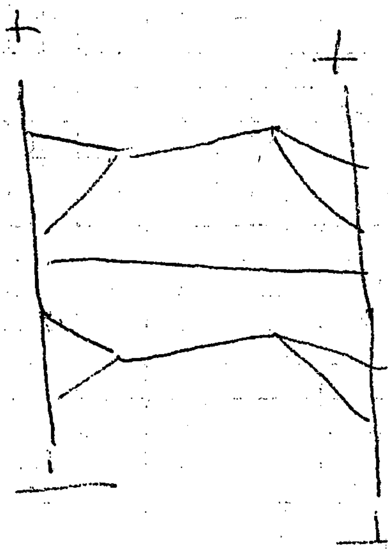
42 HIGH

attached with wire - buried white wax

OR

TOP PIECE

5H 40102   40F



64 HIGH

AB 40011 RETAG NO AREA

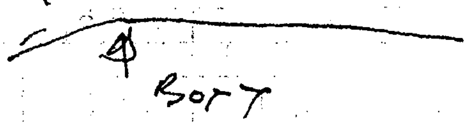
NAILS ON TOP 12 FOOT

AB 40012 40A

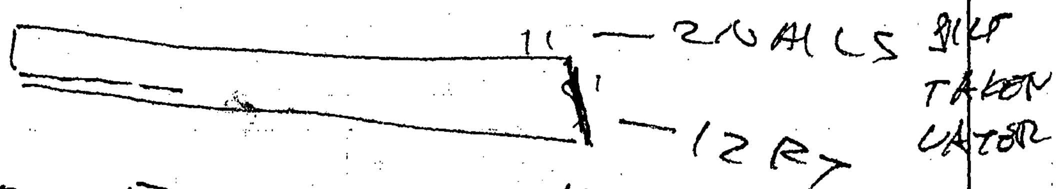
NAILS ON TOP 12FT

AB 40013 40G-F

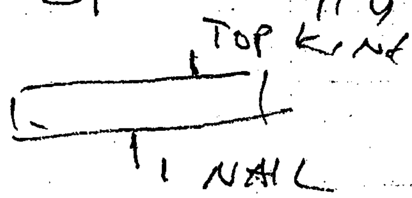
12FT



AB 40042 41G



AB 40001 41G 12FT



TB 40016 - 65"
TB 40017 - 64"
TB 40016 - 58 1/4"

PS 40002 - 137 1/4"
PS 410047 - 135 1/2
PS 40007 - 86 3/4
PS 4100-3 - 91 1/4"

SH40062 B - 45 3/4"
SH 40133 - 45 1/2
SH 40117 - 21"
SH 40128 - 45"
SH 4476 - 48"
SH 41044 - #348 - 16 1/2
349 - 60"
350 - 50"
SH 40066 - 99"
SH 40052 - 107 1/2"
SH 40055 - 107 1/2"
SH 41054 - 86 1/2
SH 40061 - 99"
SH 40064 - 99"
SH 40065 - 99"
SH 40067 - 99"
SH 41142 - 52 3/4"
SH 40112 - 40J - 19 1/2
SH 40116 - 40F - 19 1/2
SH 40071 - 40B 19 1/2

641 - 32" 3x4
234 B - 48 1/2
601 - 31"
636 - 75"
495 - 29"
642 - 32"
448 - 28"

512 3x4 - 32"

201 A 9"
201 B 9" } 4x4 (2)

395 3x4 80"
647 B 4x4 - 37"

TB 40067 3x4 - 18 3/4
TB 40068 - 20 1/2
TB 400 69 29 1/2"
TB 400 70 48"
TB 400 71 4x4 77"
TB 400 72 3x4 - 54"

TB 40061 - 3x4 40"
TB 40062 - 33"
TB 40063 (B) - 47"
TB 40063 (C) - 73"
TB 40064 18"
TB 40065 18 1/2"
TB 40066 4x4 - 39"

2 1/8" x 1"

SH 41170 41G

SH 41189 41F

SH 41178 - 41FB

SH 40044 40B

SH 41171 41G

SH 40081 40F

SH 40074 41F

NO WELD ON PIECE

— WELD ON 1 end

SH 40062B ZONE 40B

MEAS BY JAY

HAS WELDS ON BOTH END

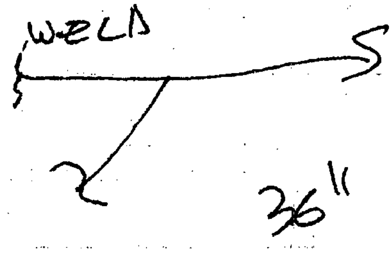
SH 40016B

40F

2 1/4 x 1" 44"

SH 40041

40B



2 1/4 X 1 SH 40086 40 F — SOME WED

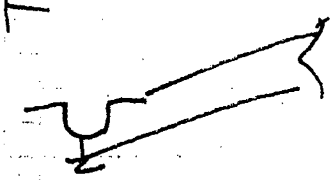
L 45'

SH 41184 41F

SF 41187 41F — 21'

AB 41041 41F

40" LG



276 Plywood 4x8

NO NAIL

SIGNED 2/25

TB 40061

63 B & C

TB 40072

SH 40134

135

140 — No 6,7,8,9

141

VGS 40003

VGS ——— 4/5

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

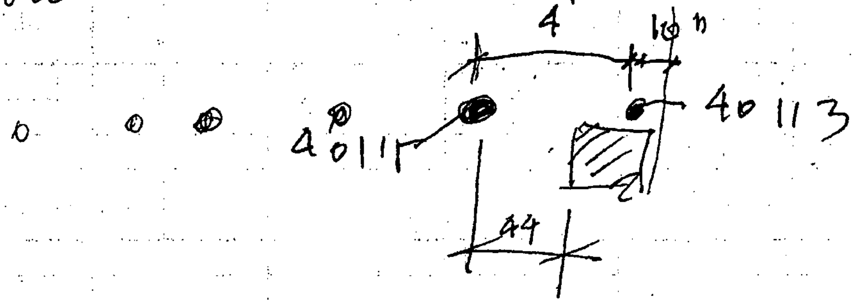
2/21/08 - JAY RUBINSTEIN
BEST SQD

429C	3x4	38½"
509B	3x4	31½"
605	3x4	32"
411		33"
415		33½"
604		18"
600		9½"
410B		15½"
593		16"
464		32½"
529		23½"
607	4x4	22½"
526	3x4	53"
474	3x4	63½"
443	4x4	72"
527	3x4	32"
468		33"
388		89"
391		90½"
386		46½"
573		32"

TB 40131	3x4	92"
TB 40132	3x4	99½"
TB 40133	3x4	43½"
TB 40134	3x4	106"
TB 40135	3x4	87"
TB 40136	3x4	20½"
TB 40137	3x4	43½"
TB 40138	3x4	24"
TB 40139	3x4	17½"
TB 40140	3x4	32"
TB 40141	3x4	43"
TB 40143	3x4	31"
TB 40144	3x4	90"
TB 40145	3x4	25½"
TB 40146	3x4	55"
TB 40147	3x4	40"
TB 40148	3x4	34"
TB 40149		29"
TB 40151	3x4	52"
TB 40152	3x4	31½"
TB 40153	3x4	51½"
TB 40154	3x4	34"
TB 40155	3x4	71"
TB 40156	3x4	28"
TB 40157	3x4	35"
TB 40158	1x4	67½"
TB 40159	3x4	54½"
TB 40160	3x4	60"

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

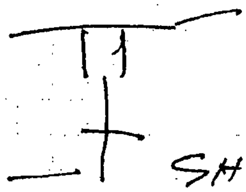
Wadin a 20



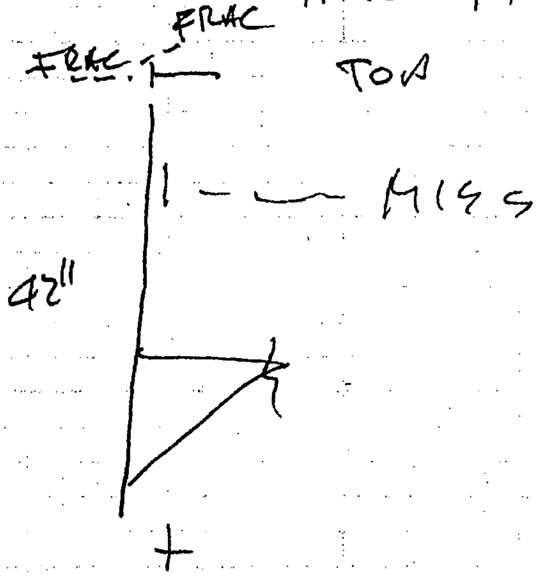
SH 40122
40123
40124
40125
40126
AB 40019
SH 40127 en broc
SH 40128 frame
SH 40139

TB40131 → TO 40160

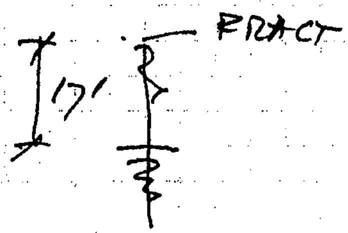
SKIP #50
142



SH 40017 40B



SH 41053 41F



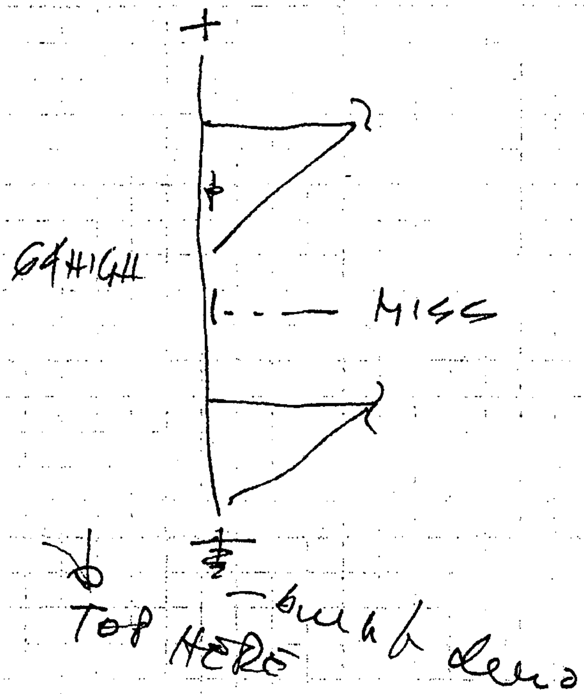
22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

591 = 31 1/2"
473 = 64"
530 = 31"
495 - 32"
438B - 40 1/2"
577 - 31 3/4"
568 - 14"
576 - 31 3/4"
432C - 34"
477 - 83"
445 32"
578 - 23"
478 - 31 1/2
515 = 14"
570 - 72"

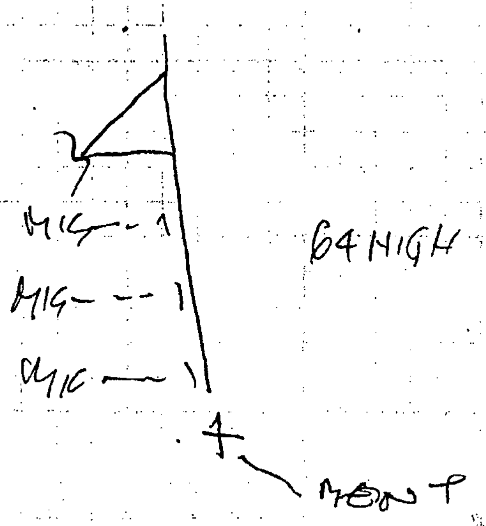
3x4

571		32"
436B		38"
442		32"
431B		41"
542		42
580		
543		41 1/2
465		48"
524		57"
580		18"
546		13 1/2
509	4x4	41"
467	3x4	36"
225		32"
437B		49"
492		30"
491		58"
540		61 1/2"
404		48"
472		32"
414	1x4	29 1/2"
418B	(2) 2x4	26 1/2
591	4x4	32"
473	1x4	64"
530	3x4	31"
495	3x4	32"
438B		41"
577		32"
439B	3x4	40 1/2

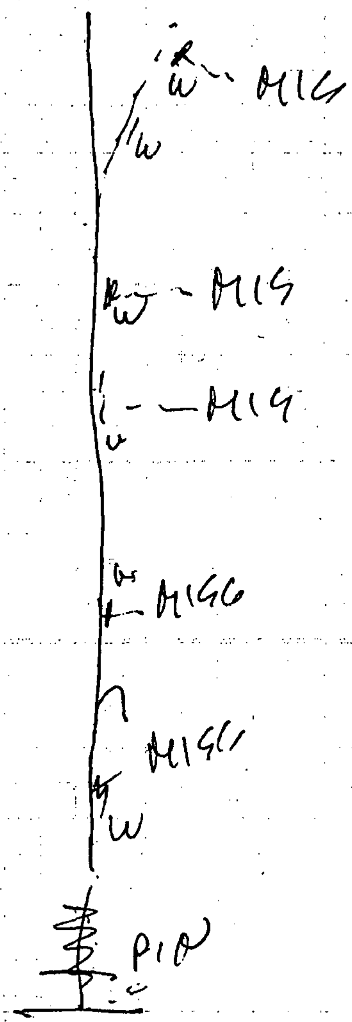
SH 41161 41G



SH 40097 90F



SH 40198 40 L
X-sheeted



62"H

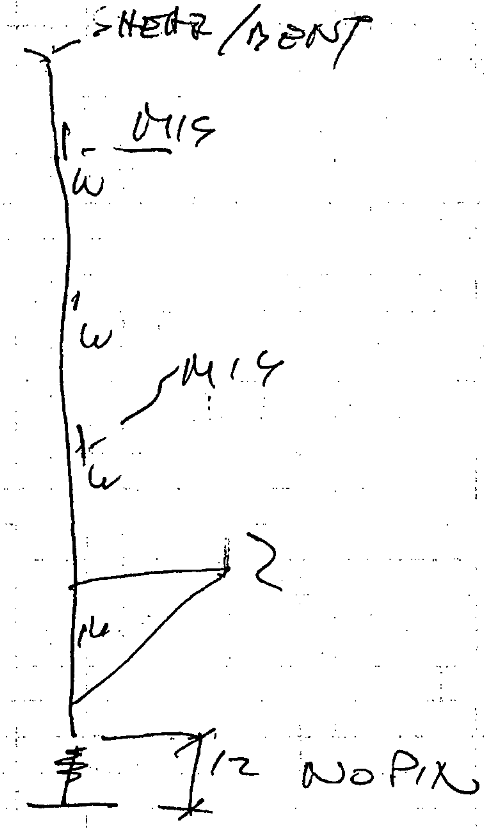
3/3 SIGNED

TB 40200 → TB 40268

SH 40217 → ~~SH~~ 40223
~~SH~~ 40224
SH

MG 40015

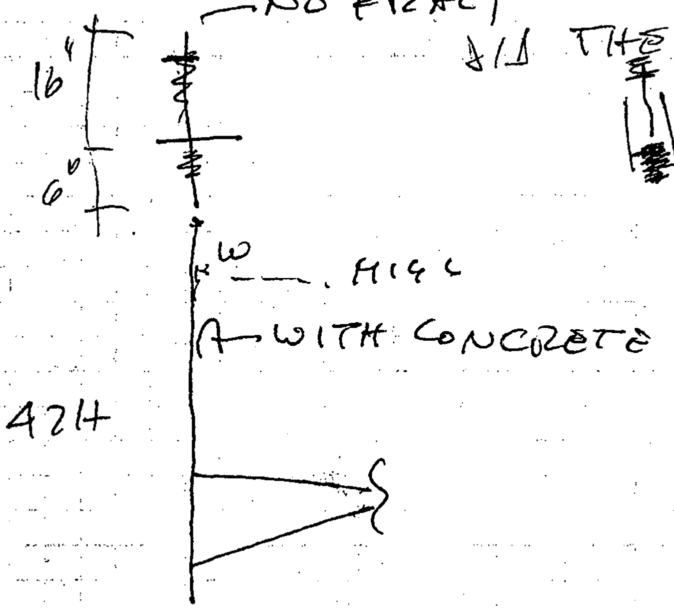
4H40202 40C



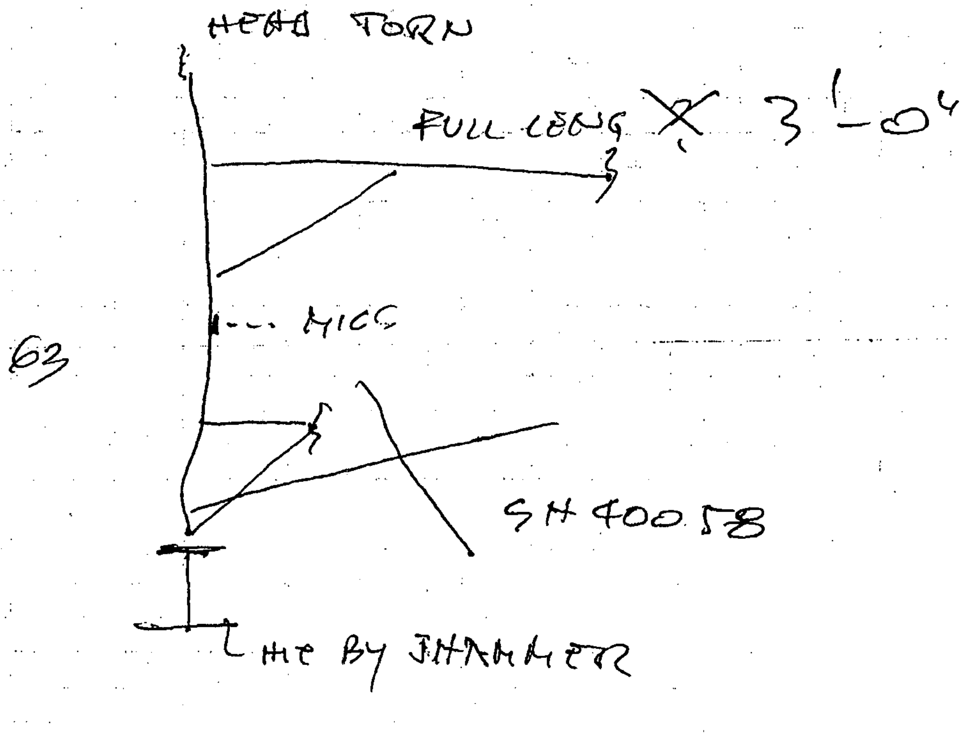
62H

SH 40204 A   40C

DID THE TOP COME OUT ?



SH 40057   40B



SH40138 CONNECT TO SH 40166

SH40120

SHEAR NO CONCRETE

MAYBE HIT BY SHAHAF!

MISS WELD

WITH CONC

42" HIGH

SH 40166 4OF

TORN

MISS

INTACT

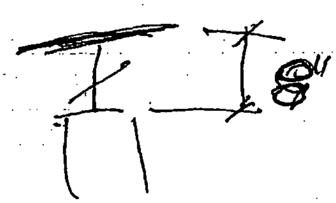
FULL CONC

SH40120

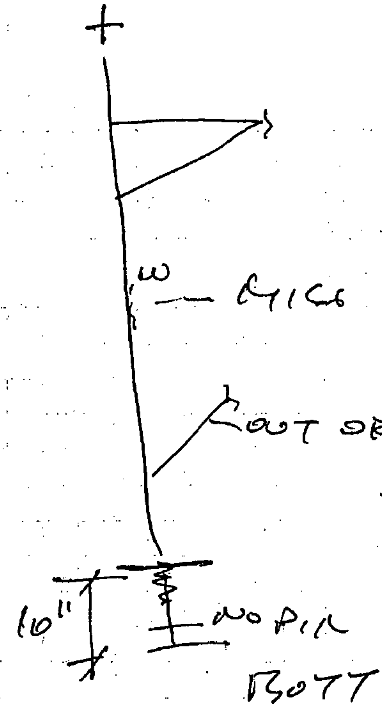
TOP

PIN

SH40138

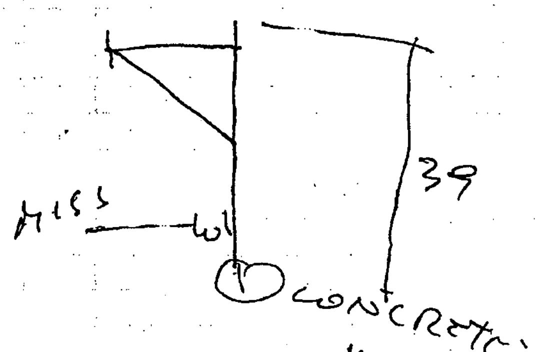


SH 40062 40B



63H

SH40072 40B

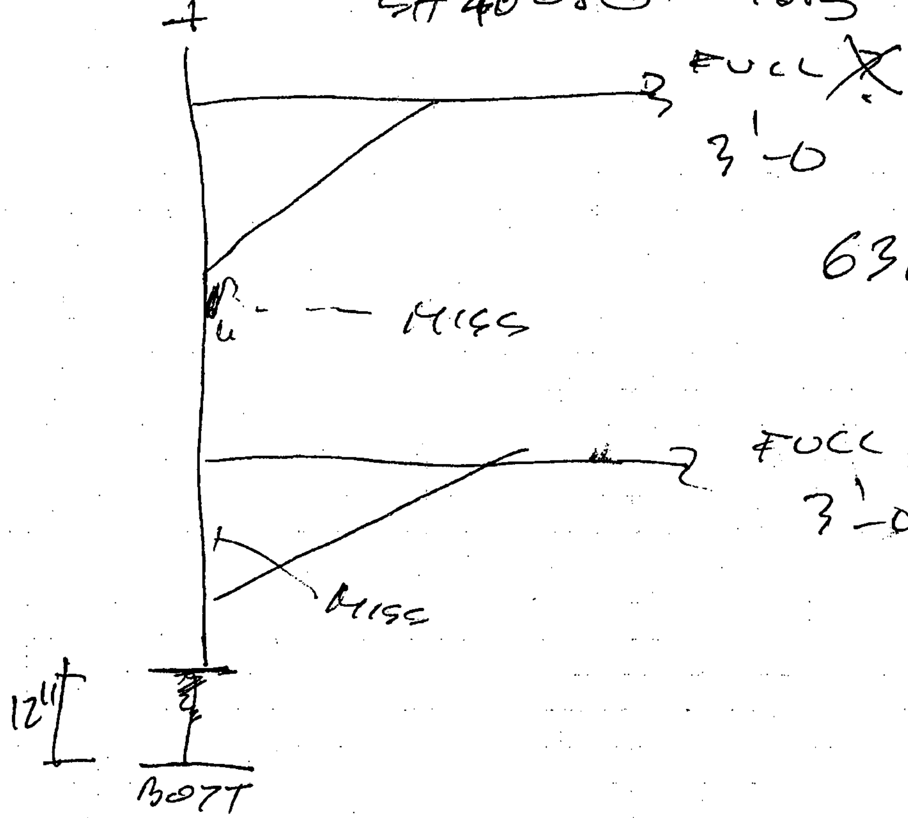


42H ?

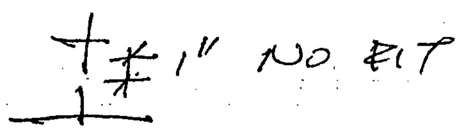
BROKE AND FOUND SHEHRED END

SH 40050 40B

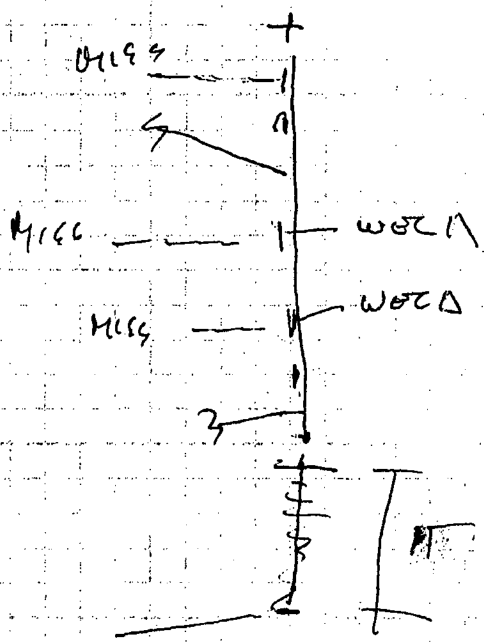
63H



SH 40120 40E



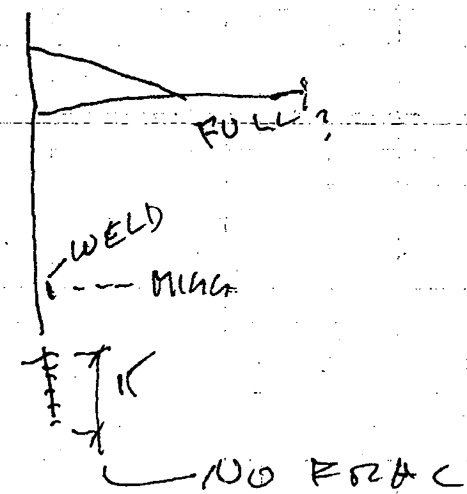
SH 40143 406



64 HIGH ~~I~~
NOT CLEAR
TOP OR BOTT
PROBABLY

NO FRACT - NOT CLEAR NO HOLE FOR PIN

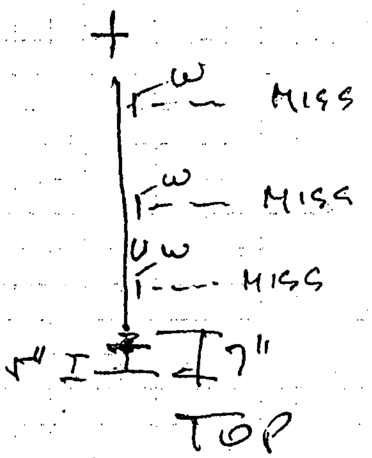
SH 40088 40F



42" HIGH

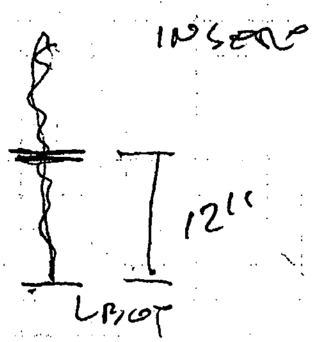
TOP

GH40191
40G-E



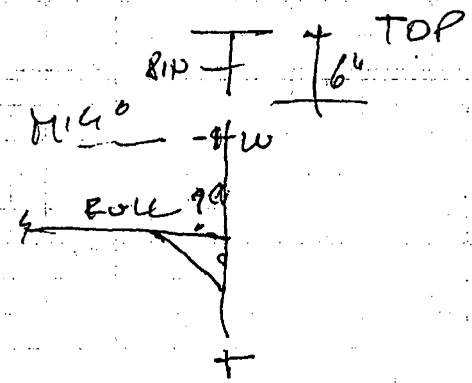
42 H

GH40150 40G



GH40103

40'F

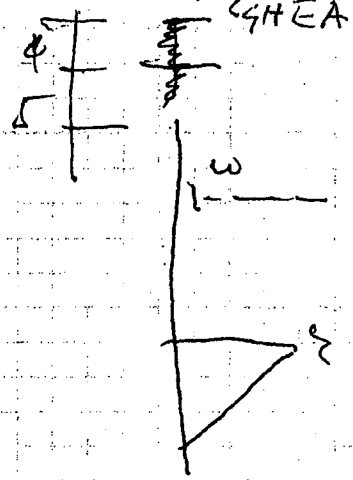


42" H

9H 40021 40G

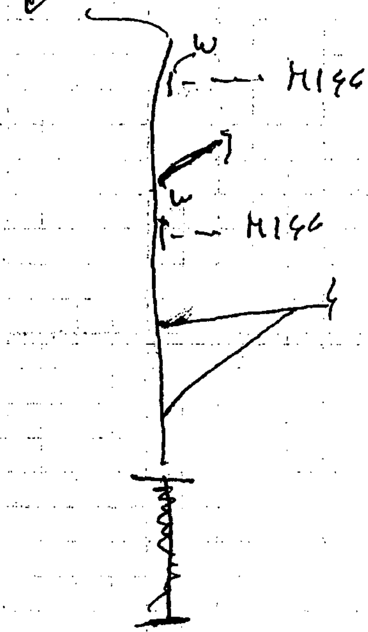
NO SHEAR - HOLE PIN PRESENT

PAINT MARK

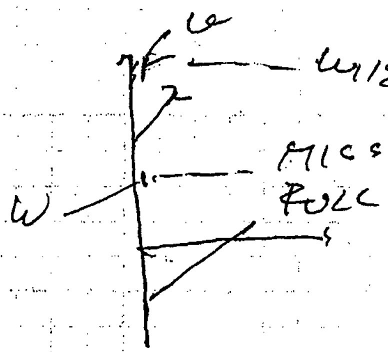


9H 40180 40G

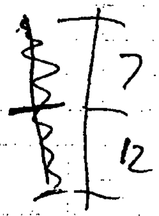
~~[illegible]~~ (demo?)



SH40163 40 G



63H

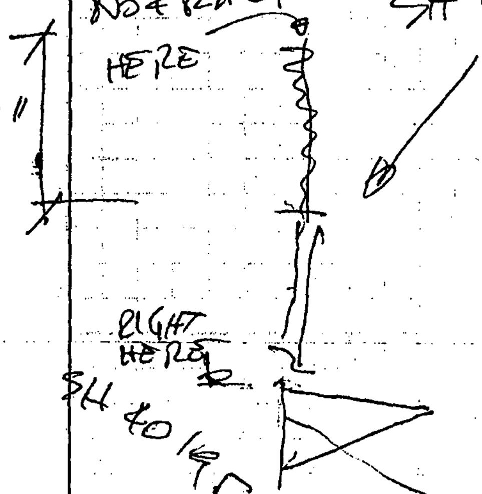


NO PAINT (ROT CONCRETE)

NO SHEAR - HOLE FOR PIN EXISTS

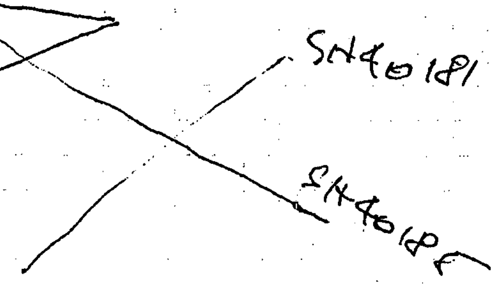
NO FRACT HERE

SH 40196 40C

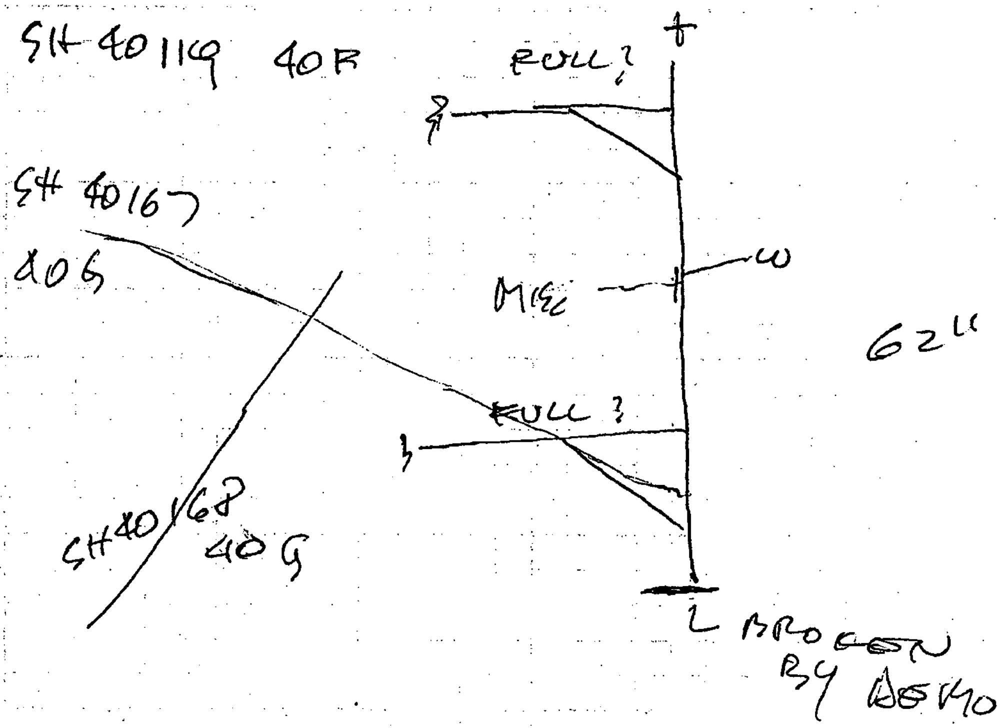


SEEMS THIS ONE WAS FRACTURED AT ACCIDENT

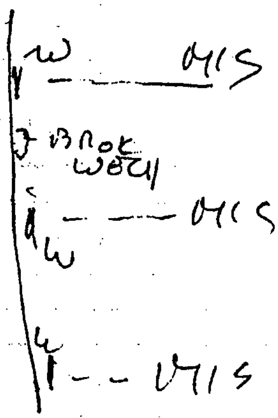
42141/61



broken by jack hammer

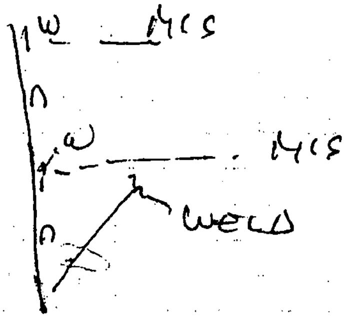

SH 40119 40R
ROLL?
SH 40167
40G
MISC
62"
ROLL ?
SH40168
40G
BROKEN
BY DEMO

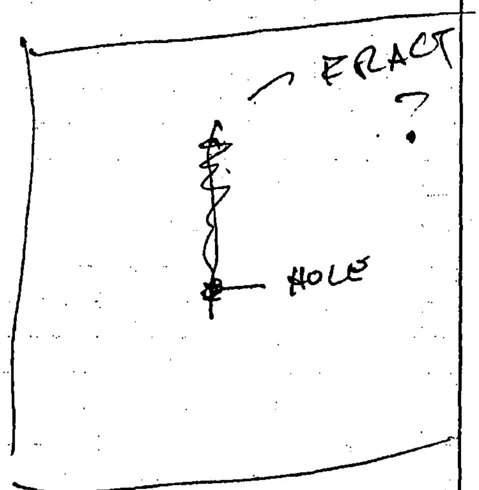
SH 40 135 40G



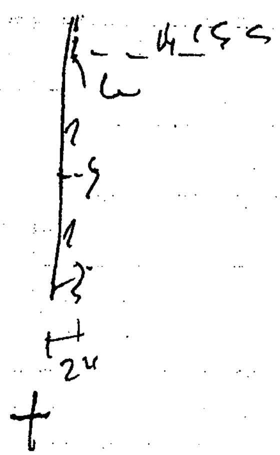
42"

SH 40124 40F





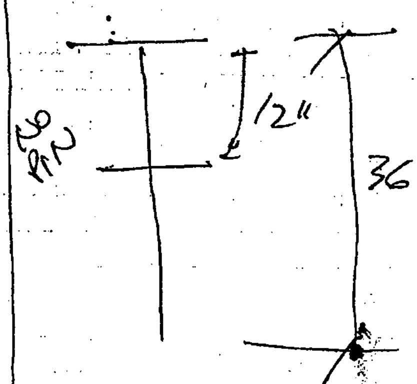
SH 40165 40G



42H

SH 40131 40G

A LITTE BENT



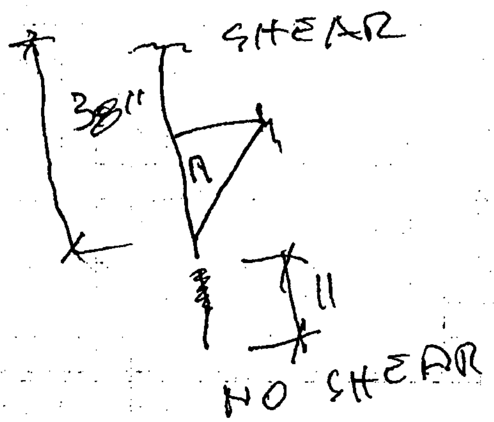
TOP INSERT

T/4 6 MG 40012
40 BC
8 FT

MG 40012
40 B-C
8 FT

AB 40014 B 40 C-B

SH 40207



BOT ?

2/29 SIGNED

SH 40207 A+B
201

MG 40012
13

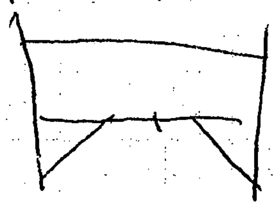
TB 40067
95

(71 A+B) 87 ABC

12 FT
AB 40015 40F
AB 40016 40F
AB 41041 41G

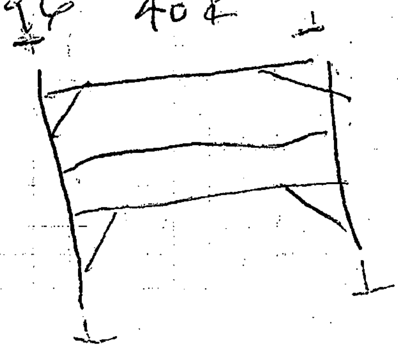
SH 41028 41G

BOTT? T SHEAR



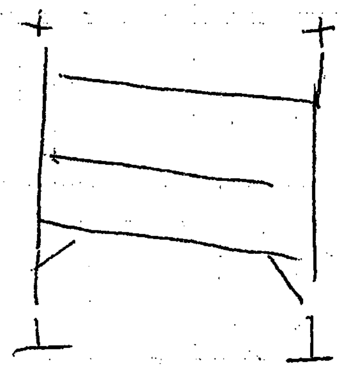
42H

SH40096 40K

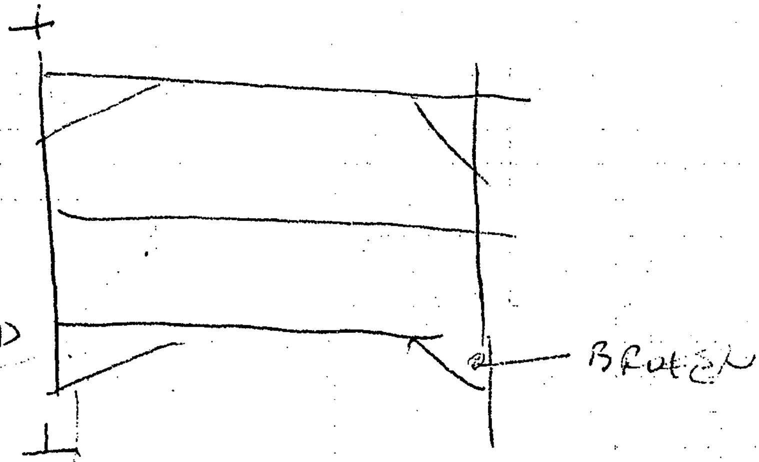


ck 41H

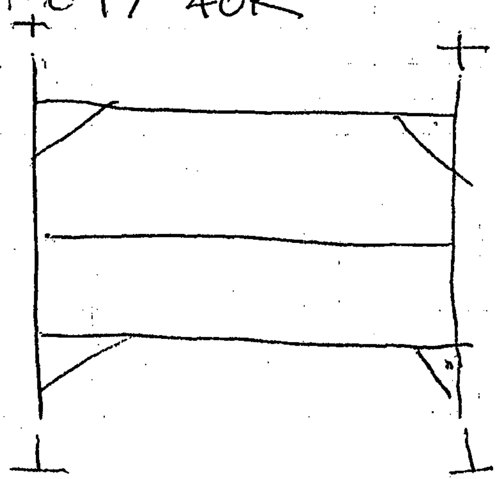
SH40069 40A



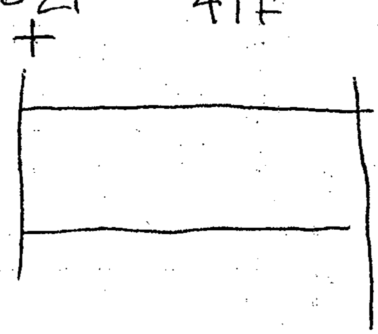
9H 40076 40F

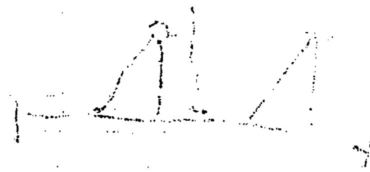


9H40097 40K



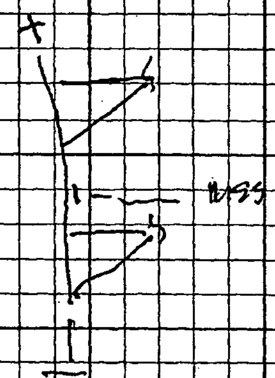
9H41024 41F





SH 40079

40f



2/28 SIGN

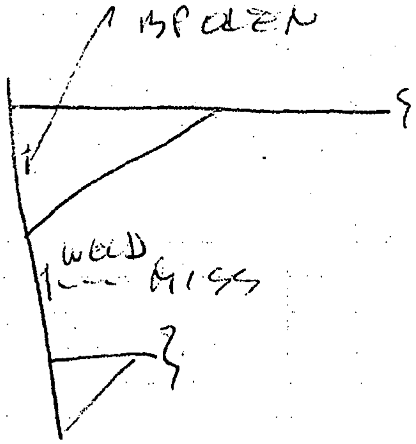
SH 40183

SH 40204

MS 40005 B

SH 40148 B

SH 41173 41G



GH

SH 40105 40G.

1 WELD —— MISS

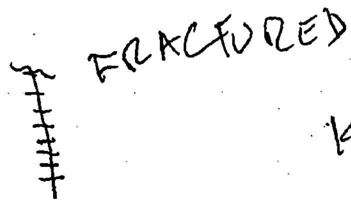
1 WELD BROK PIECE —— MISS

1 WELD ——— MISS

PIN

SH 41193 41F SHOE

MG 40005 40B



14" LG

MG 40005 40B LAT SUP

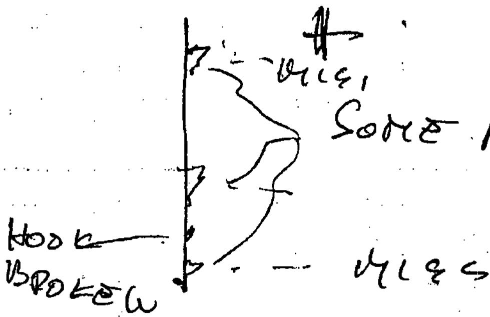
MG 40C04 40G "

SH 40145 PIECE 2 1/4 x 1 15" LG

AB 41044 41F

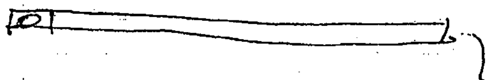
104 LG.

SH 40134 40G



SOME MAT REMAINING

SH 40144 40G



33" LG

FRACTURED
WITH CONCRETE
ON IT.

SH 40077 40F

SH 41151 41G

SH 40136 40G

BRACE
111" lg

SONOTUBE CUT -

MS 40003 A&B 51 - 55
40F

MS 40009 A&B
40F
20 - 24

MS 40011 A&B 40F
45"

2/22
SIGNE

SH 40 149
SH 150
MS 40008

SH 40151
SH 40163

MS 40009
MS 10
MS 40 11

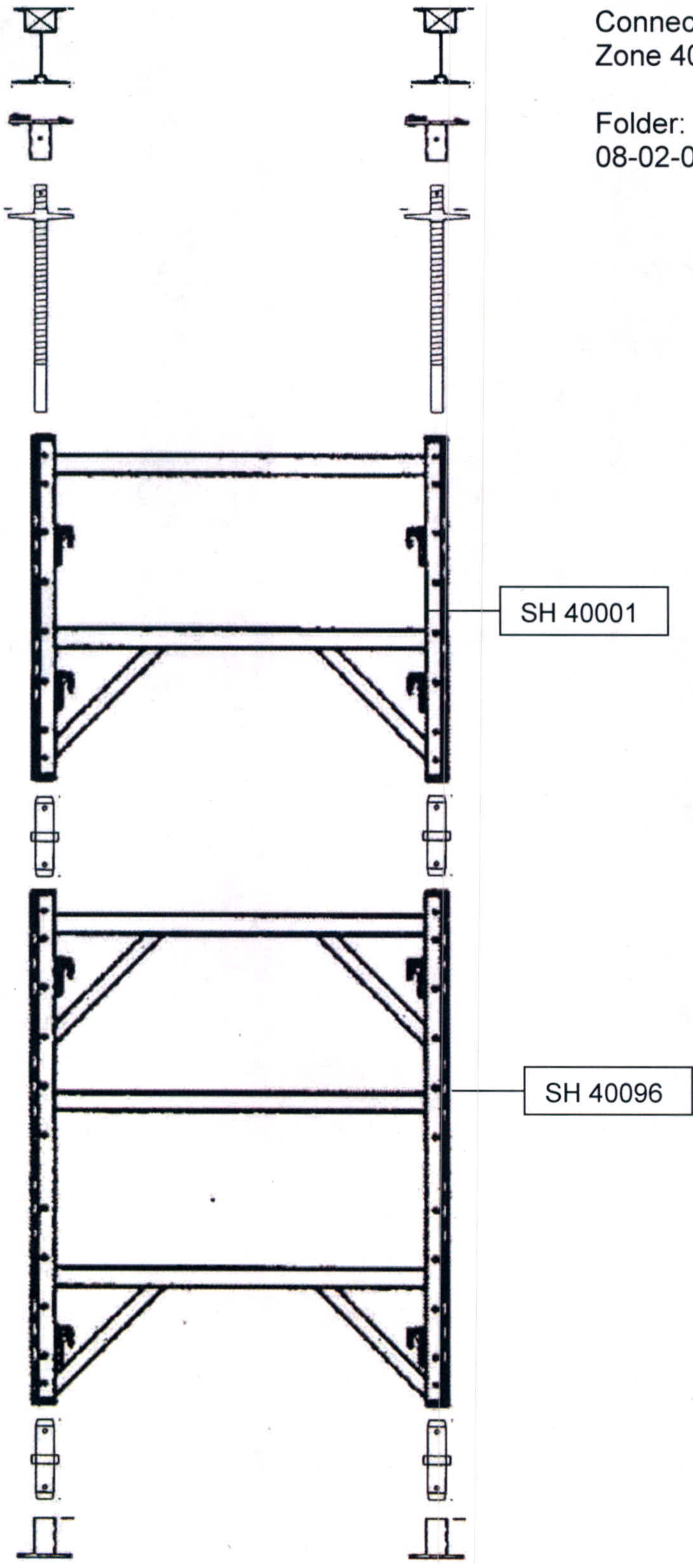
246 Spring Street Investigation

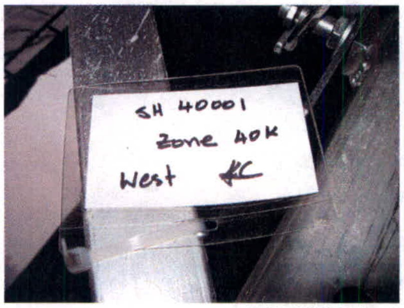
APPENDIX D

Documentation and Preparatory Documents for Shoring Layout

SH 40001
Whole Shore
Connect to SH 40002
Zone 40K

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40001





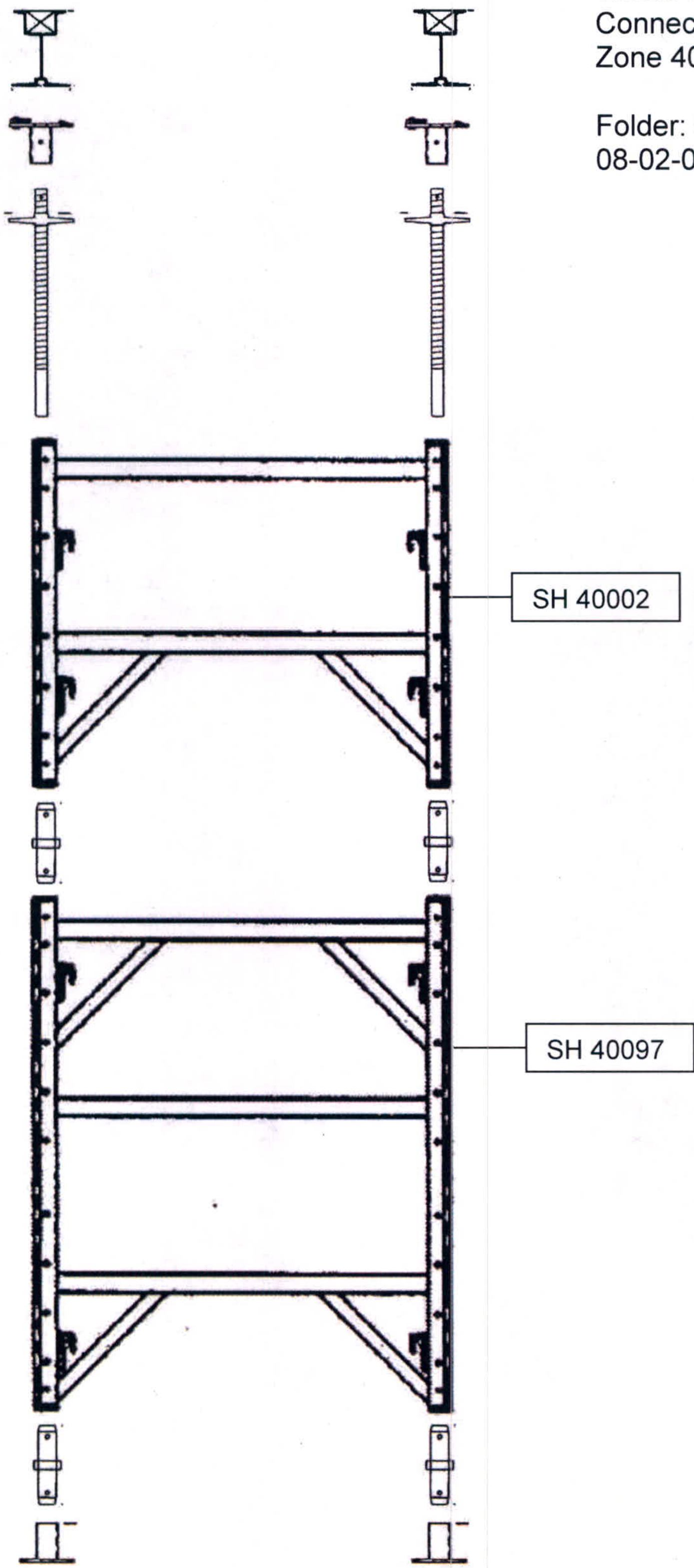
SH40001_1

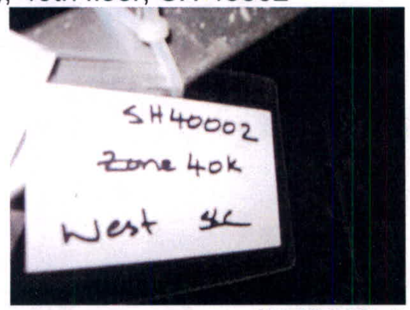
SH40001_2

SH40001_3

SH 40002
Whole Shore
Connect to SH 40001
Zone 40K

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40002





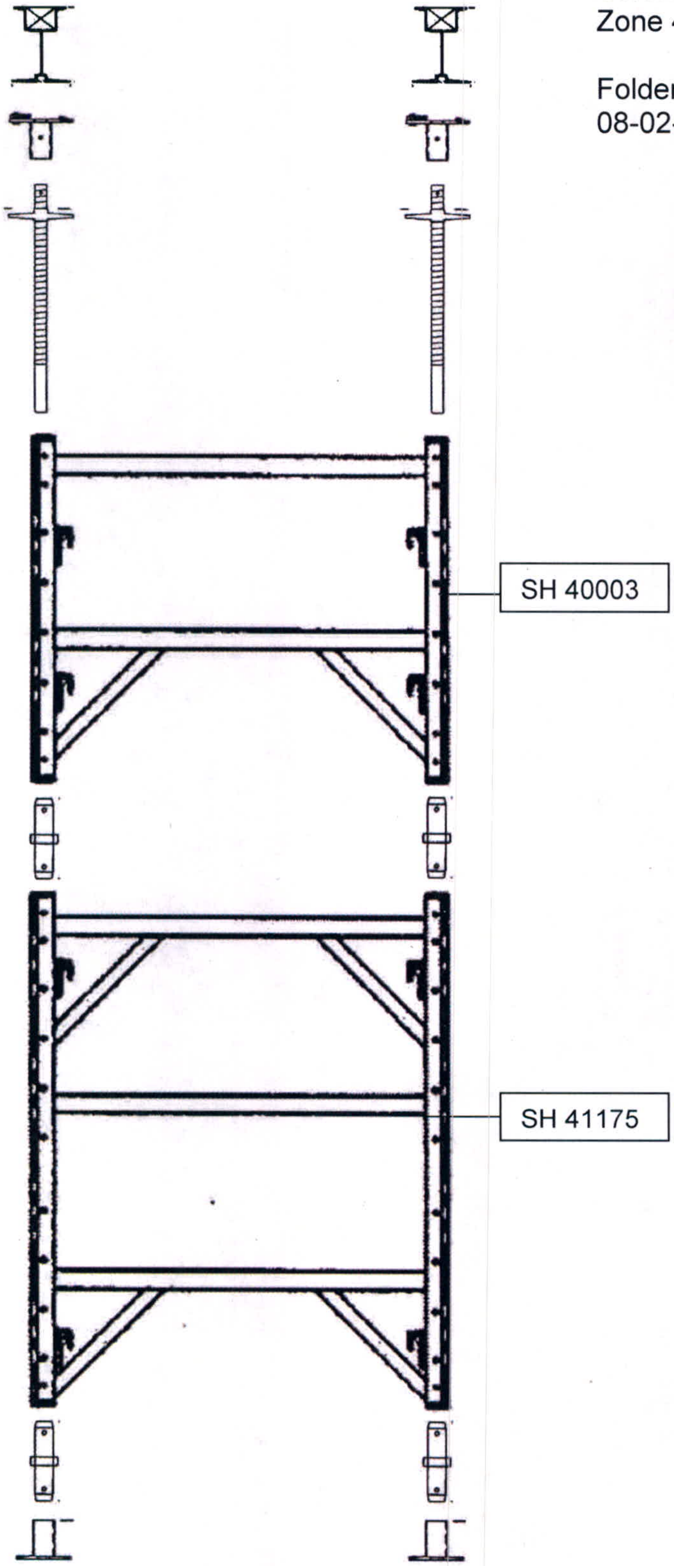
SH40002_2

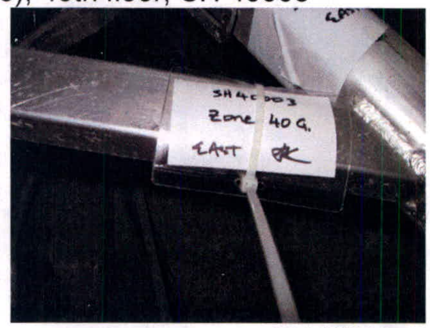
SH40002_3

SH40002_4

SH 40003
Connect to SH 40004
Zone 40G

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40003





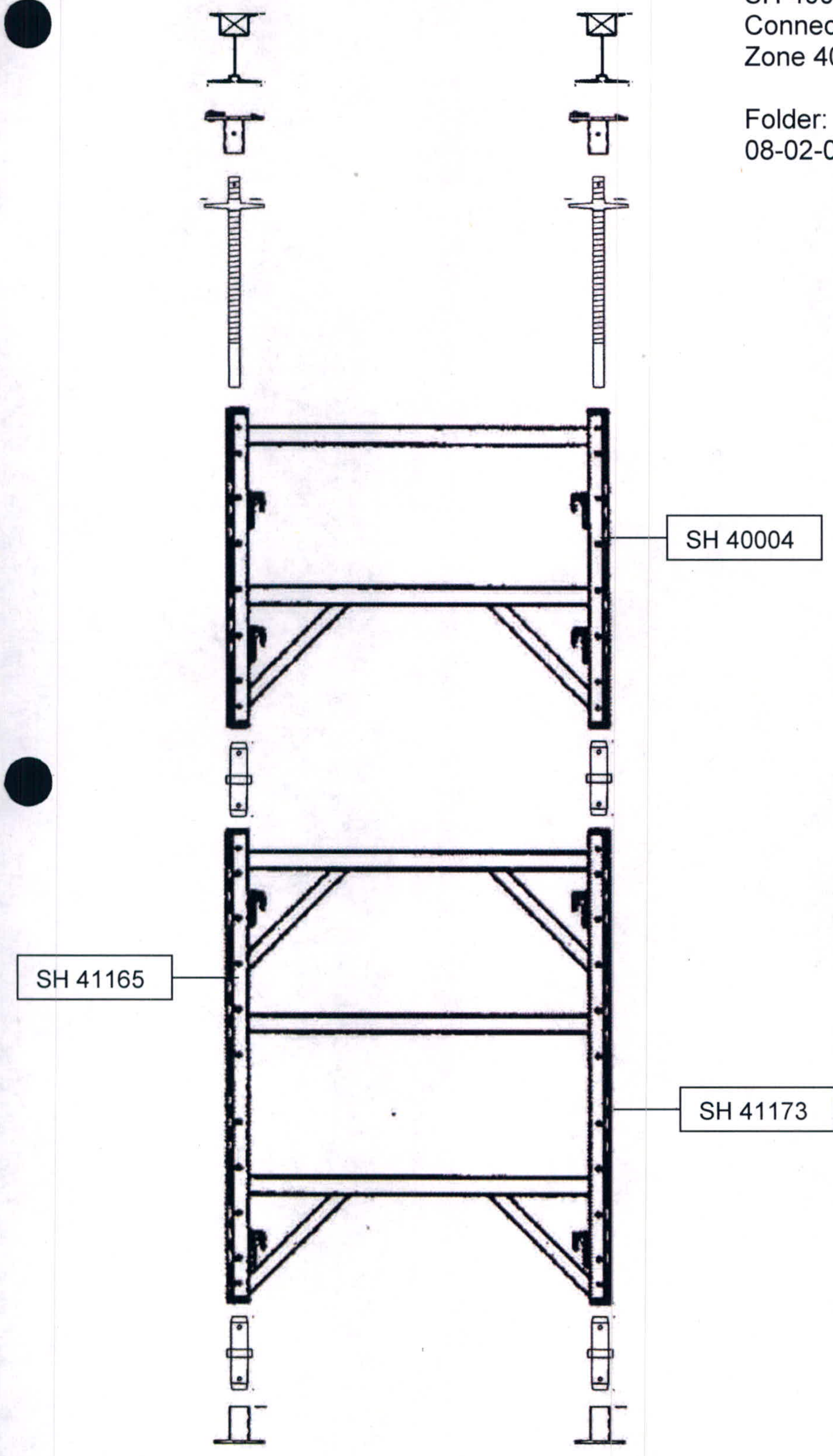
SH40003_2

SH40003_4

SH40003_5

SH 40004
Connect to SH 40003
Zone 40 G

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40004



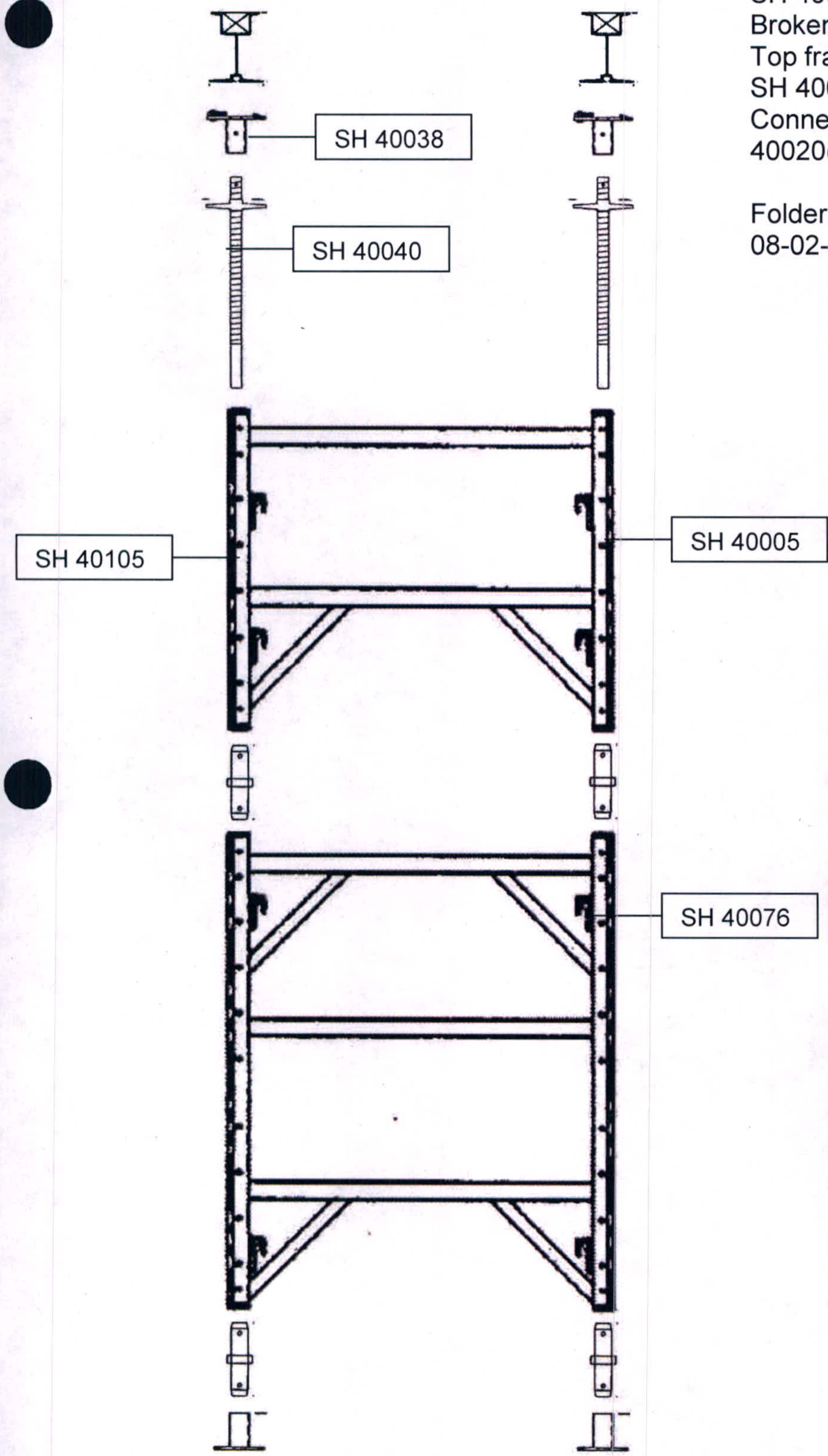
SH40004_1

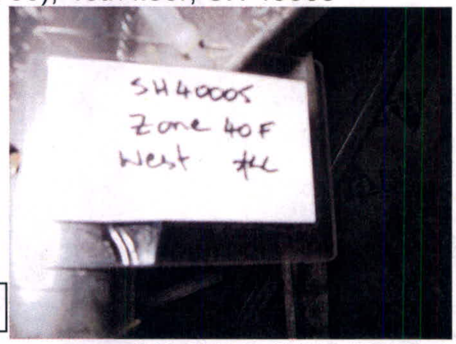
SH40004_5

SH40004_4

SH 40005
Broken and Bent with SH 40105
Top frame of SH 40076
SH 40015 & SH 40020(Frame)
Connected to SH 40015 & SH 40020(Tower)

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40005



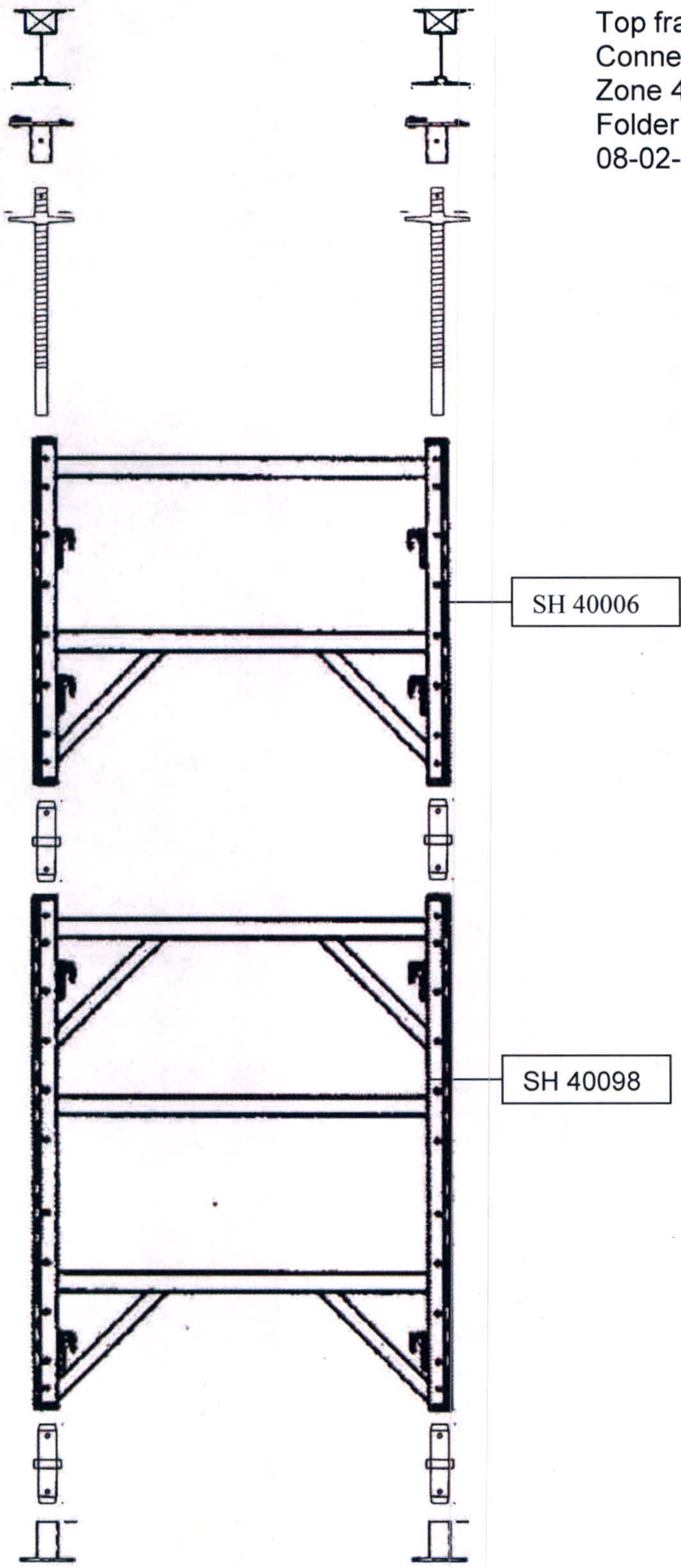


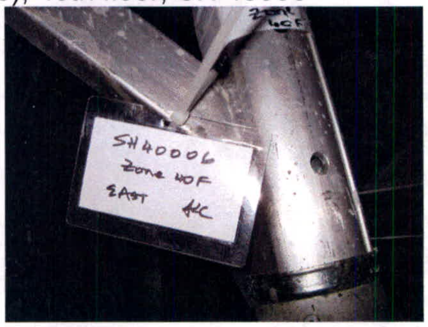
SH40005_2 SH40005_3

SH40005_3

SH40005_4

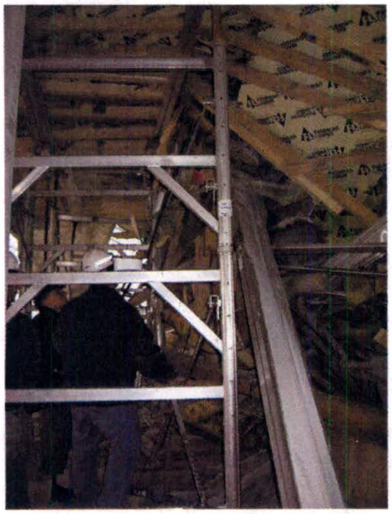
SH 40006
Top frame of SH 40098
Connected to SH 40007(Tower)
Zone 40F
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40006





SH40006_1

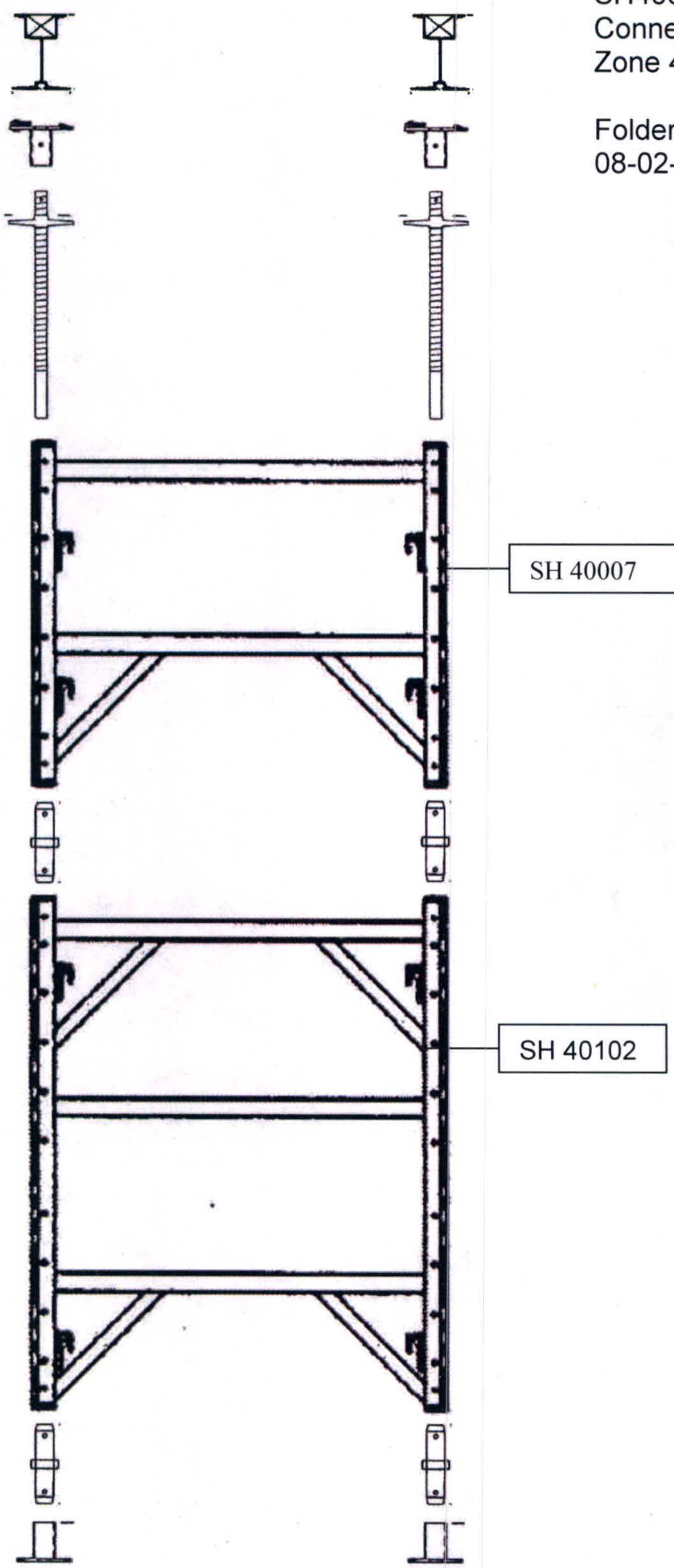
SH40006_3

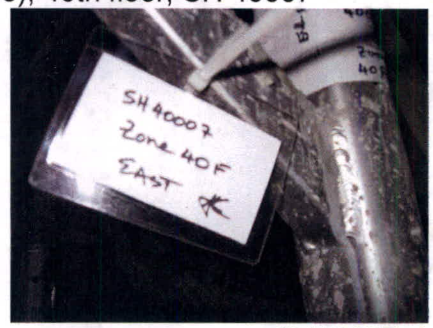


SH40006_4

SH 40007
SH40007
Connected to SH 40006(Tower)
Zone 40K

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40007





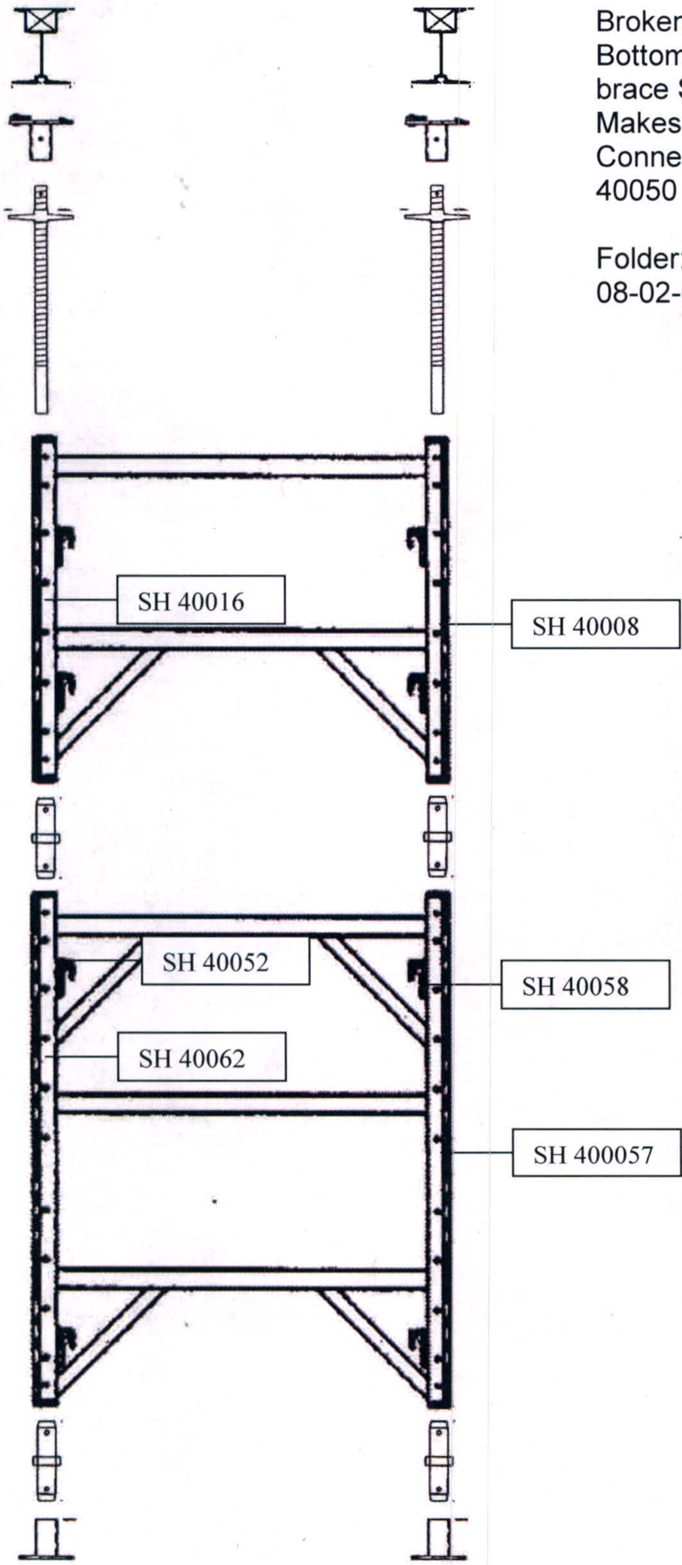
SH40007_1

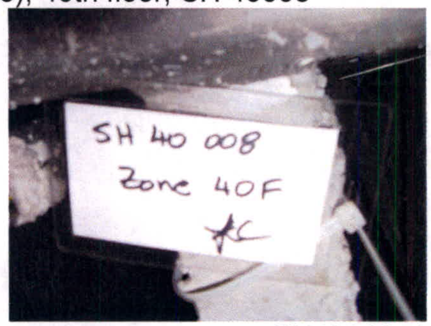
SH40007_3

SH40007_4

SH 40008
Broken
Bottom frame SH 40057 and bottom brace SH 40058
Makes a frame with SH 40016
Connects to SH 40010, SH 40017, SH 40050 & SH 40072(Tower)

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40008



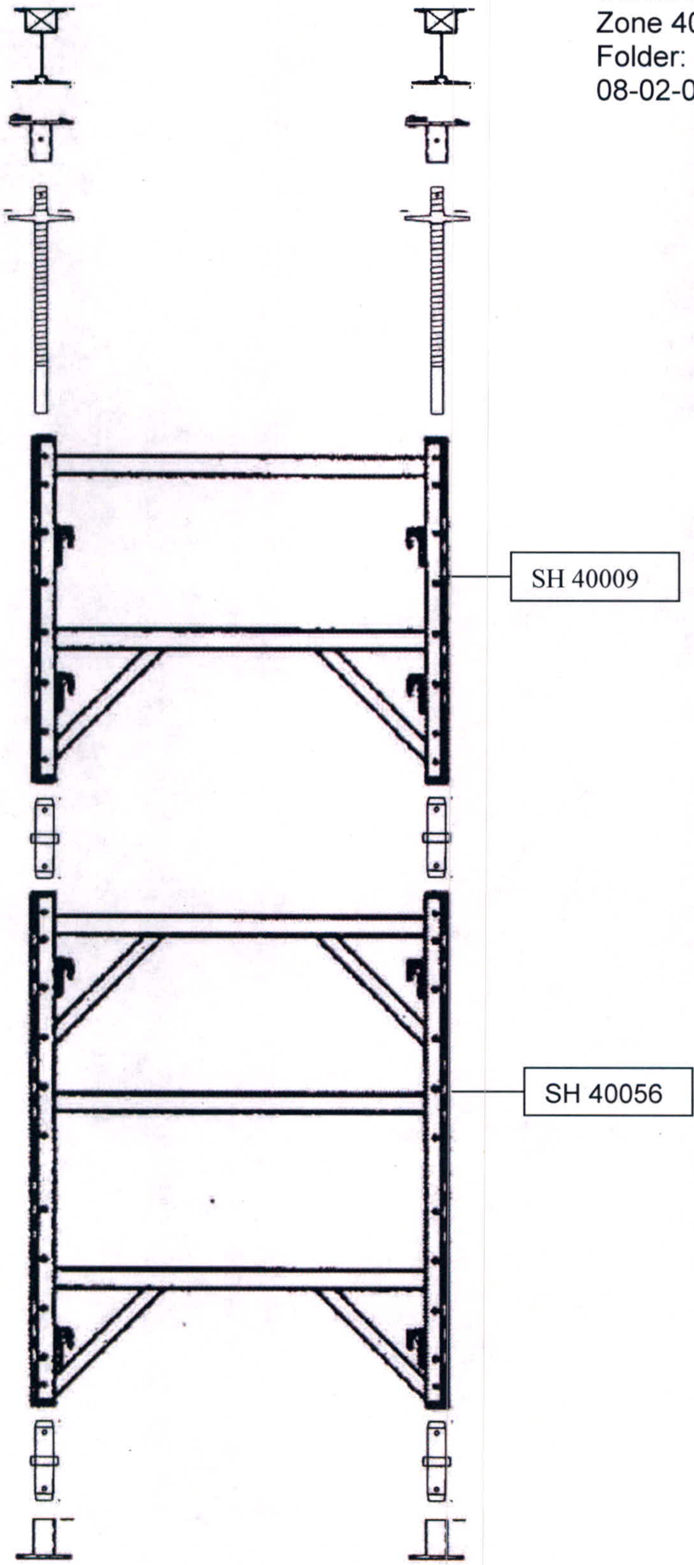


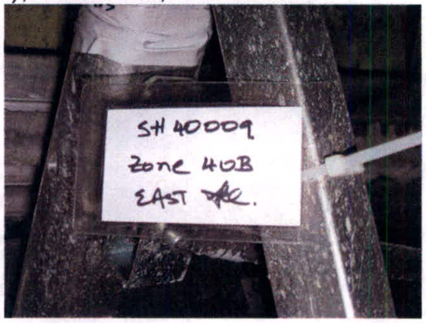
SH40008_1


SH40008_2


SH40008_3

SH 40009
Connect to SH 40011(Tower)
Zone 40B
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40009





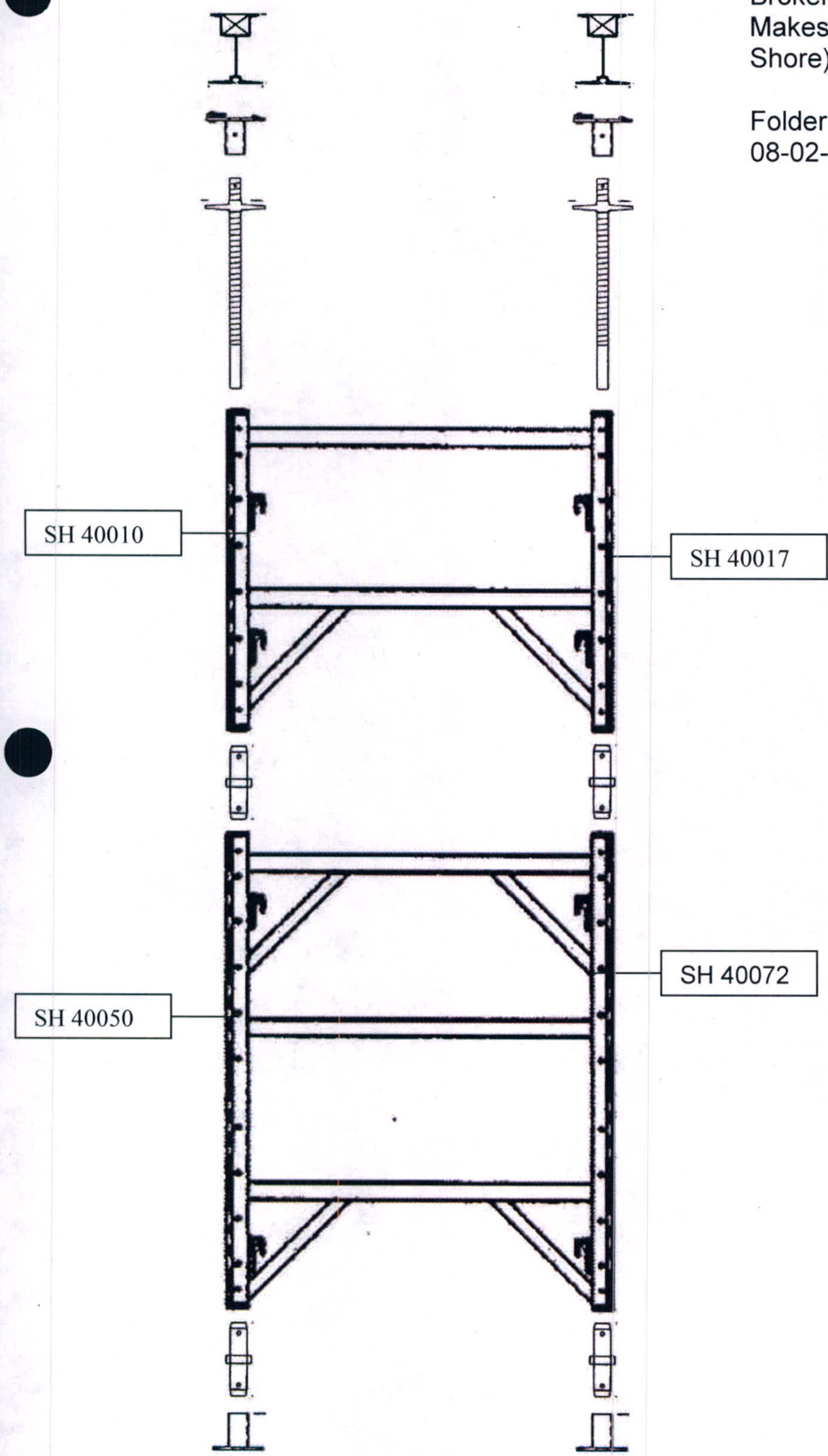
SH40009_1

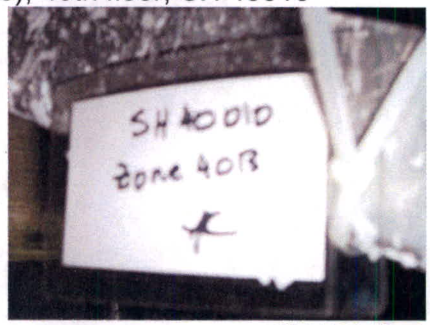
SH40009_3

SH40009_4

SH 40010 (West Shore)
Broken
Makes a frame with SH 40017(East Shore)

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40010



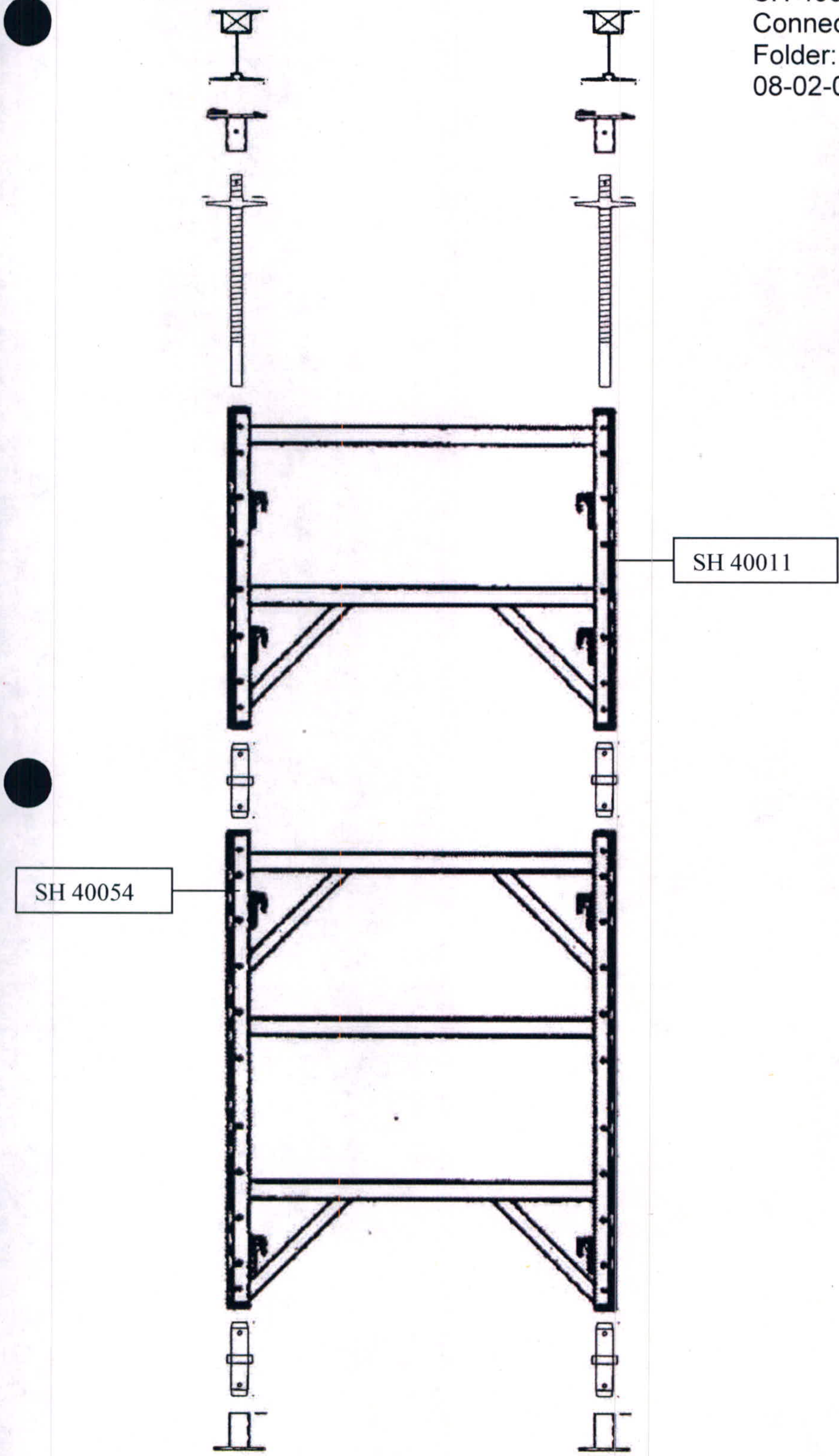


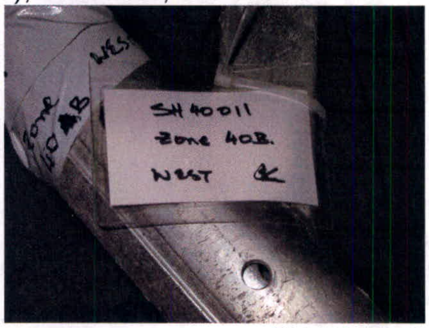
SH40010_1

SH40010_2

SH40010_3

SH 40011
Connect to SH 40009 (Tower)
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40011





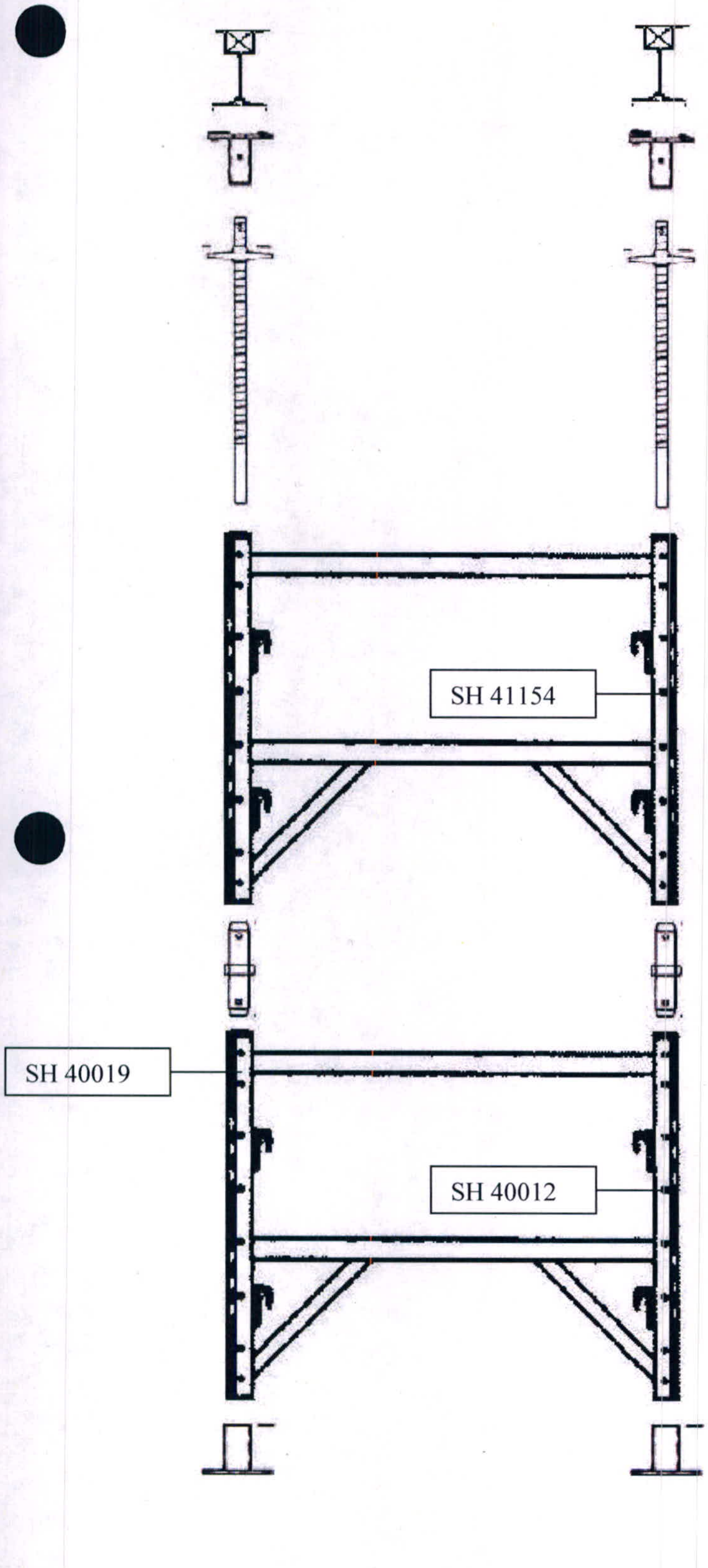
SH40011_2

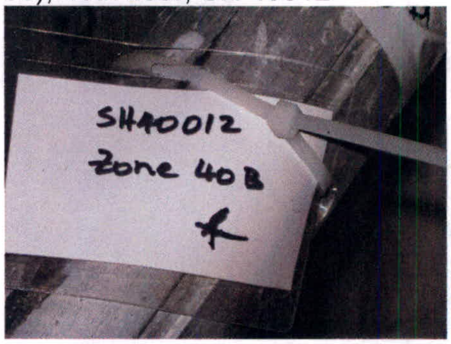
SH40011_5

SH40011_3

SH 40012 (on 41st floor)
Broken Bottom frame with SH 40019 (3'-6") Zone 40B

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40012





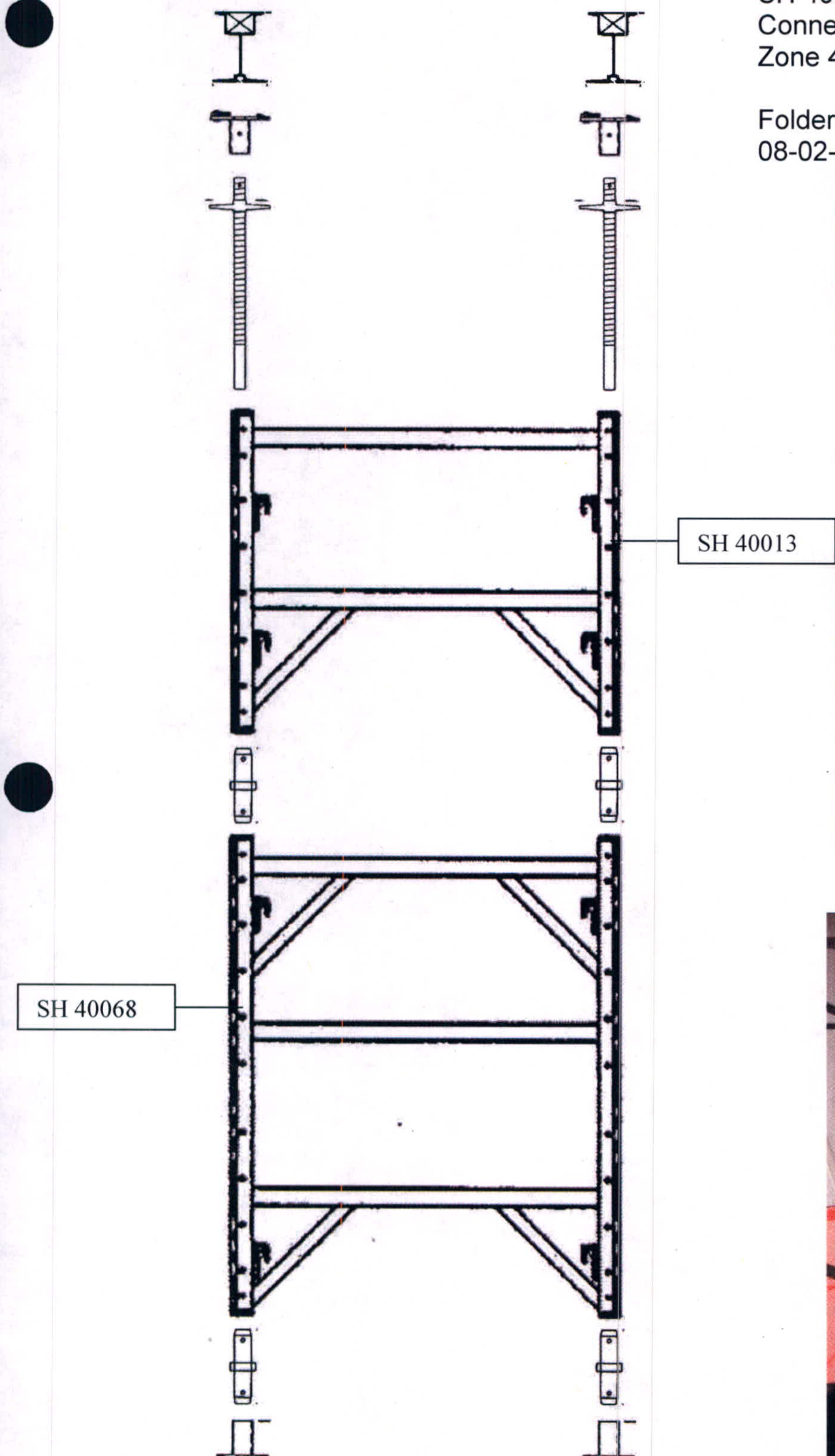
SH40012_1

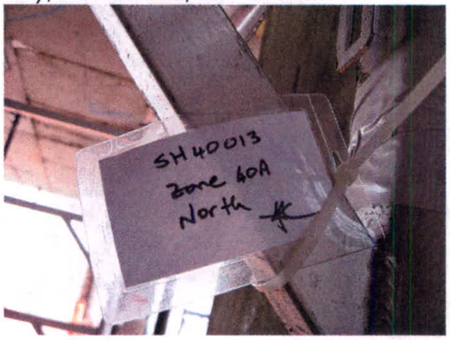

SH40012_2


SH40012_4

SH 40013
Connect to SH 40014(Tower)
Zone 40A

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40013



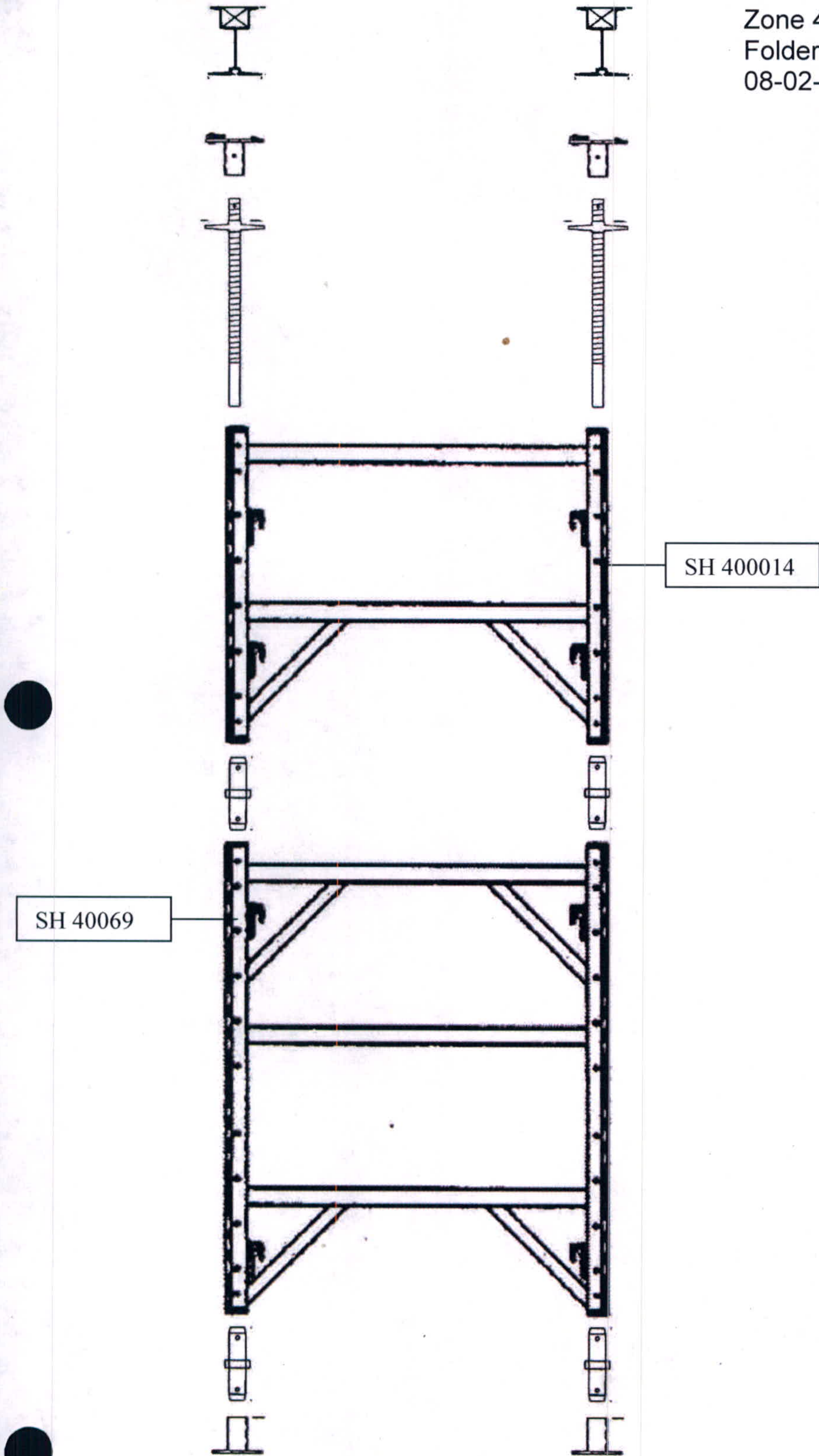


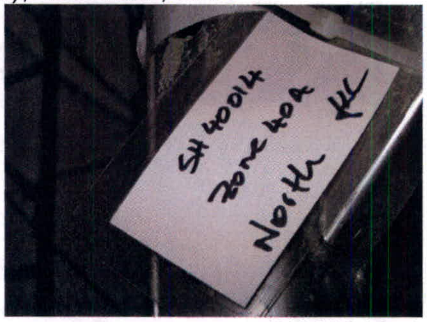
SH40013_2

SH40013_3

SH40013_4

SH 40014
Connect to SH 40014(Tower)
Zone 40A
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH





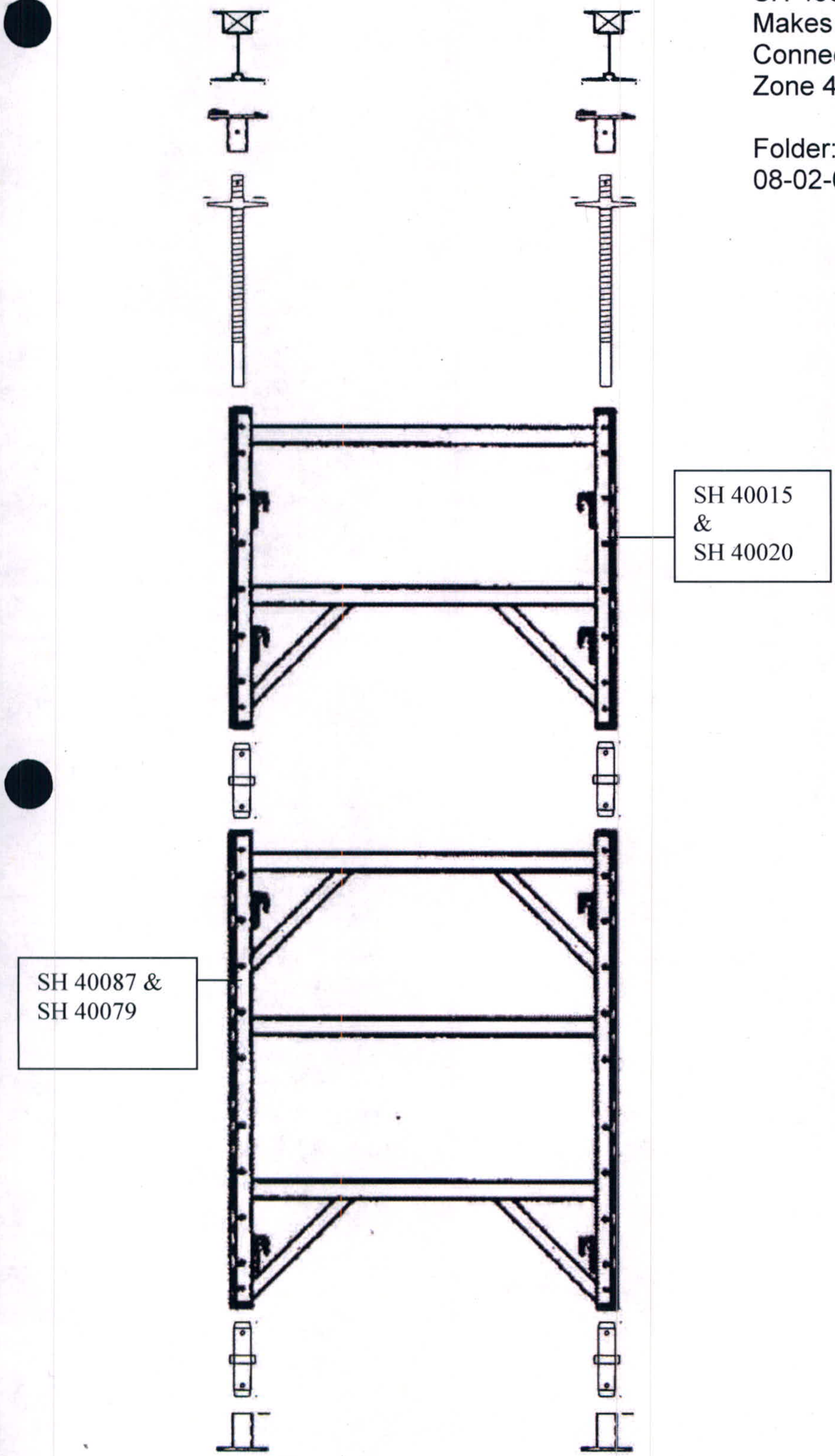
SH40014_1

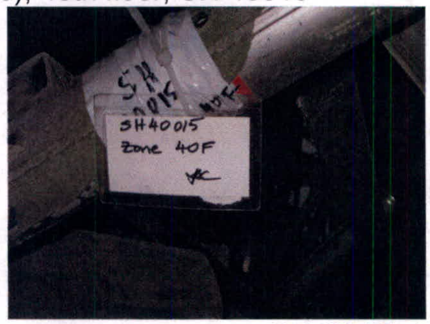
SH40014_2

SH40014_3

SH 40015
Makes a frame with SH 40020
Connects to SH 40005(Tower)
Zone 40F

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40015



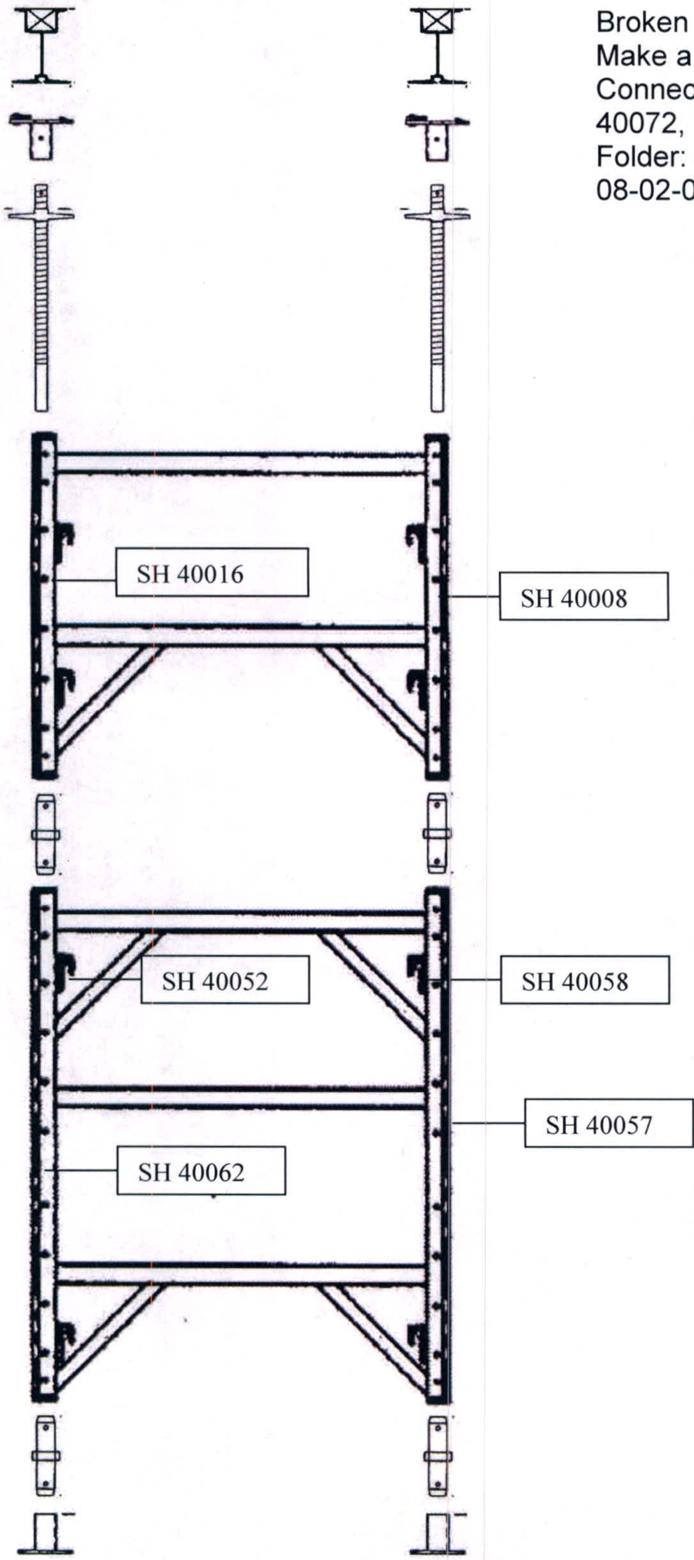


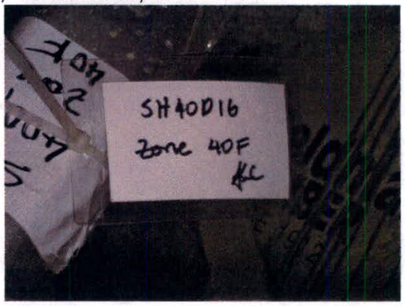
SH40015_1


SH40015_2


SH40015_3

SH 40016
Broken
Make a frame with SH 40008
Connects SH 40010, SH 40017, & SH 40072, SH 40062(Tower)
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40016





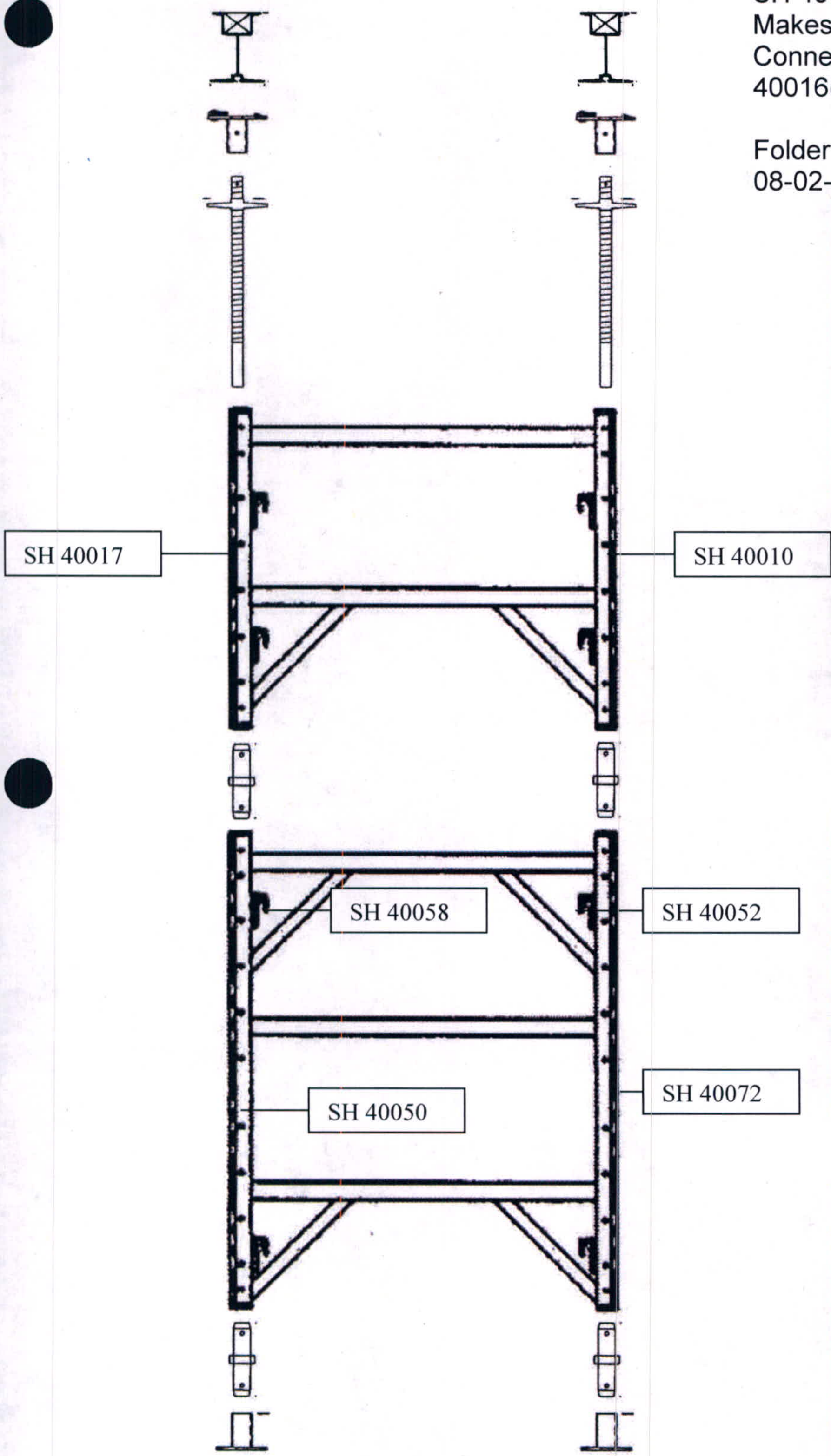
SH40016_1

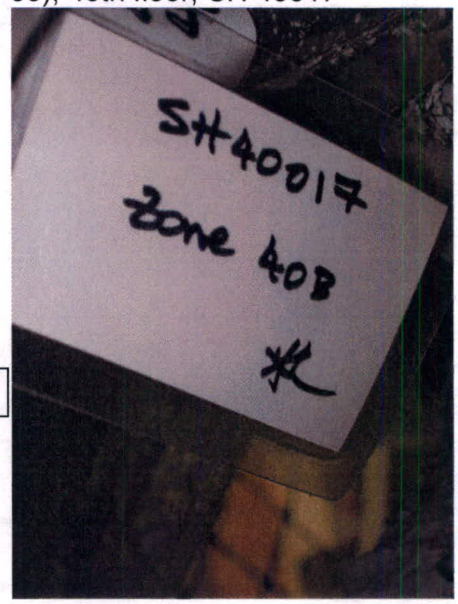

SH40016_3


SH40016_2

SH 40017(East)
Makes one frame with SH 40010(West)
Connects to SH 40008 & SH 40016(Tower)

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40017





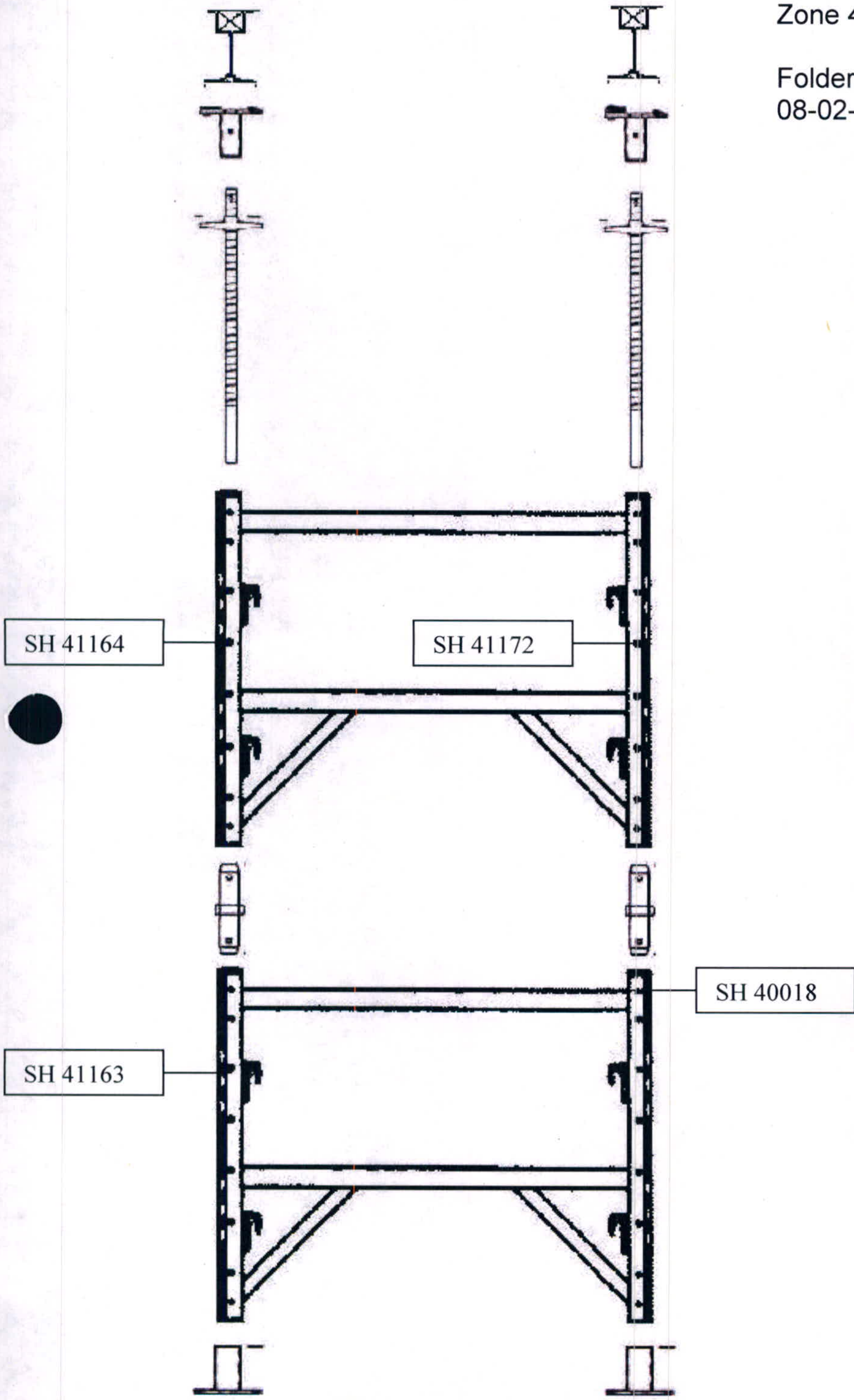
SH40017_1

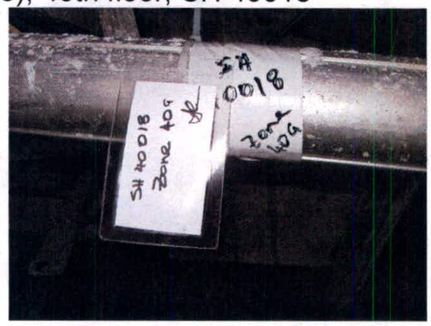

SH40017_2


SH40017_3

SH 40018
Broken
Bottom Frame on 41st floor
Zone 40G

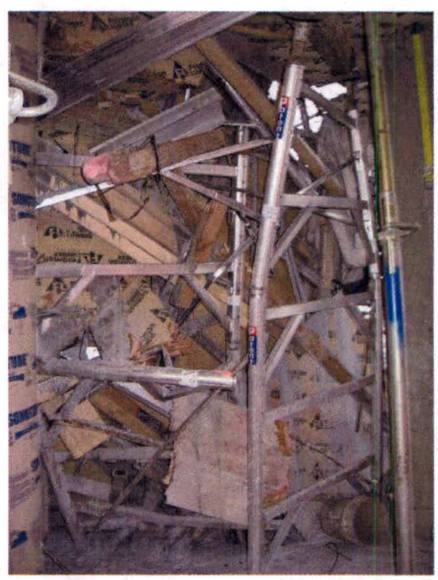
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40018





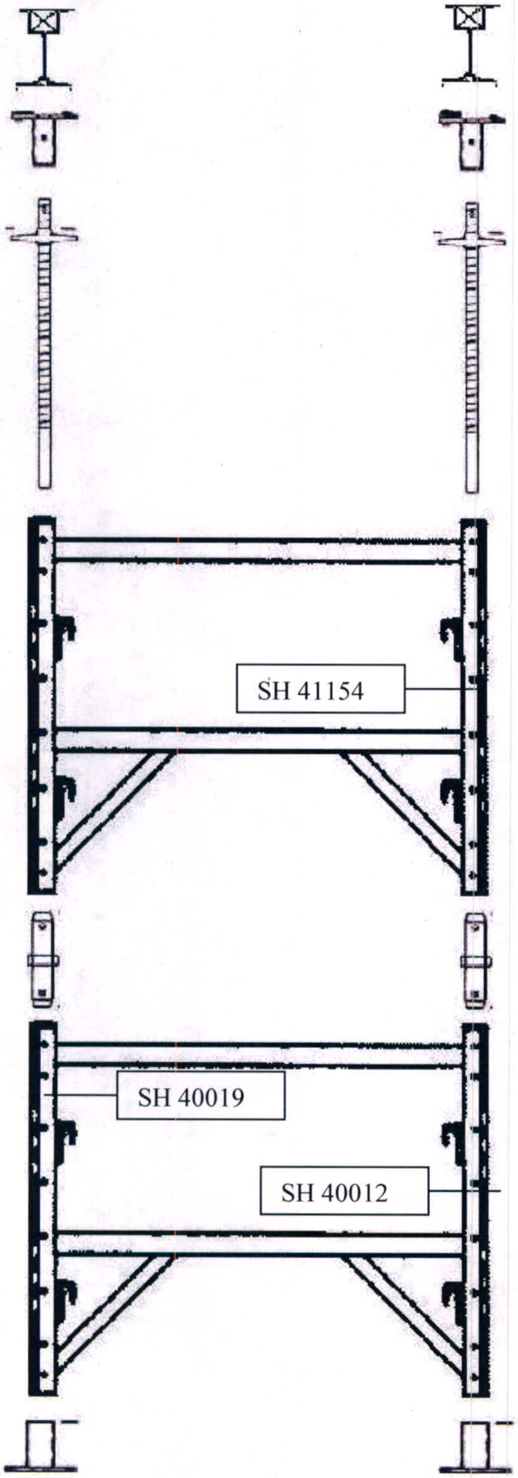
SH40018_1

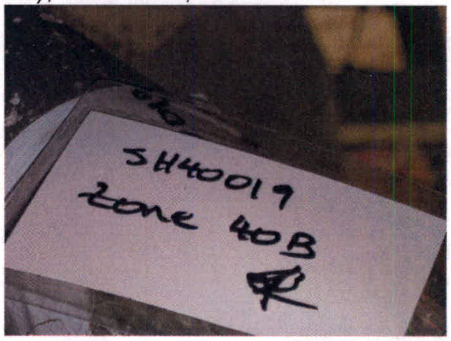

SH40018_2


SH40018_3

SH 40019
41St Floor
Bottom Frame
Broken with SH 40012

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 40th floor; SH 40019





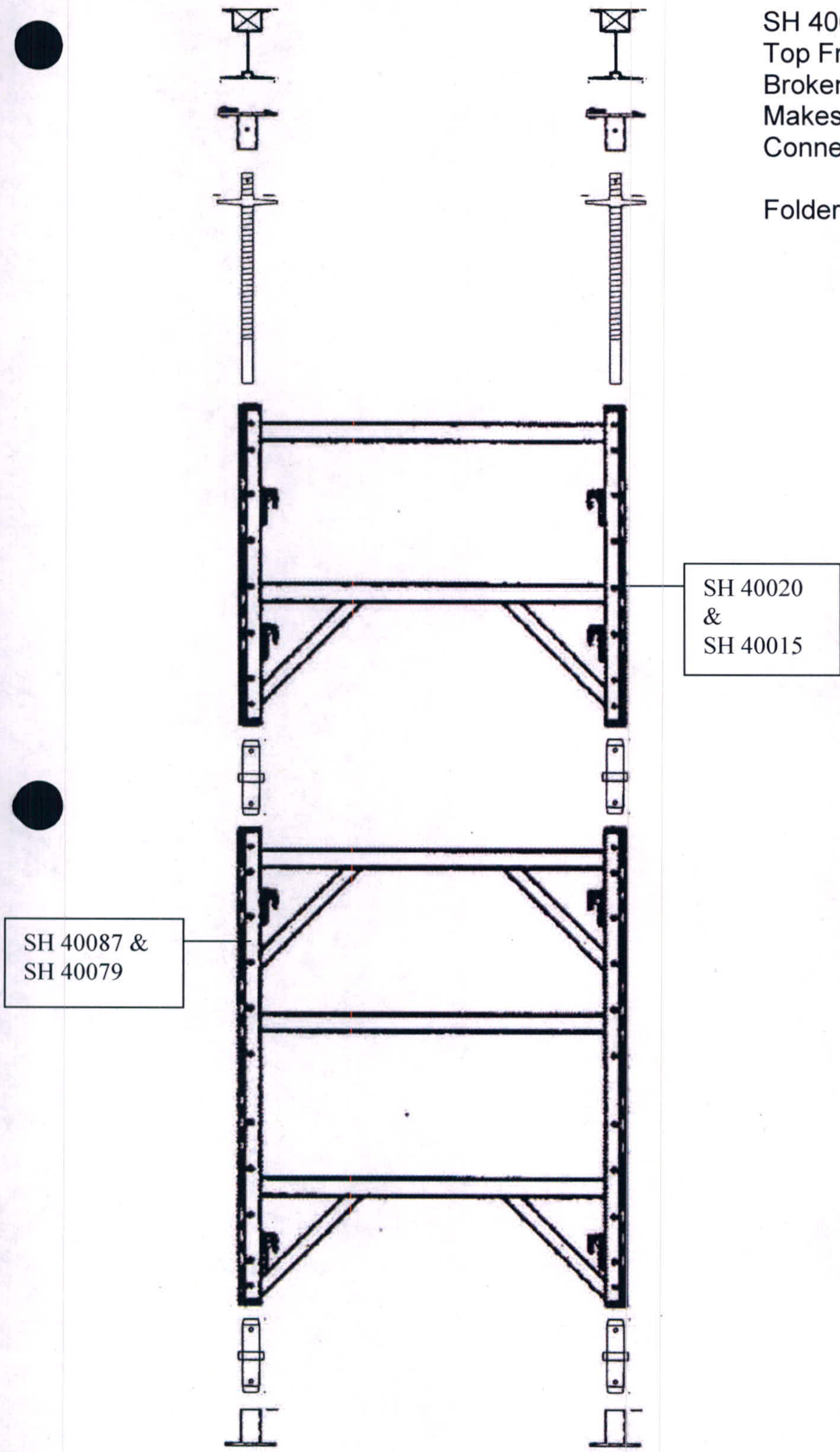
SH40019_1

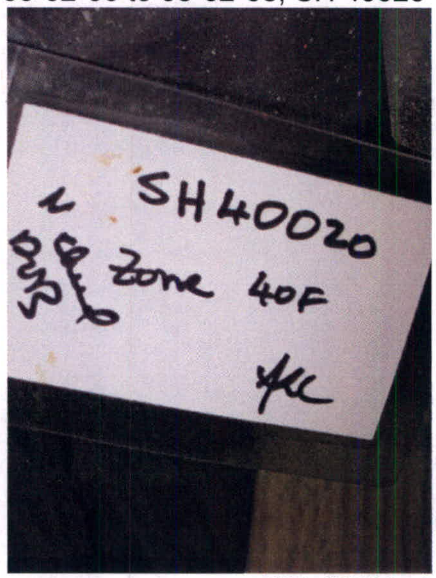
SH40019_3

SH40019_2

SH 40020
Top Frame
Broken
Makes a frame with SH 40015
Connects to SH 40005(Tower)

Folder: 08-02-06 to 08-02-08; SH 40020

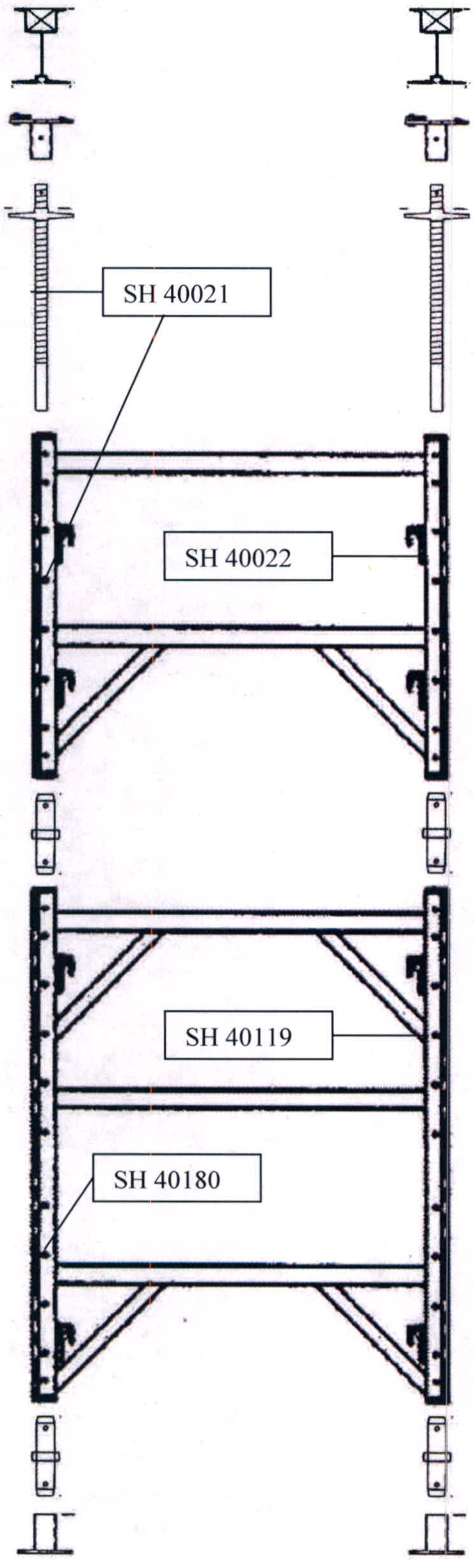




SH 40020_1

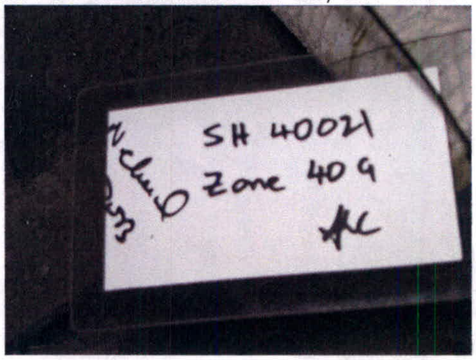
SH 40020_3

SH 40020_2



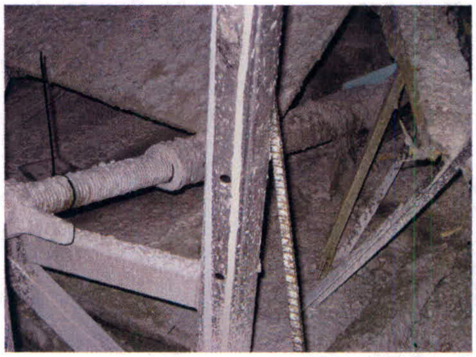
SH 40021
Zone 40G
Found on Floor
Collapse Zone
Screw Jack and top shore broken

Folder: 08-02-06 to 08-02-08; SH 40021



SH 40021_1

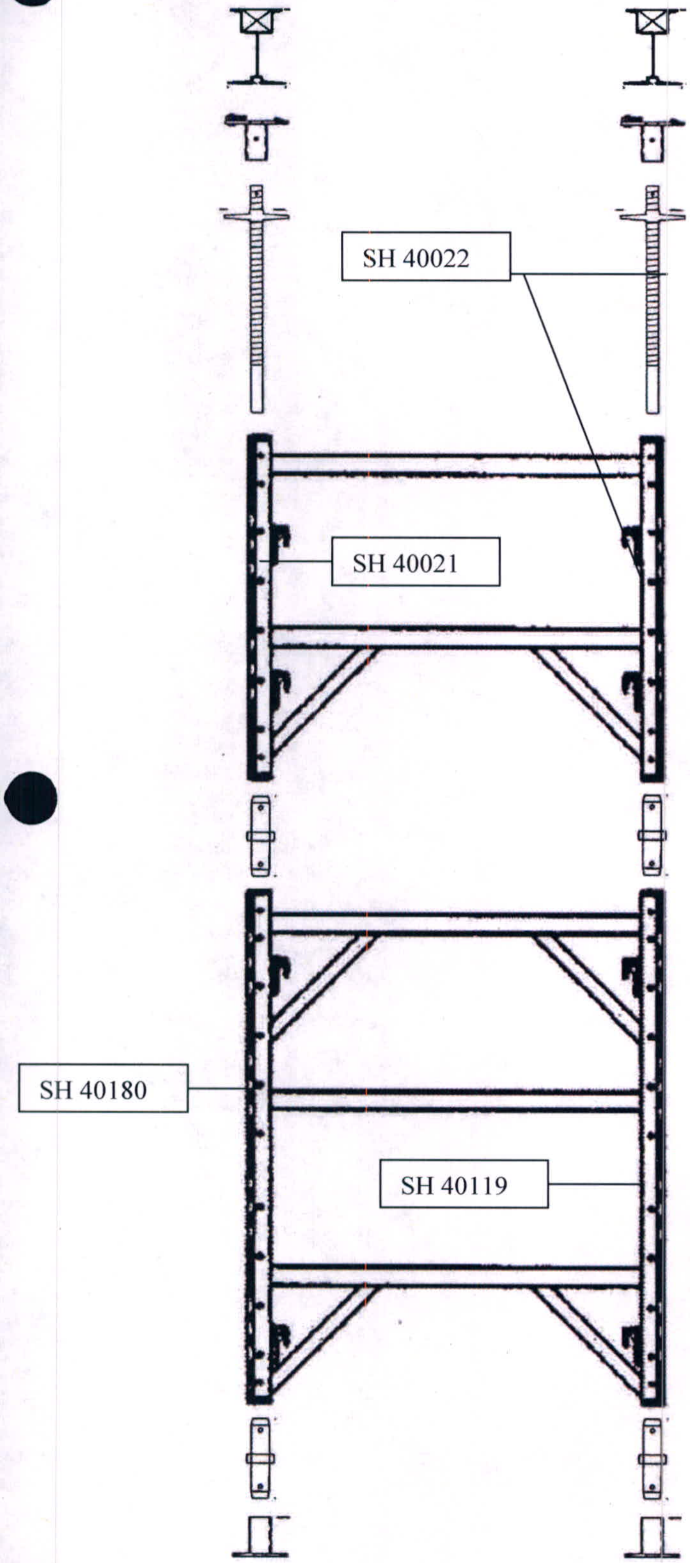
SH 40021_3

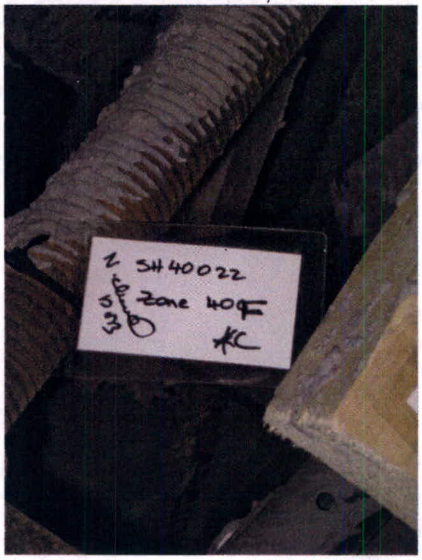


SH 40021_2

SH 40022
Screw Jack and Shore
Zone 40F
Connection to other shore leg

Folder: 08-02-06 to 08-02-08; SH 40022

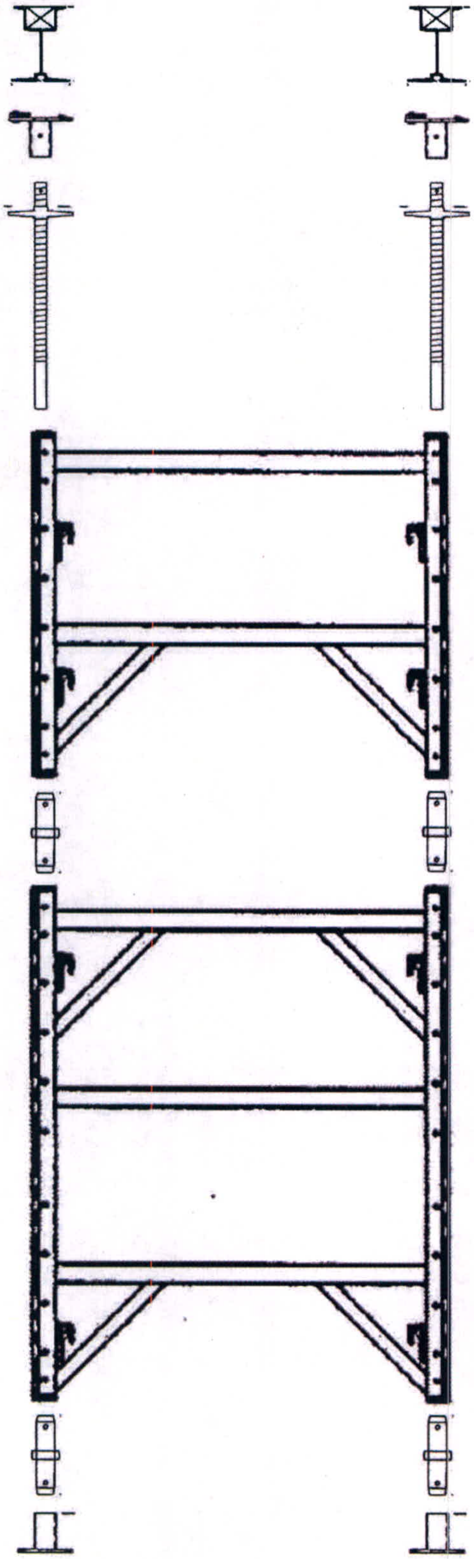




SH 40022_1

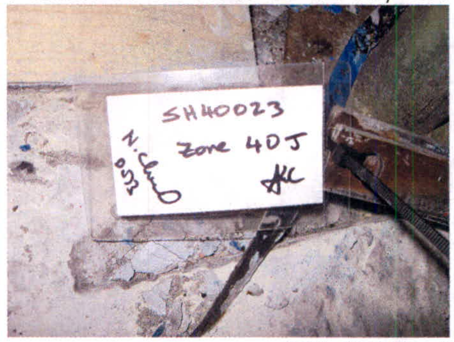

SH 40022_2


SH 40022_3



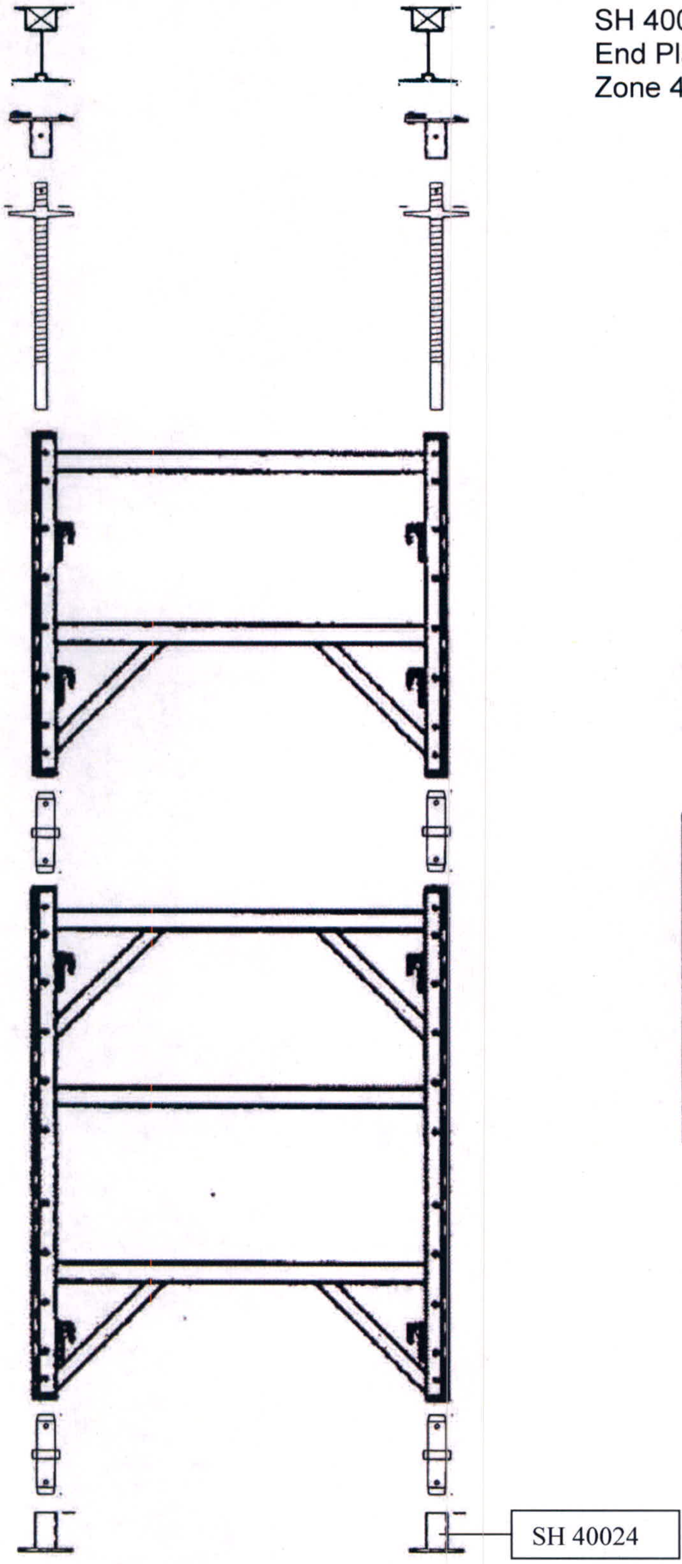
SH 40023
X Brace
Zone 40J

Folder: 08-02-06 to 08-02-08; SH



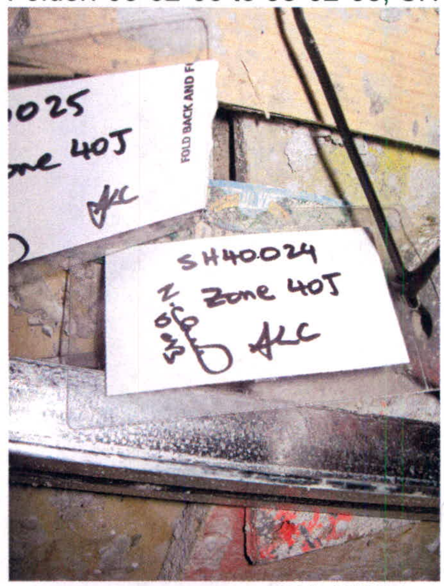
SH 40023_1

SH 40023_2



SH 40024
End Plate
Zone 40J

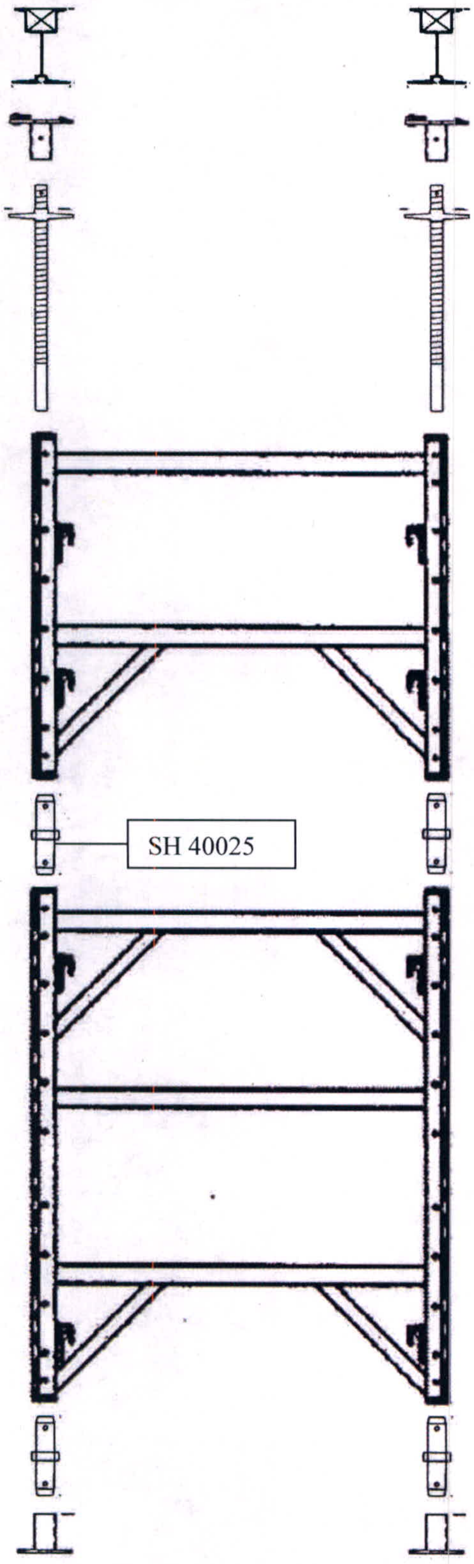
Folder: 08-02-06 to 08-02-08; SH



SH 40024_1

SH 40024_2

SH 40025
Tower Coupler
Zone 40J



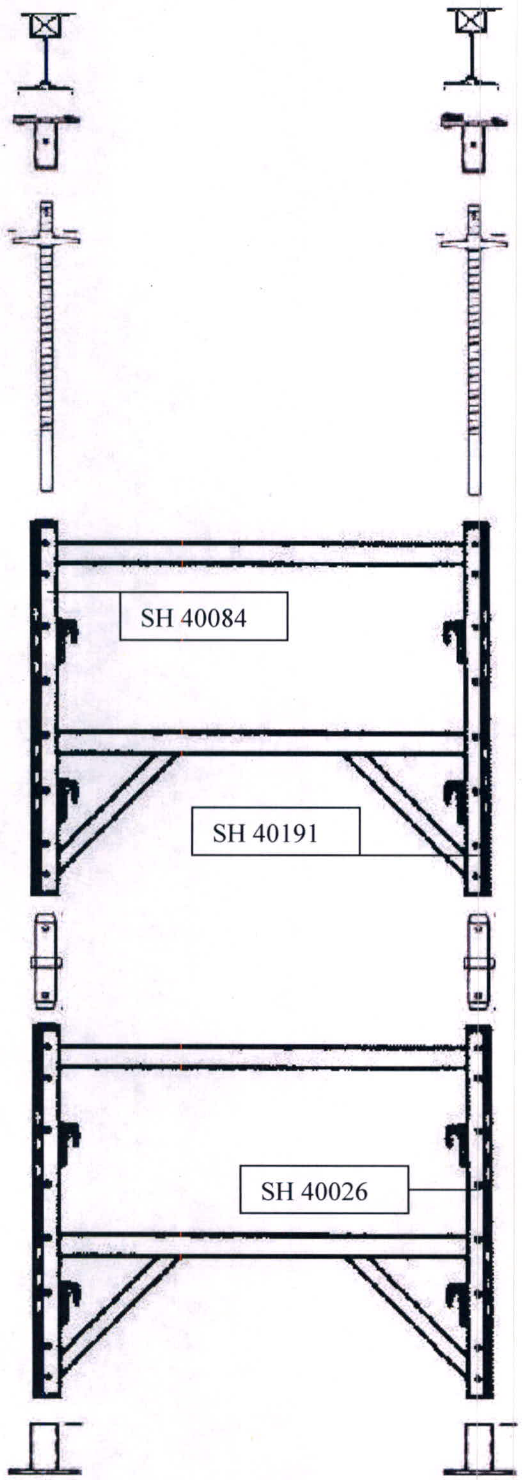
Folder: 08-02-06 to 08-02-08; SH

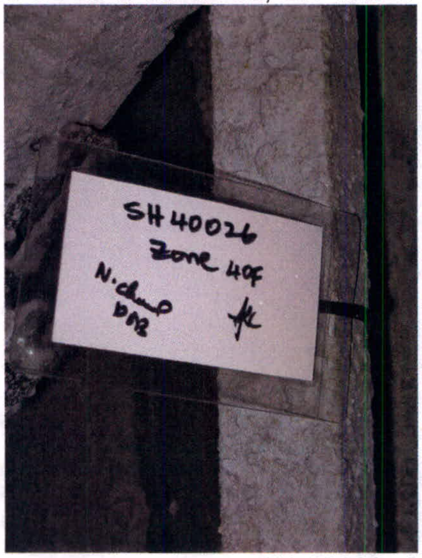
SH 40025_1

SH 40025_2

SH 40026 (41st floor bottom frame 3'-6")
Found on floor
Bott Frame
Zone 40F

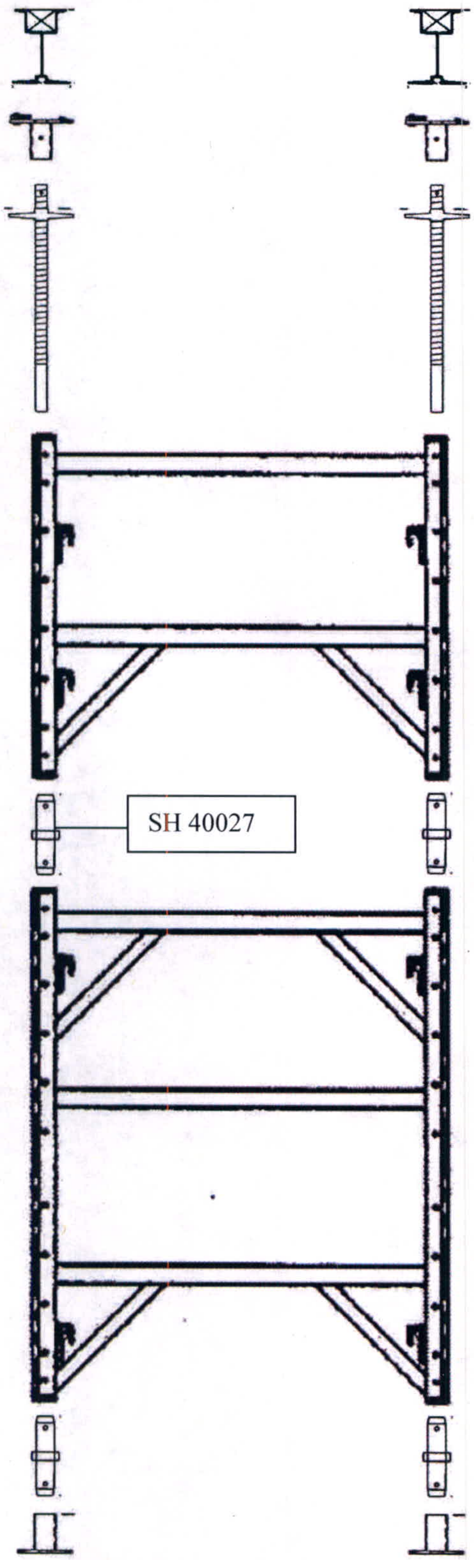
Folder: 08-02-06 to 08-02-08; SH 40026





SH 40026_1


SH 40026_2



SH 40027
Tower Coupler
Zone 40F

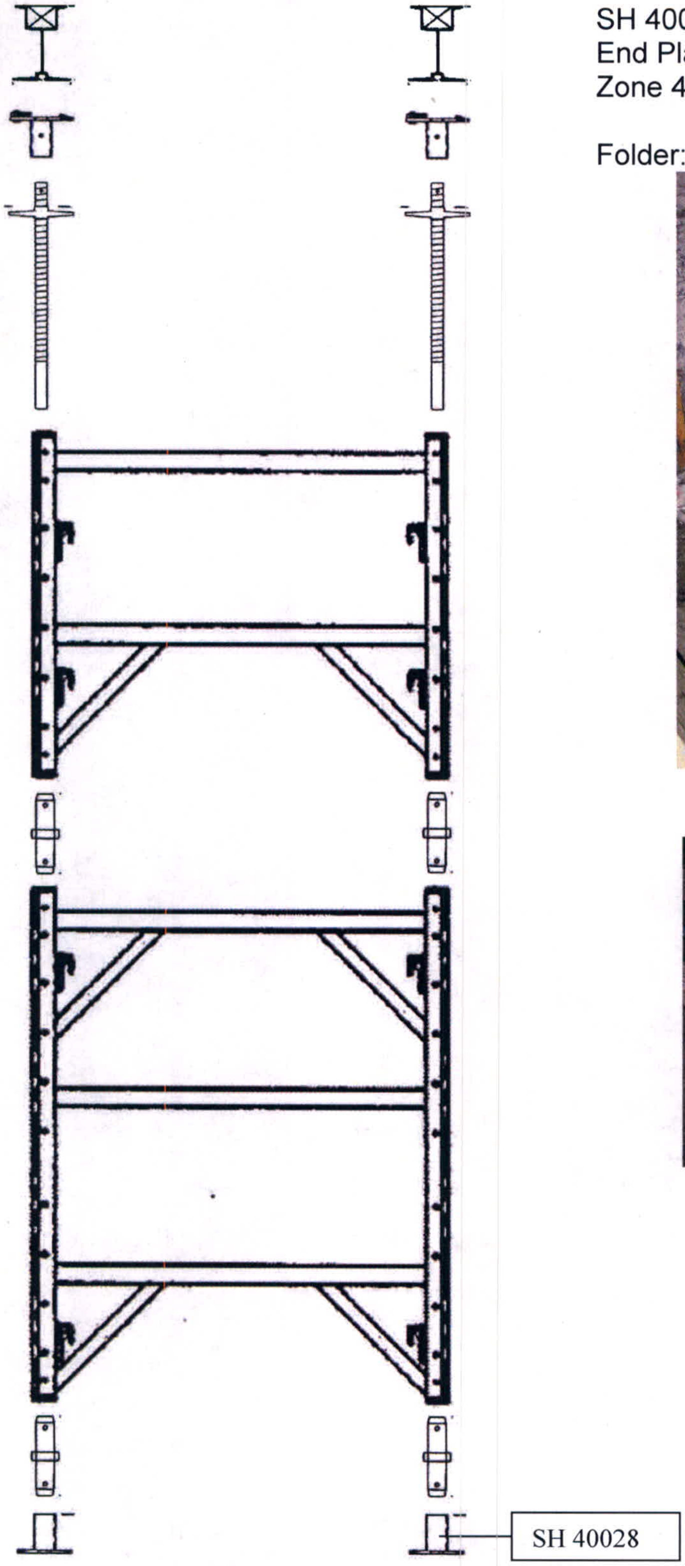
Folder: 08-02-06 to 08-02-08; SH 40027

SH 40027_1

SH 40027_2

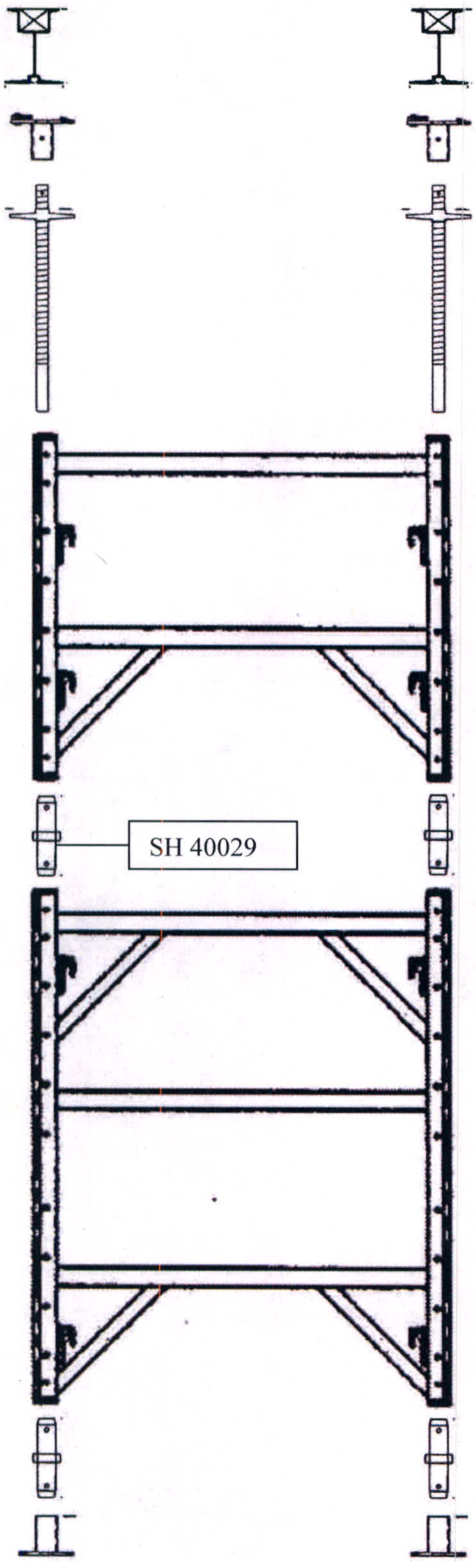
SH 40028
End Plate
Zone 40F

Folder: 08-02-06 to 08-02-08; SH 40028



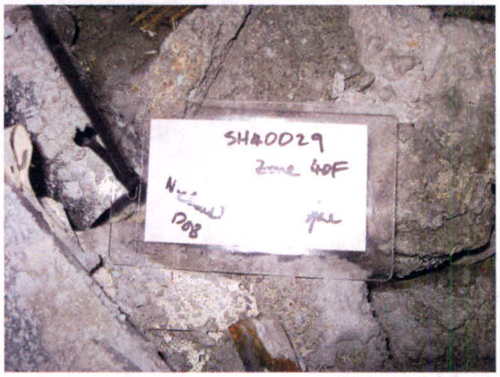
SH 40028_1

SH 40028_2



SH 40029
Coupler
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-06; SH 40029

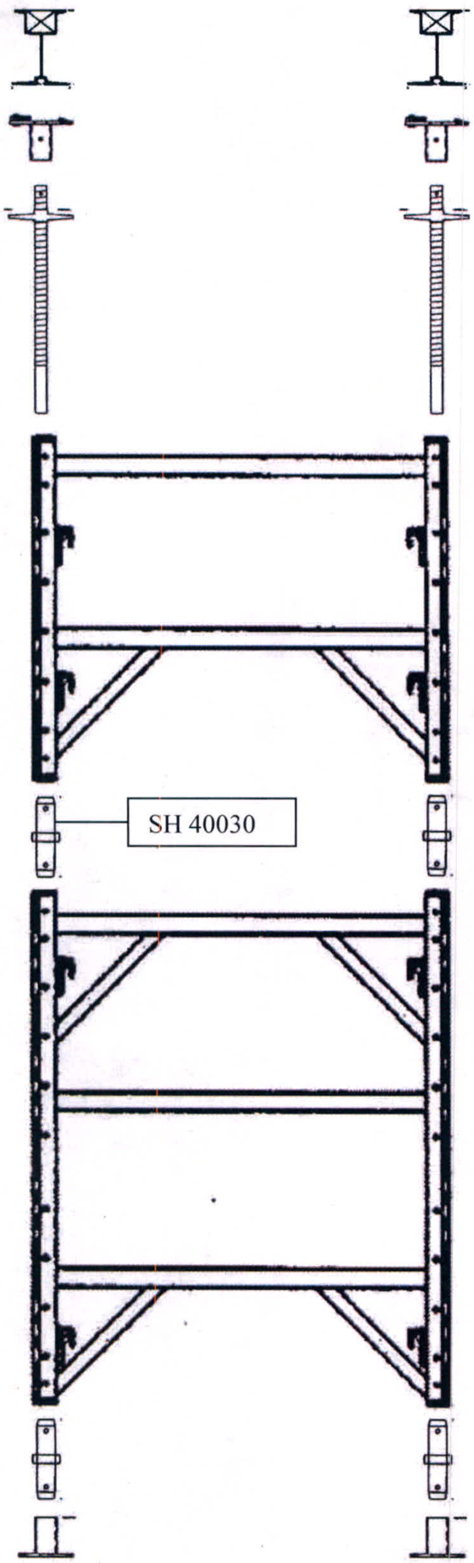


SH 40029_1

SH 40029_2

SH 40030
Coupler
40F

Folder: 08-02-06 to 08-02-08; 08-02-06;
SH 40030

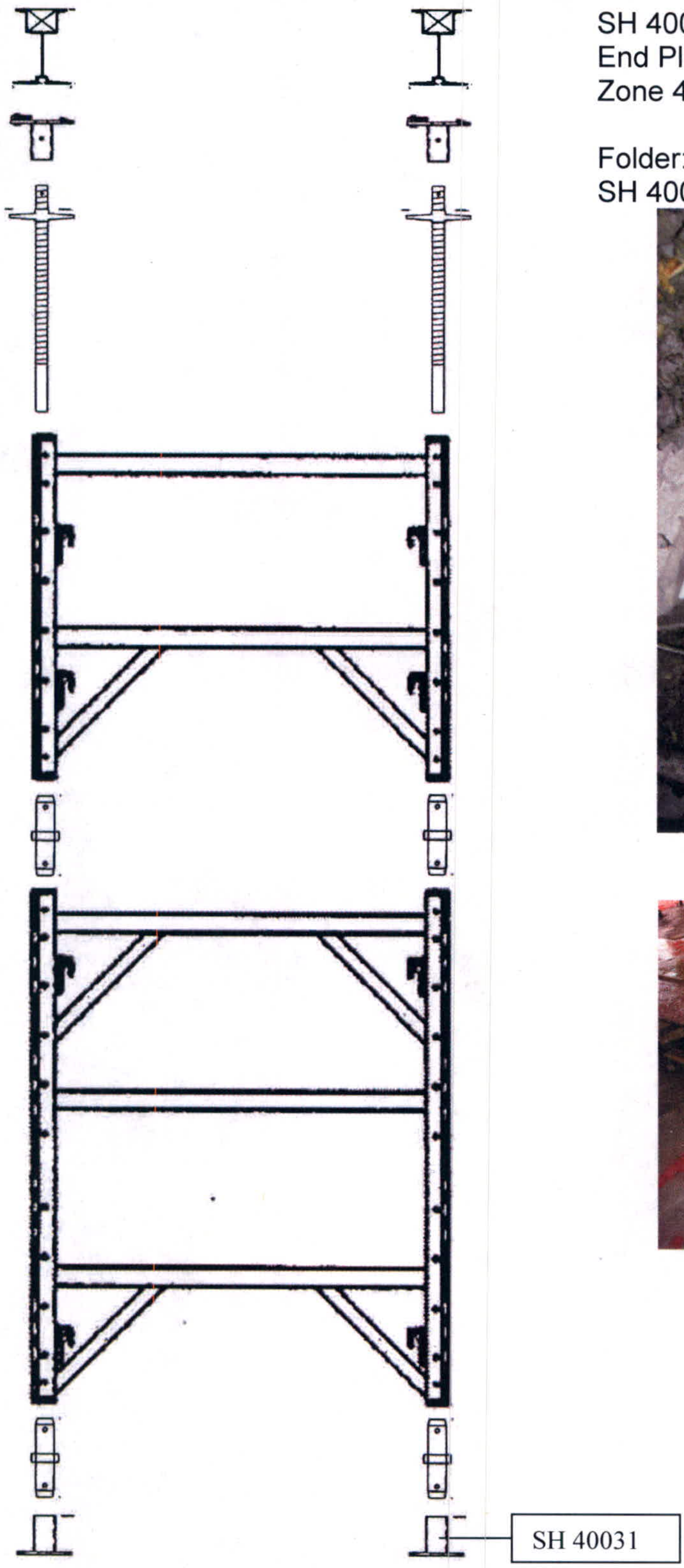


SH 40030_1

SH 40030_2

SH 40031
End Plate
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-06; SH 40031

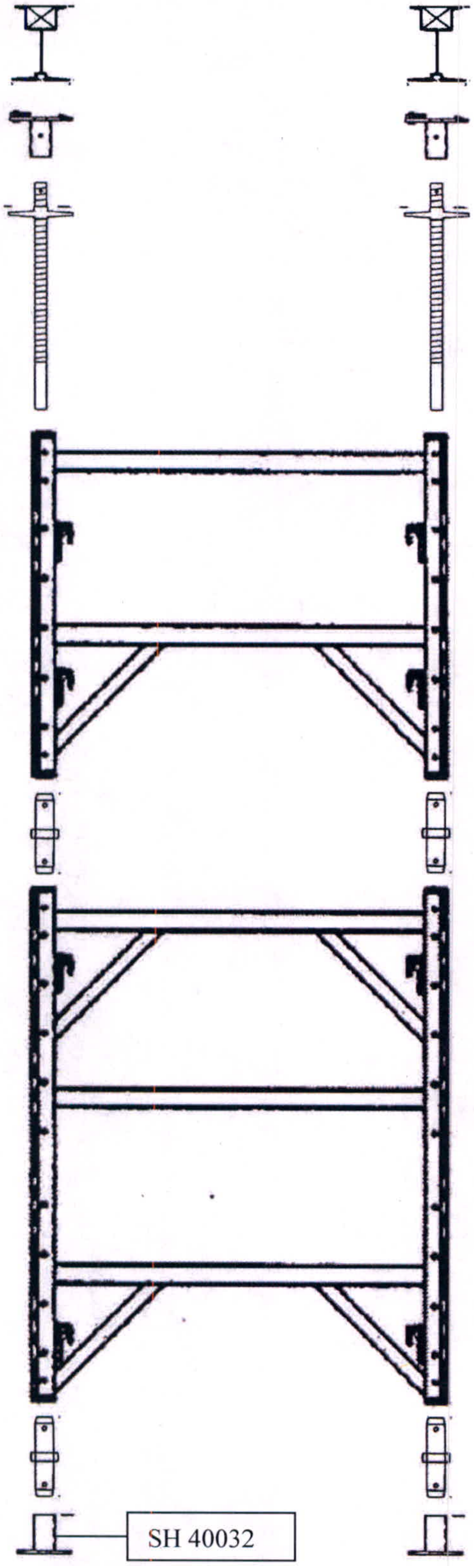


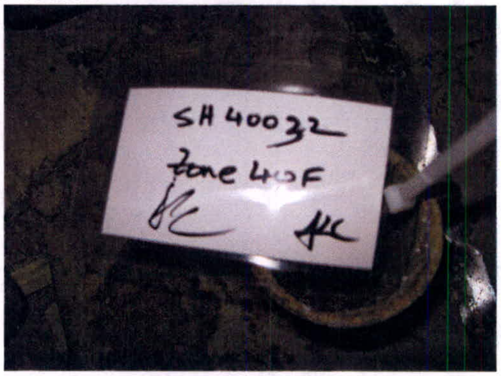
SH 40031_1

SH 40031_2

SH 40032
End Plate
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-07; SH 40032

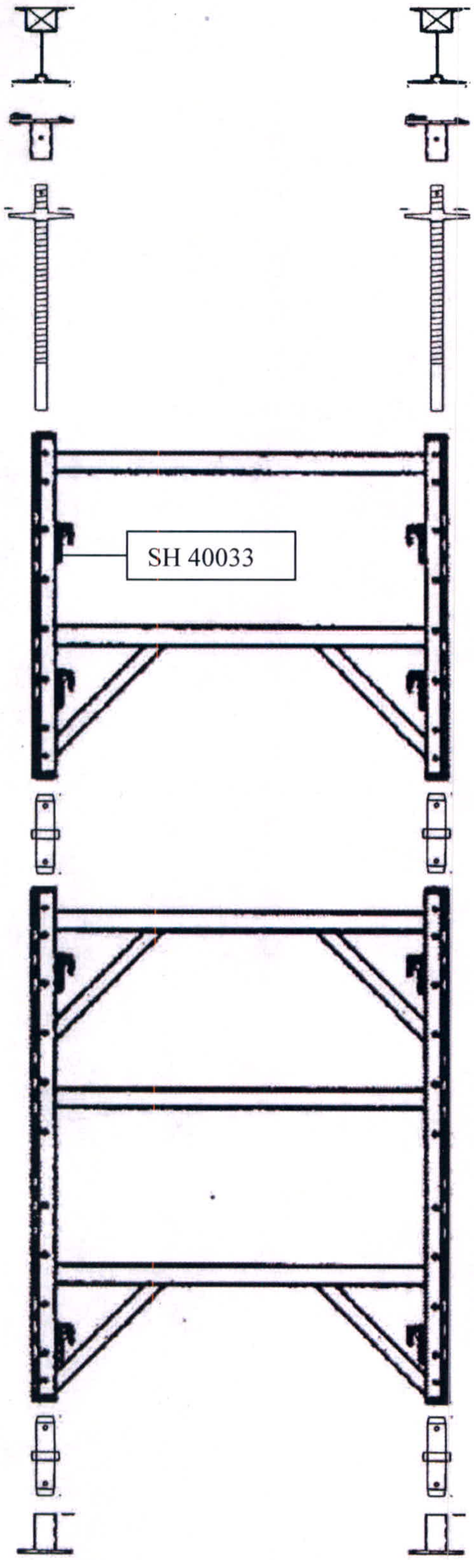




SH 40032 1

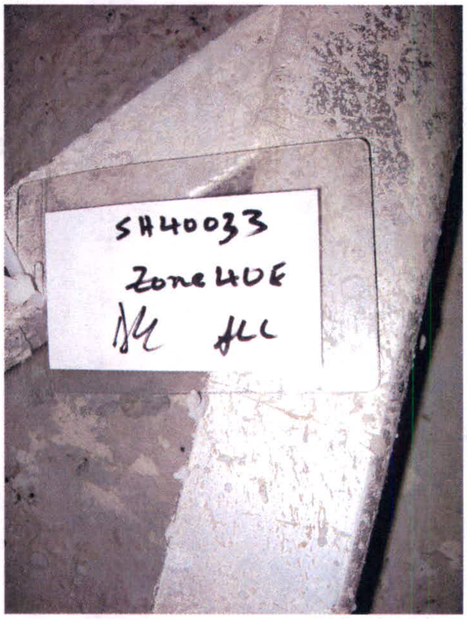
SH 40032 2

SH 40032 3



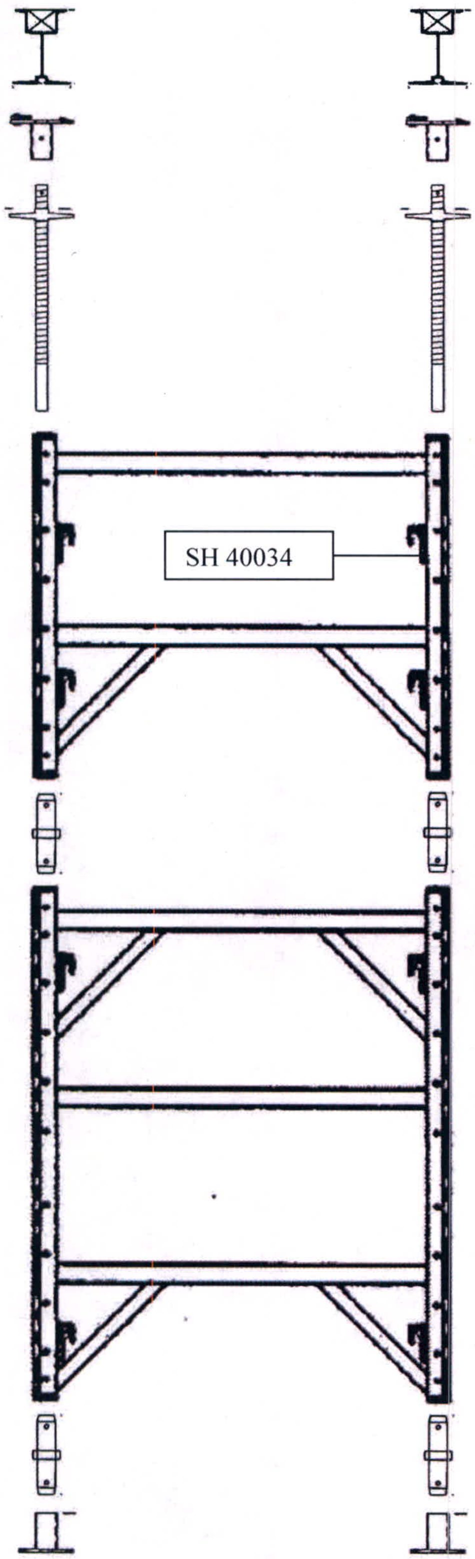
SH 40033
Zone 40E
Top Frame neat at Col. 101
Moved from Original Position-Unknown

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40033



SH 40033 1

SH 40033 2

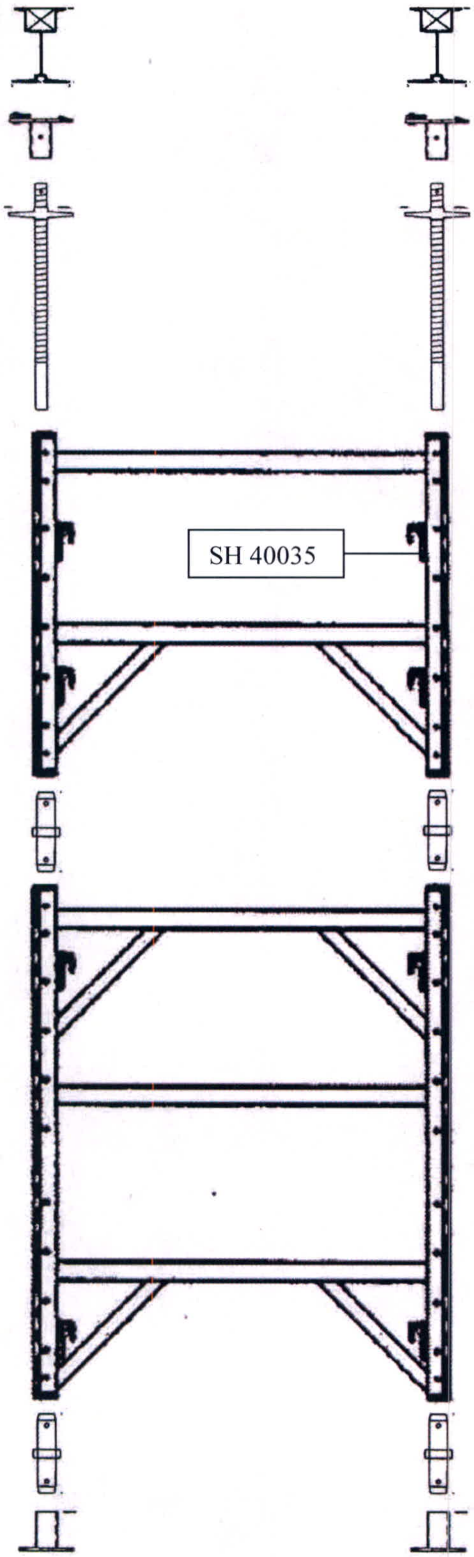


SH 40034
Zone 40E
At location of Column 101
Top Frame
Move from Original Location-Unknown

Folder: 08-02-06 to 08-02-08; 08-02-07; SH 40034

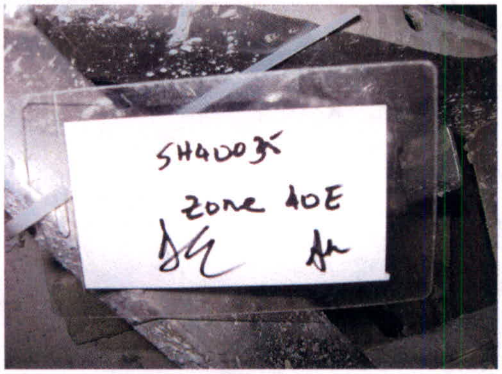
SH 40034 1

SH 40034 2

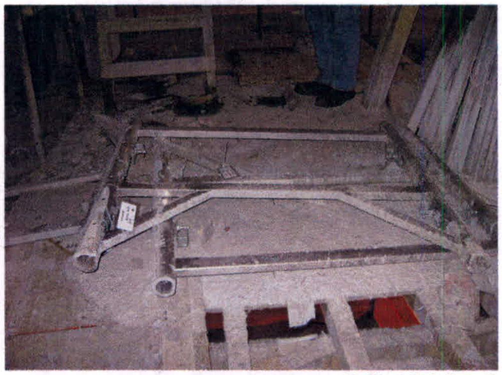


SH 40035
Top Frame near Col. 101
Zone 40E
Move from original location-Unknown

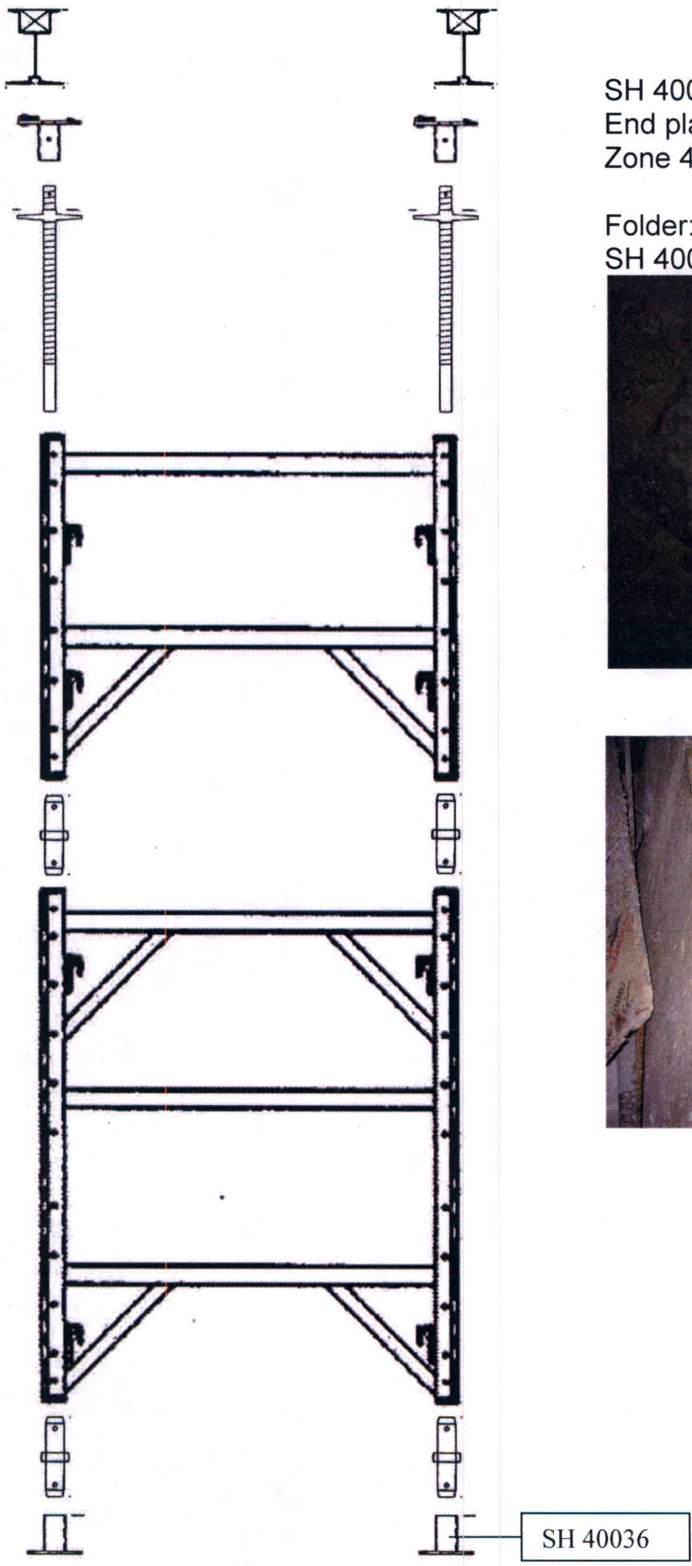
Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40035



SH 40035 1

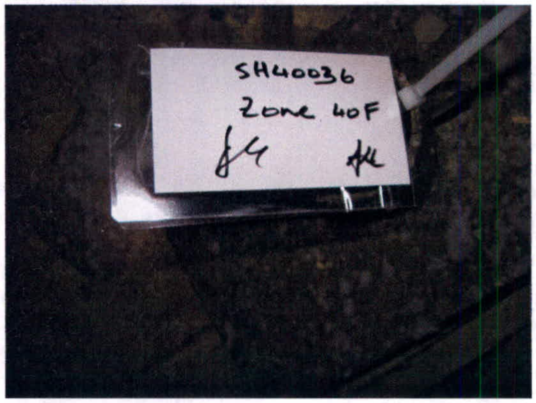


SH 40035 2



SH 40036
End plate
Zone 40F

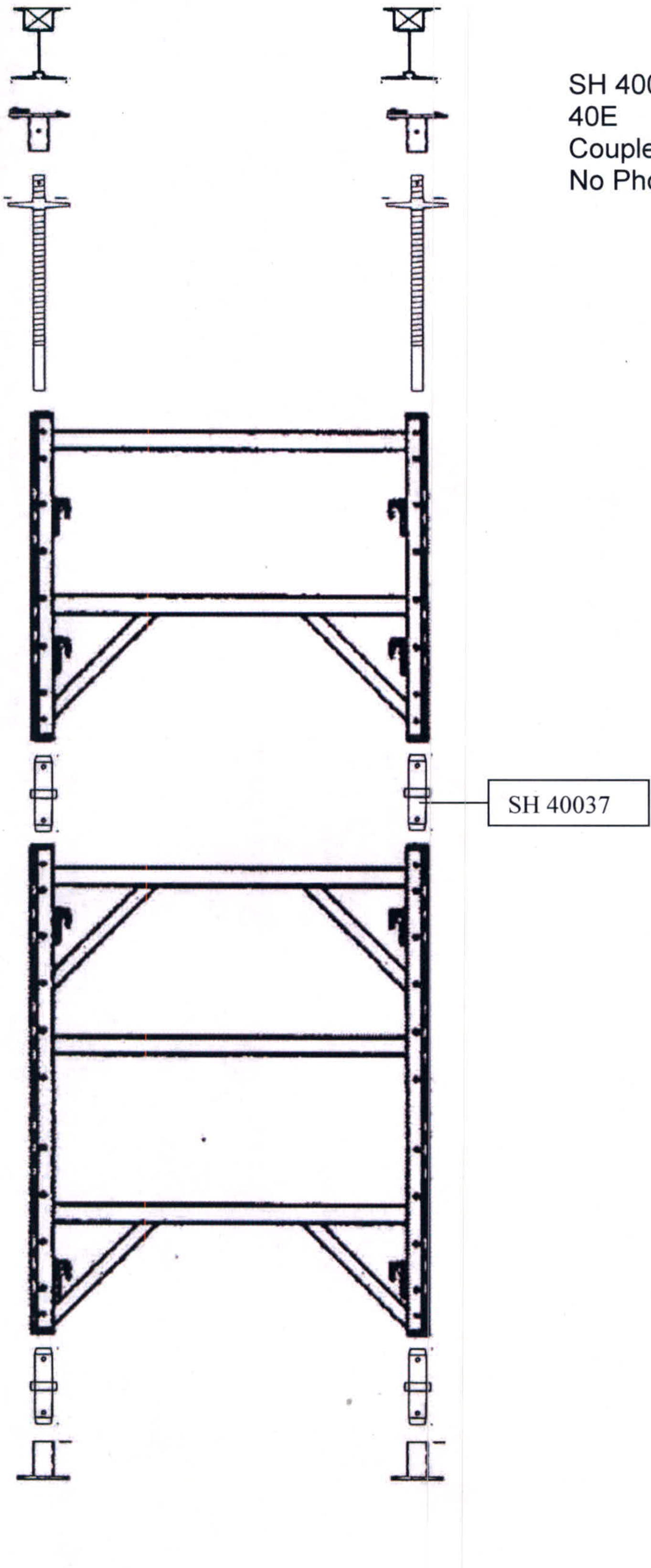
Folder: 08-02-06 to 08-02-08; 08-02-07; SH 40036

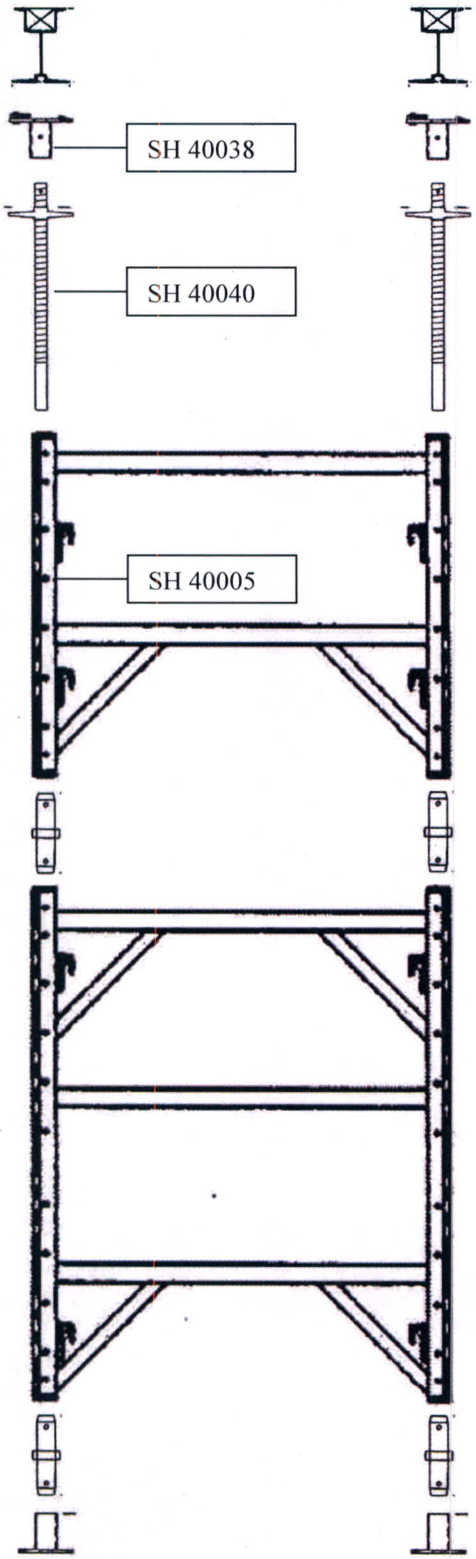


SH 40036 1

SH 40036 2

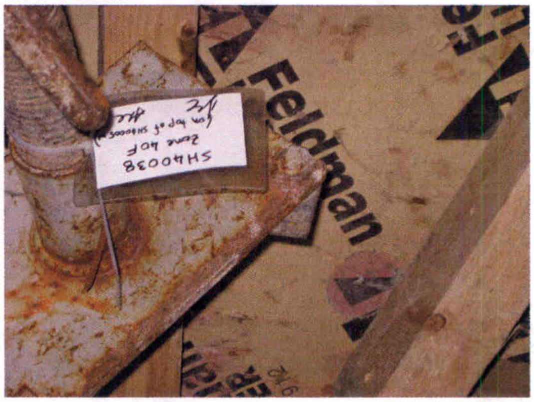
SH 40037
40E
Coupler
No Photo





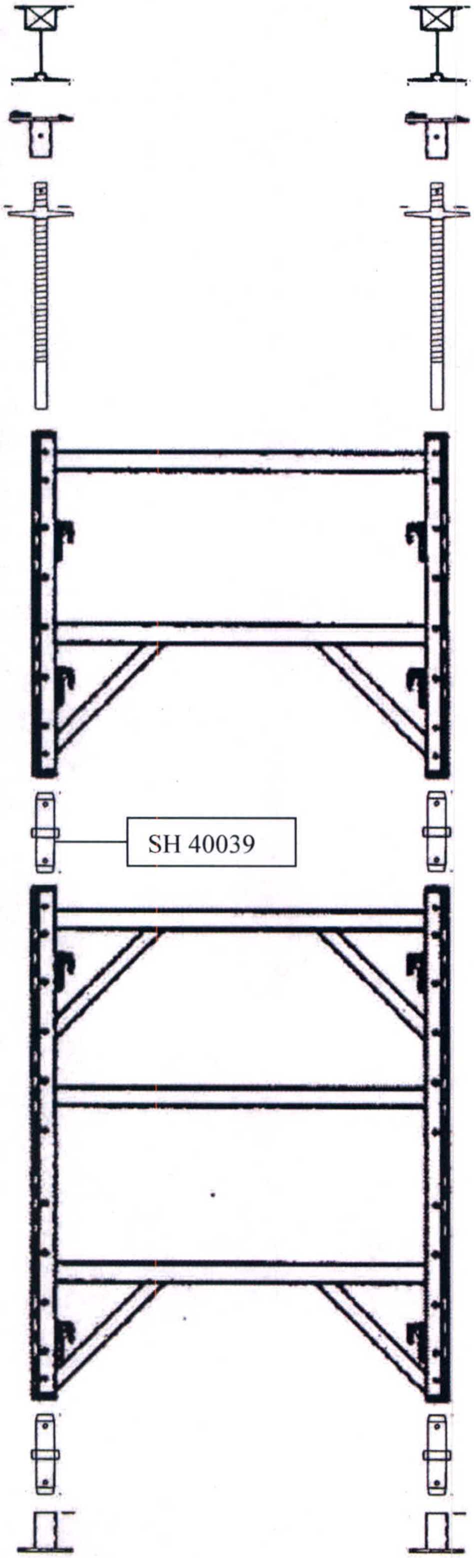
SH 40038
Top Plate
Zone 40F
Attached to SH 40005

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40038



SH 40038 1

SH 40038 2

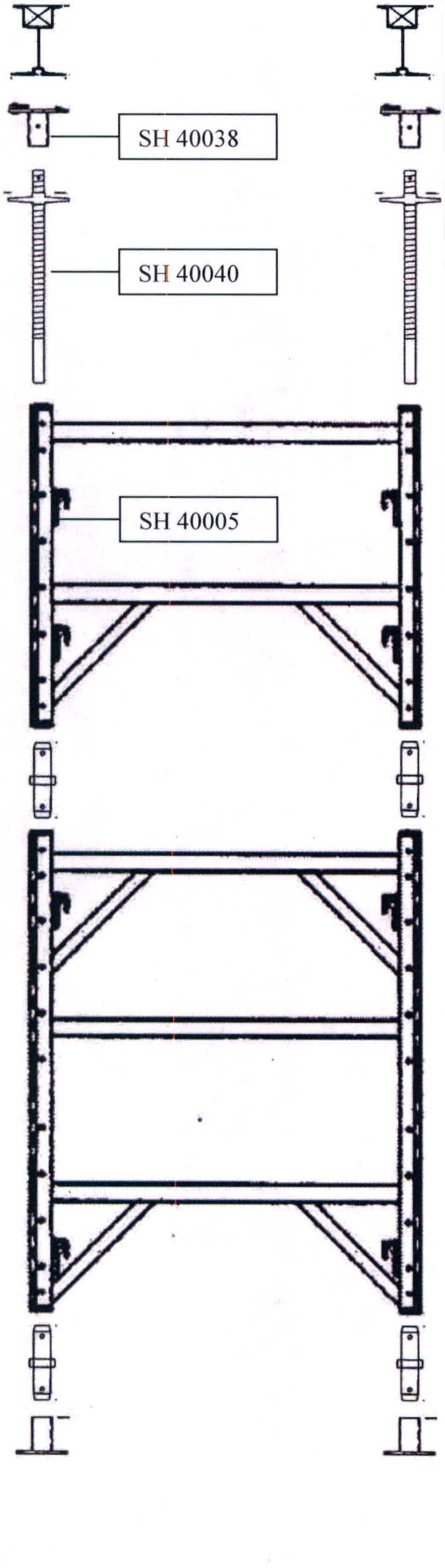


SH 40039
Coupler
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40039

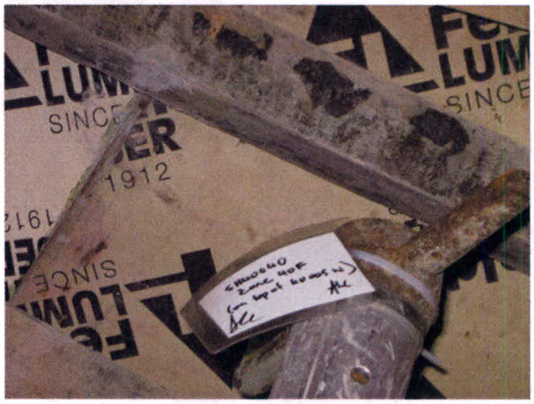
SH 40039 1

SH 40039 3



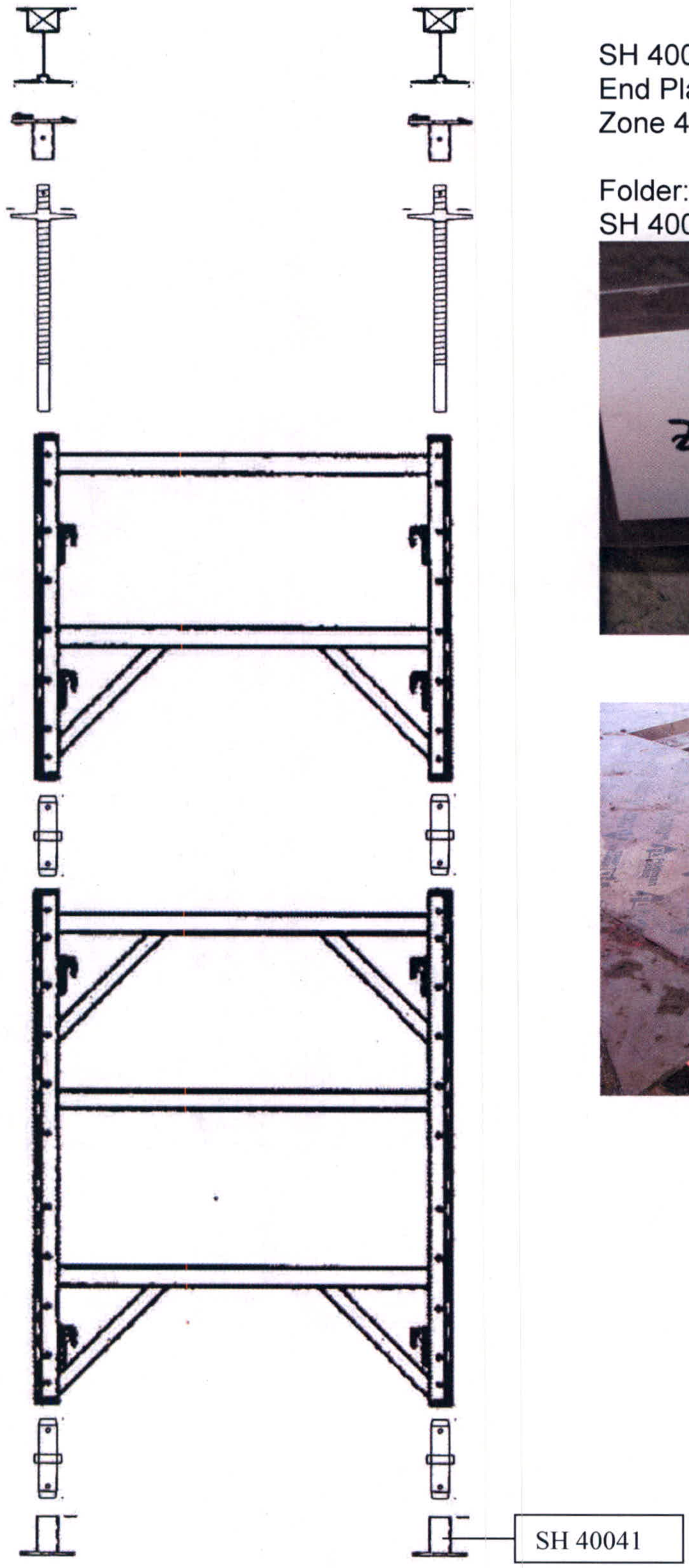
SH 40040
Screw Jack
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40040



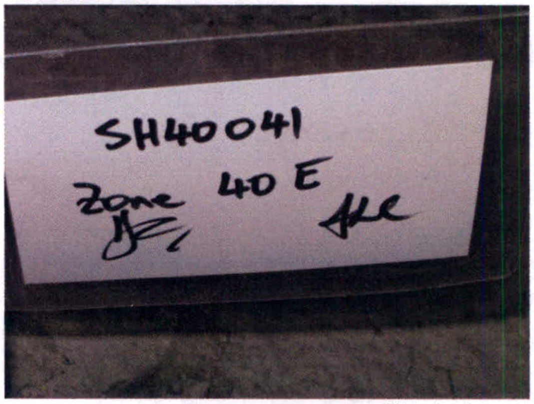
SH 40040 1

SH 40040 2



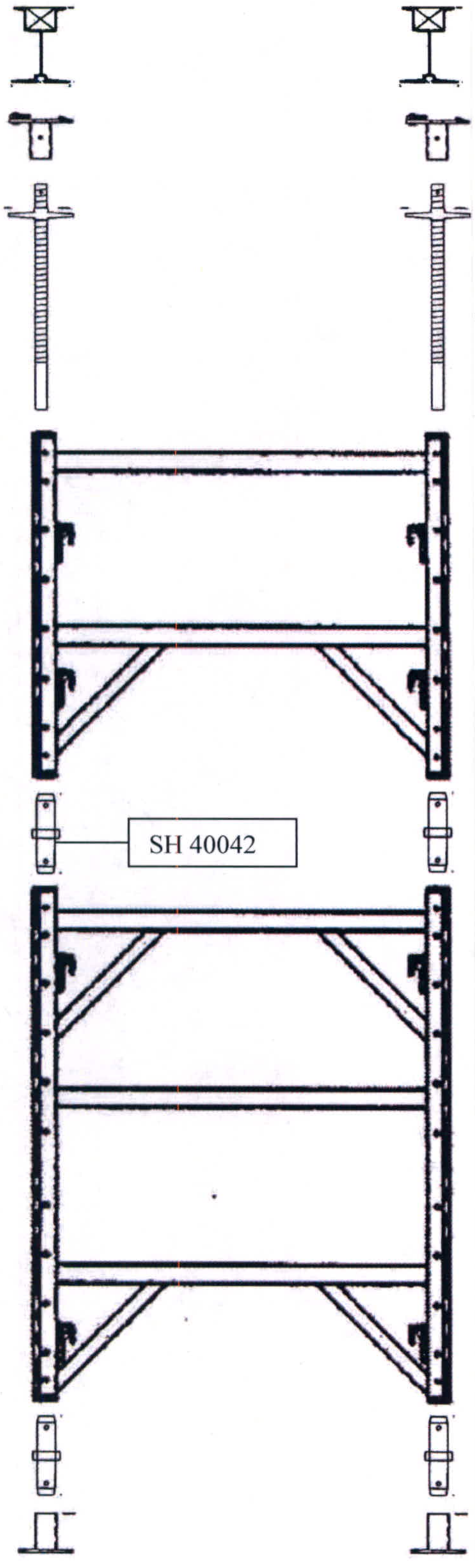
SH 40041
End Plate
Zone 40F

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40041



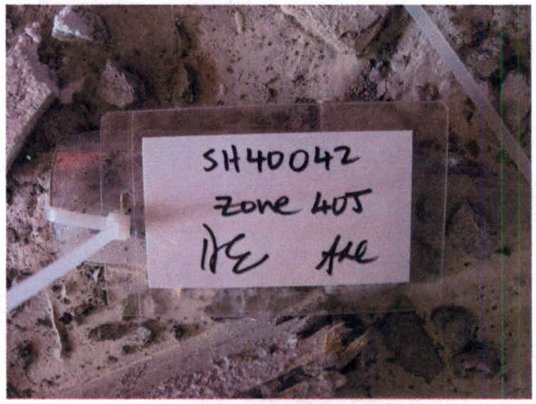
SH 40041 1

SH 40041 2



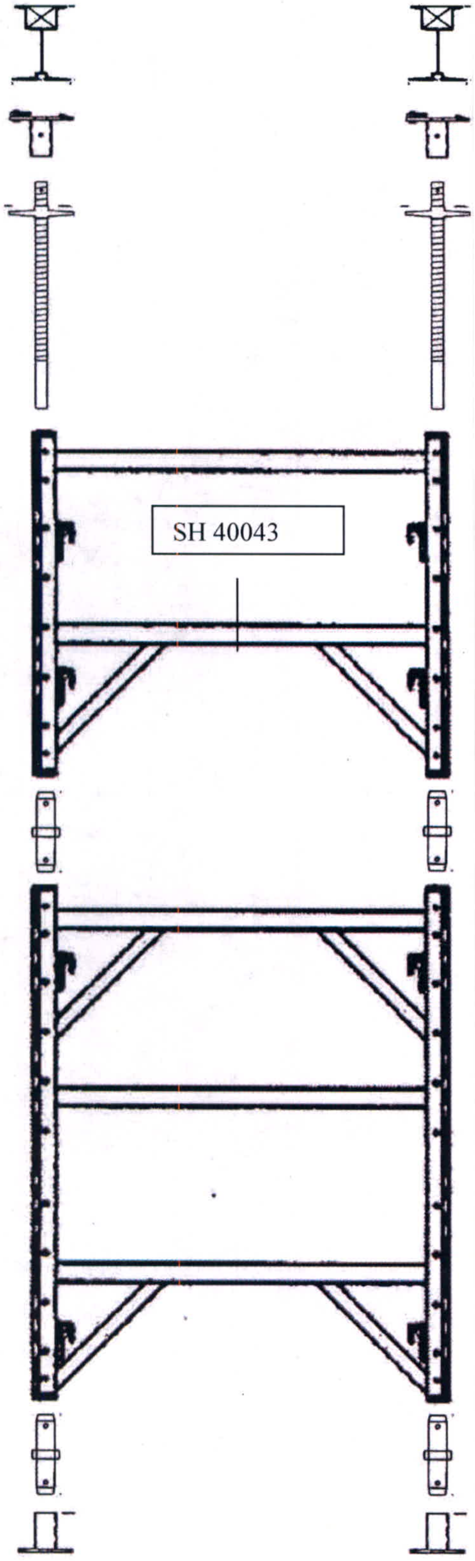
SH 40042
Coupler
Zone 40E

Folder: 08-02-06 to 08-02-08; 08-02-07;
SH 40042



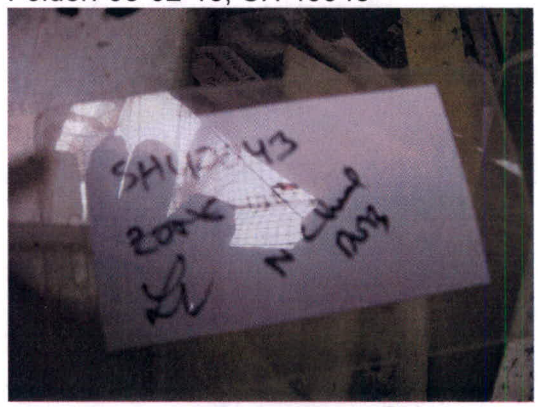
SH 40042 1

SH 40042 2



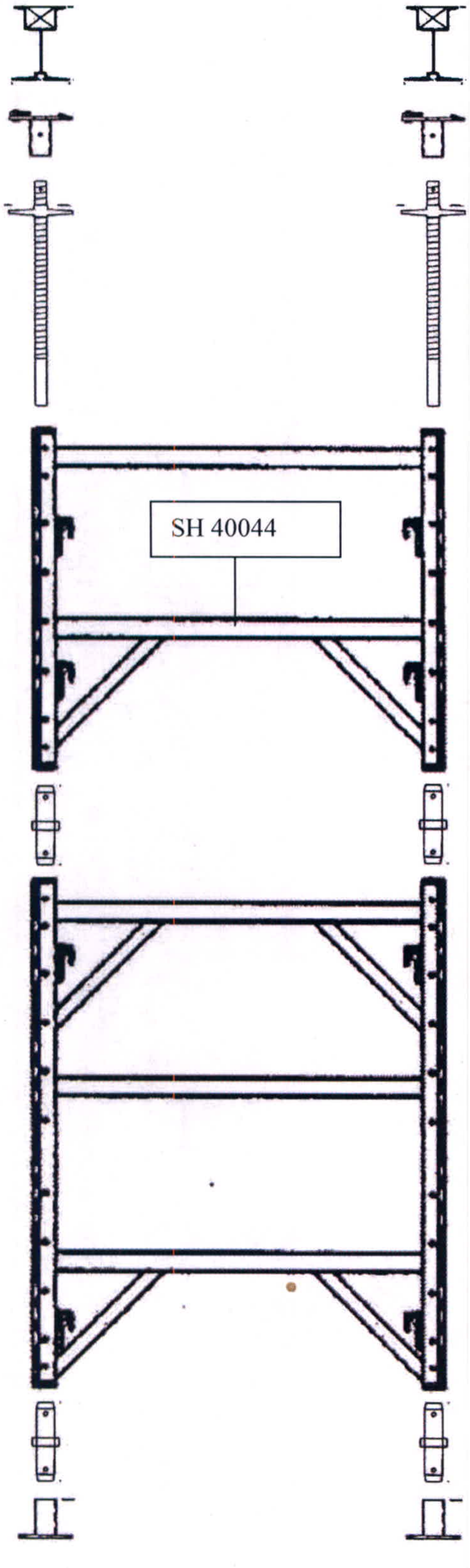
SH 40043
Piece of Shore
Zone 40B

Folder: 08-02-15; SH 40043



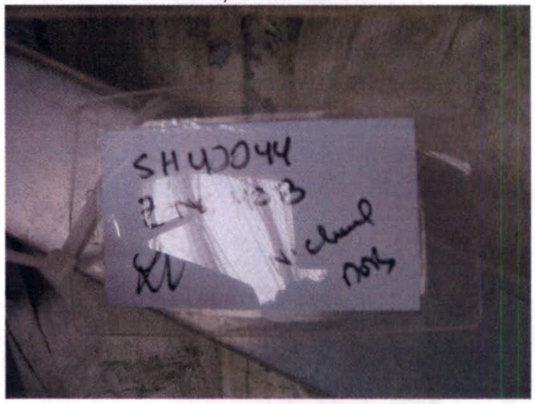
SH40043_1

SH40043_2



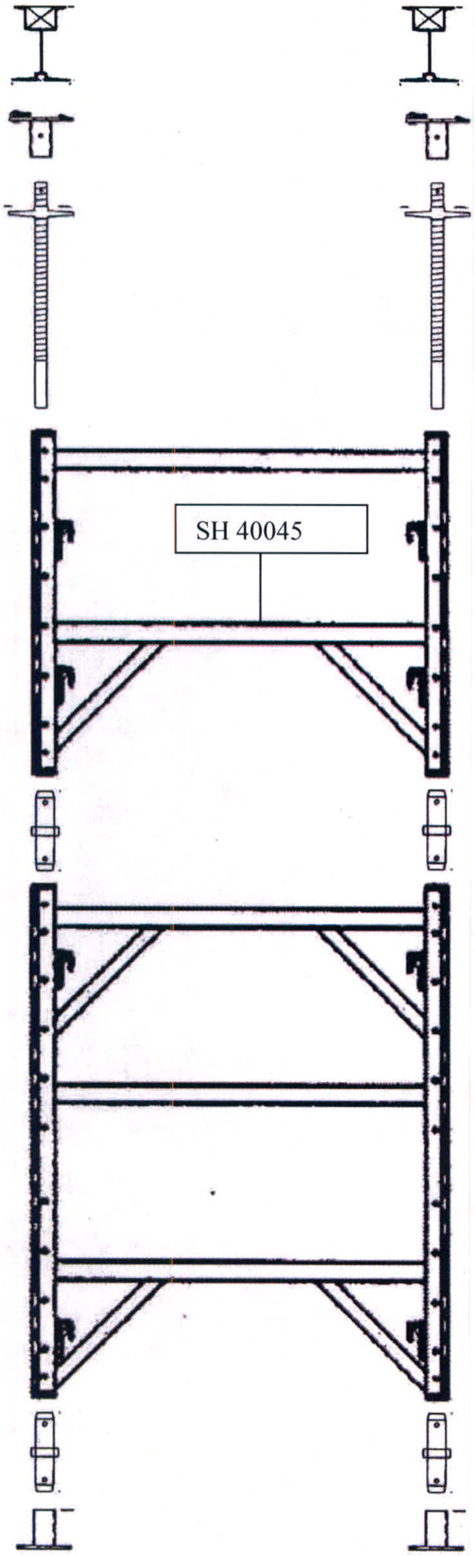
SH 40044
Piece of Shore
Zone 40B

Folder: 08-02-15; SH 40044



SH40044_1

SH40044_2

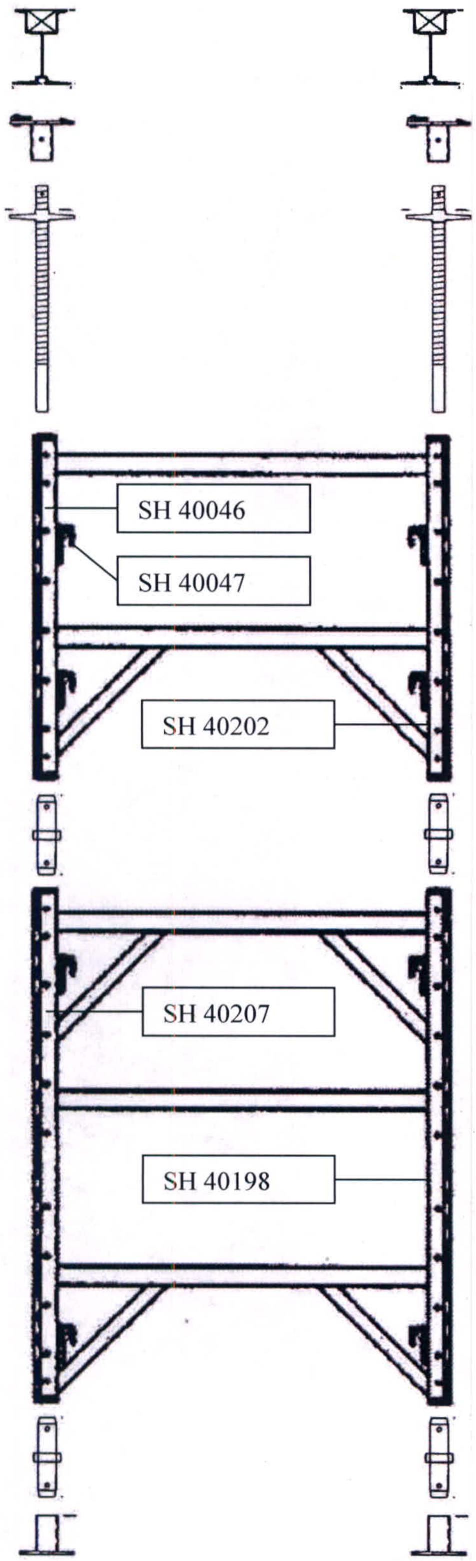


SH 40045
Piece of Shore
Zone 40B

Folder: 08-02-15; SH 40045

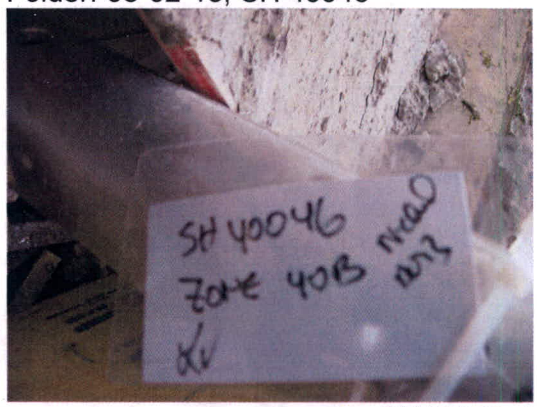
SH40045_1

SH40045_2



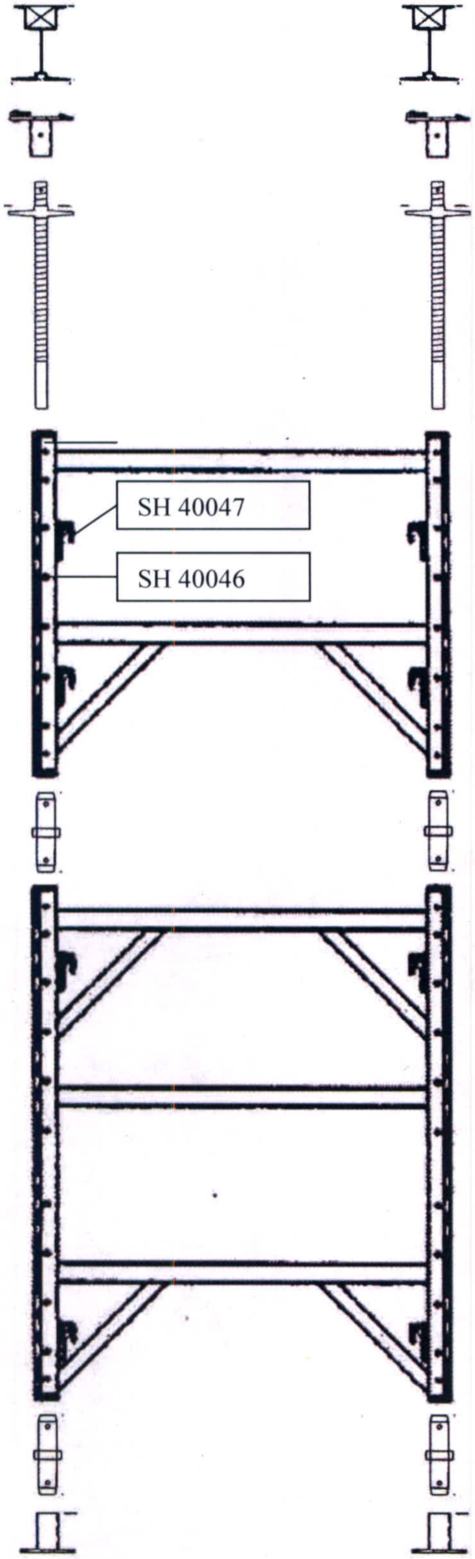
SH 40046
Top Shore
Attached Screw Jack and Top Plate
Attached to X Brace SH 40047
Zone 40B

Folder: 08-02-15; SH 40046



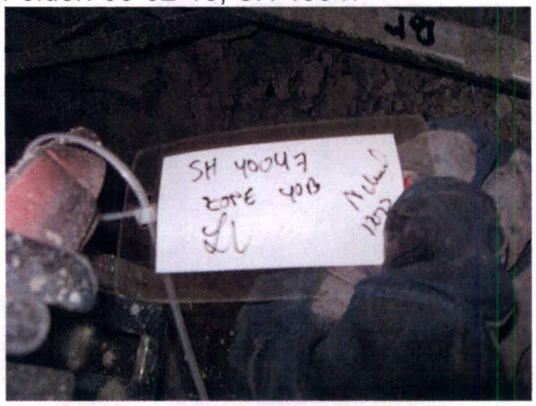
SH40046_1


SH40046_2



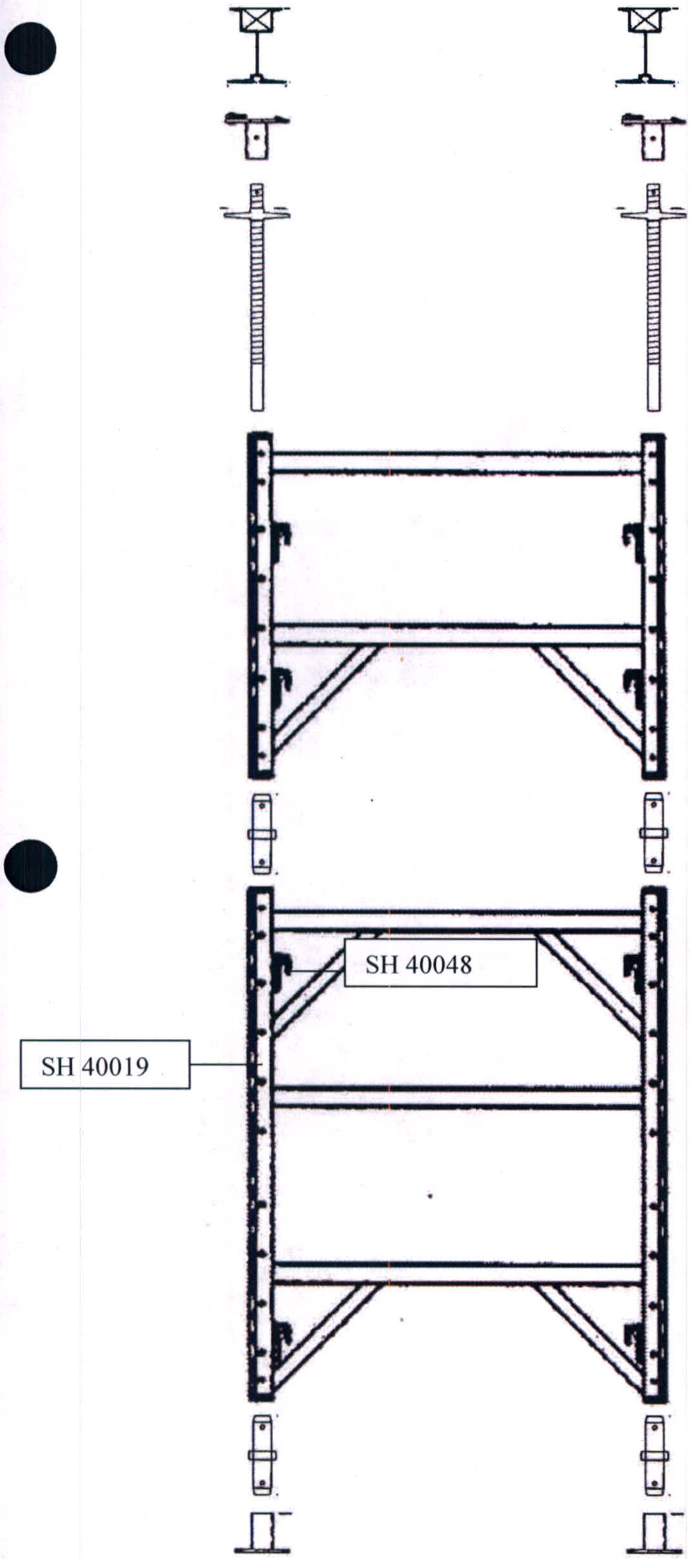
SH 40047
X Brace
Attached to SH 40046

Folder: 08-02-15; SH 40047



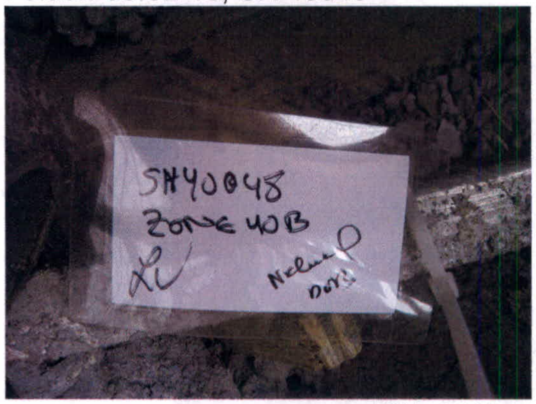
SH40047_1

SH40047_2

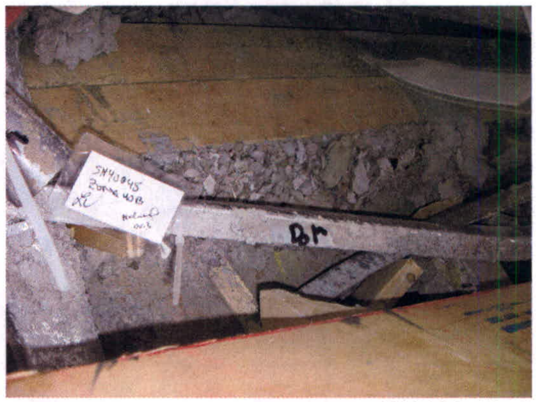


SH 40048
X Brace
Attached to bottom frame SH 40019

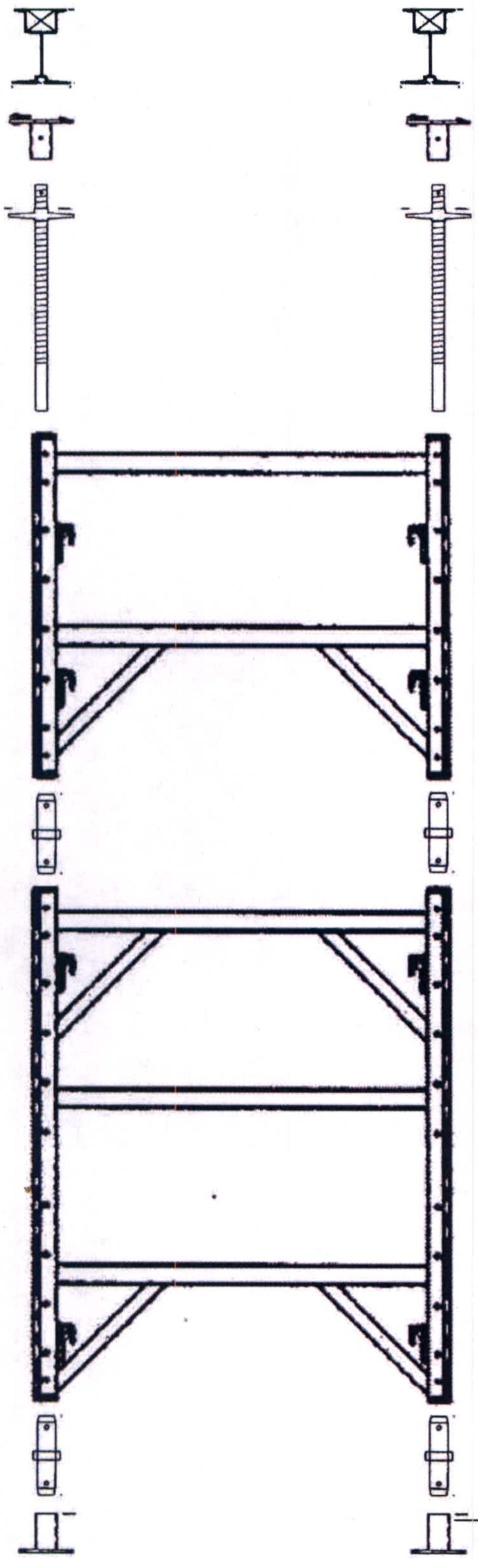
Folder: 08-02-15; SH 40048



SH40048_1



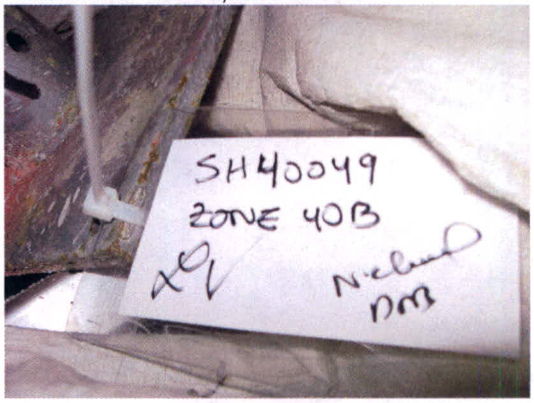
SH40048_2



SH 40049

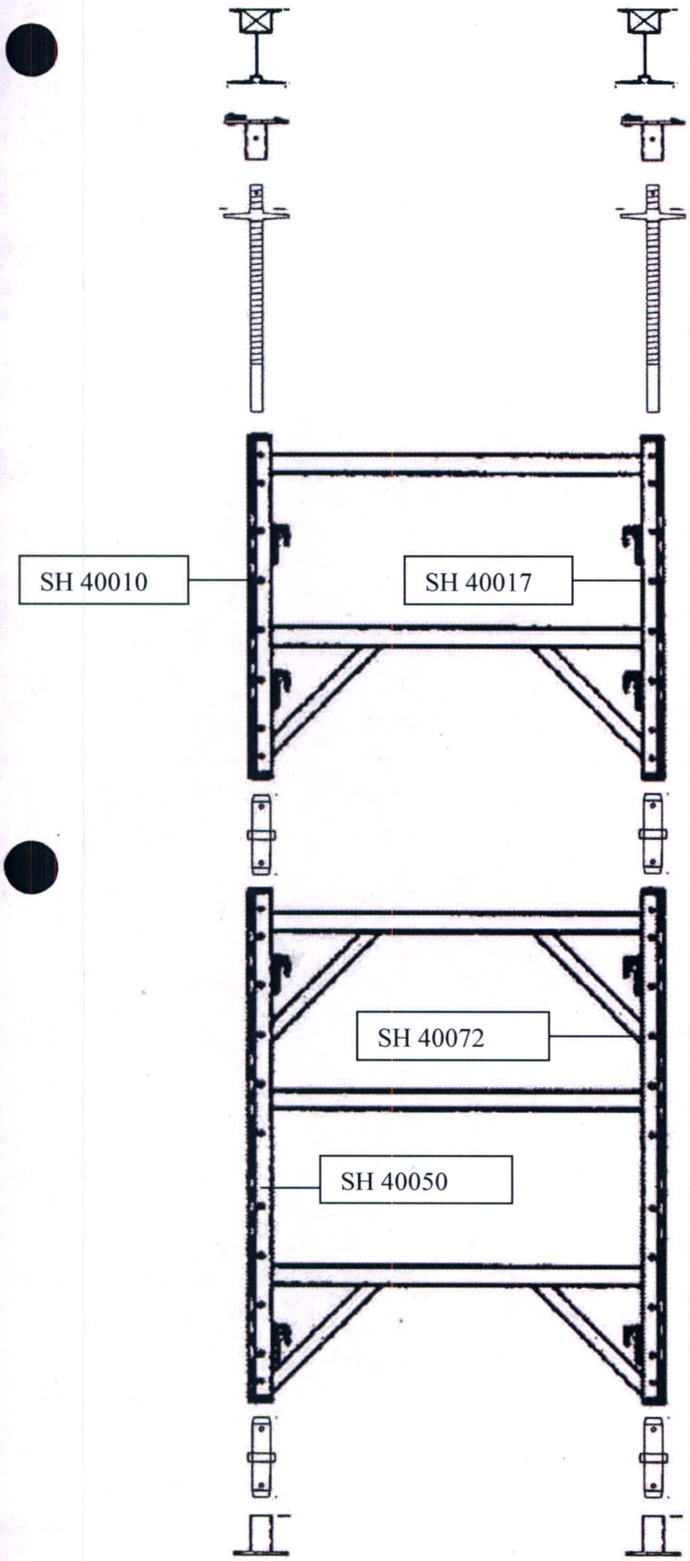
SH 40049
Bottom Plate
Zone 40B

Folder: 08-02-15; SH 40049



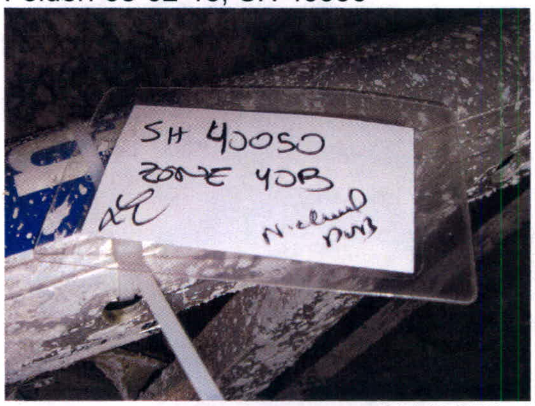
SH40049_1

SH40049_2



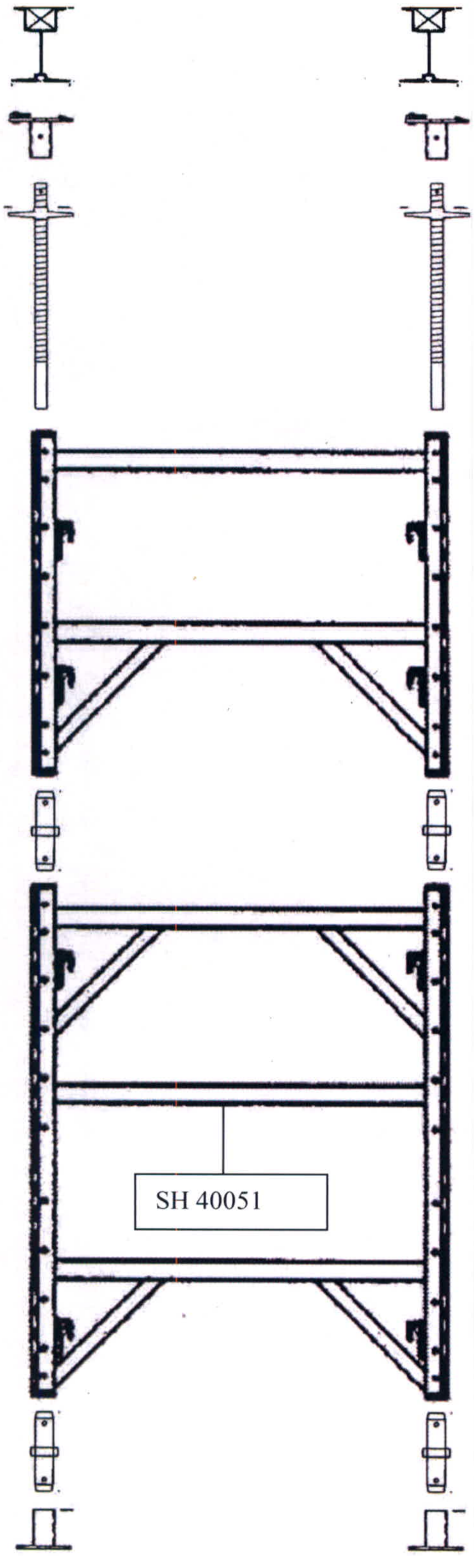
SH 40050
Bottom shore broken connect SH 40072 to make full frame
Shore Connect to SH 40010 with brace to make tower
Zone 40B

Folder: 08-02-15; SH 40050



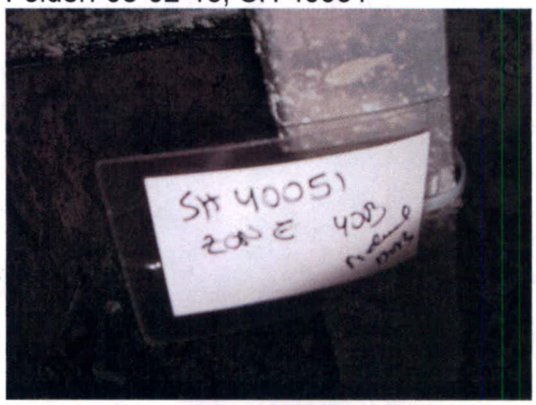
SH40050_1

SH40050_2



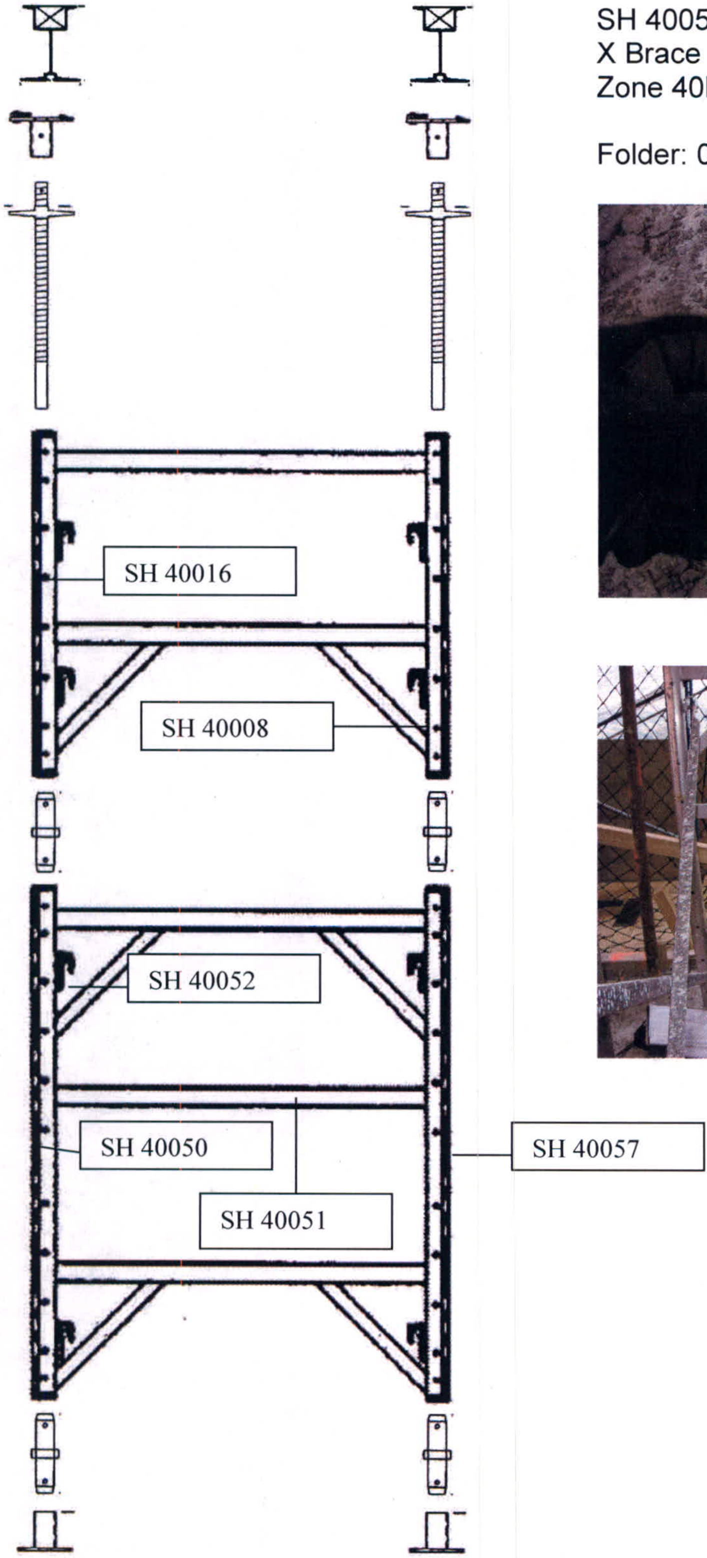
SH 40051
Piece of frame that is part of SH 40050
Zone 40B

Folder: 08-02-15; SH 40051



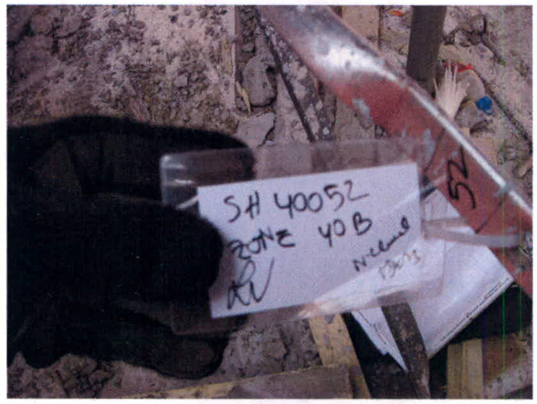
SH40051_1

SH40051_2

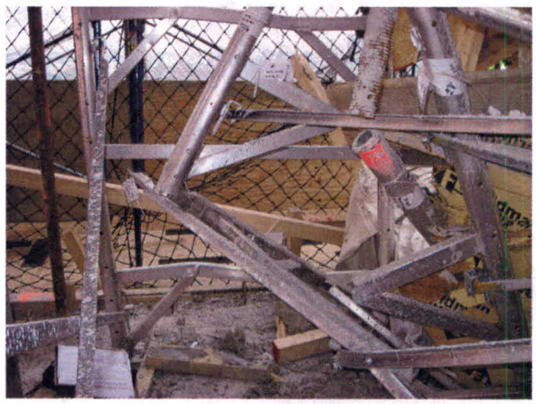


SH 40052
X Brace
Zone 40B

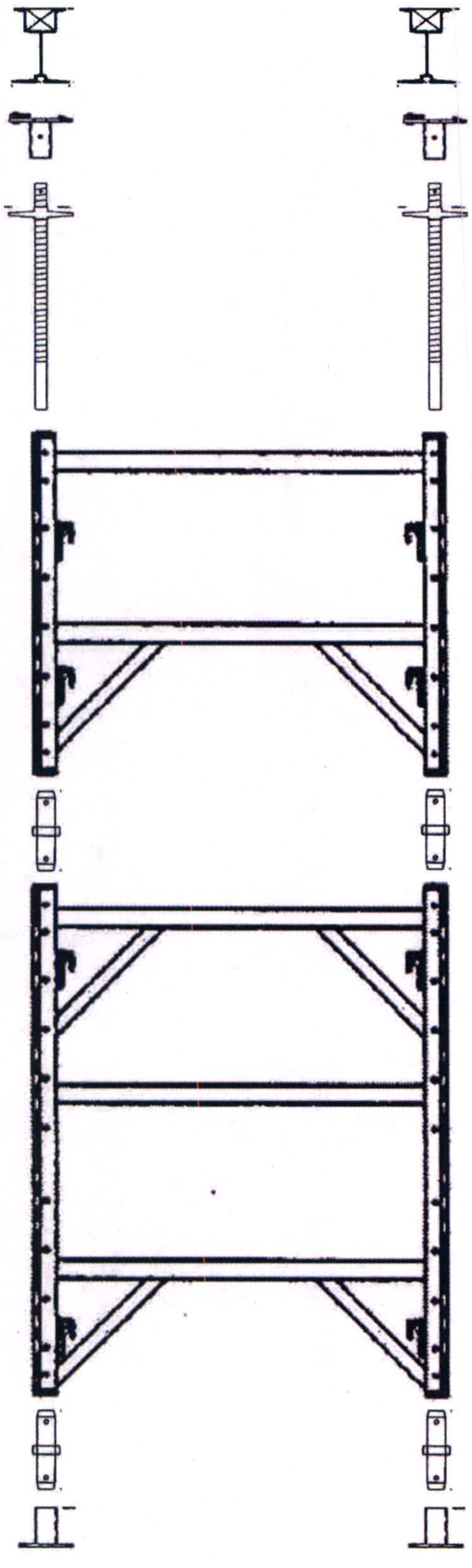
Folder: 08-02-15; SH 40052



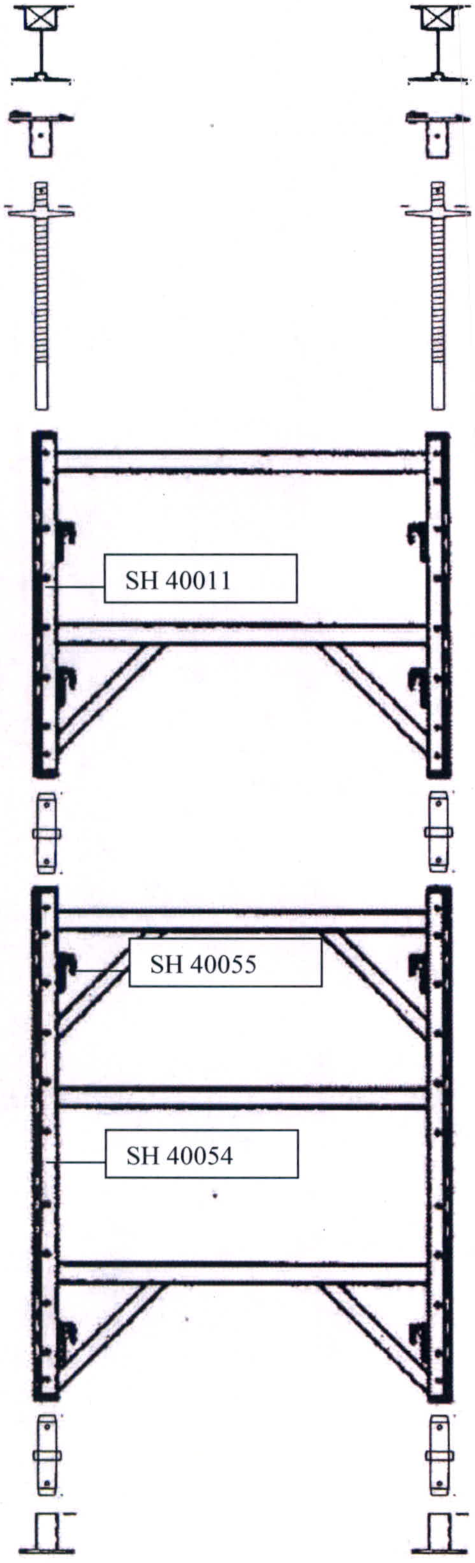
SH40052_1



SH40052_2

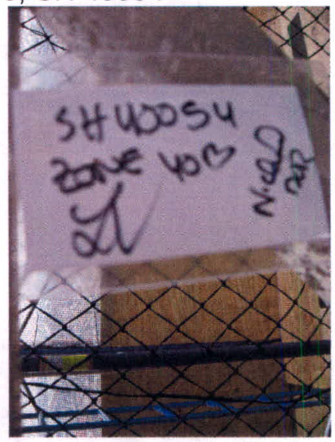


SH 40053
X Brace
No Photo
Zone 40B



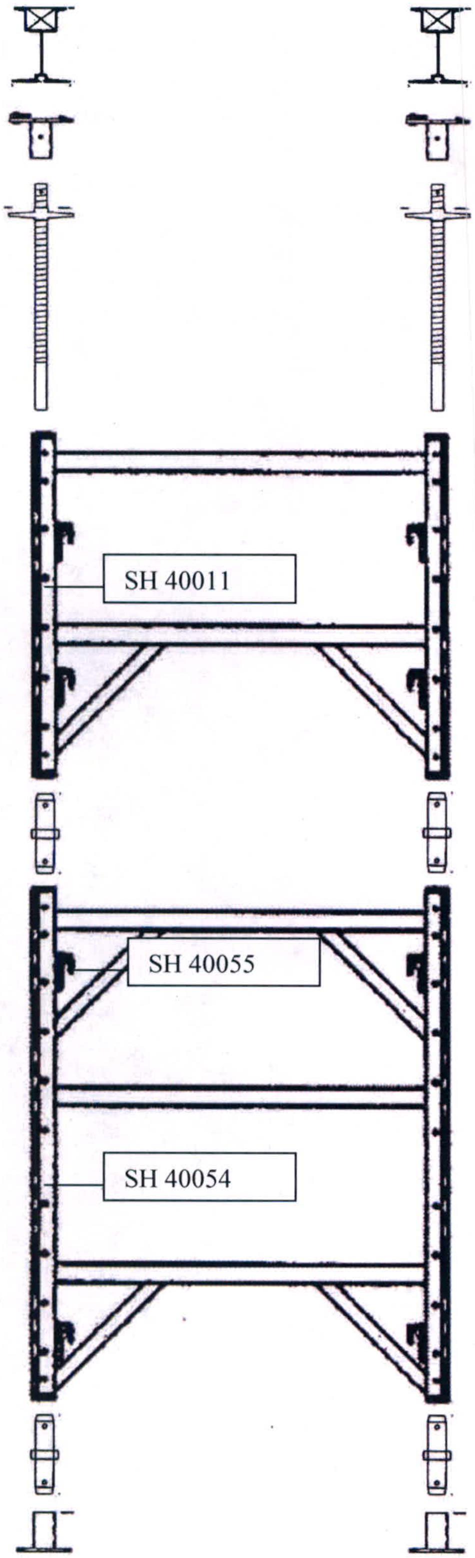
SH 40054
Bottom Frame of SH 40011

Folder: 08-02-15; SH 40054


SH40054_1

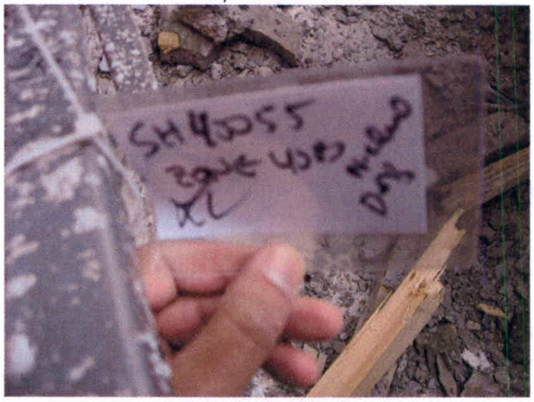

SH40054_2


SH40054_4

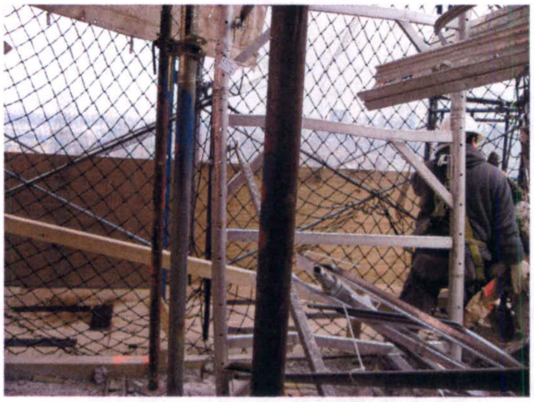


SH 40055
X Brace Connect with SH 40054 for Bottom Shore and Top Shore SH 40011
Zone 40B

Folder: 08-02-15; SH 40055

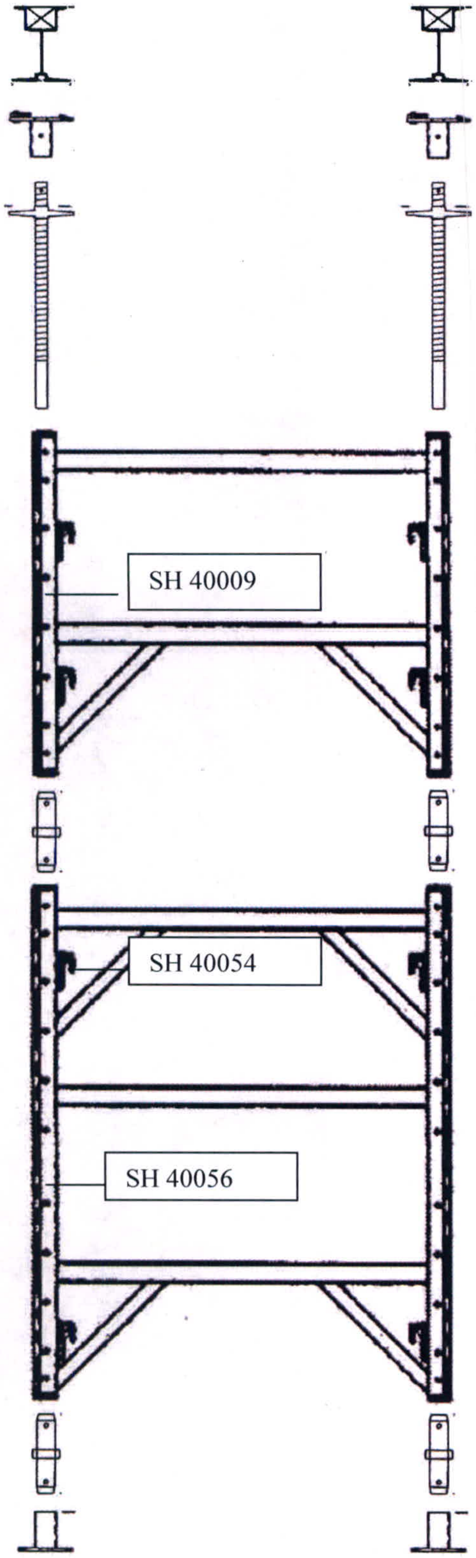


SH40055_1

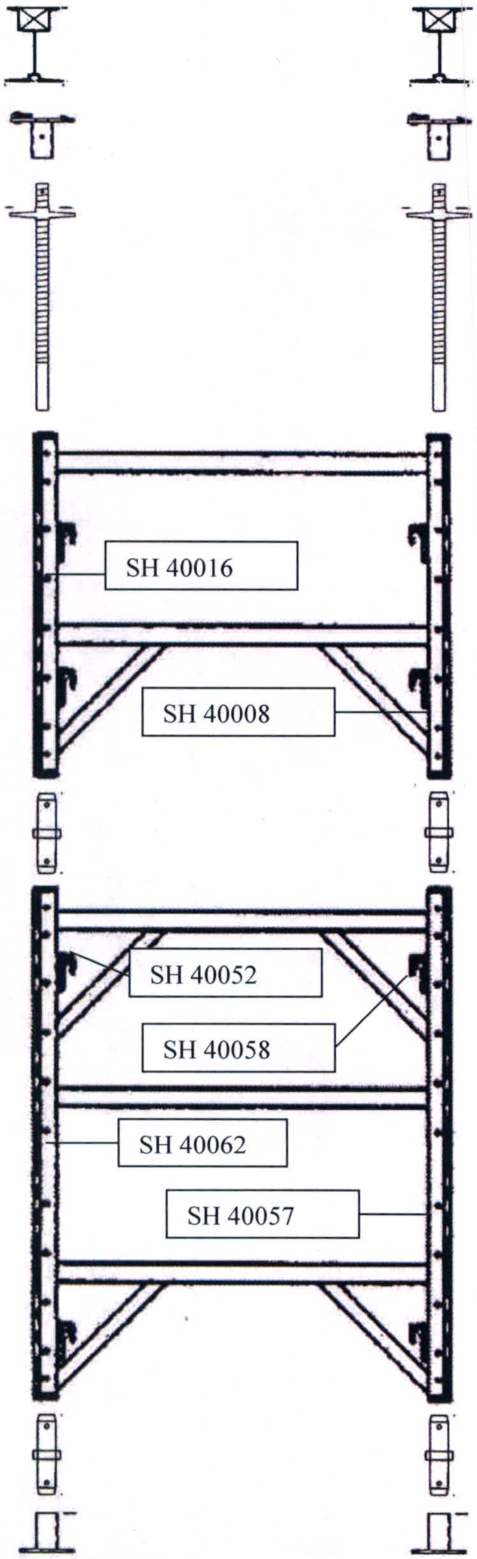


SH40055_5

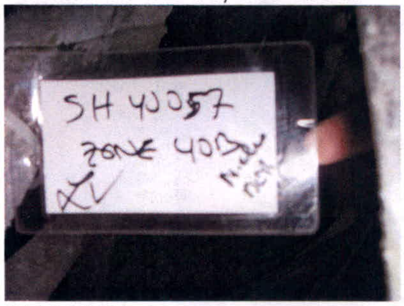
SH40055_3



SH 40056
See SH 4009 for Photos
The bottom Frame of SH 40009
Connected to SH 40055
No Photo
Zone 40B



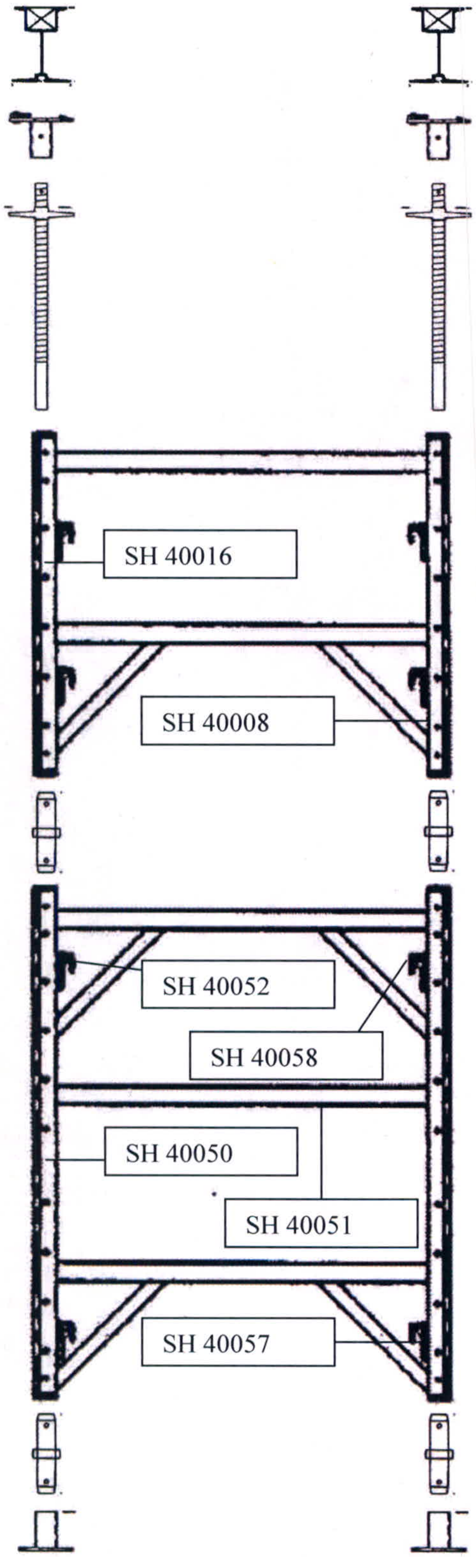
SH 40057
Broken
Bottom Frame of SH 40008
Connected to Bottom Brace of SH 40058
Zone 40B
Folder: 08-02-15; SH 40057



SH40057_1

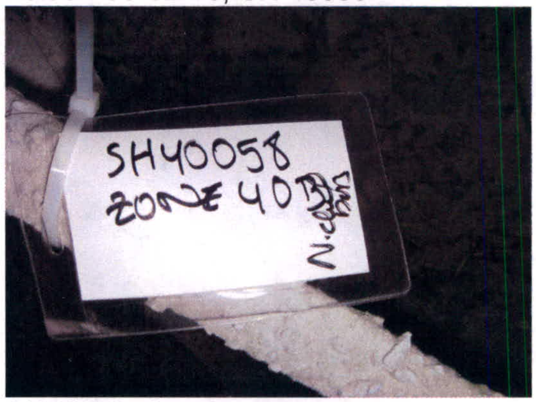
SH40057_3

SH40057_5



SH 40058
Bottom Brace connected to Bottom
Frame SH 40057
Zone 40B

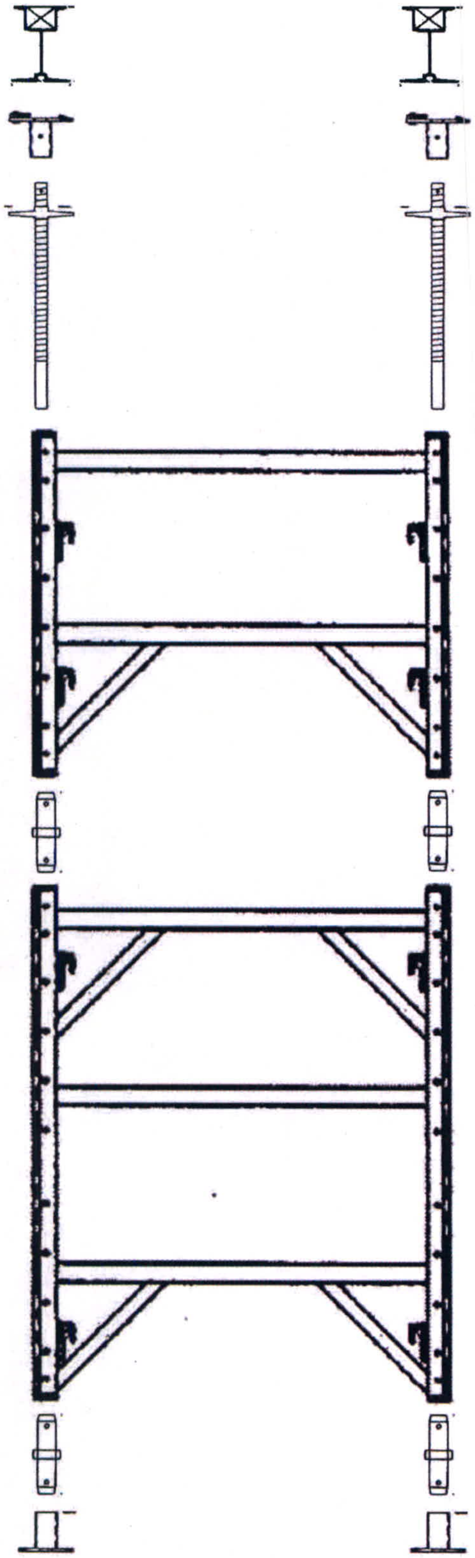
Folder: 08-02-15; SH 40058



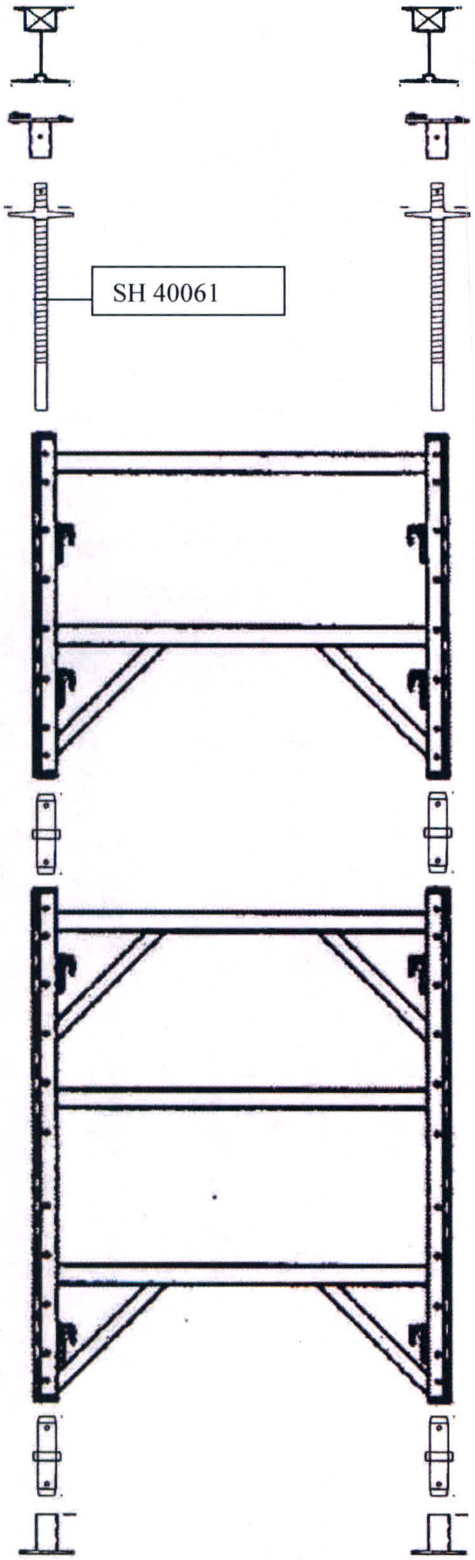
SH40058_1

SH40058_3

SH40058_2

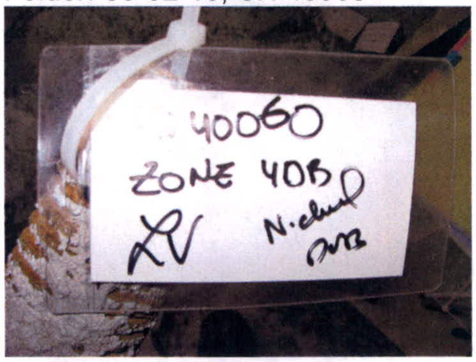


SH 40059
Skipped
No tag



SH 40060
Screw Jack
Zone 40B

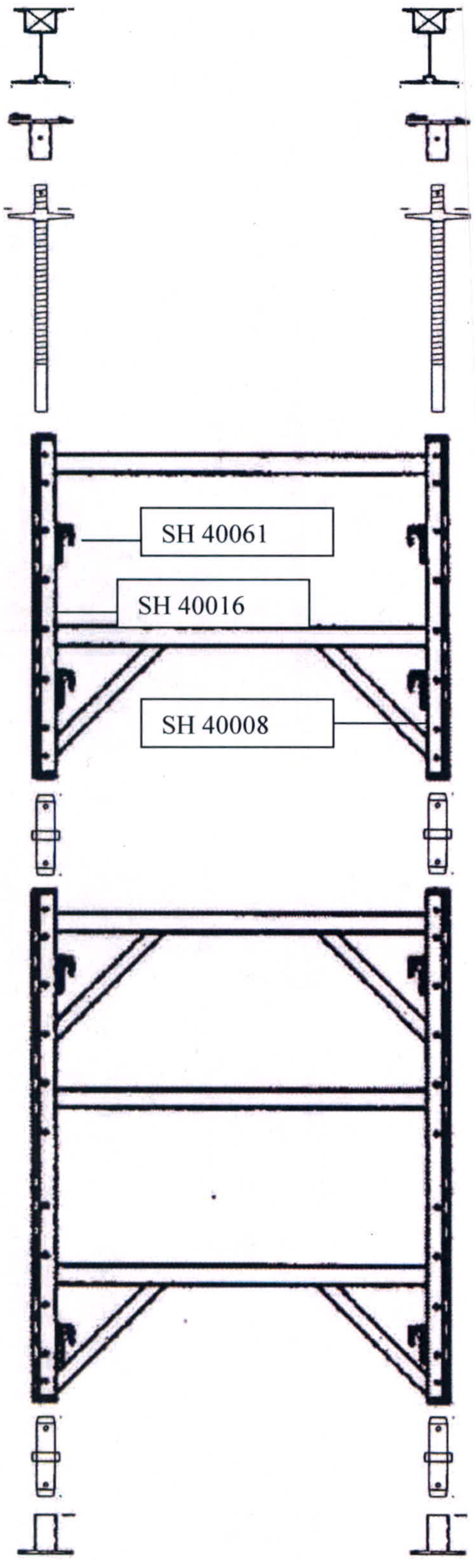
Folder: 08-02-15; SH 40060



SH40060_1

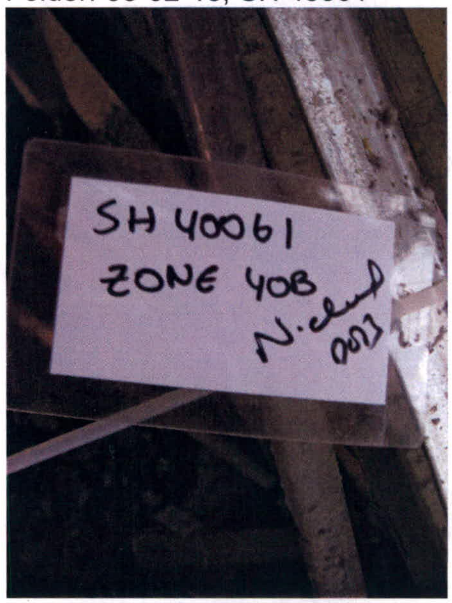
SH40060_2

SH40060_3



SH 40061
X Brace
Connects to SH 40016 and SH 40017
Zone 40B

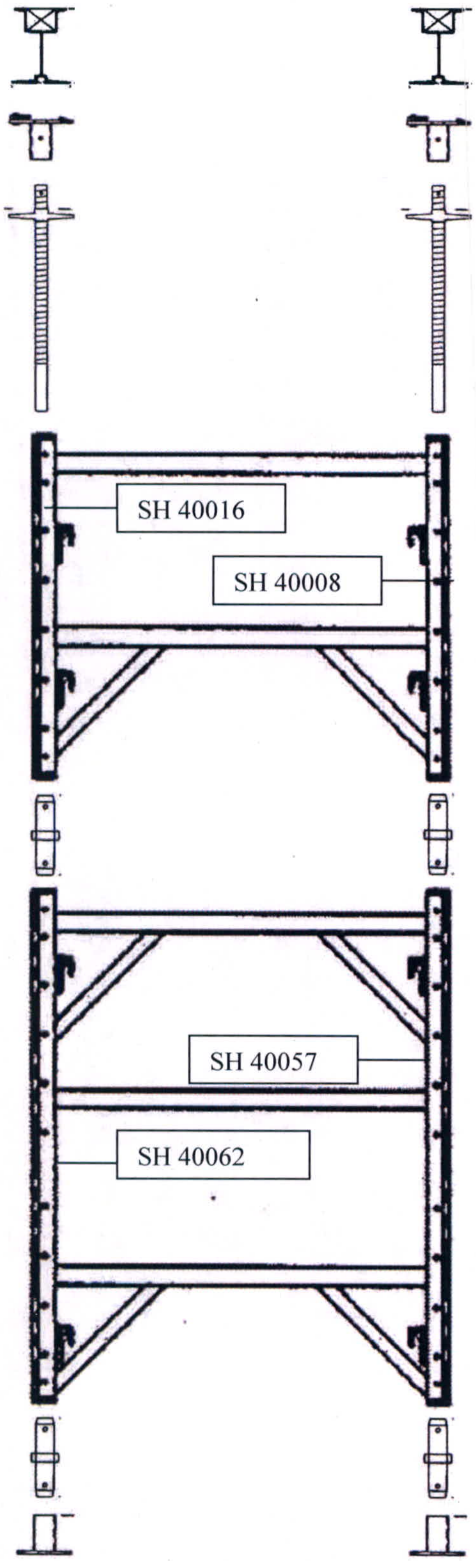
Folder: 08-02-15; SH 40061



SH40061_1

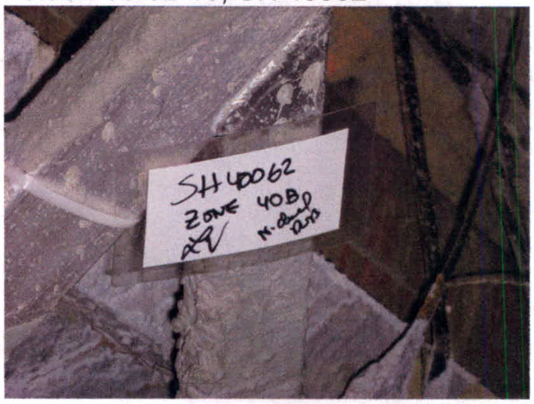
SH40061_2

SH40061_4



SH 40062
Bottom Frame of SH 40016
Zone 40B

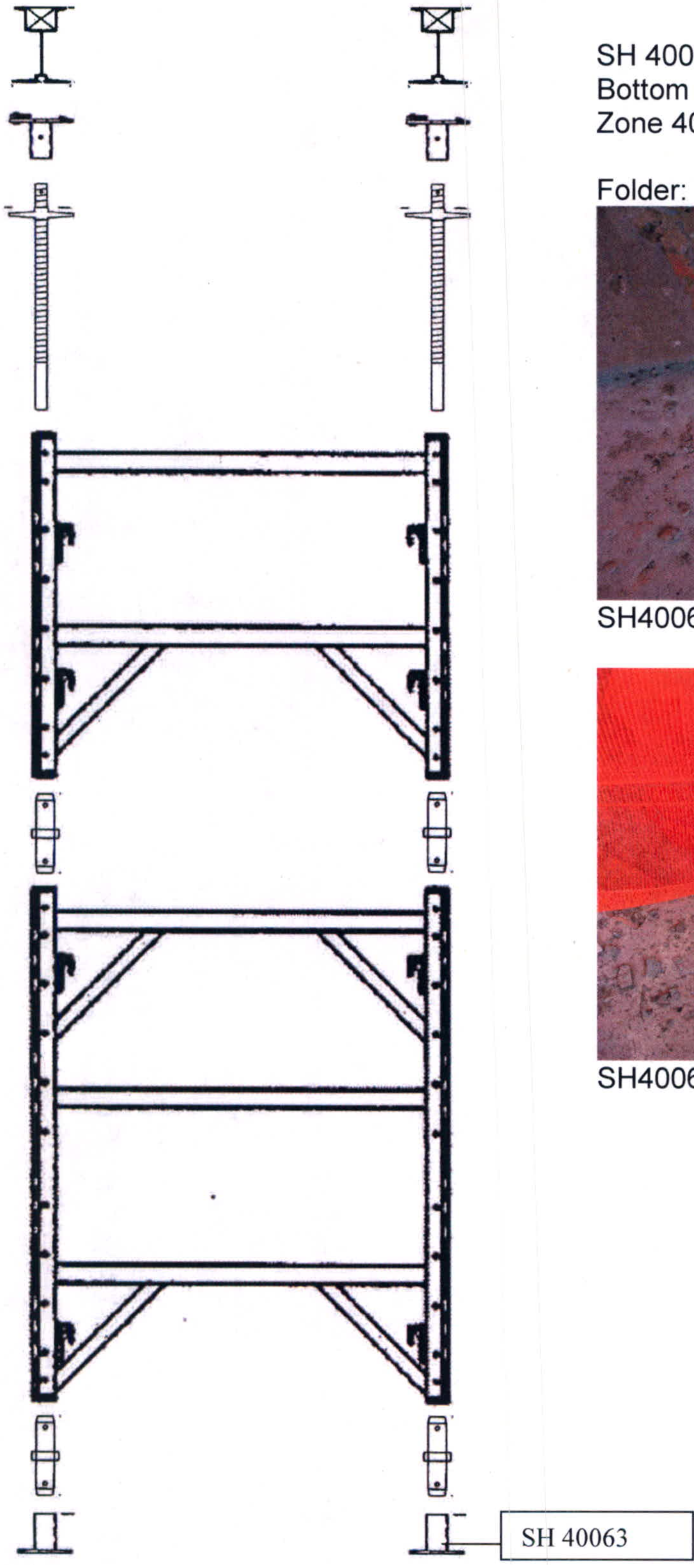
Folder: 08-02-15; SH 40062



SH40062_1

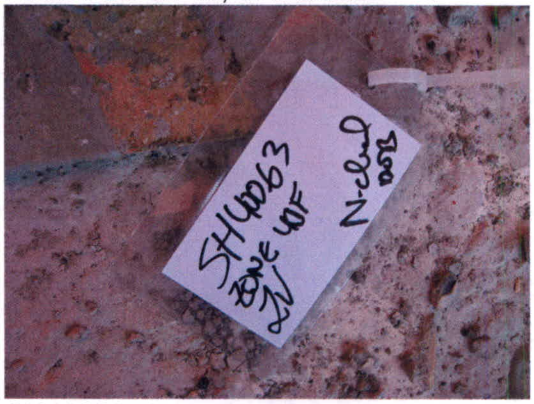
SH40062_5

SH40062_3



SH 40063
Bottom Plate
Zone 40F

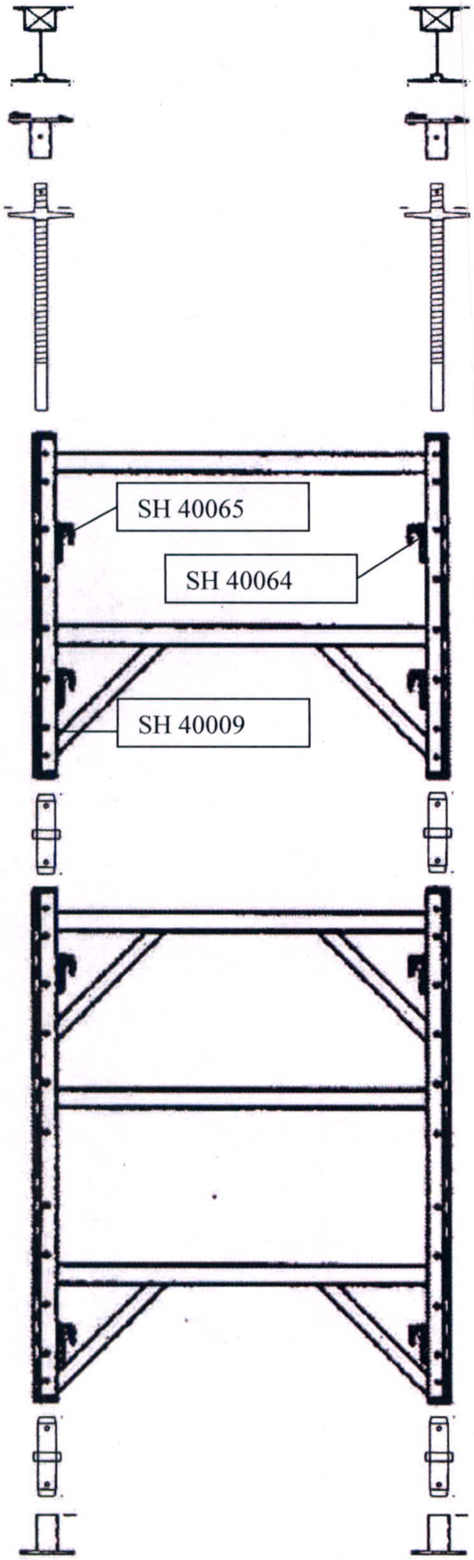
Folder: 08-02-15; SH 40063



SH40063_1


SH40063_2

SH 40064
Top X Brace
Connects SH 40009 to SH 40011
Zone 40B

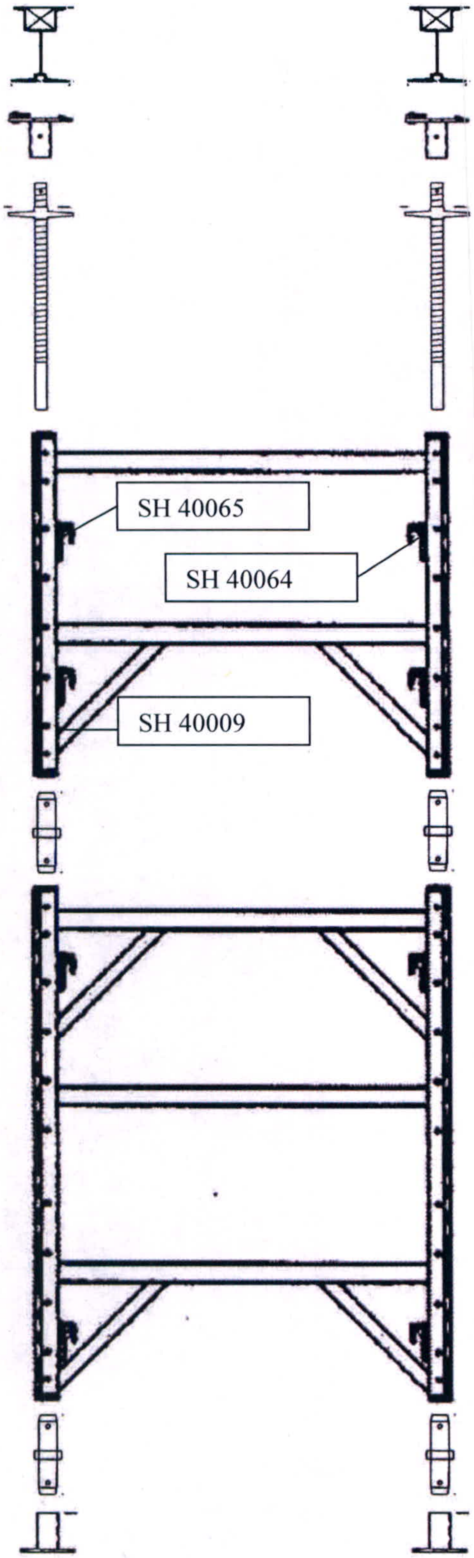


Folder: 08-02-15; SH 40064

SH40064_1

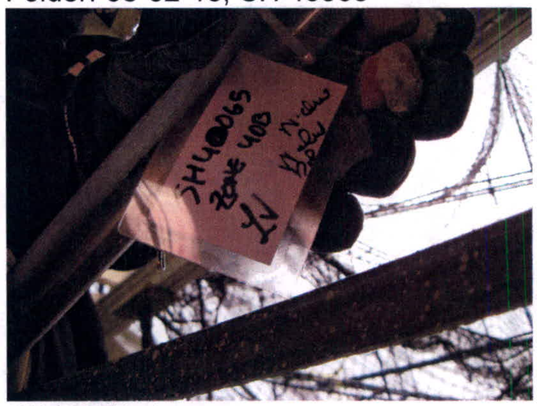
SH40064_2

SH40064_3



SH 40065
X Brace
Connect SH 40009 to SH 40011
Zone 40B

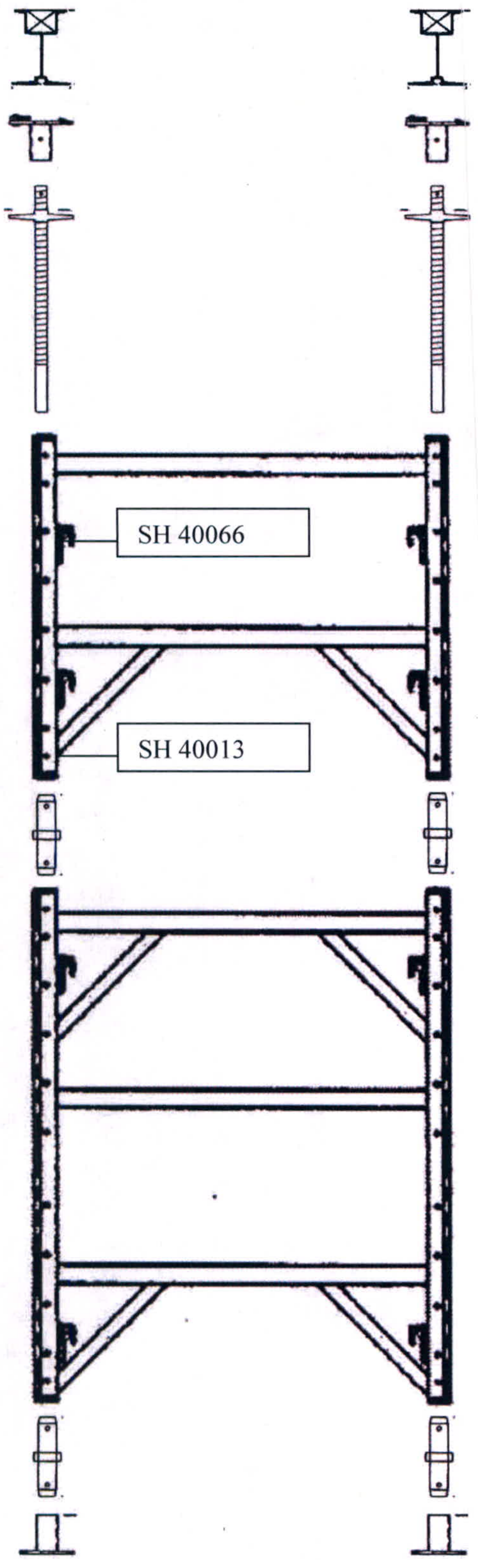
Folder: 08-02-15; SH 40065



SH40065_1

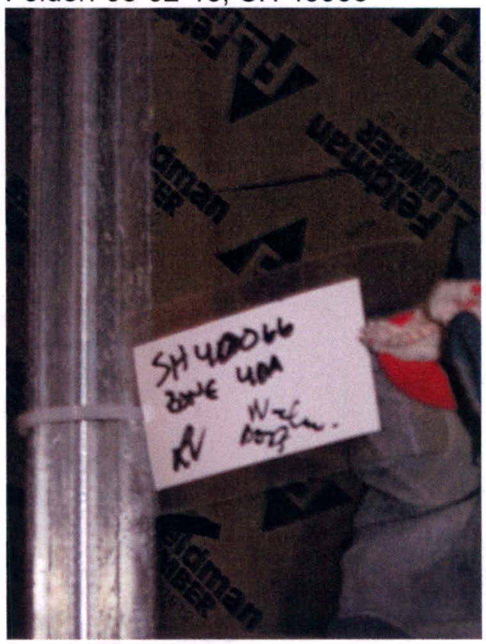
SH40065_2

SH40065_3



SH 40066
X Brace
Connect SH 40013 to SH 40014
Zone 40A

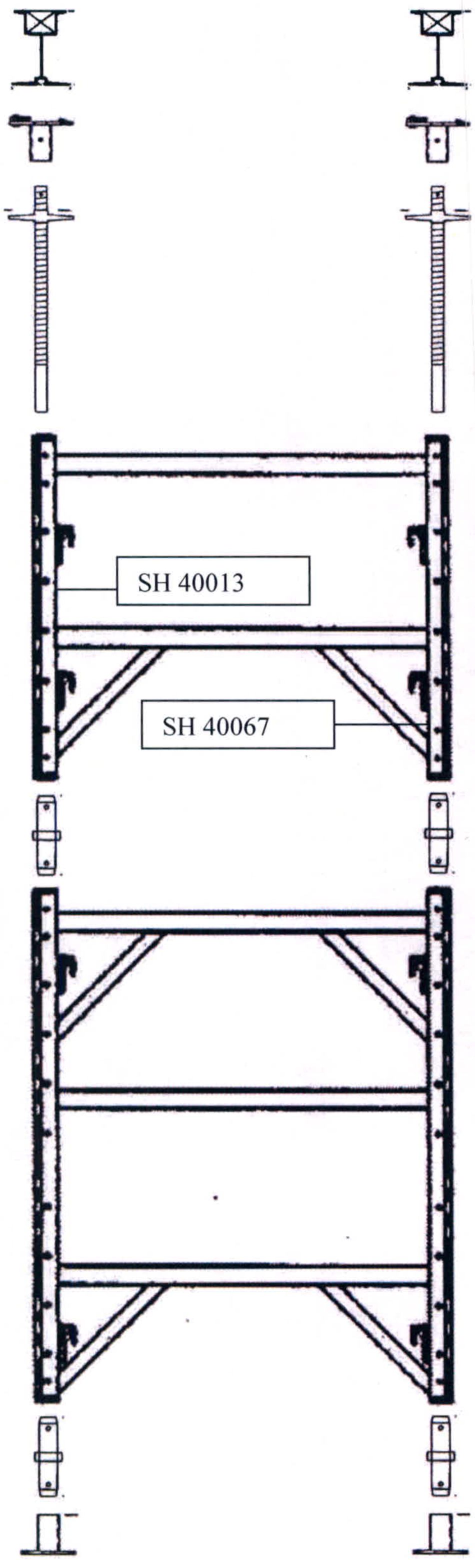
Folder: 08-02-15; SH 40066



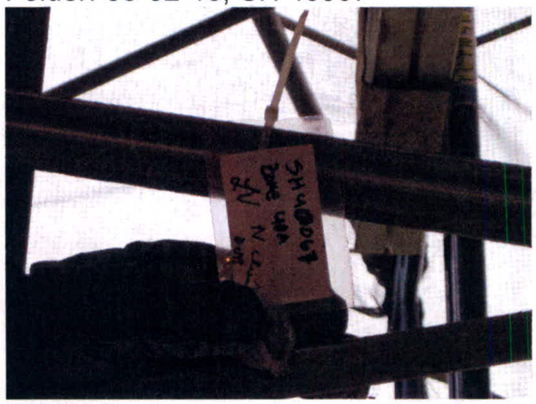
SH40066_1

SH40066_2

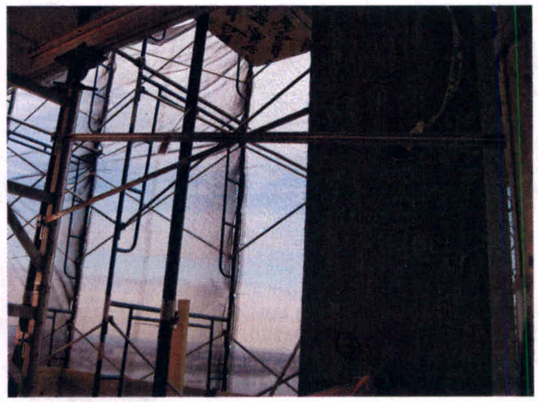
SH 40067
X Brace
Connect SH 40013 to SH 40014
Zone 40A



Folder: 08-02-15; SH 40067



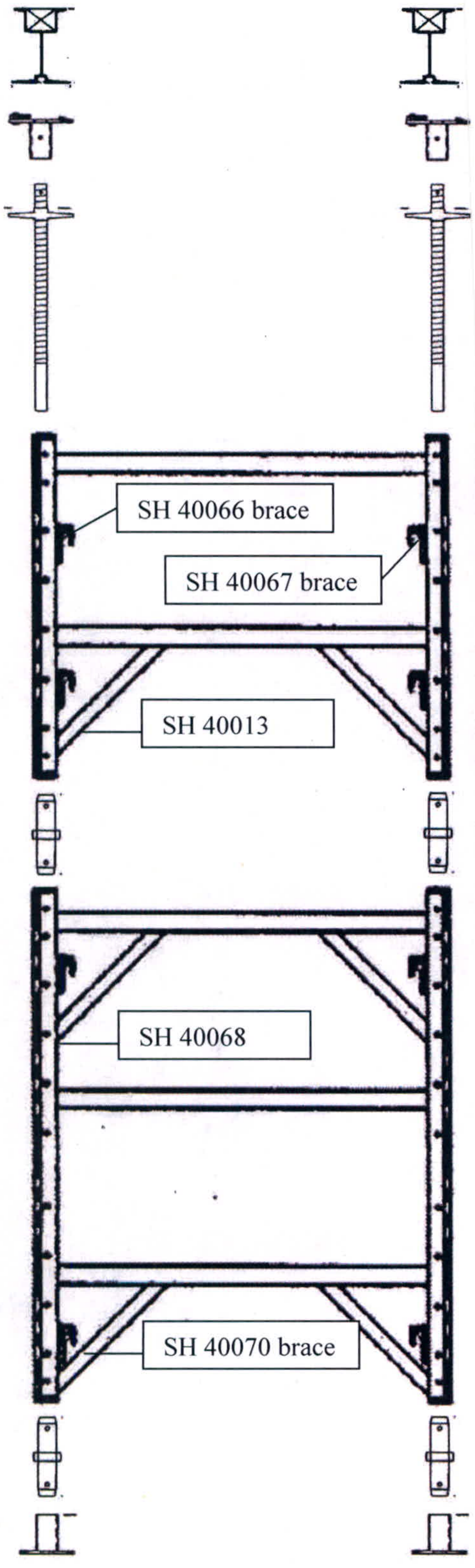
SH40067_1



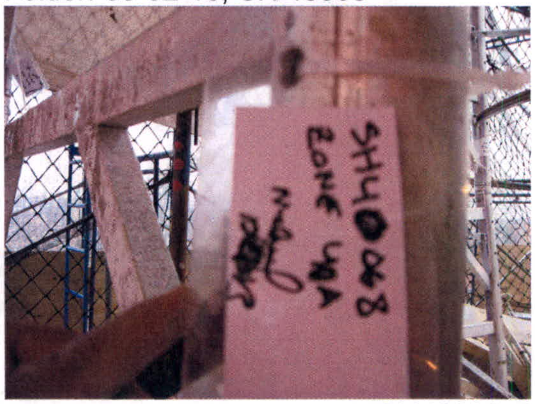
SH40067_2

SH40067_3

SH 40068
Bottom Frame of SH 40013
Zone 40A

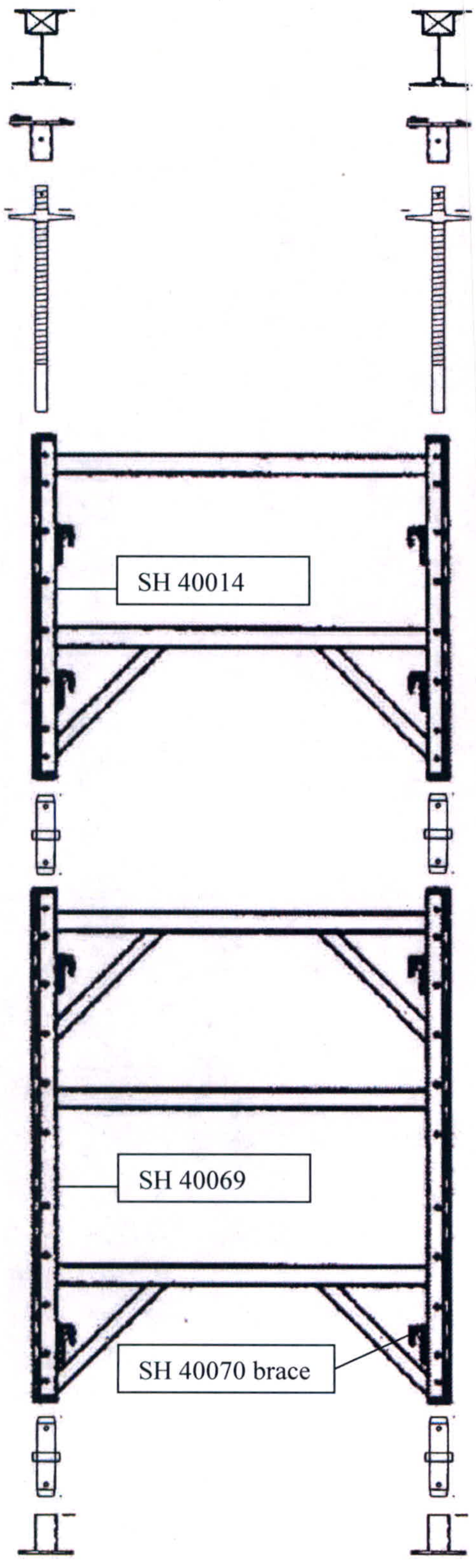


Folder: 08-02-15; SH 40068



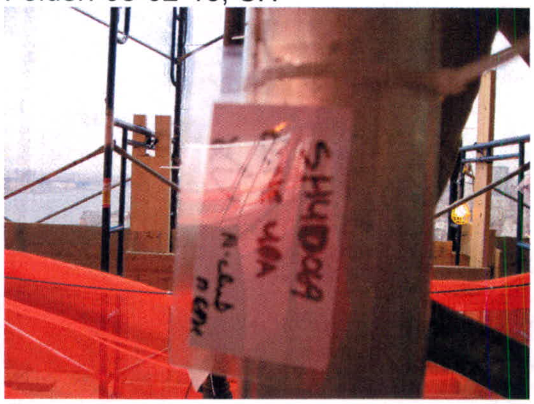
SH40068_1

SH40068_2

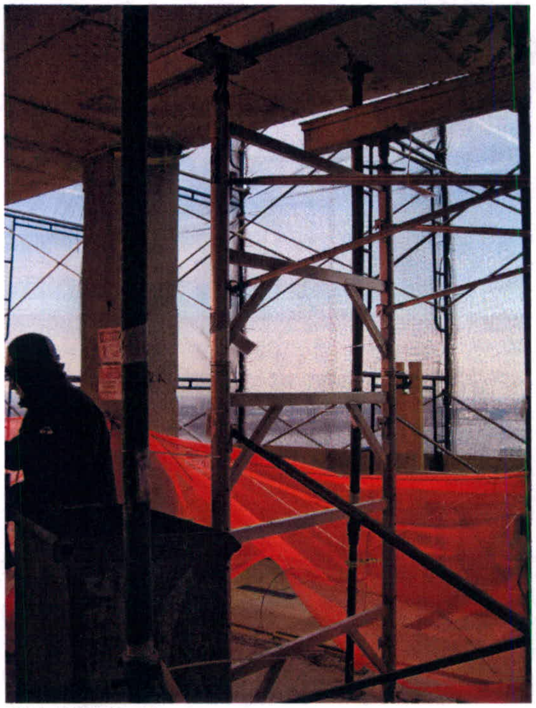


SH 40069
Bottom Frame of SH 40014
Zone 40A

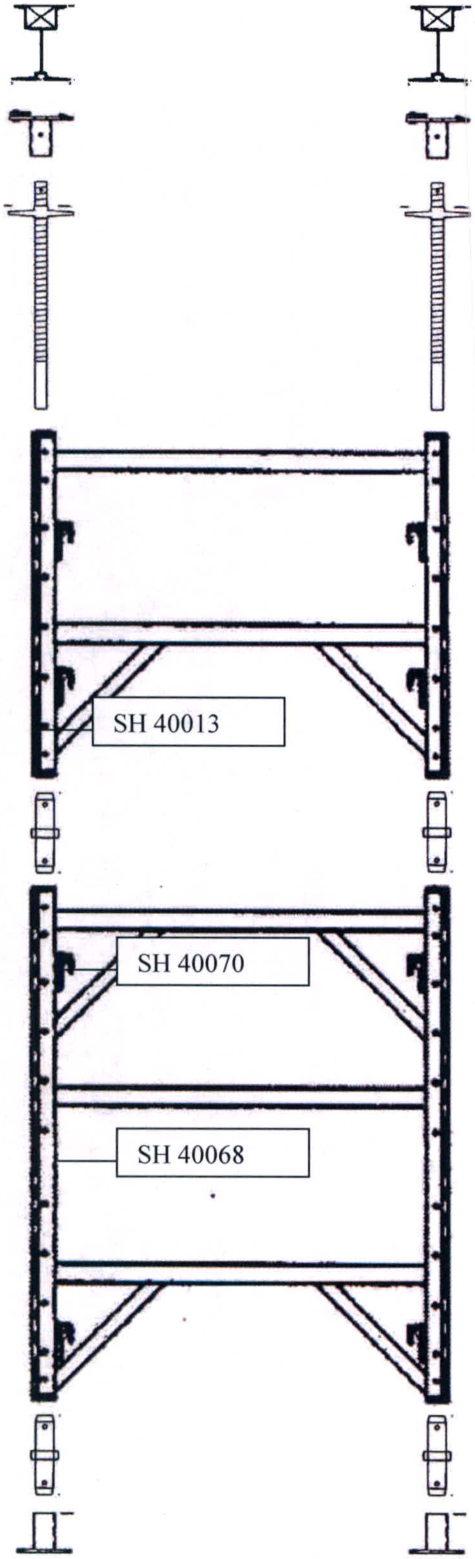
Folder: 08-02-15; SH



SH40069_1

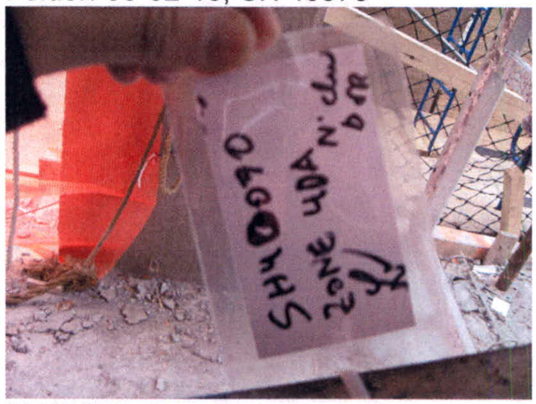


SH40069_2



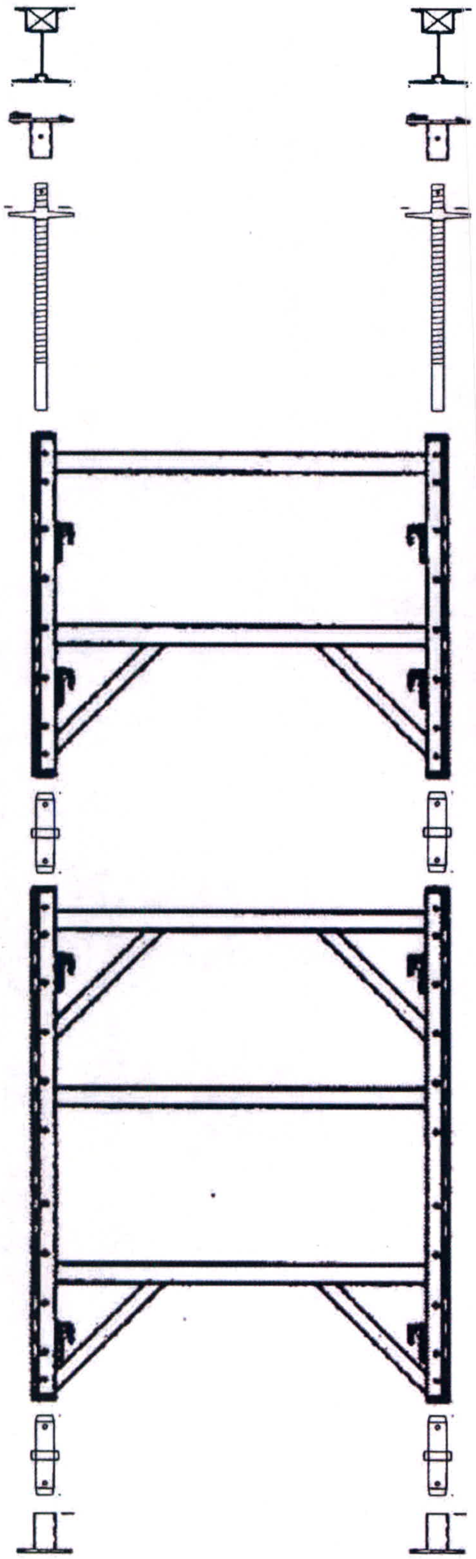
SH 40070
Bottom X Brace
Connects SH 40068 to SH 40069

Folder: 08-02-15; SH 40070


SH40070_1


SH40070_2


SH40070_3

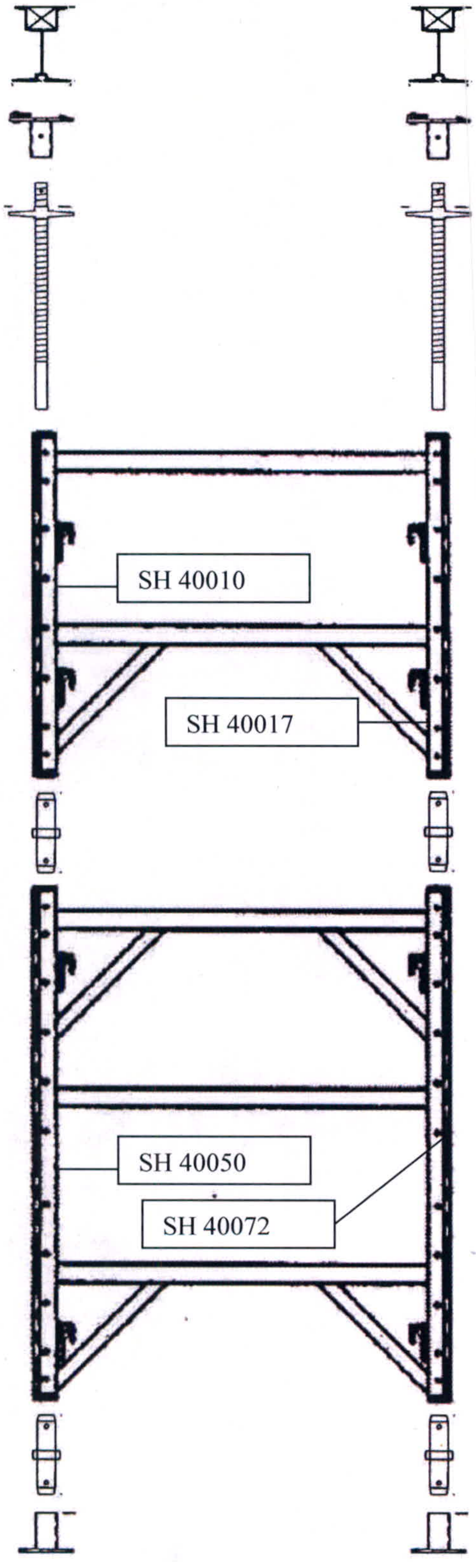


SH 40071
Broken Piece of Frame
Might Connect to SH 40072
Zone 40B

Folder: 08-02-16; SH

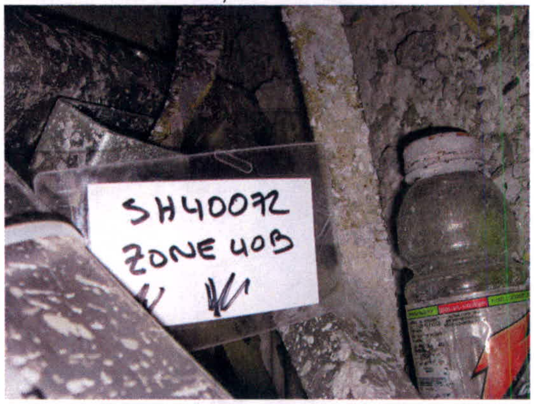
SH40071_1

SH40071_2



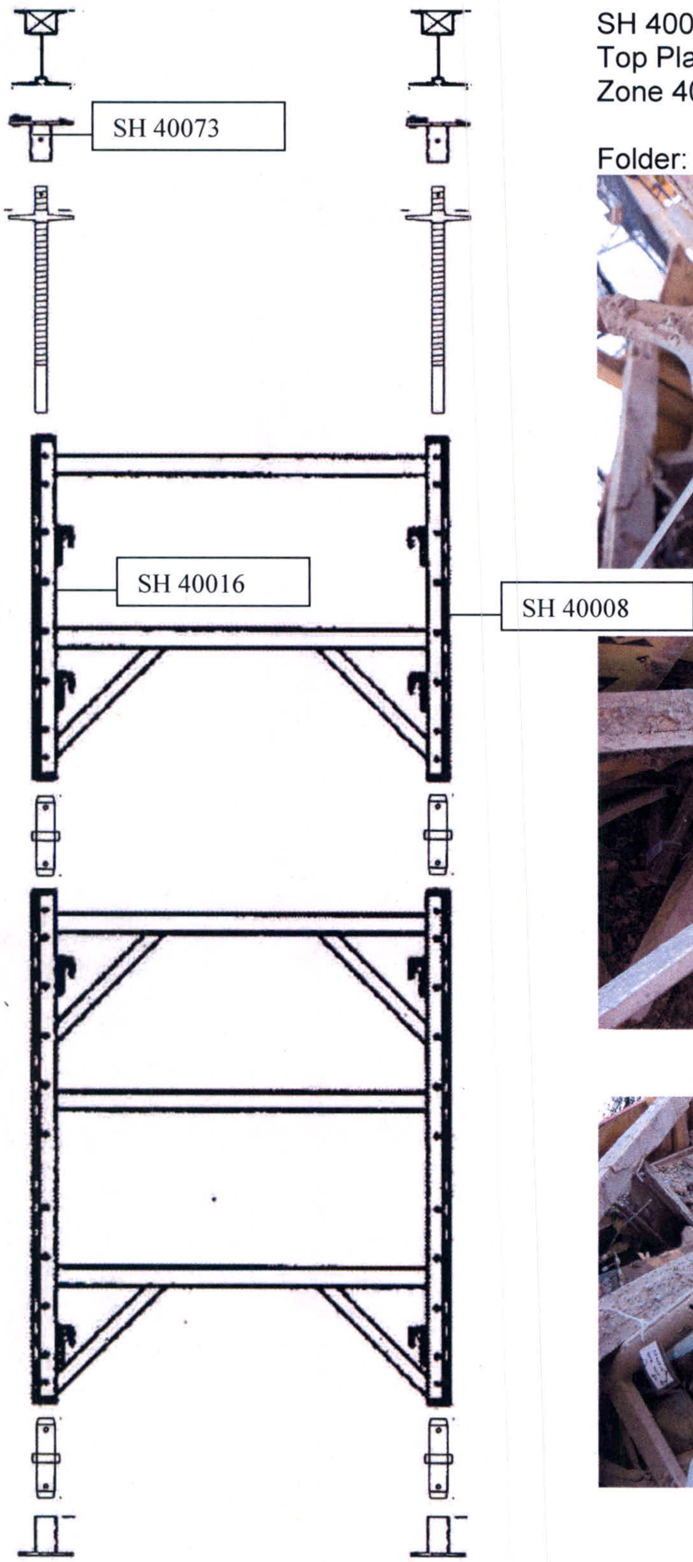
SH 40072
Bottom Frame of SH 40017
Broken
Zone 40B

Folder: 08-02-16; SH 470072



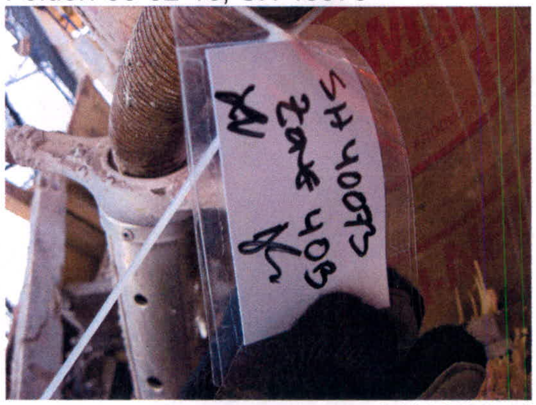
SH40072_1

SH40072_2



SH 40073
Top Plate of SH 40016
Zone 40B

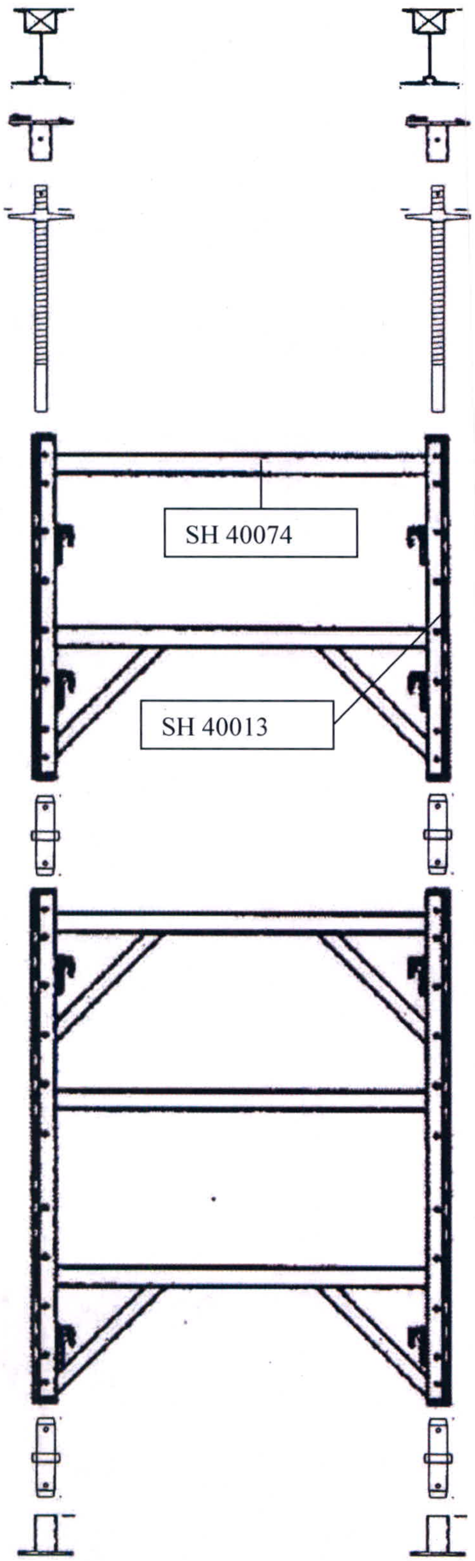
Folder: 08-02-16; SH 40073



SH40073_1

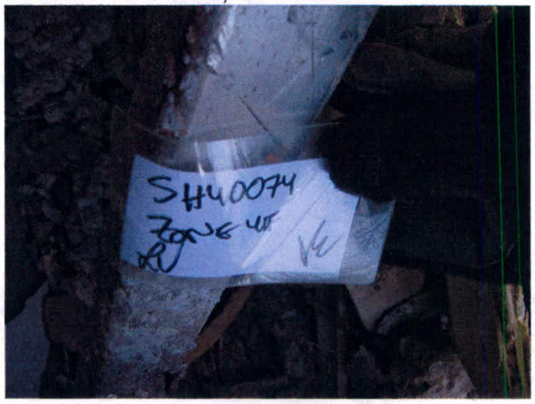
SH40073_3

SH40073_2



SH 40074
Piece of Frame SH 40013
Zone 40F

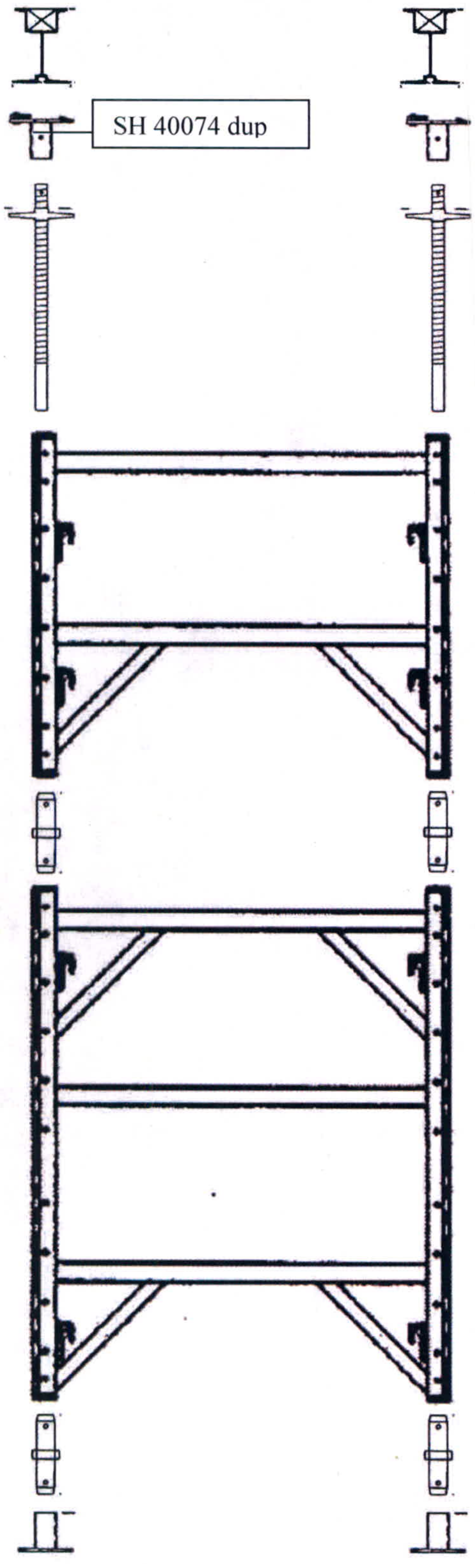
Folder: 08-02-16; SH 40074



SH40074_1

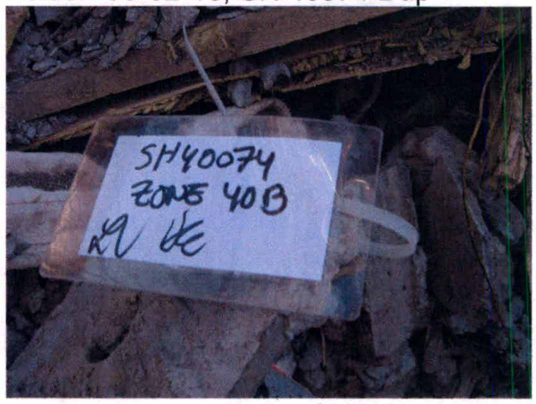
SH40074_2

SH40074_3



SH 40074 Dup
Component should have been labeled
as SH 40075
Plate
Zone 40B

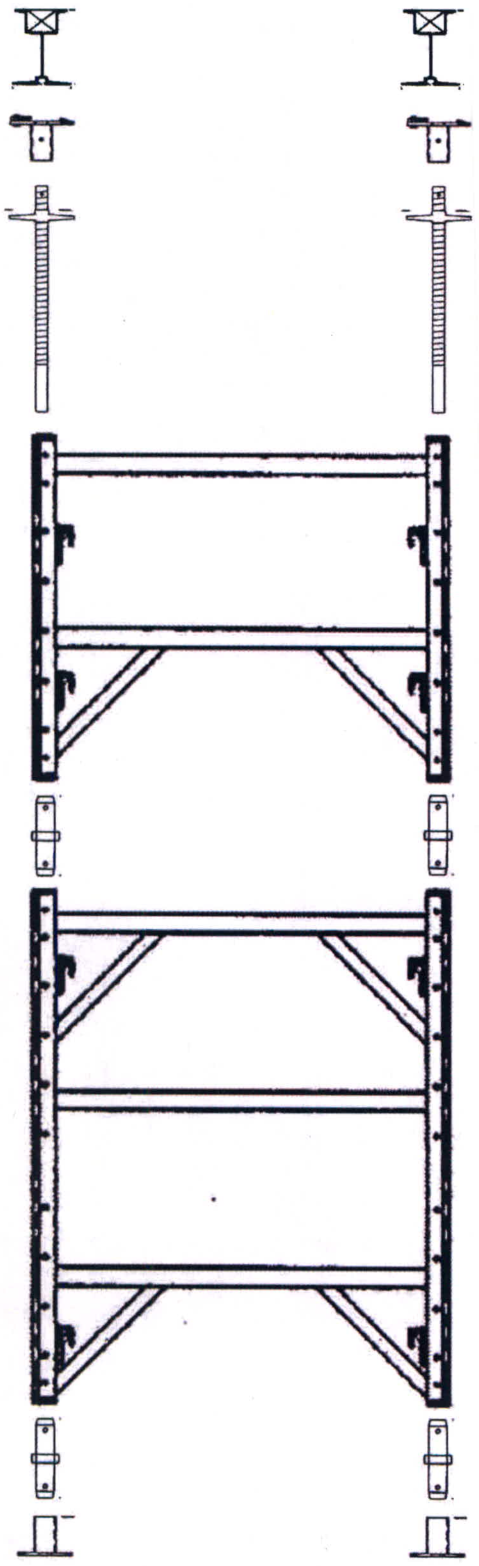
Folder: 08-02-16; SH 40074 Dup



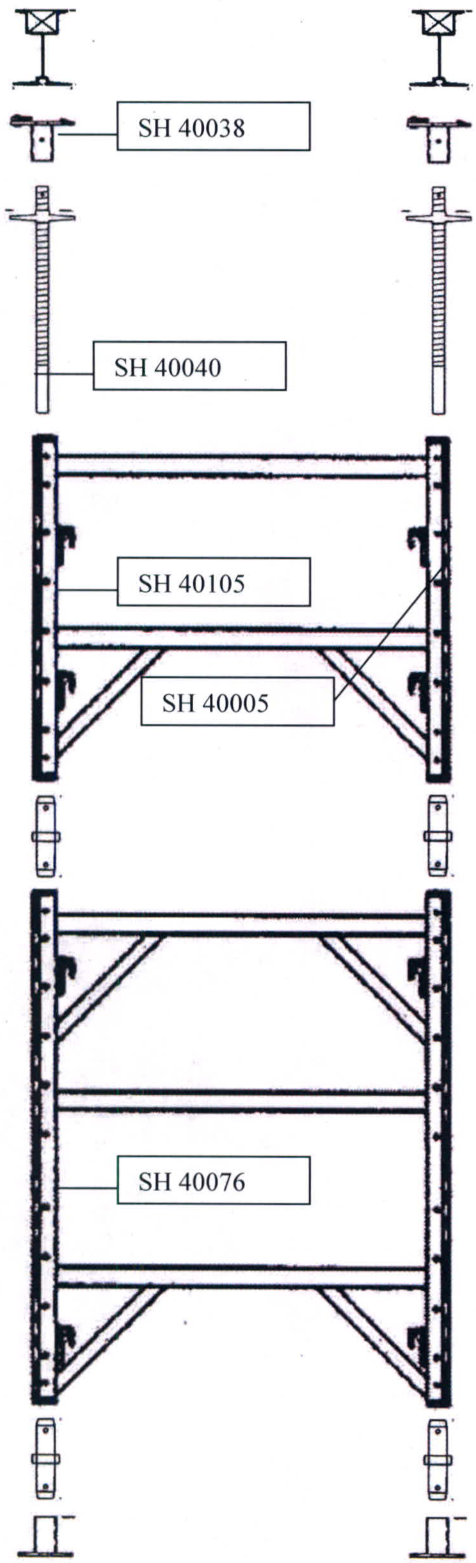
SH40074(dup)_1

SH40074(dup)_2

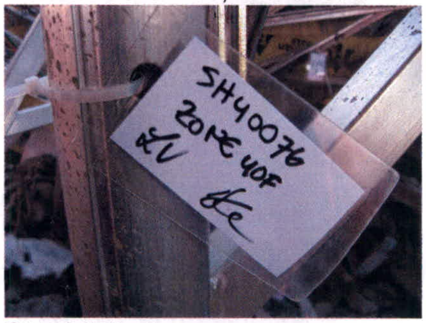
SH40074(dup)_3



SH 40075
Tag Skipped



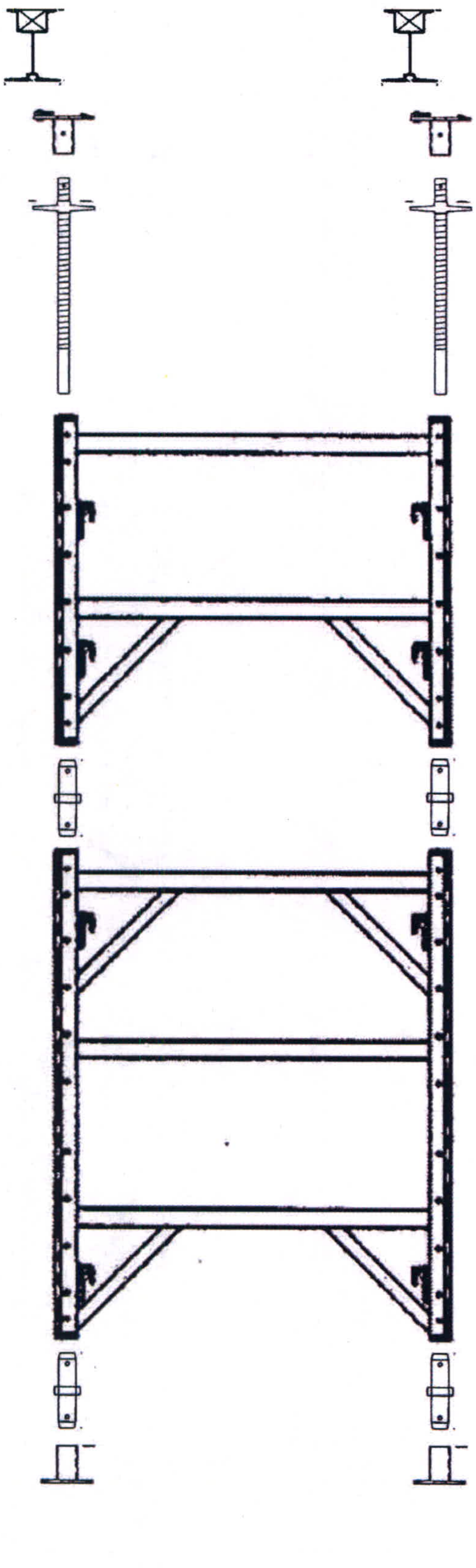
SH 40076
Bottom Frame of SH 40005
Zone 40F
Folder: 08-02-16; SH 40076



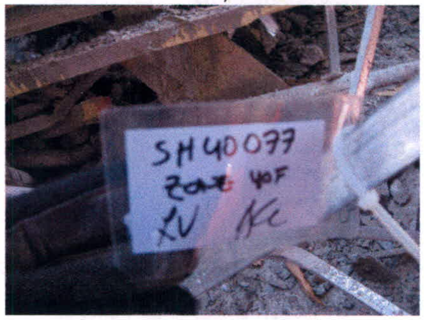
SH40076_1


SH40076_2


SH40076_3

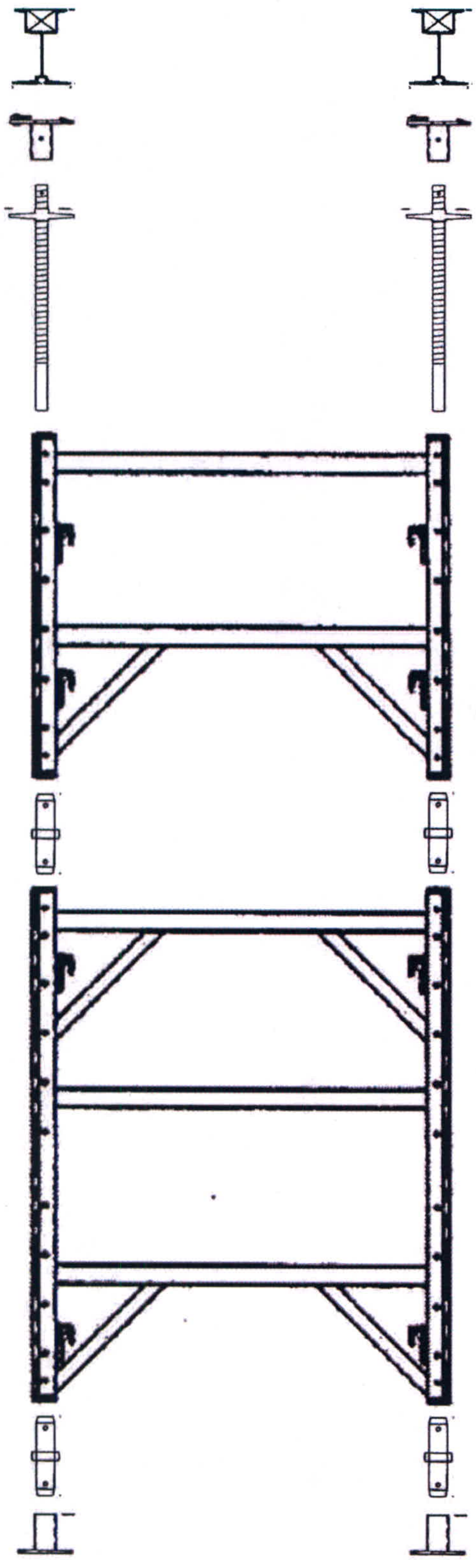


SH 40077
Brace conn to SH 40076
Zone 40F
Folder: 08-02-16; SH 40077

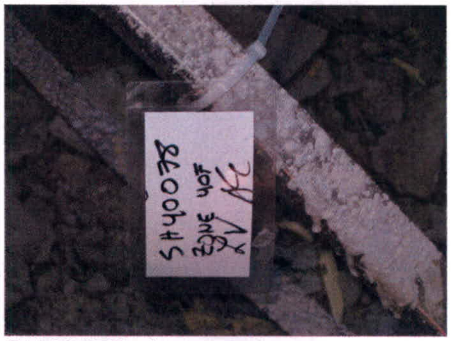


SH40077_1

SH40077_2

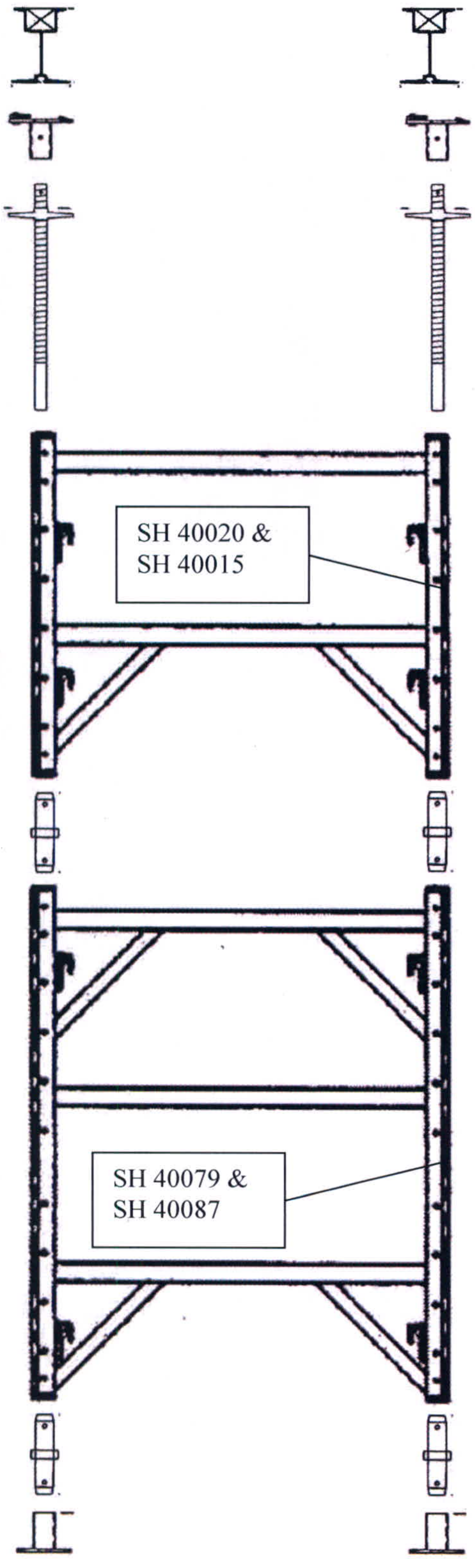


SH 40078
Brace conn to SH 40076
Zone 40F
Folder: 08-02-16; SH 40078



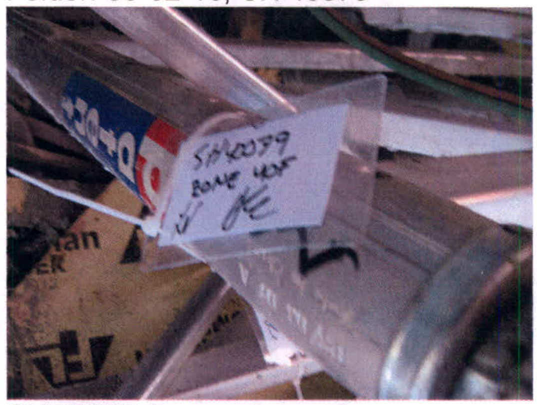
SH40078_1

SH40078_2



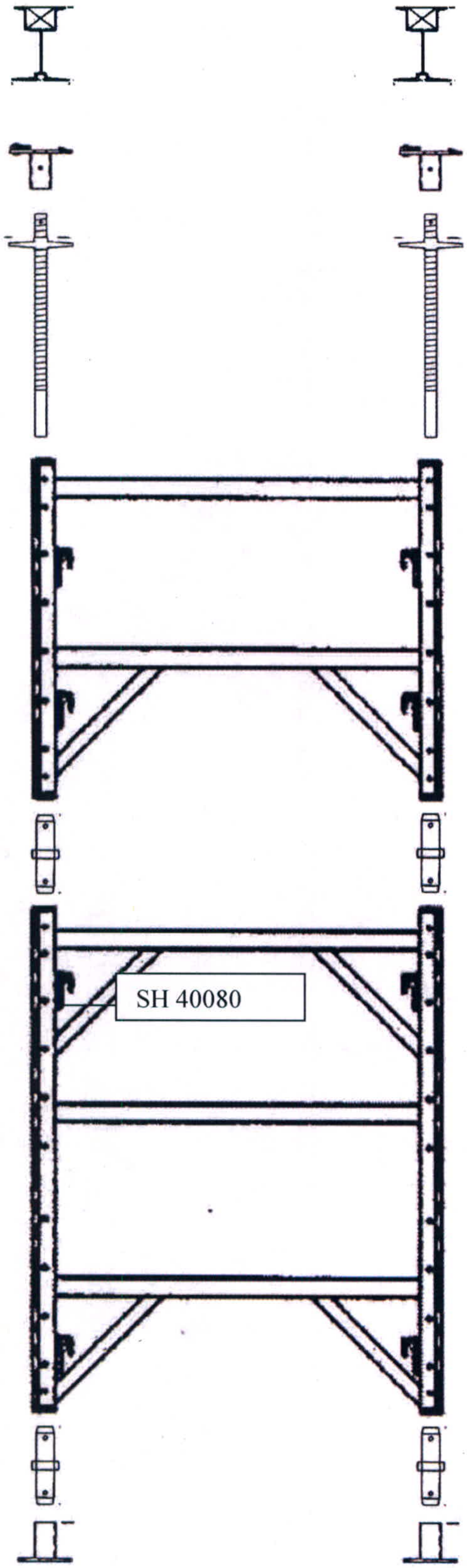
SH 40079
Bottom Frame of SH 40020
Zone 40F

Folder: 08-02-16; SH 40079



SH 40079 1

SH 40079 2

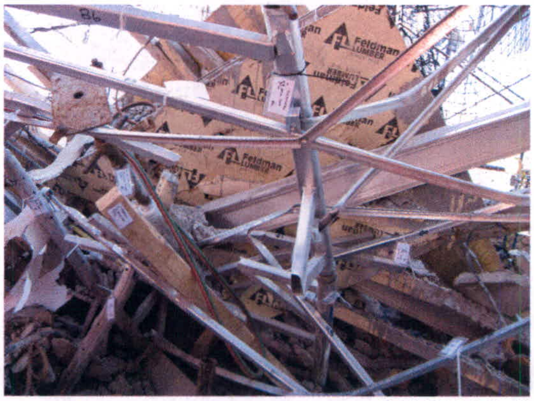


SH 40080
X Brace
Zone 40F

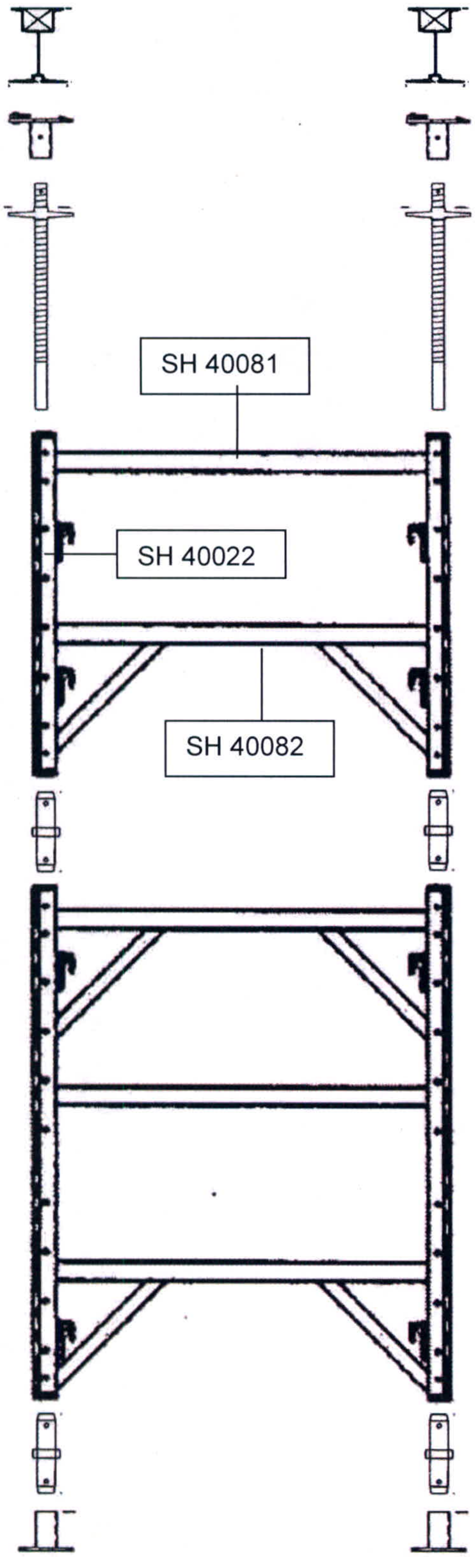
Folder: 08-02-16; SH 40080

SH40080_1

SH40080_2

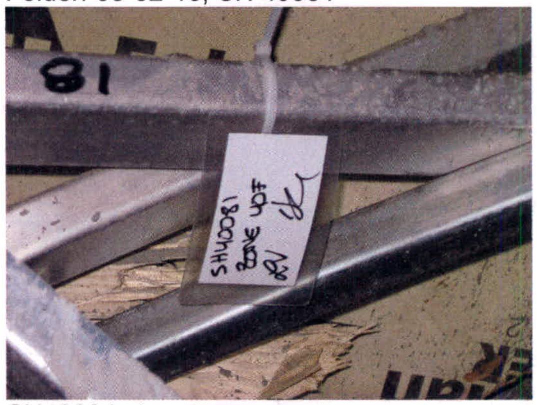


SH40080_3



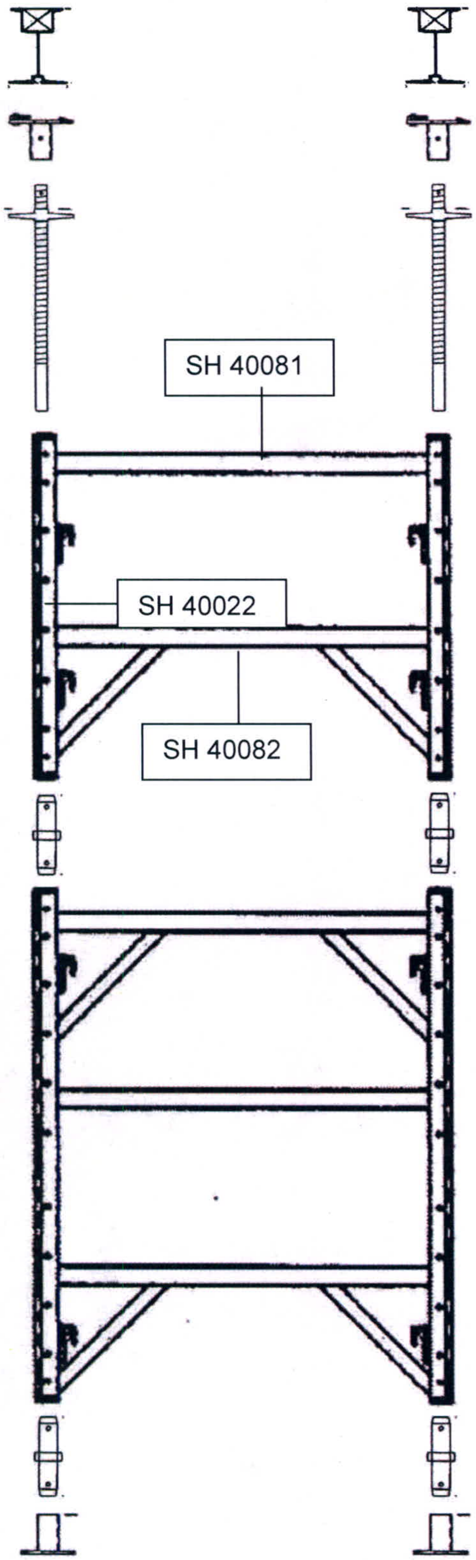
SH 40081
Broken piece of frame
Might Connect to SH 40022
Zone 40F

Folder: 08-02-16; SH 40081



SH40081_1

SH40081_2



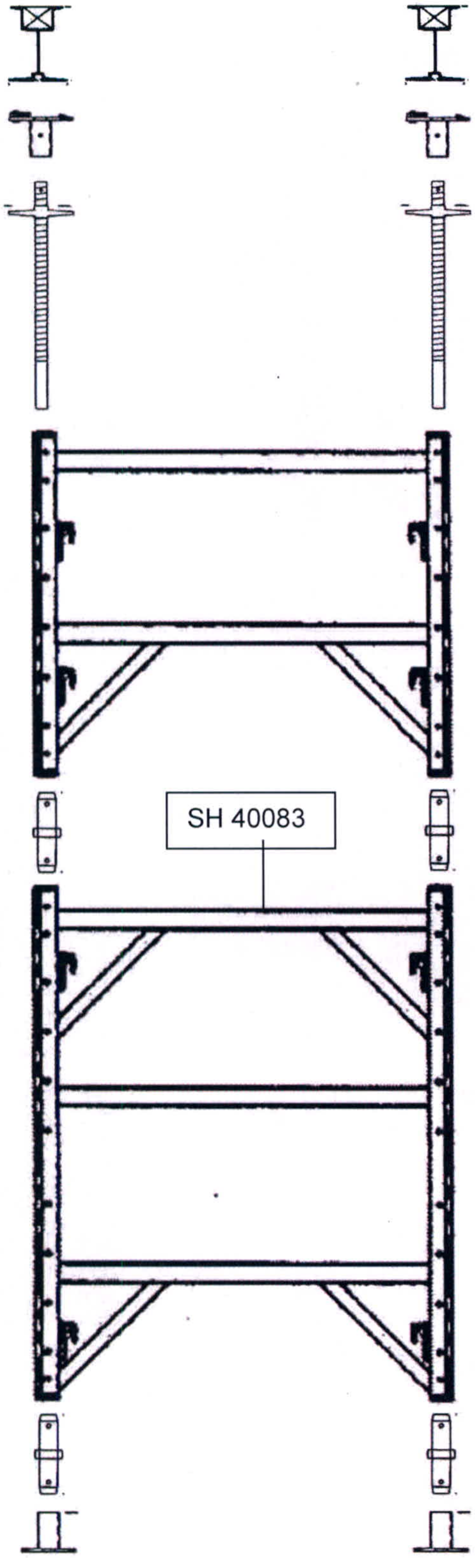
SH 40082
Broken piece of Frame
Might connect to SH 40022
Zone 40F

Folder: 08-02-16; SH 40082

SH40082_1

SH40082_3

SH40082_2



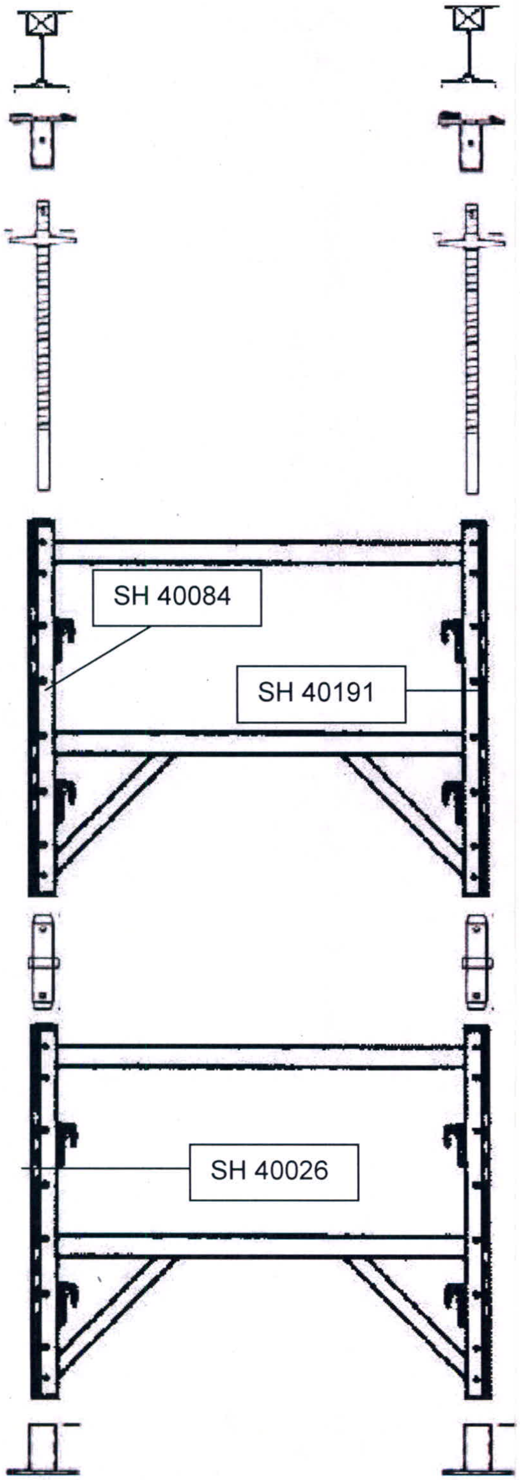
SH 40083
Broken Piece
Zone 40F

Folder: 08-02-16; SH 40083

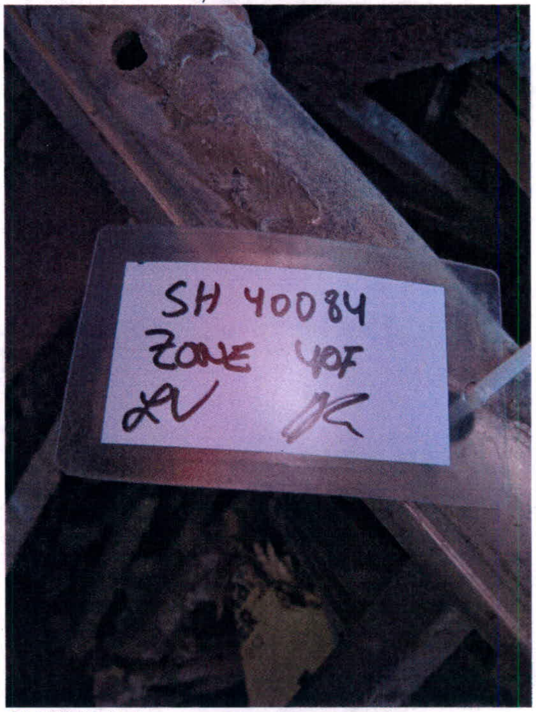
SH40083_1


SH40083_2

SH 40084 on 41st floor
Top Shore near SH 40015 & SH 40087
Zone 40F (3'-6")

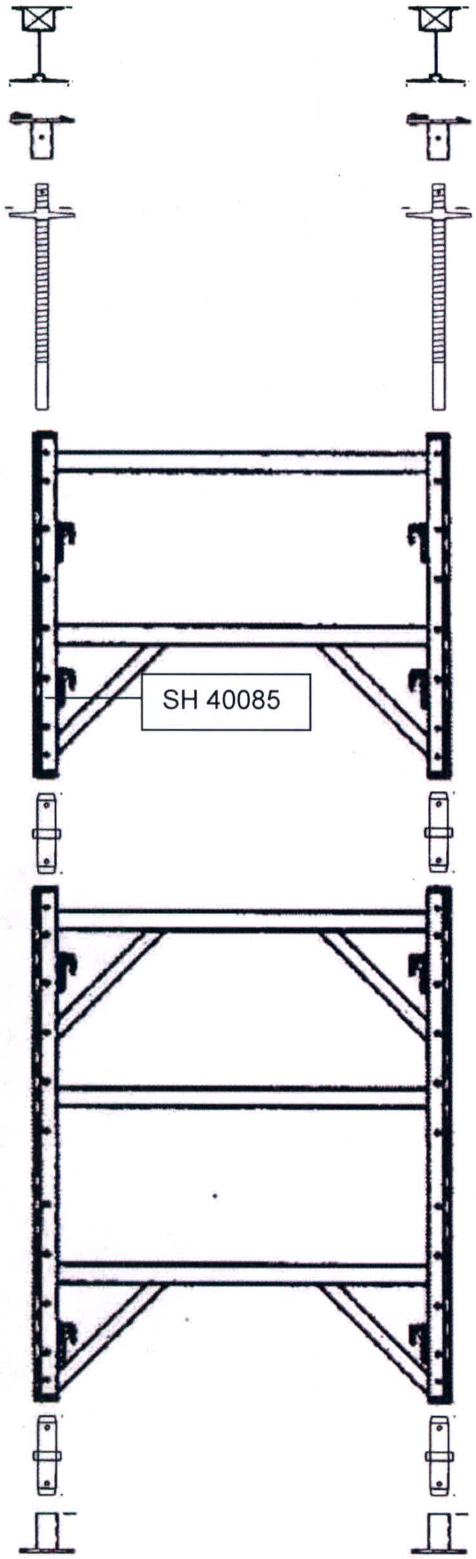


Folder: 08-02-16; SH 40084



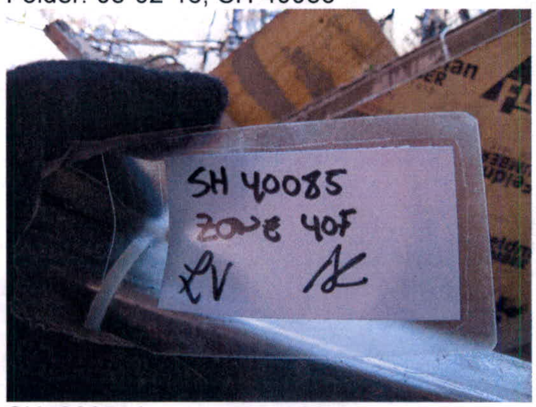
SH40084_1


SH40084_2



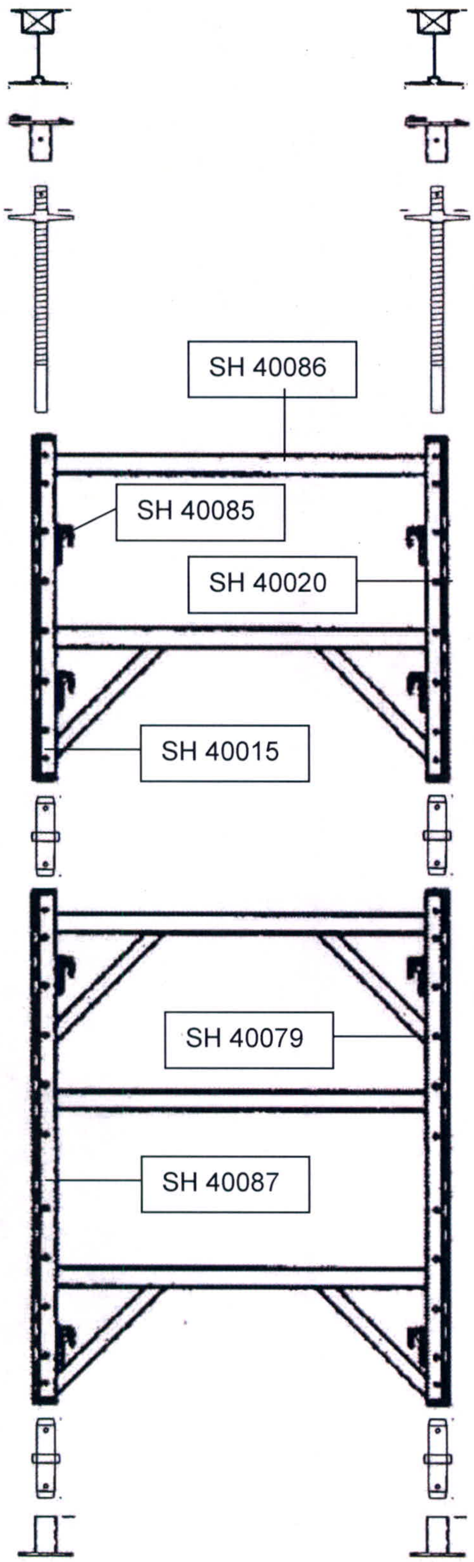
SH 40085
Top X Brace
Attached to SH 40015
Zone 40F

Folder: 08-02-16; SH 40085



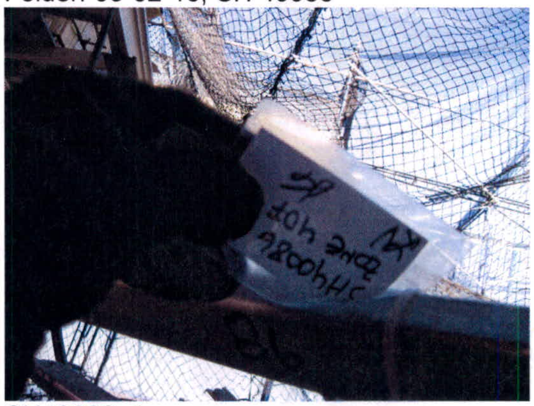
SH40085_1

SH40085_2



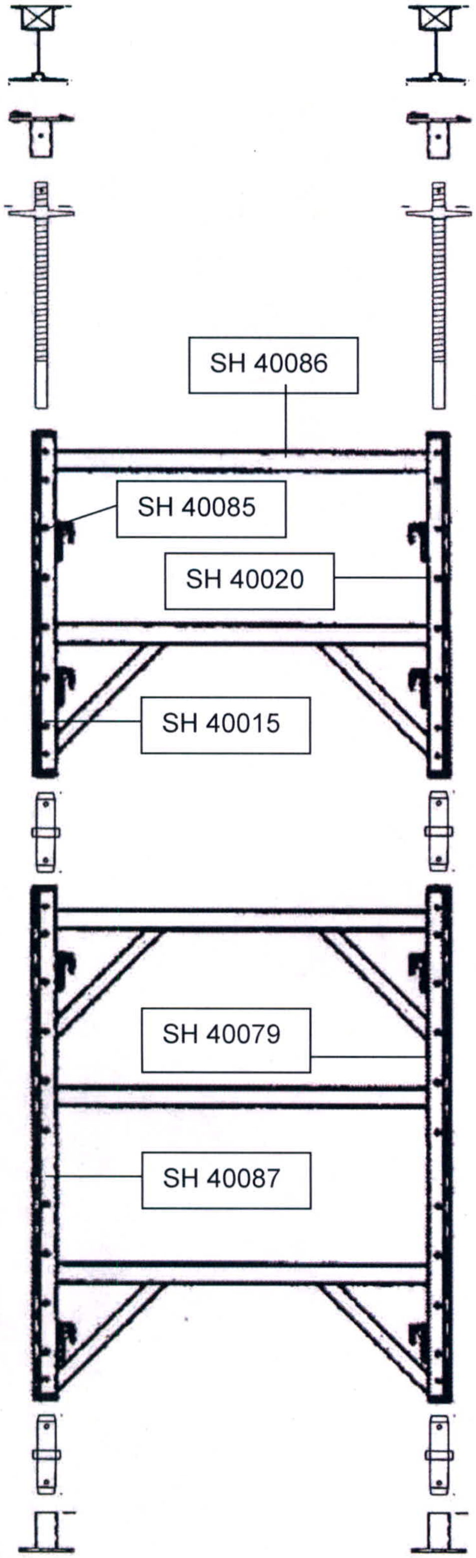
SH 40086
Piece of Frame SH 40020

Folder: 08-02-16; SH 40086


SH40086_1


SH40086_2


SH40086_3



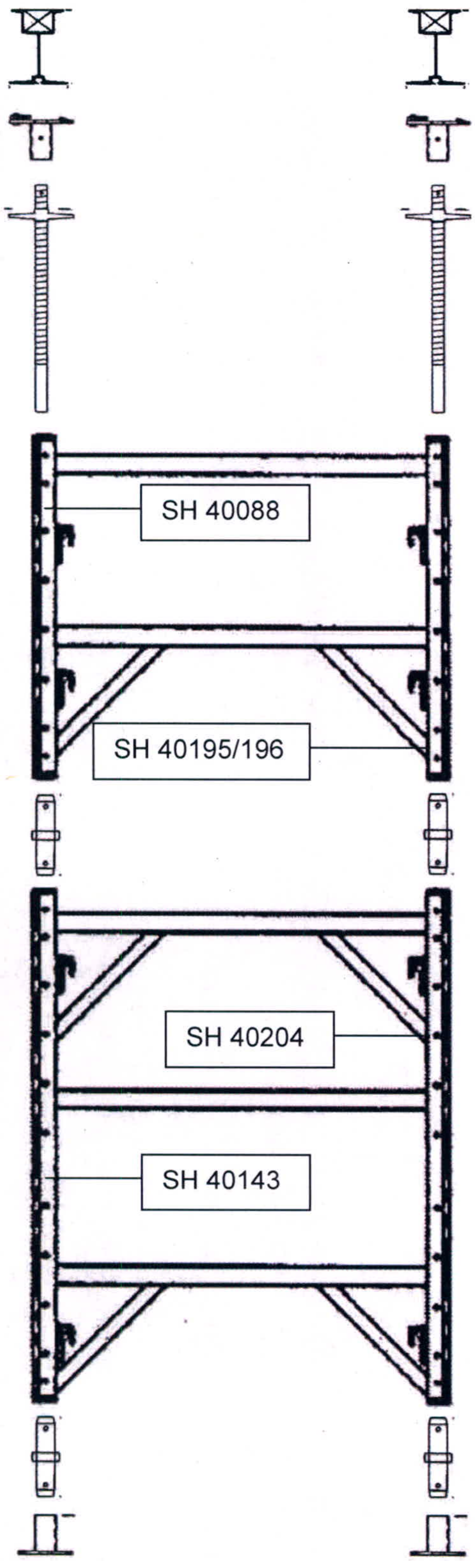
SH 40087
Bottom Frame of SH 40015
Makes a bottom frame with SH 40079
Zone 40F

Folder: 08-02-16; SH 40087

SH40087_1


SH40087_4

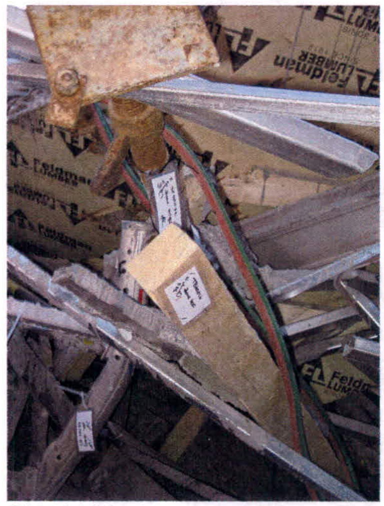

SH40087_3



SH 40088
Top Frame (Shore and Screw jack)
Zone 40F

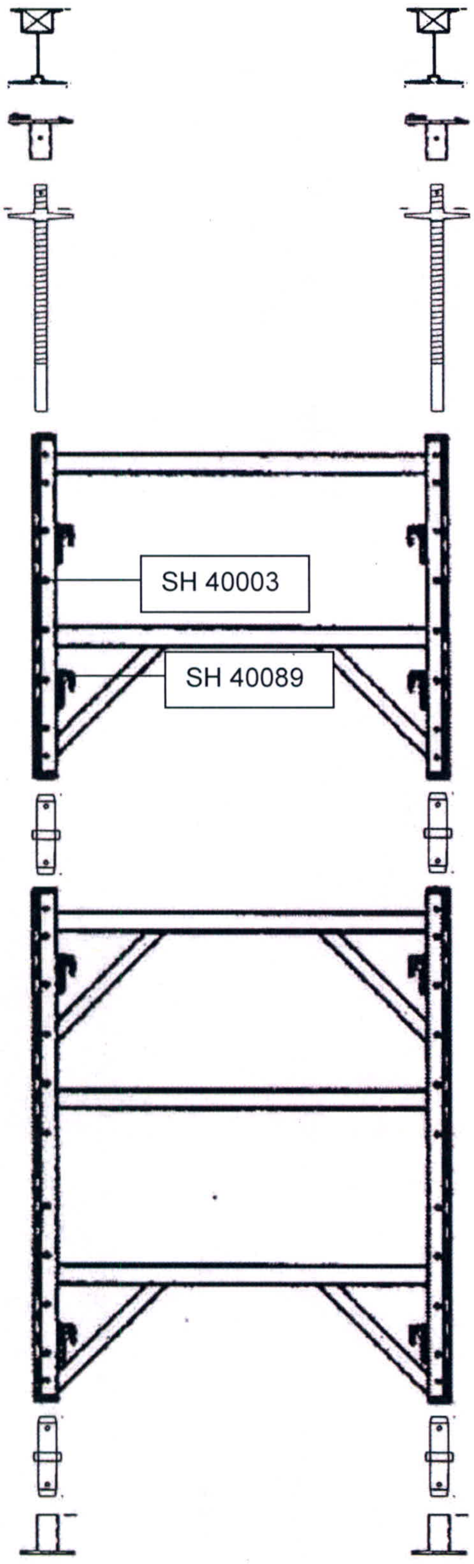
Folder: 08-02-16; SH 40088

SH40088_2



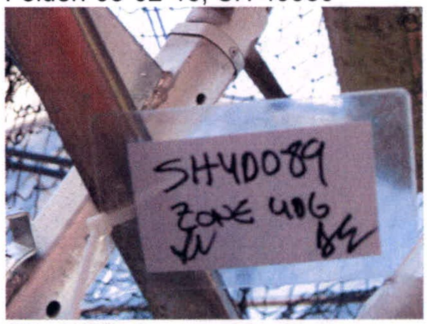
SH40088_4

SH40088_3

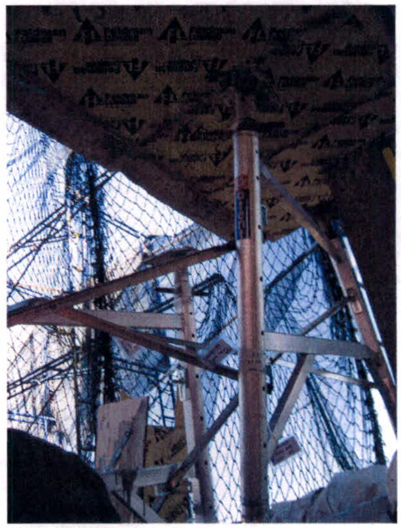


SH 40089
X Brace
Connects SH 40003 to SH 40004

Folder: 08-02-16; SH 40089

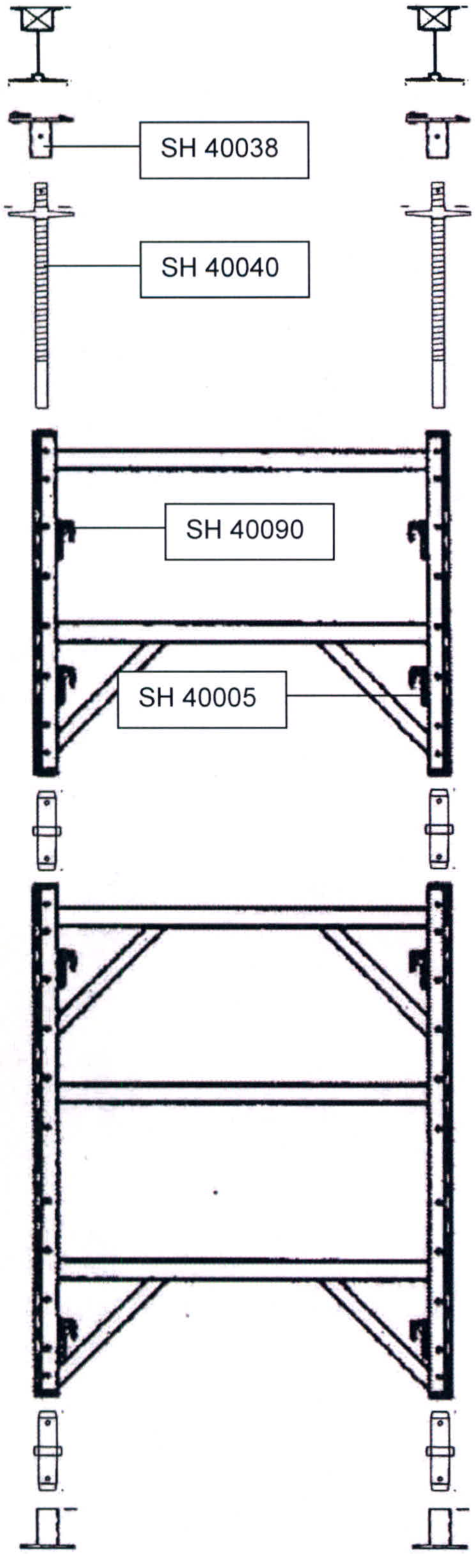


SH40089_1



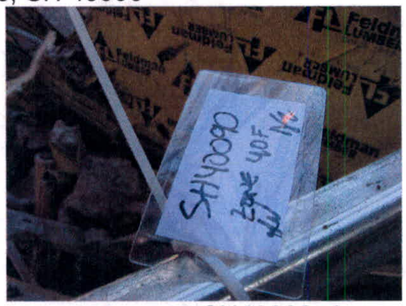
SH40089_2

SH40089_5



SH 40090
X Brace
Connects SH 40005 to SH 40020
Zone 40F

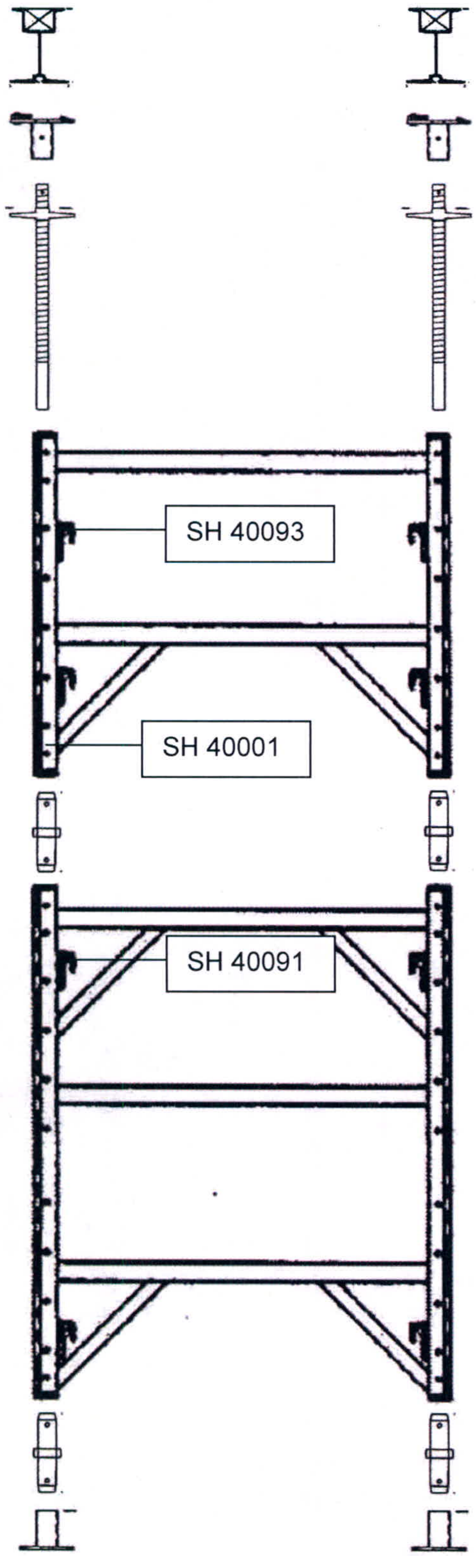
Folder: 08-02-16; SH 40090



SH40090_1

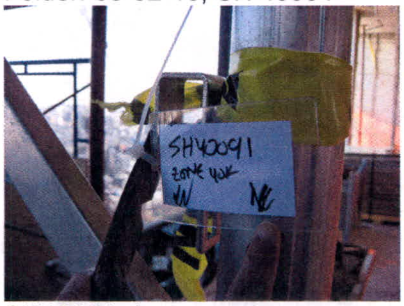
SH40090_2

SH40090_3



SH 40091
Bottom X Brace
Top Brace is SH 40093 (Connects SH 40001 and SH 40002)
Zone 40K

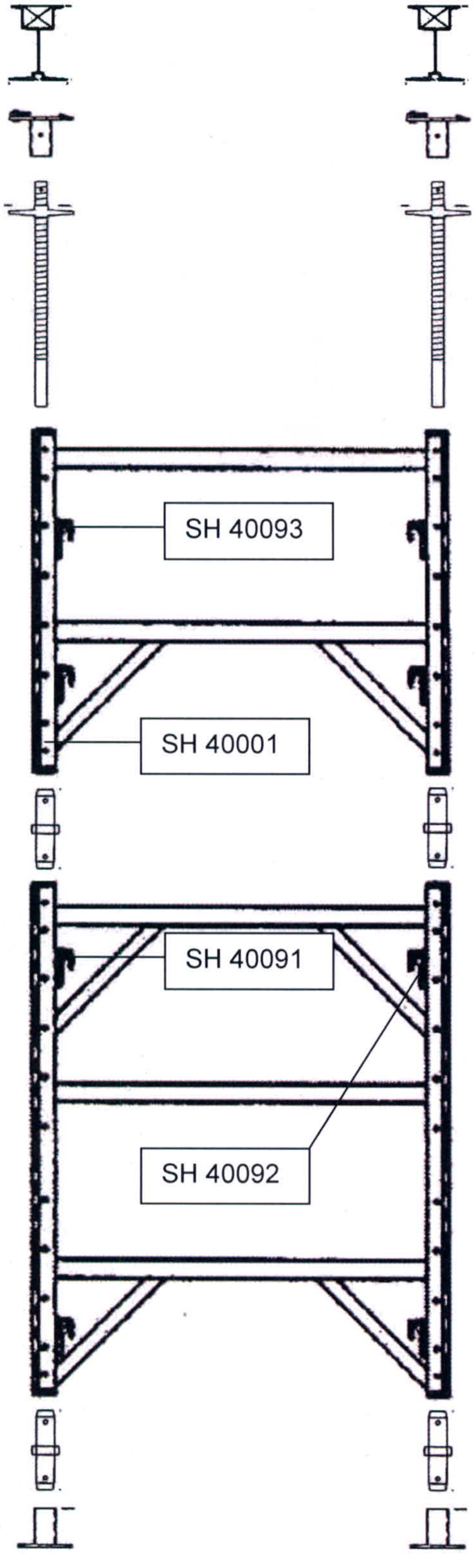
Folder: 08-02-16; SH 40091



SH40091_1

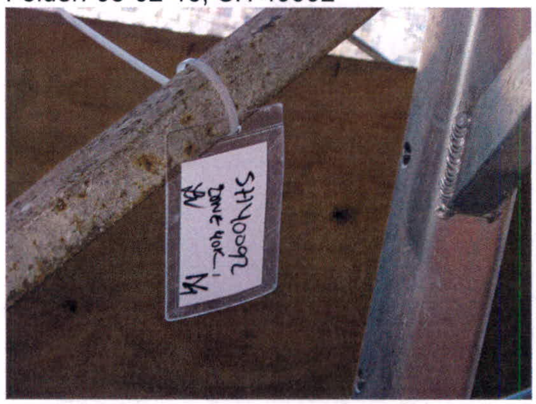
SH40091_3

SH40091_2



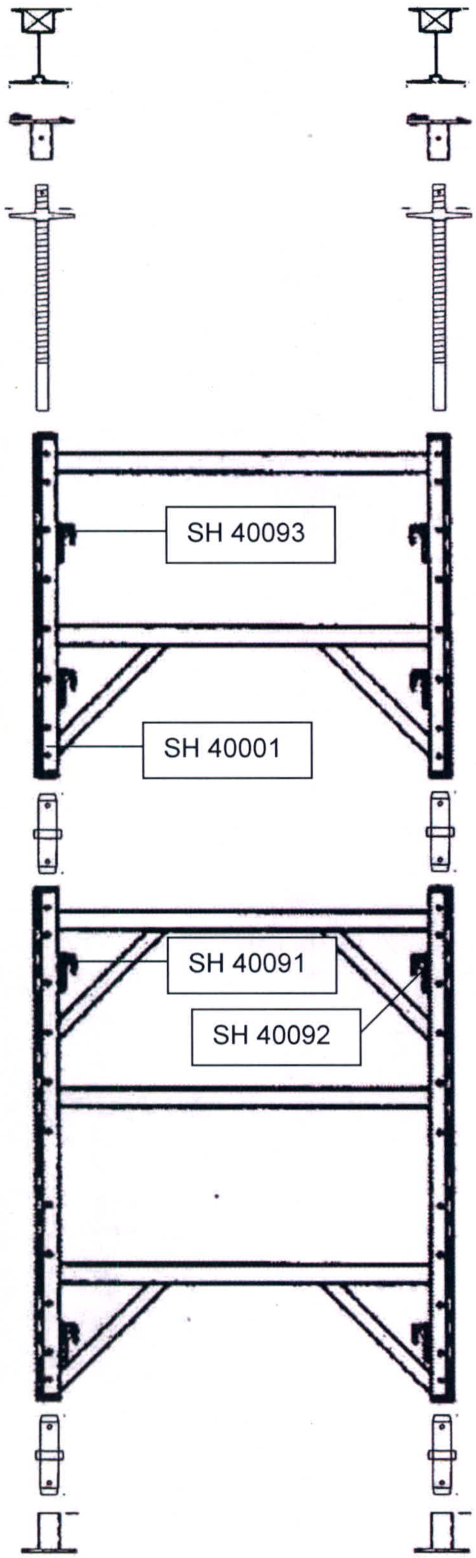
SH 40092
Bottom X Brace parallel to SH 40091
Zone 40K

Folder: 08-02-16; SH 40092



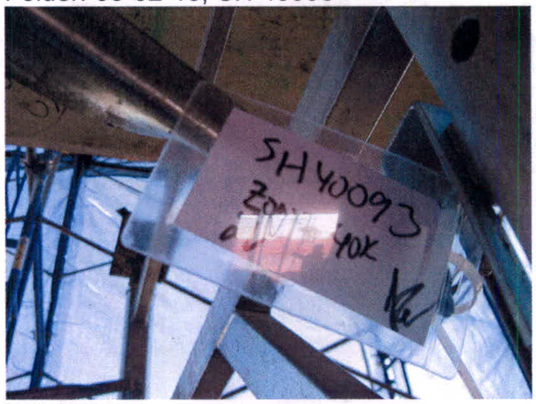
SH40092_1

SH40092_2



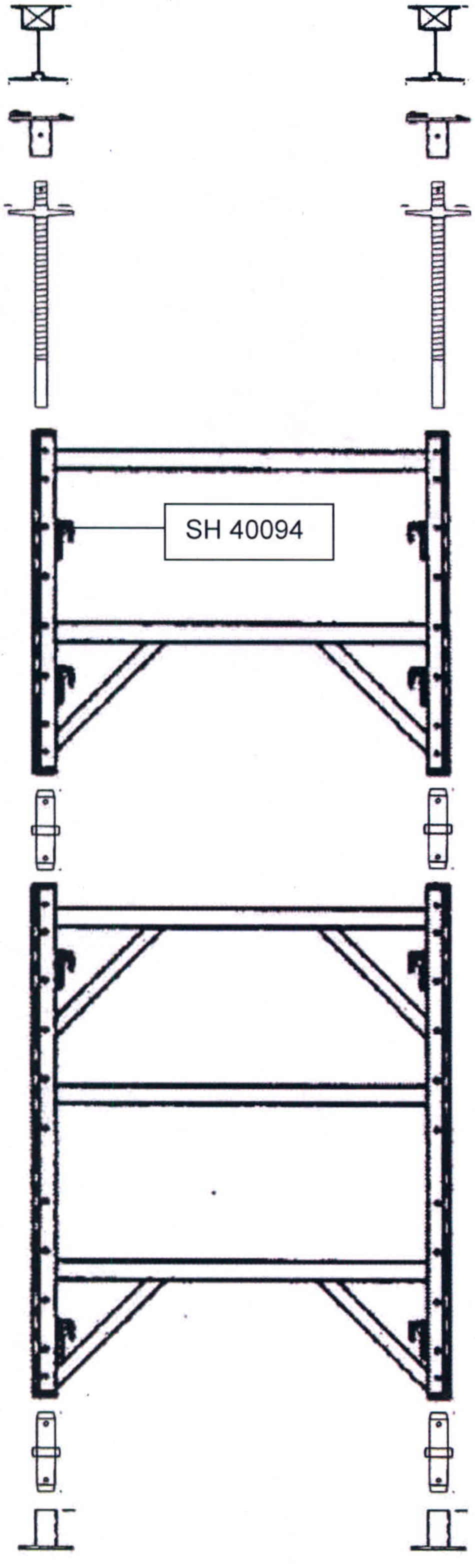
SH 40093
Top X Brace connects SH 40001 & SH 40002
Zone 40K

Folder: 08-02-16; SH 40093



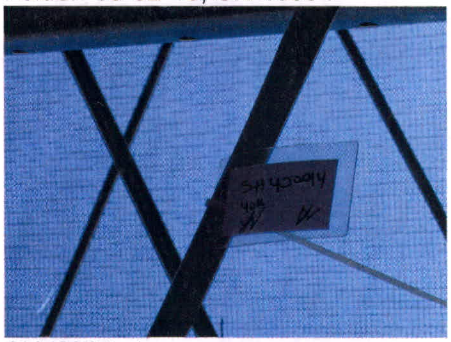
SH40093_1

SH40093_2



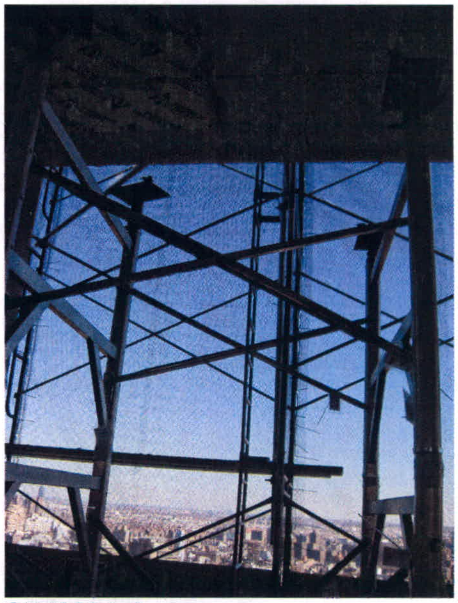
SH 40094
X Brace
Zone 40K

Folder: 08-02-16; SH 40094

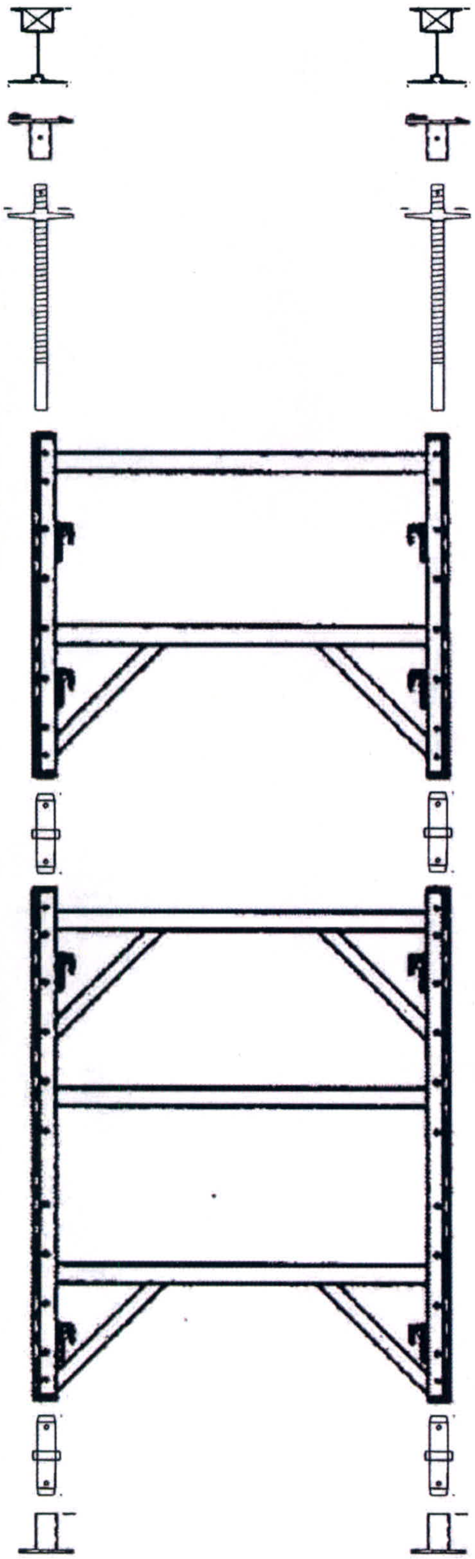


SH40094_1

SH40094_3

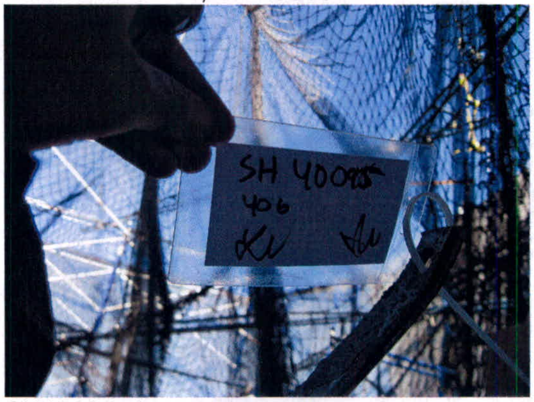


SH40094_2



SH 40095
X Brace
Leaning on SH 40004
Zone 40K

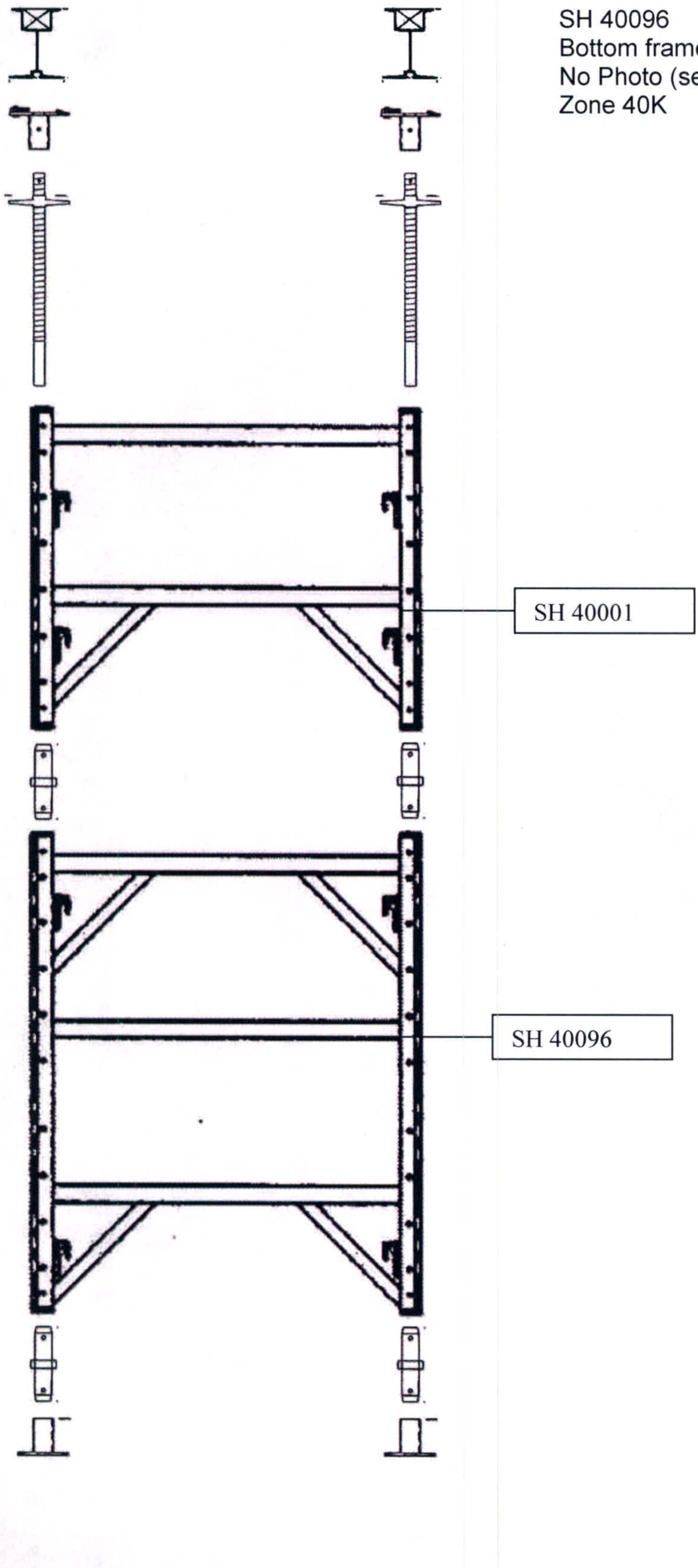
Folder: 08-02-16; SH 40095



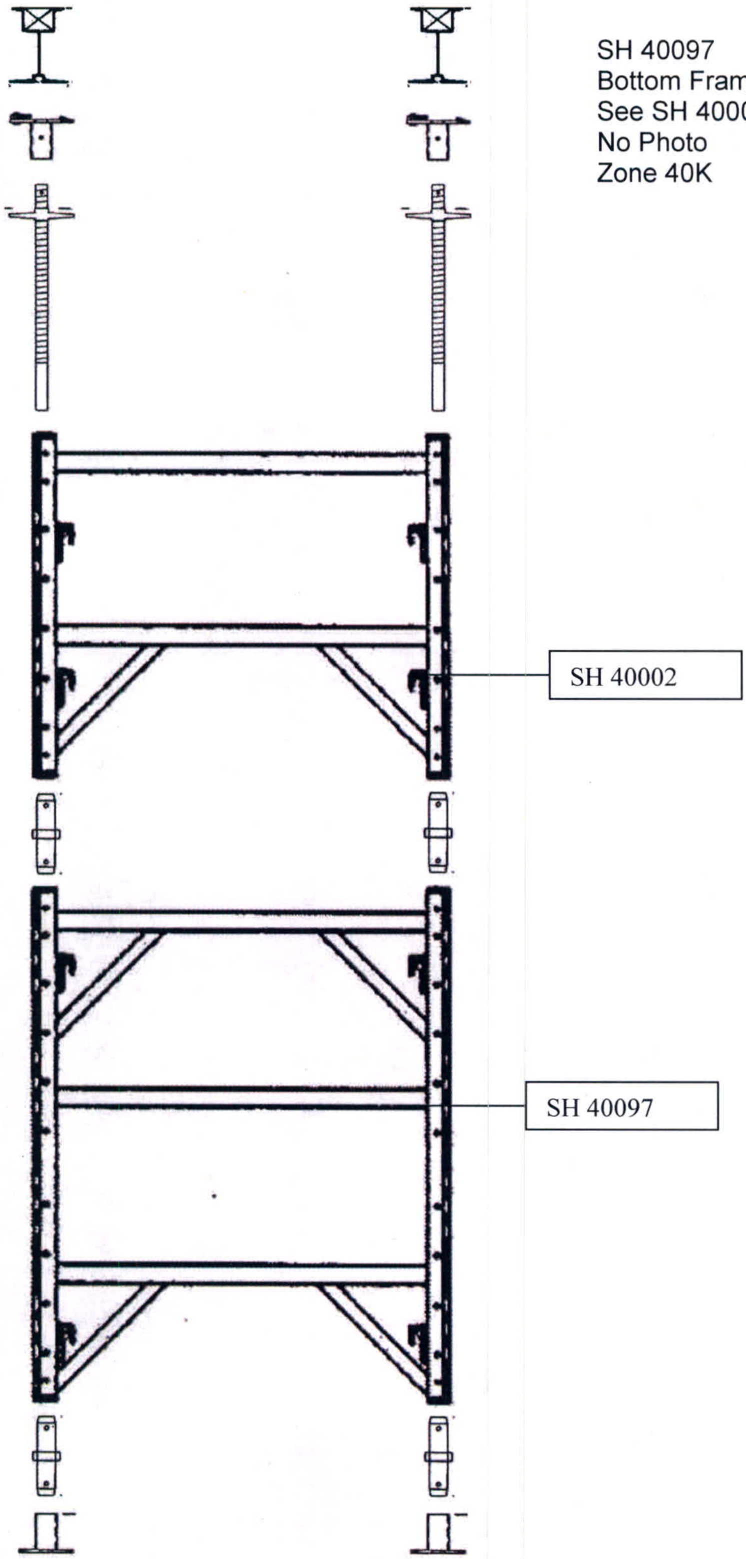
SH40095_1

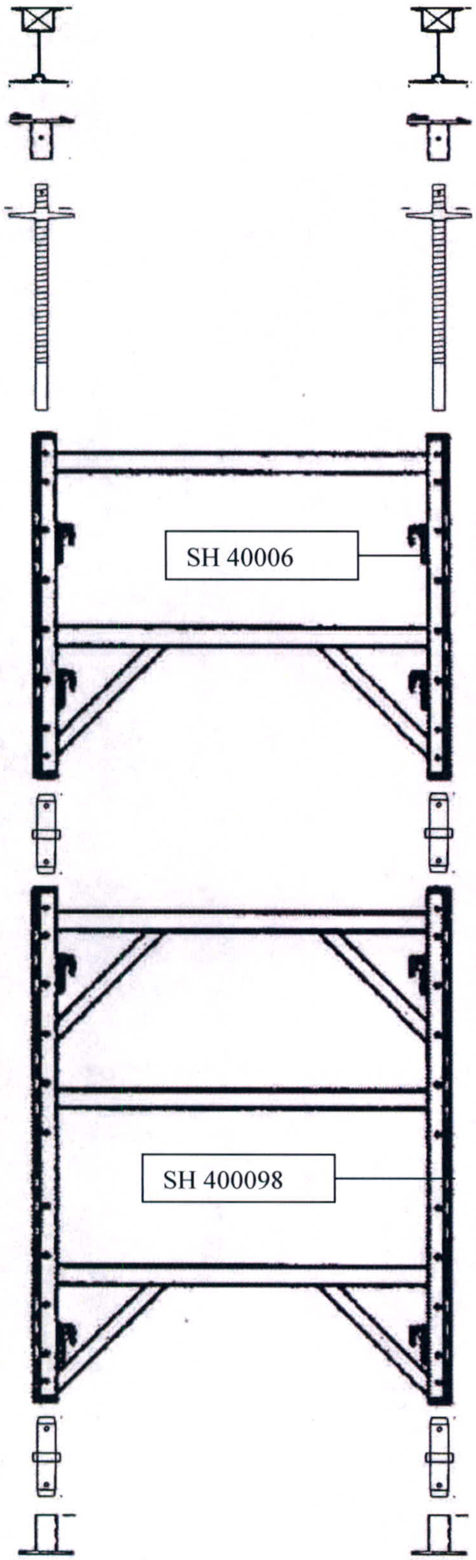
SH40095_2

SH 40096
Bottom frame of SH 40001
No Photo (see SH 40001 for photos)
Zone 40K



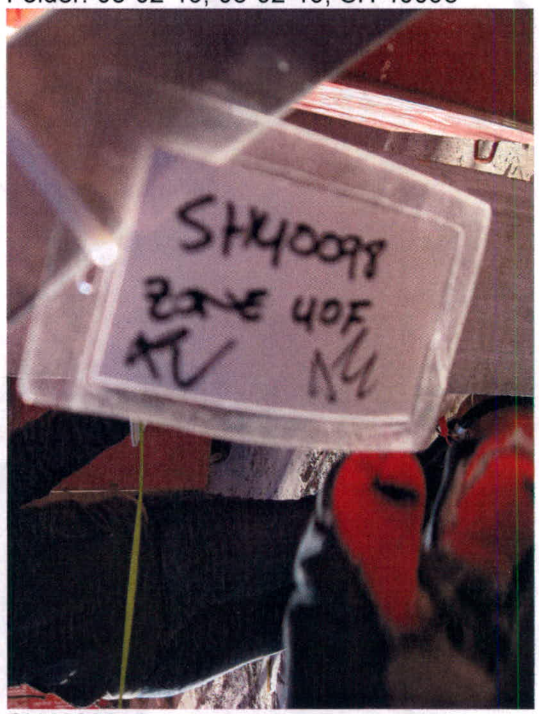
SH 40097
Bottom Frame of SH 40002
See SH 40002 for photos
No Photo
Zone 40K





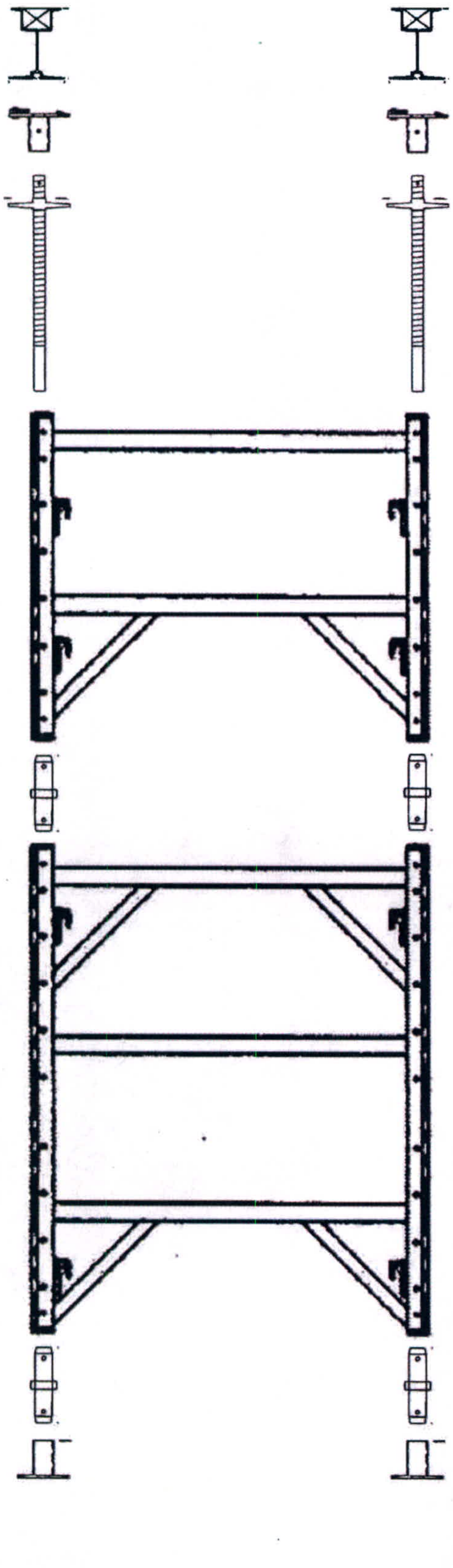
SH 40098
Bottom Frame of SH 40006
Zone 40F

Folder: 08-02-19; 08-02-19; SH 40098



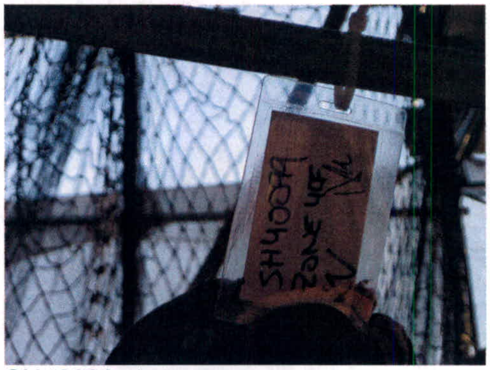
SH40098_1

SH40098_2



SH 40099
Top East Brace
Attached to SH 40007
Relocated from original position
See SH 40007
Zone 40F

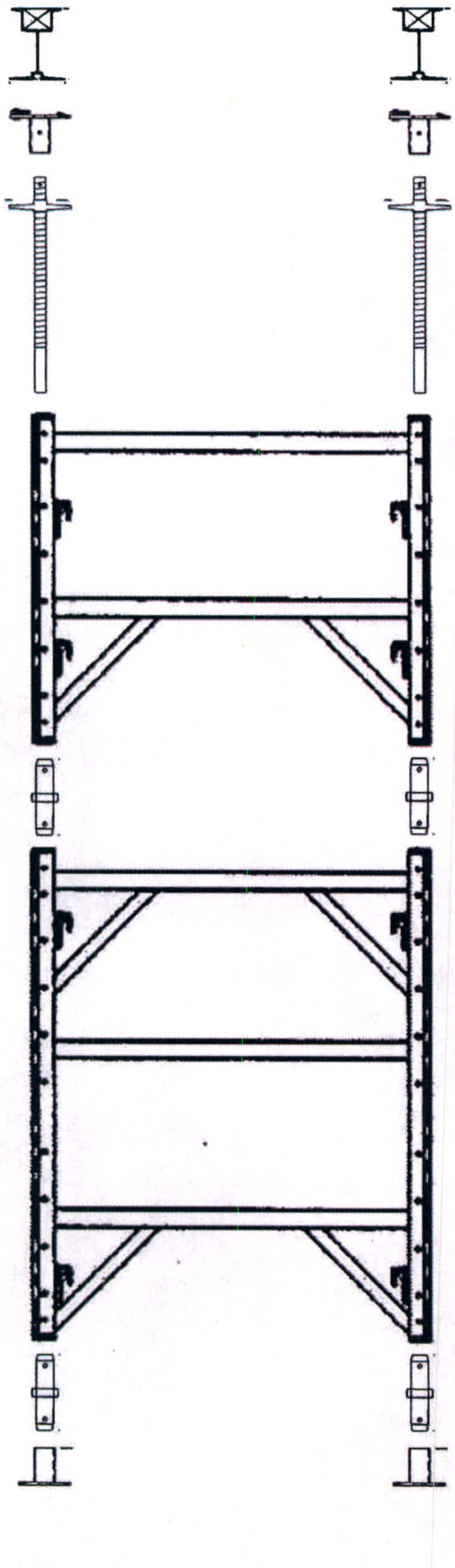
Folder: 08-02-19; 08-02-19; SH 40099



SH40099_1

SH40099_2

SH40099_3



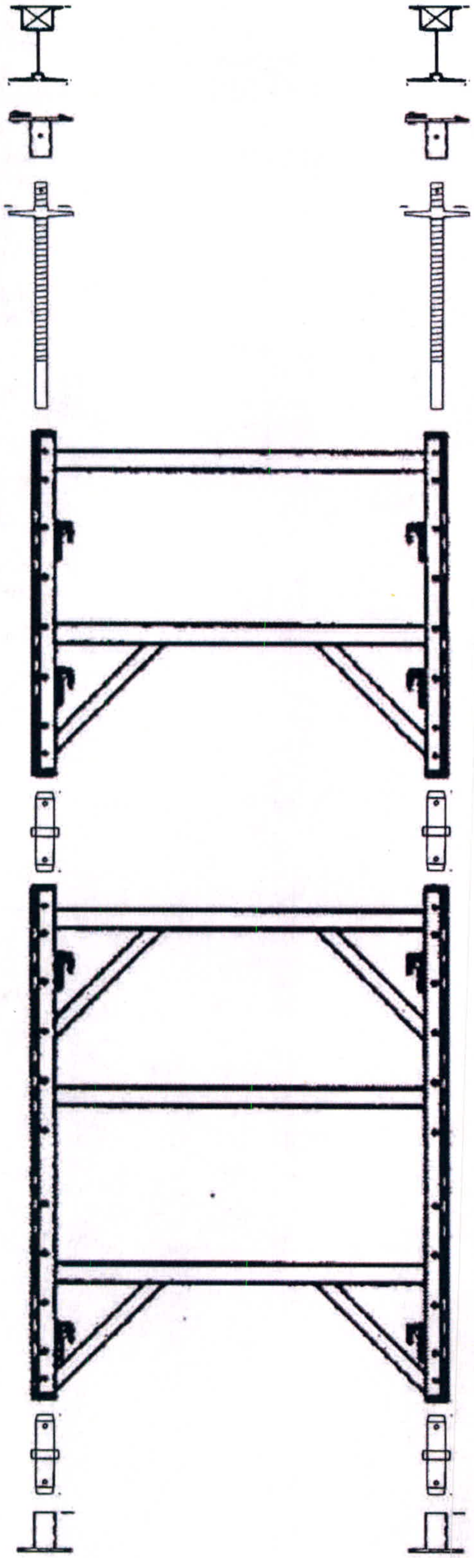
SH 40100
Top X Brace of SH 40007
Zone 40F

Folder: 08-02-19; 08-02-19; SH 40100

SH40100_1

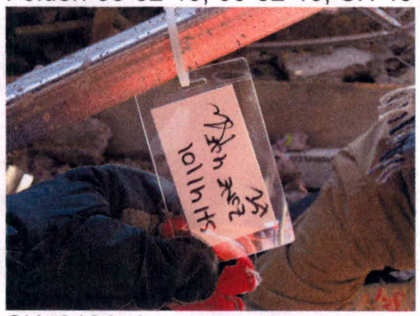
SH40100_2

SH40100_3



SH 40101
Bottom X Brace for Bottom Frame SH 40102
Zone 40F

Folder: 08-02-19; 08-02-19; SH 40101

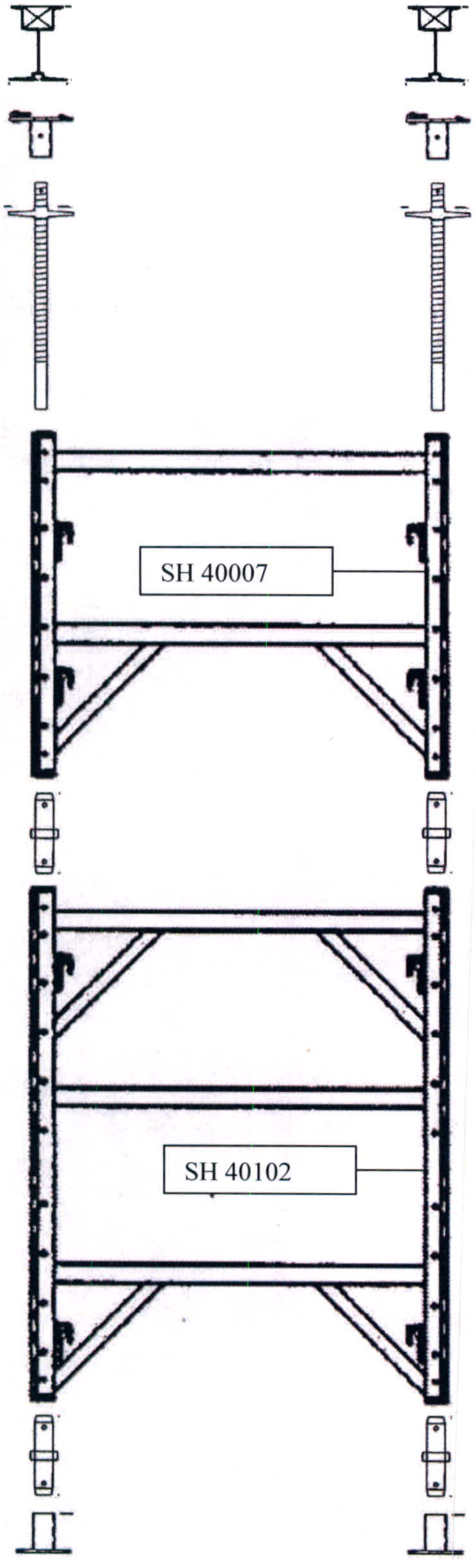


SH40101_1

SH40101_2

SH40101_3

SH 40102
Bottom Frame of SH 40007
Connected to Brace SH 40101
Zone 40F



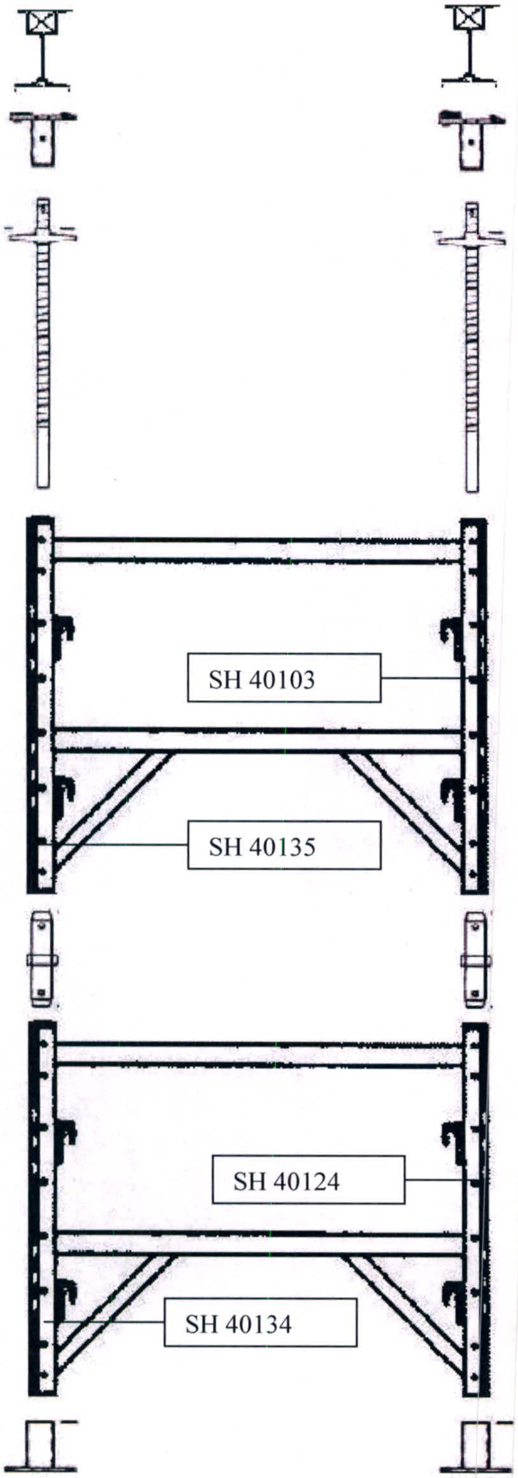
Folder: 08-02-19; 08-02-19; SH 40102

SH40102_1


SH40102_2


SH40102_3

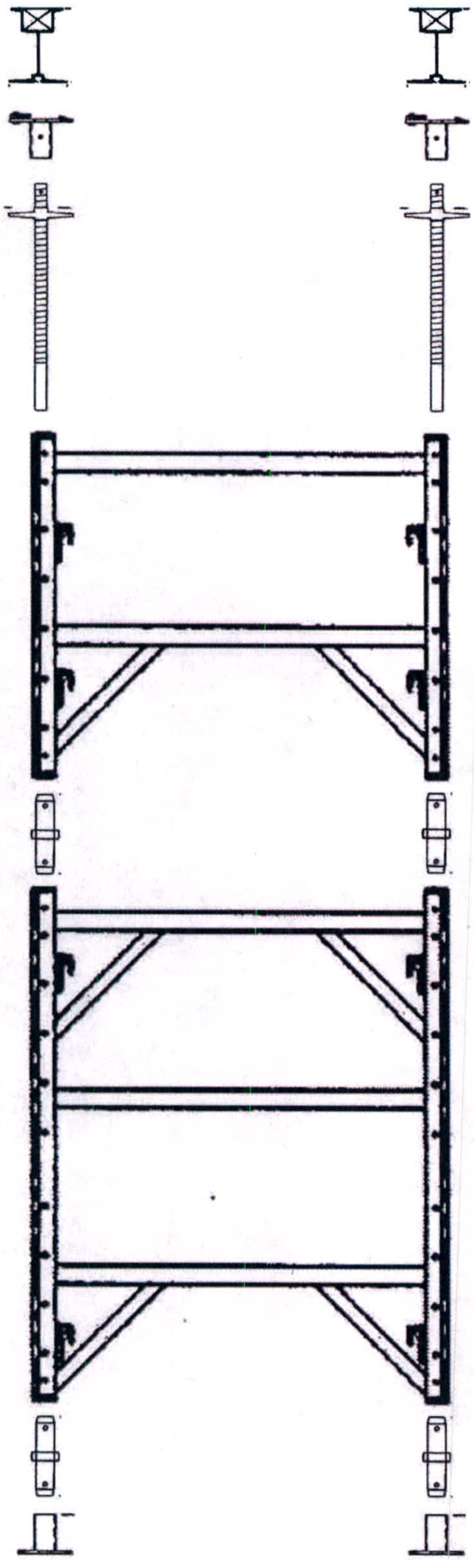
SH 40103 on 41st floor
Top Shore of SH 40124
Support AB 40014
Zone 40F



Folder: 08-02-19; SH 40103

SH40103_4

SH40103_2



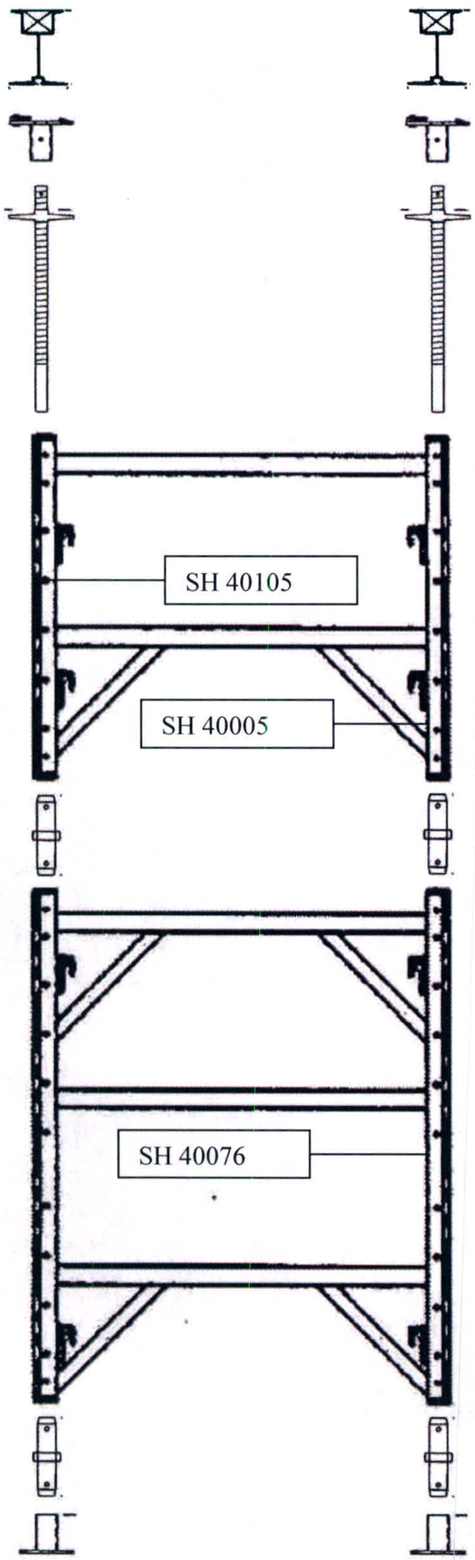
SH 40104
X Brace collapsed near SH 40103
Zone 40F

Folder: 08-02-19; 08-02-19; SH 40104

SH40104_1

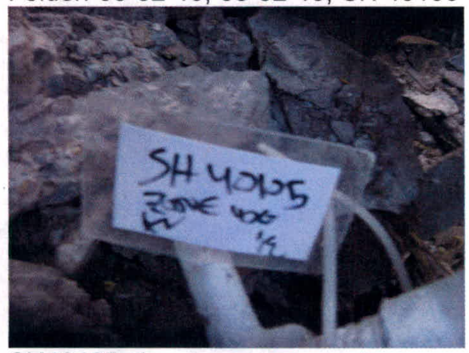
SH40104_2

SH40104_3



SH 40105
Attached to Screw Jack & Top plate
Supports AB 40042
Zone 40F

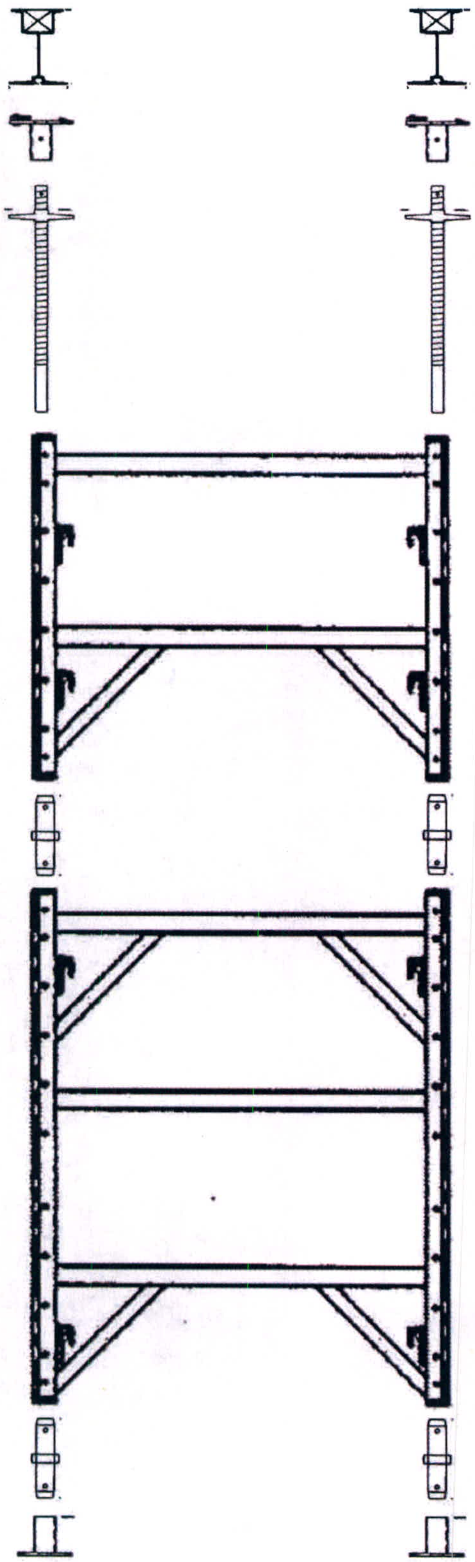
Folder: 08-02-19; 08-02-19; SH 40105



SH40105_1

SH40105_2

SH40105_3

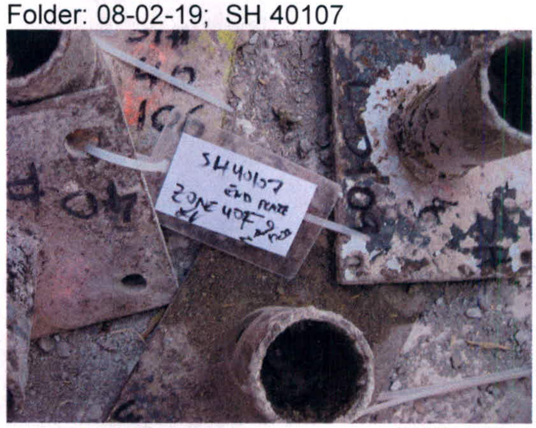


SH 40106-SH 40108

SH 40106 Bottom Plate
Zone 40F
Folder: 08-02-19; SH 40106

SH40106_1

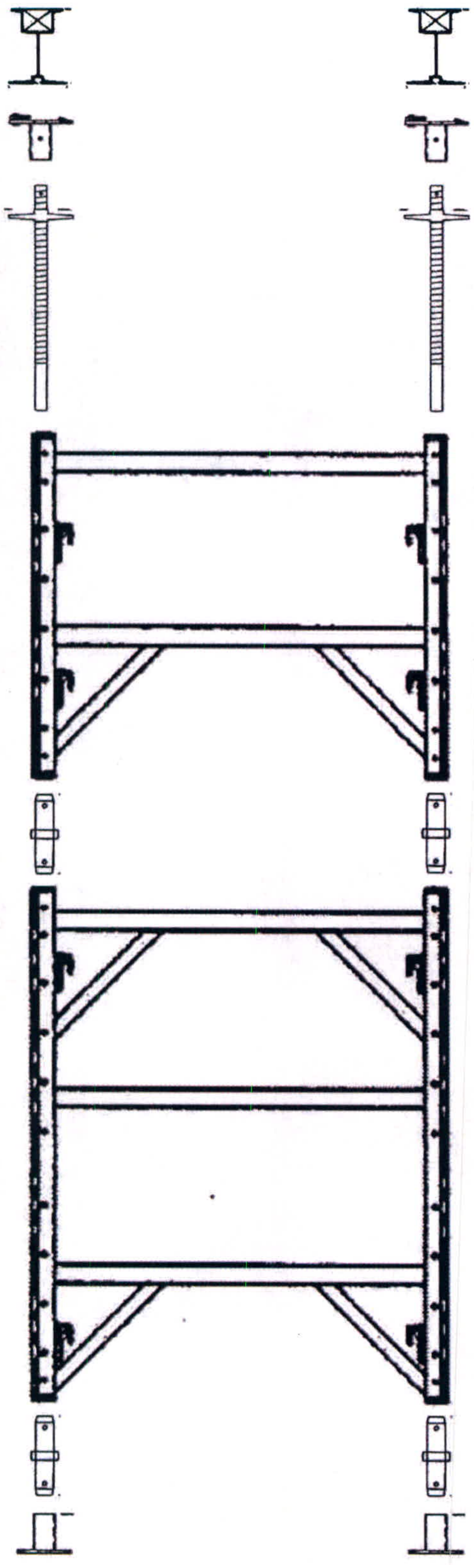
SH 40107
Bottom Plate
Zone 40F
Folder: 08-02-19; SH 40107



SH40107_1

SH 40108
Bottom Plate
Zone 40F
Folder: 08-02-19; SH 40108

SH40108_1

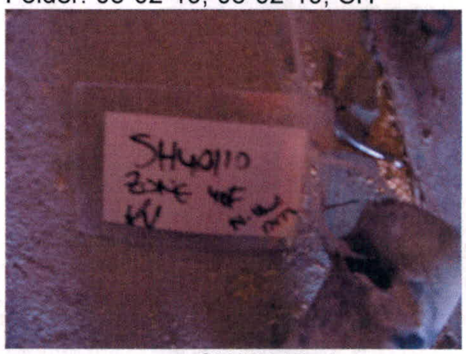


SH 40109 – SH 40110

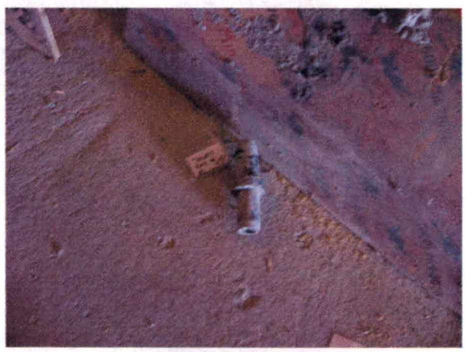
SH 40109
Bottom Plate
Zone 40F
Folder: 08-02-19; 08-02-19; SH

SH40109_1

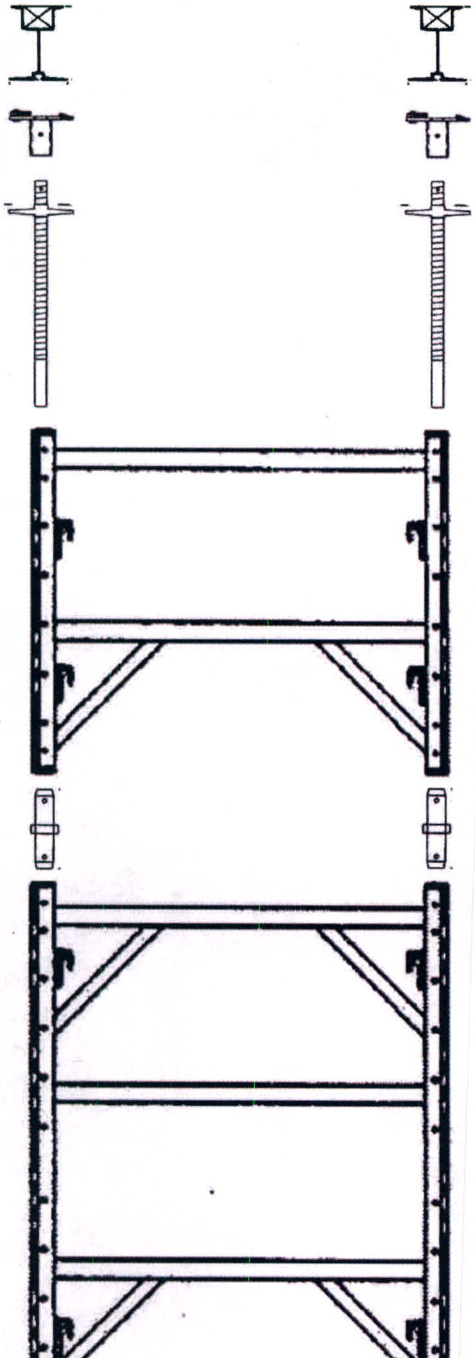
SH 40110
Coupler
Moved from original position
Zone 40F
Folder: 08-02-19; 08-02-19; SH



SH40110_1

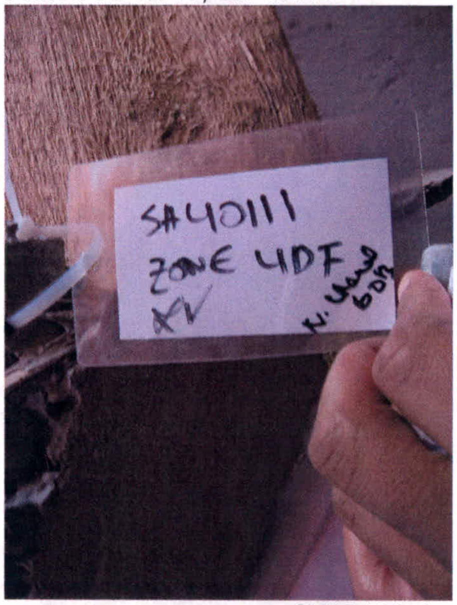


SH40110_2

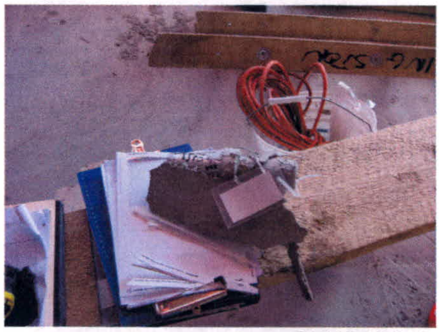


SH 40111
Coupler
Moved from original position
Zone 40F

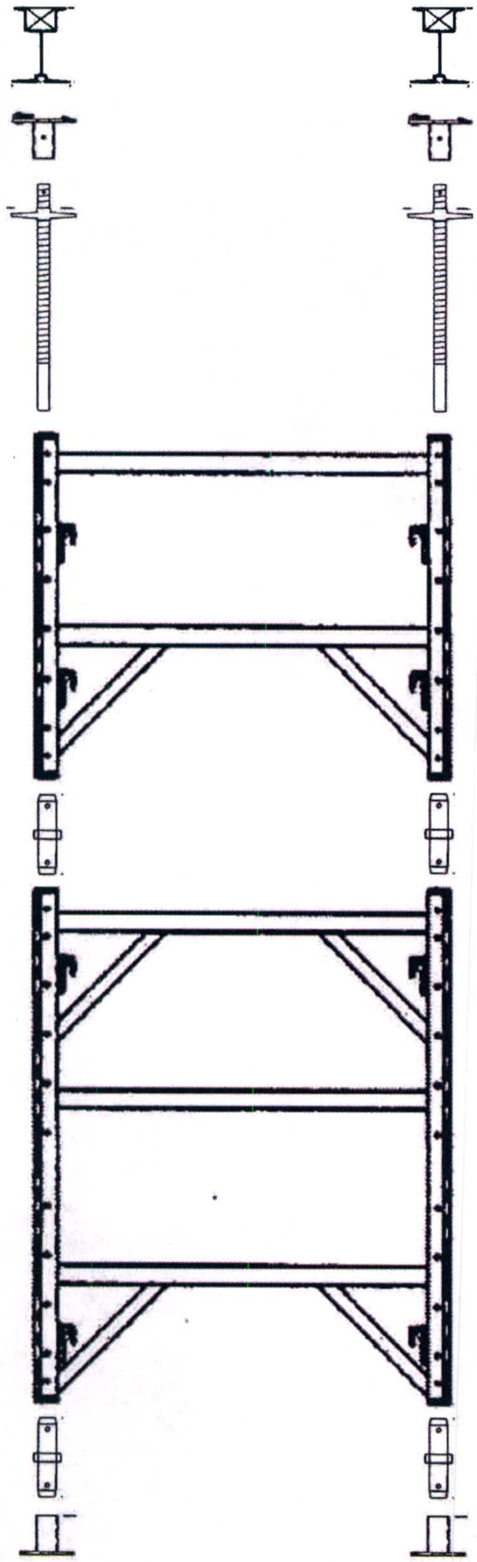
Folder: 08-02-19; SH 40111



SH40111_1



SH40111_2

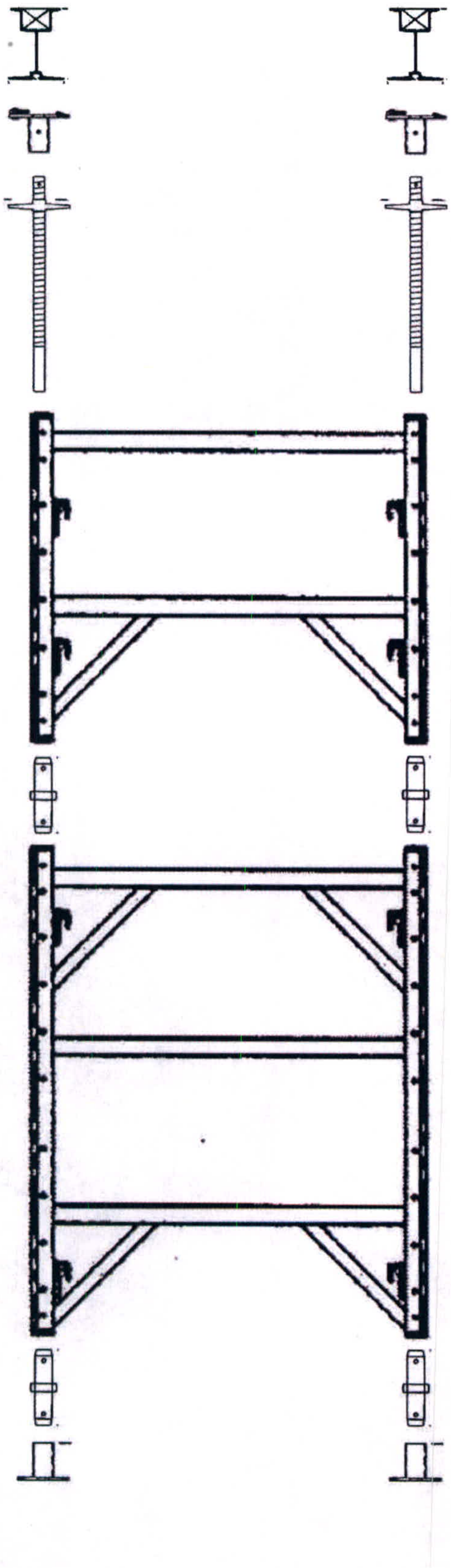


SH 40112
Broken piece of frame
Moved from original position
Zone 40J

Folder: 08-02-19; SH 40112

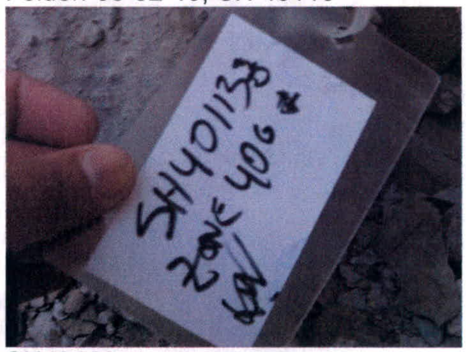
SH40112_1

SH40112_2



SH 40113
Coupler
Zone 40G

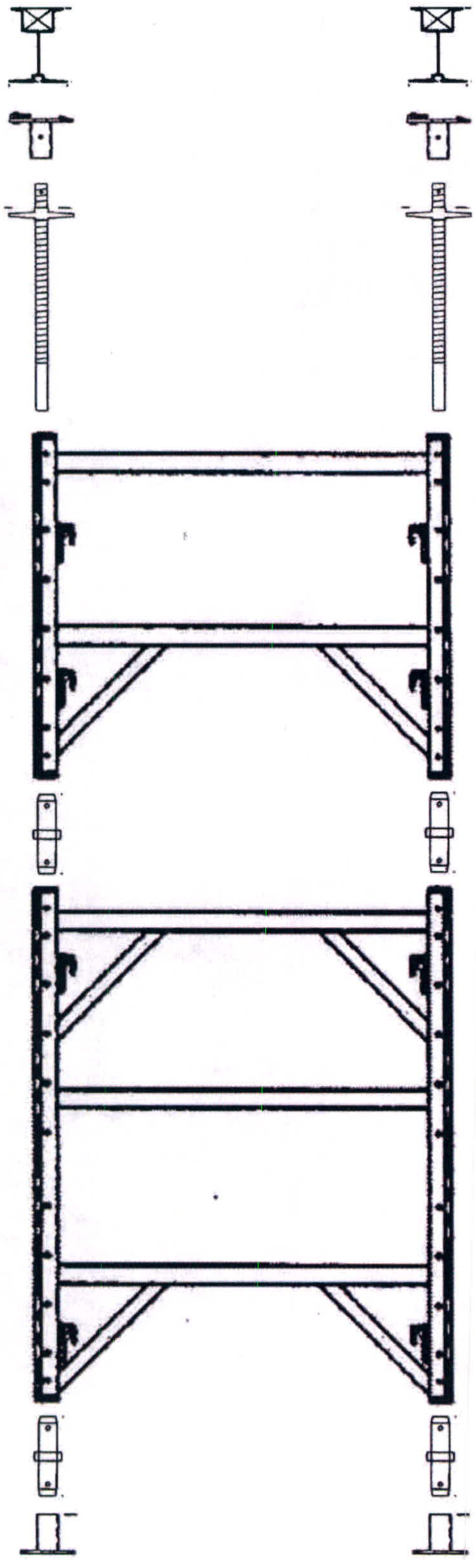
Folder: 08-02-19; SH 40113



SH40113_1

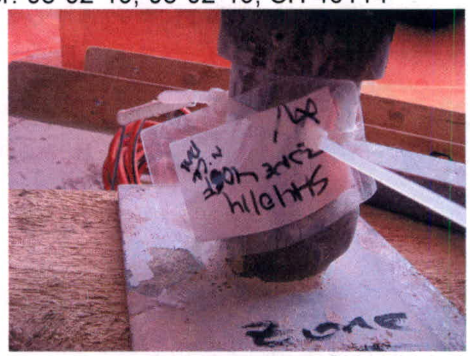

SH40113_2

SH40113_3

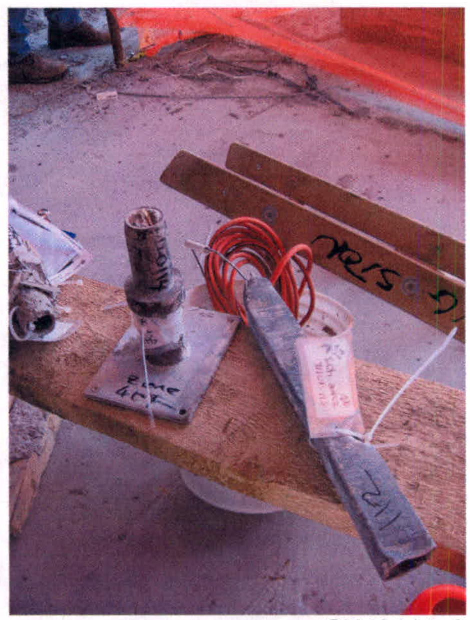


SH 40114
Coupler with Bottom Plate
Zone 40F

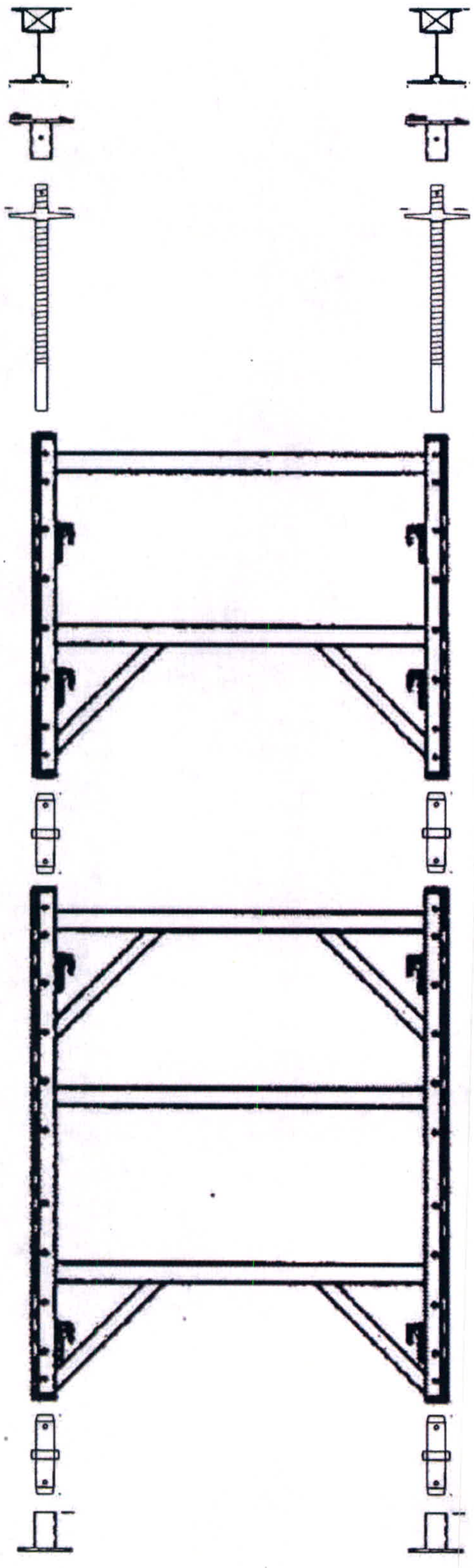
Folder: 08-02-19; 08-02-19; SH 40114



SH40114_1



SH40114_2

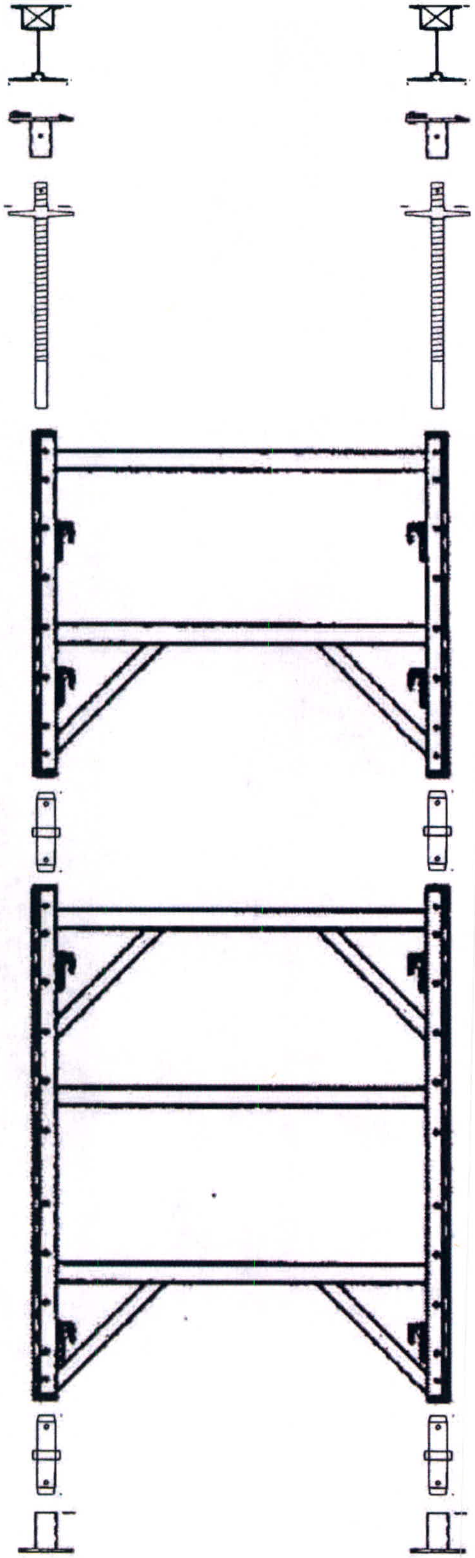


SH 40115
Coupler
Zone 40G

Folder: 08-02-19; 08-02-19; SH 40115

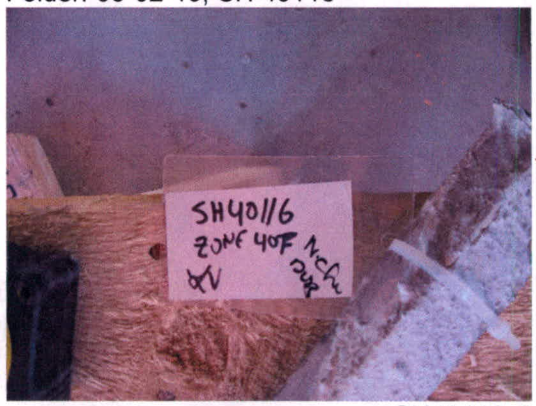
SH40115_1

SH40115_2



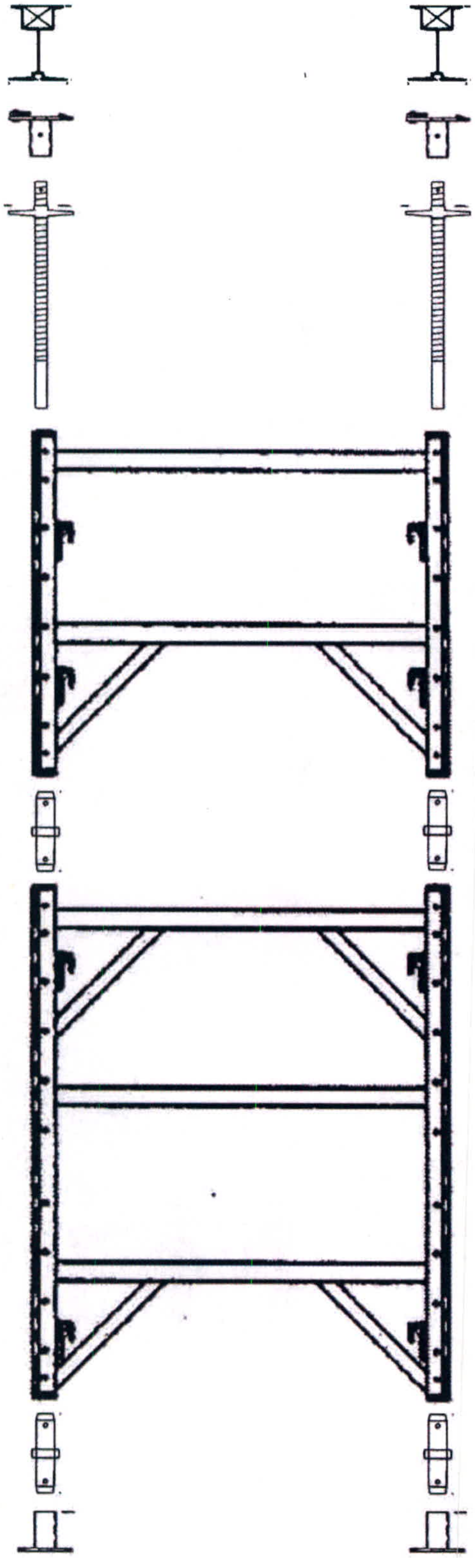
SH 40116
Piece of frame
Moved from original position
Zone 40F

Folder: 08-02-19; SH 40116



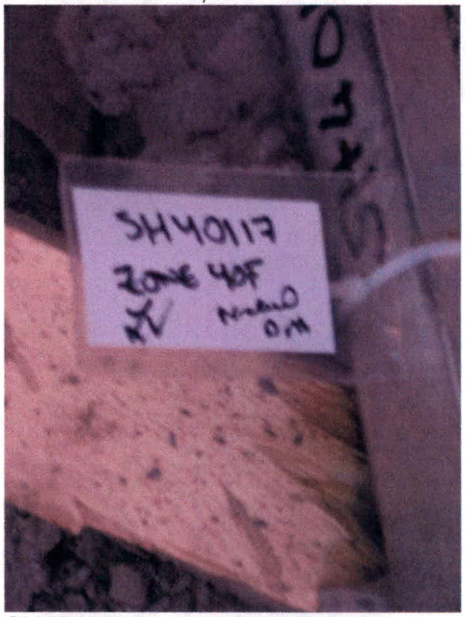
SH40116_1

SH40116_2



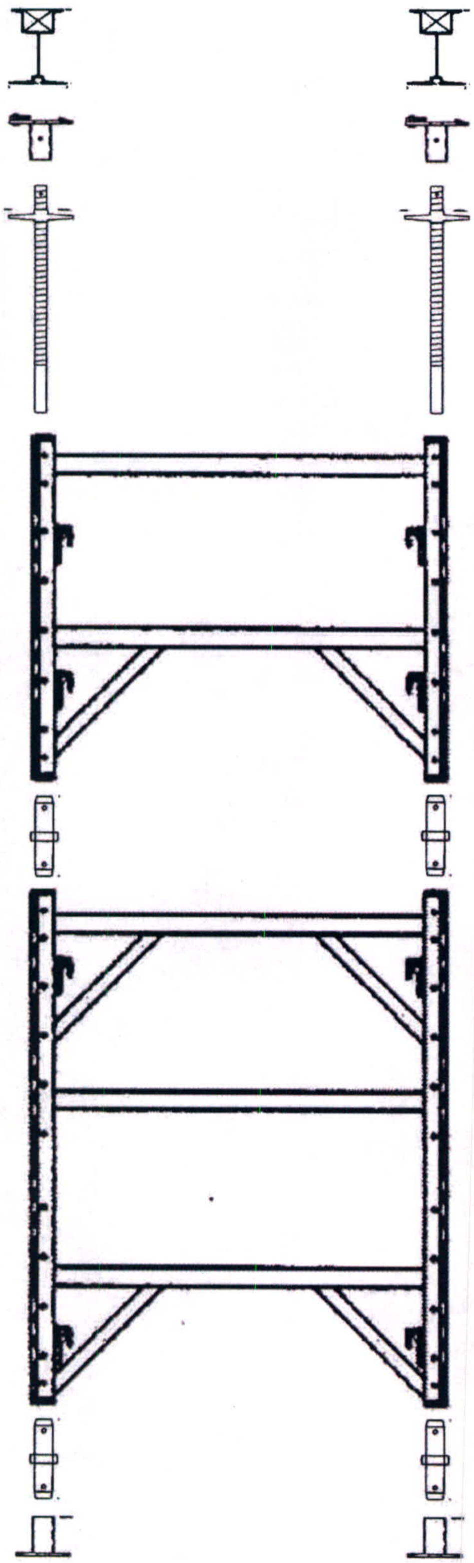
SH 40117
Broken piece of frame found under SH 40088 (retagged)
Zone 40F

Folder: 08-02-19; SH 40117



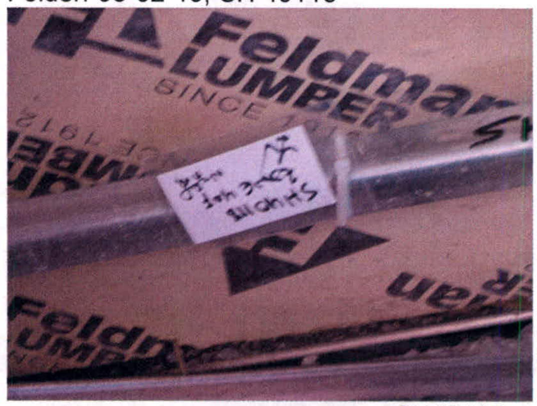
SH40117_1

SH40117_2



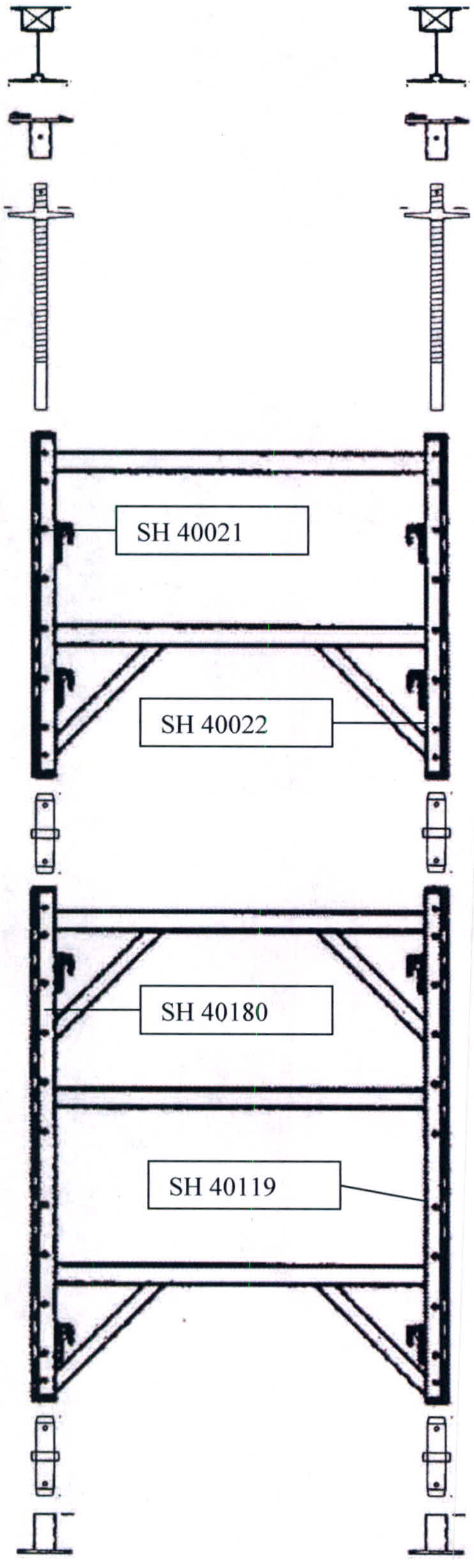
SH 40118
Brace Connected to SH 40088
Zone 40F

Folder: 08-02-19; SH 40118



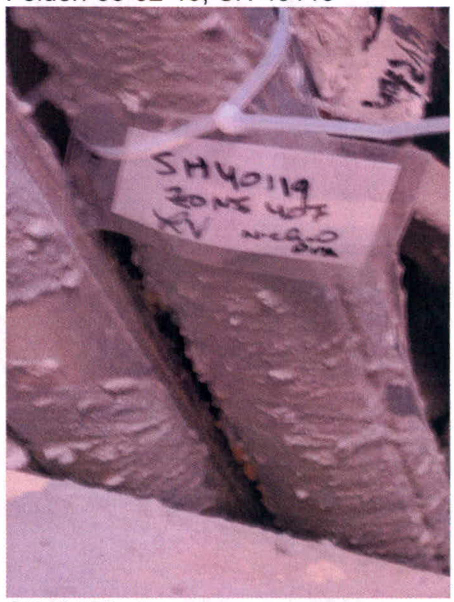
SH40118_1

SH40118_2



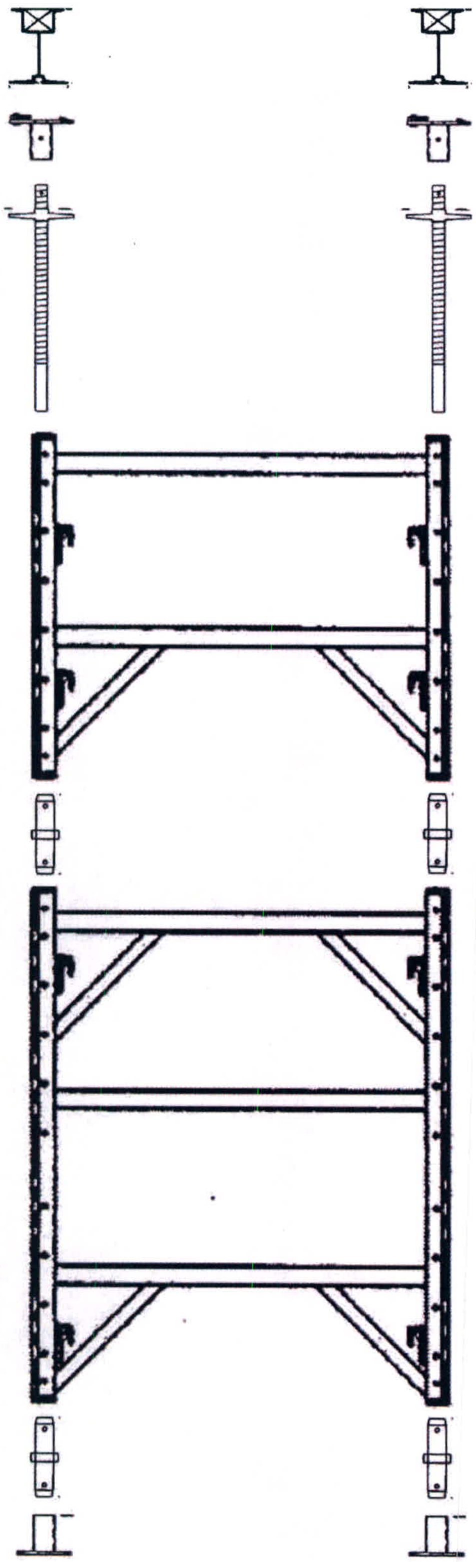
SH 40119 Bottom Shore
Broken with SH 40180
Zone 40F

Folder: 08-02-19; SH 40119



SH40119_1

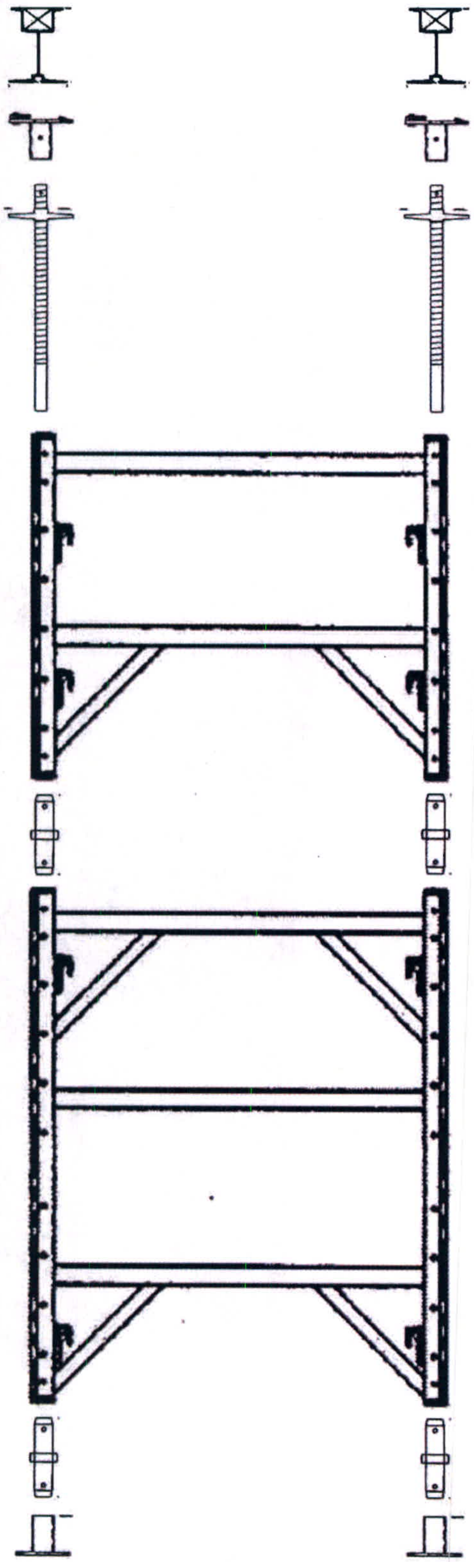
SH40119_2



SH 40120
Coupler
Zone 40F

Folder: 08-02-20; SH 40120

SH40120_1

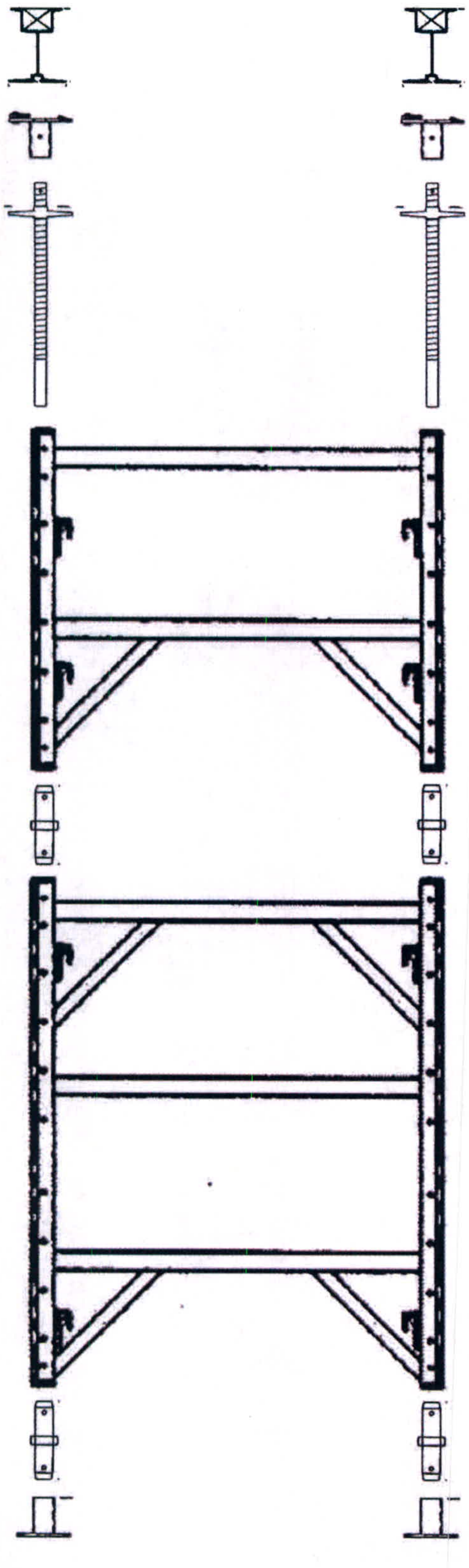


SH 40121
X Brace connect to SH 40119
Zone 40F

Folder: 08-02-19; SH 40121

SH40121_2

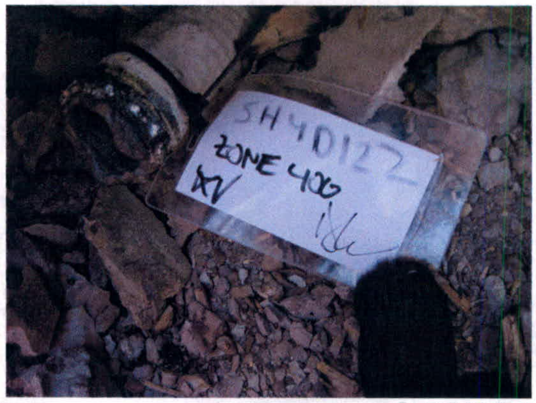
SH40121_3



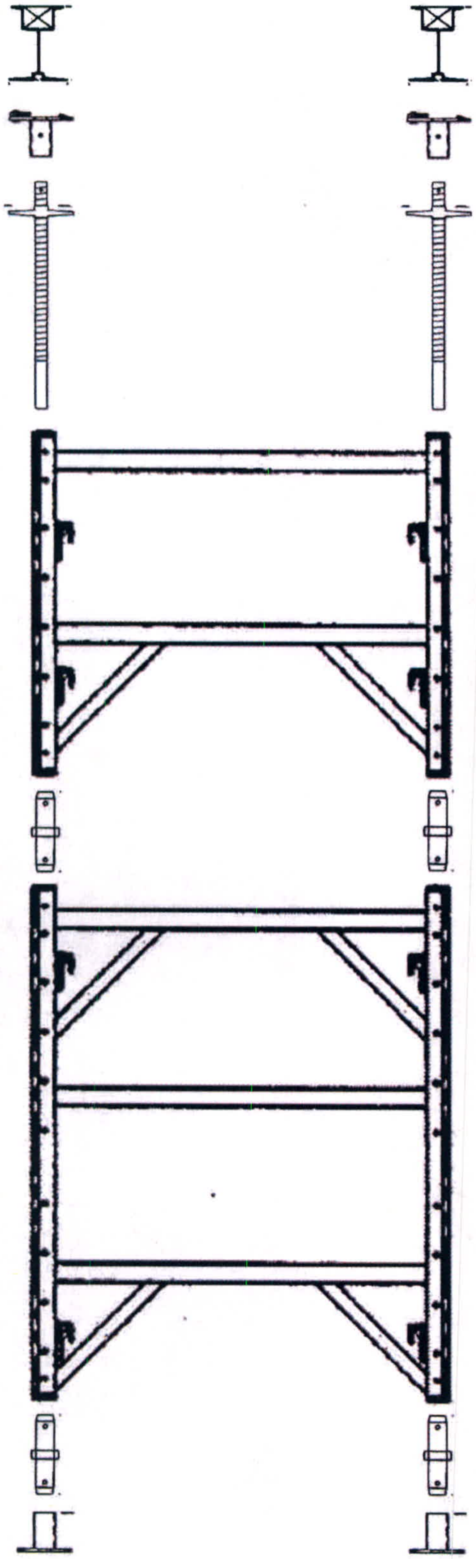
SH 40122
Screw Jack cut
Zone 40G

Folder: 08-02-20; SH 40122

SH40122_2



SH40122_1

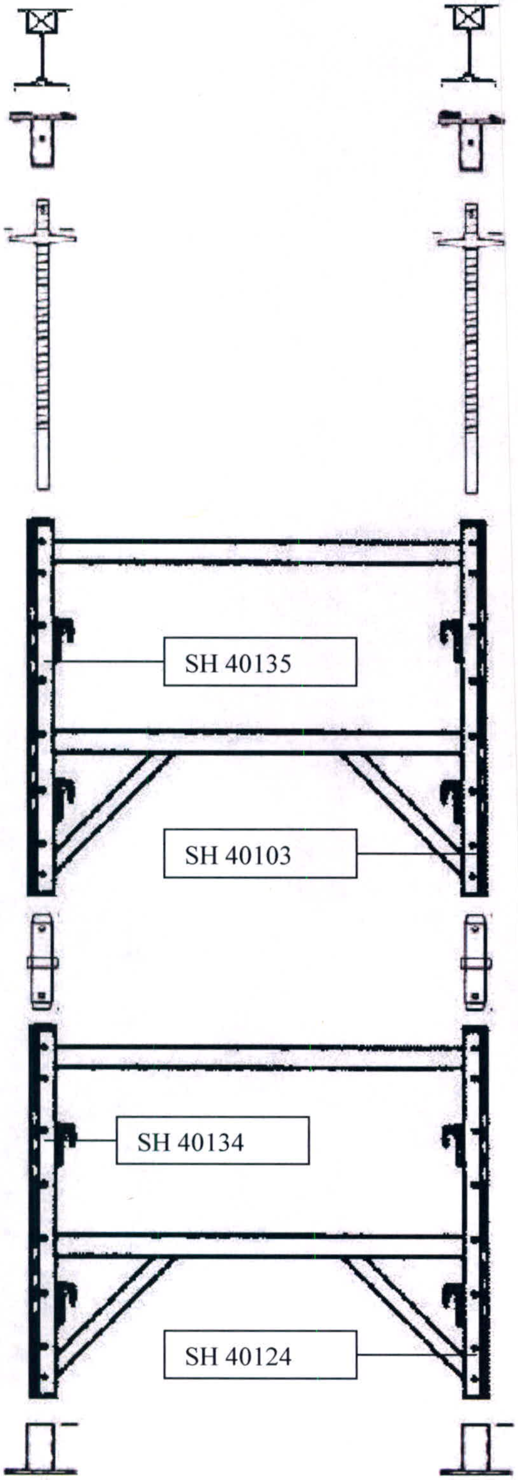


SH 40123
X Brace
Zone 40G

Folder: 08-02-20; SH 40123

SH40123_1

SH40123_2

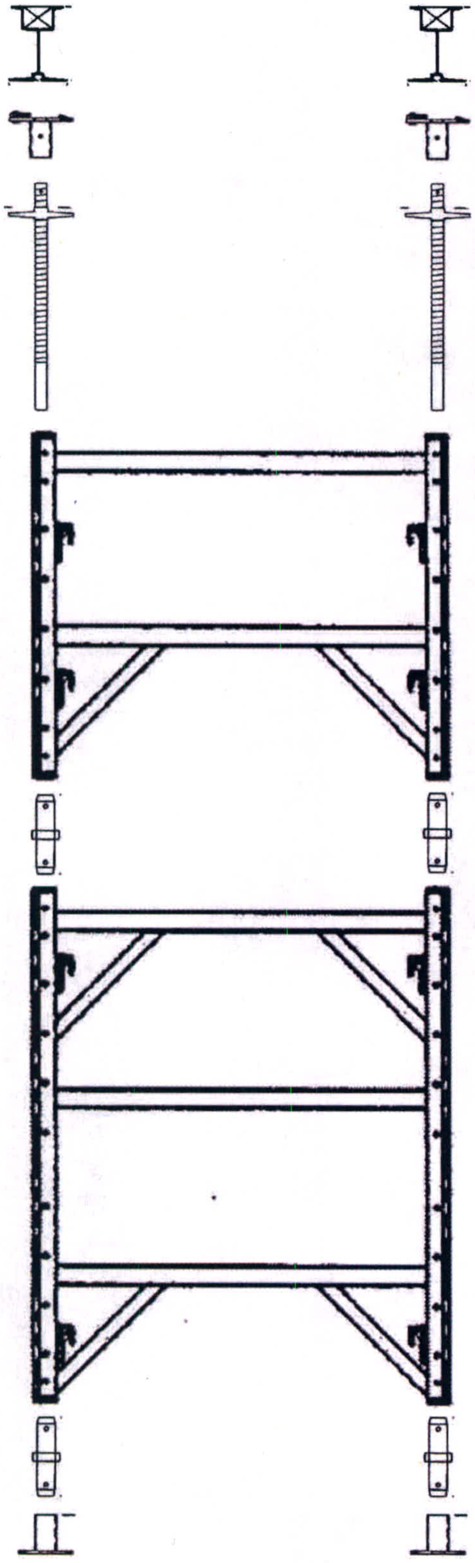


SH 40124 on 41st floor
Bottom frame of SH 40103
Attached to bottom brace SH 40126
Zone 40F

Folder: 08-02-20; SH 40124

SH40124_1

SH40124_3



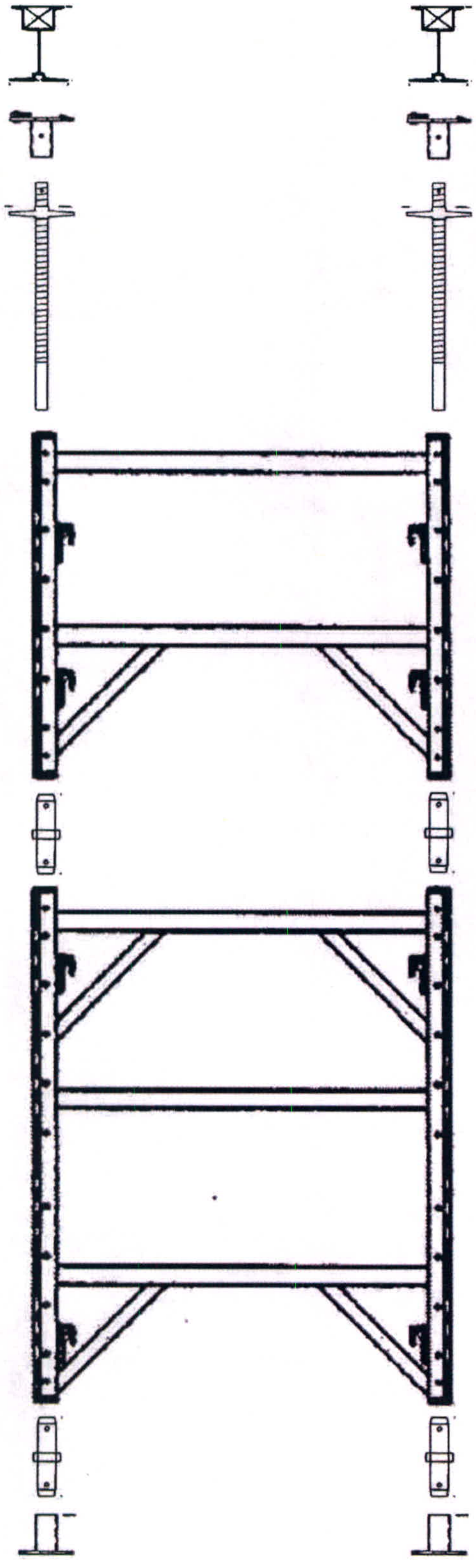
SH 40125
Broken piece near SH 40124
Zone 40G

Folder: 08-02-20; SH 40125

SH40125_1

SH40125_2

SH40125_3

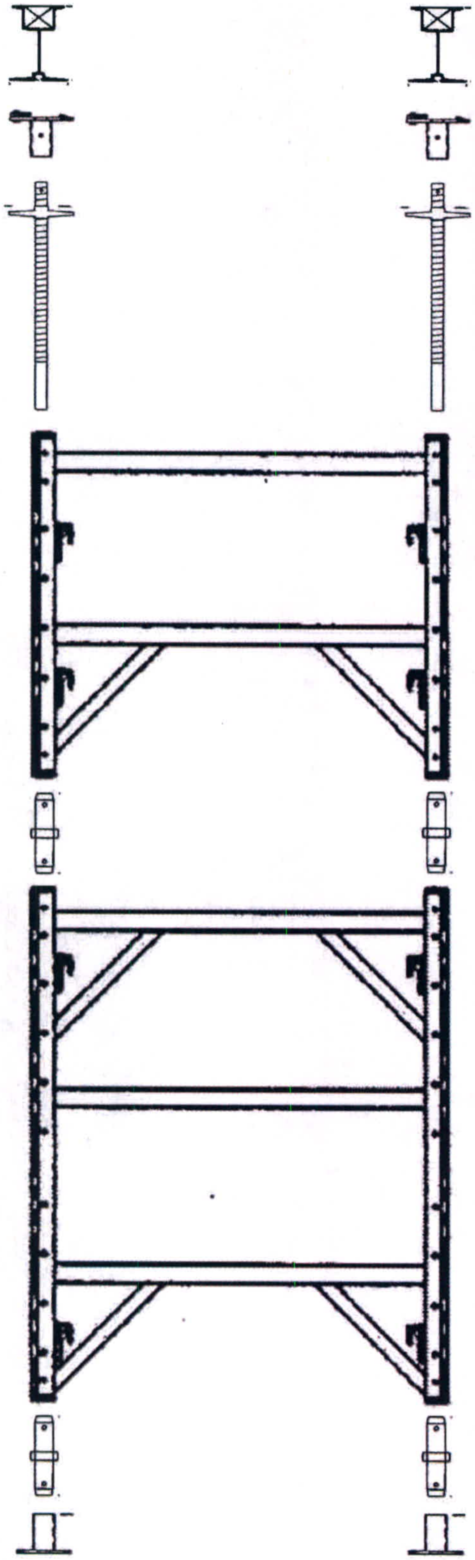


SH 40126
Brace
Zone 40G-F

Folder: 08-02-20; SH 40126

SH40126_2

SH40126_3

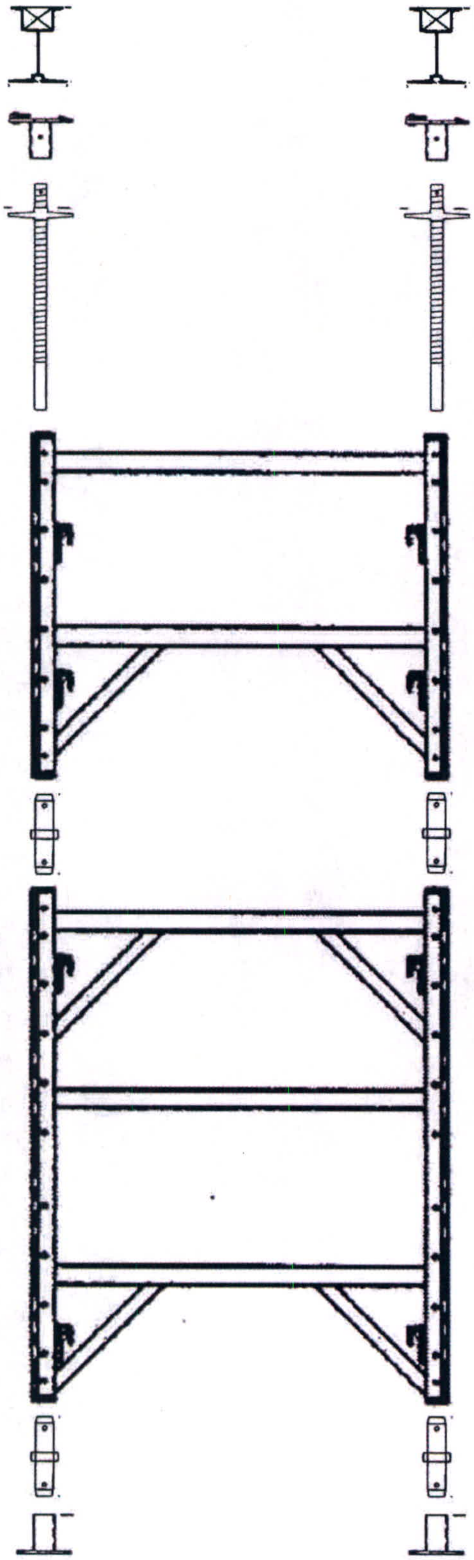


SH 40127
Broken piece
Zone 40F

Folder: 08-02-20; SH 40127

SH40127_1

SH40127_2



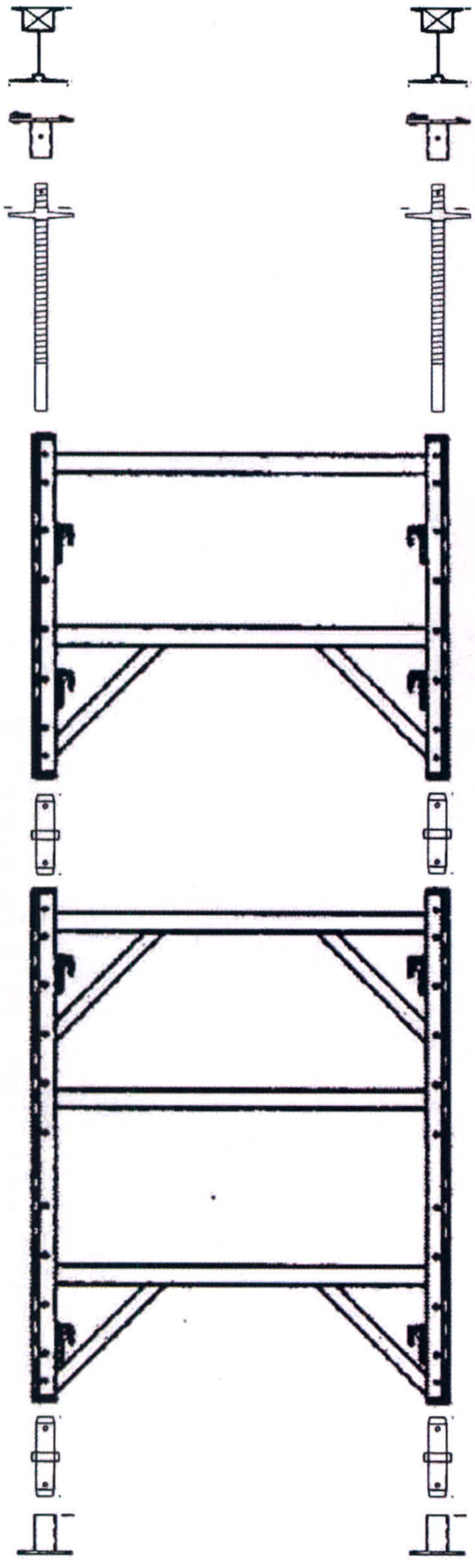
SH 40128
Broken Piece
Zone 40G

Folder: 08-02-20; SH

SH40128_2

SH40128_3

SH40128_4

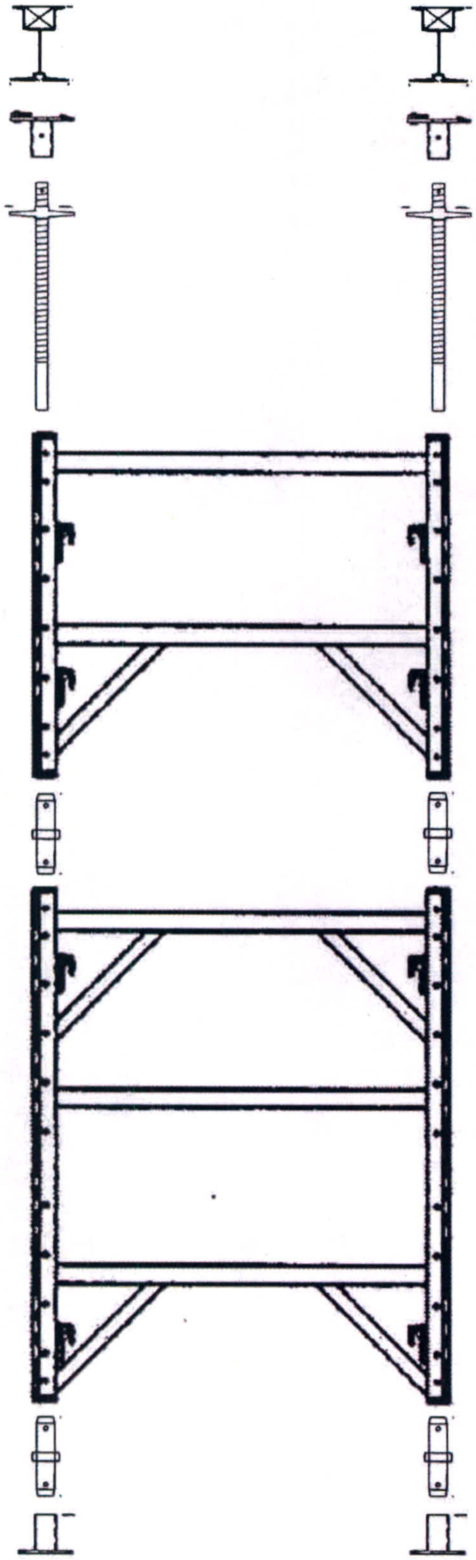


SH 40129
Top plate
Zone 40F

Folder: 08-02-20; 08-02-20; SH

SH40129_1

SH40129_2

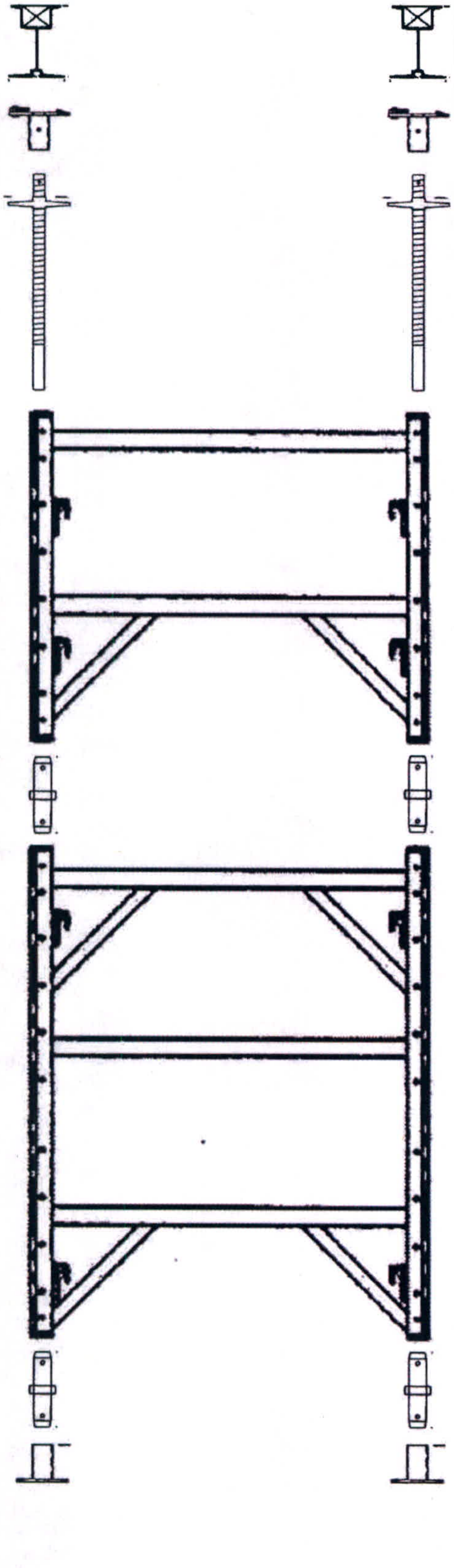


SH 40130
Top plate
Zone 40F

Folder: 08-02-20; SH 40130

SH40130_1

SH40130_2



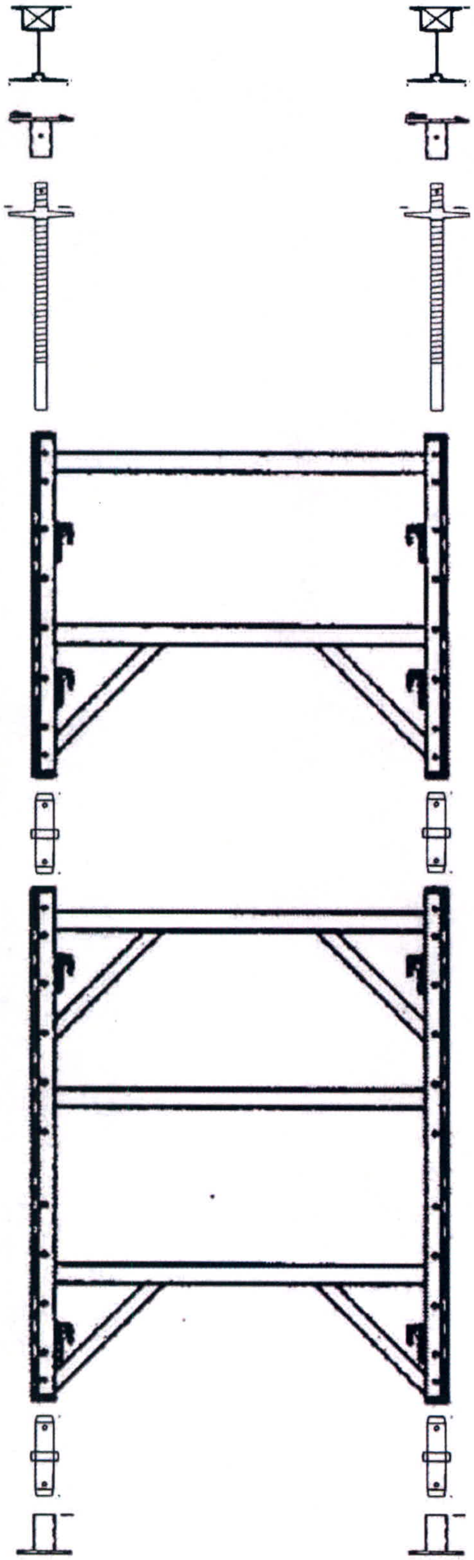
SH 40131
Top plate
Zone 40F

Folder: 08-02-20; SH 40131

SH40131_1

SH40131_2

SH40131_3



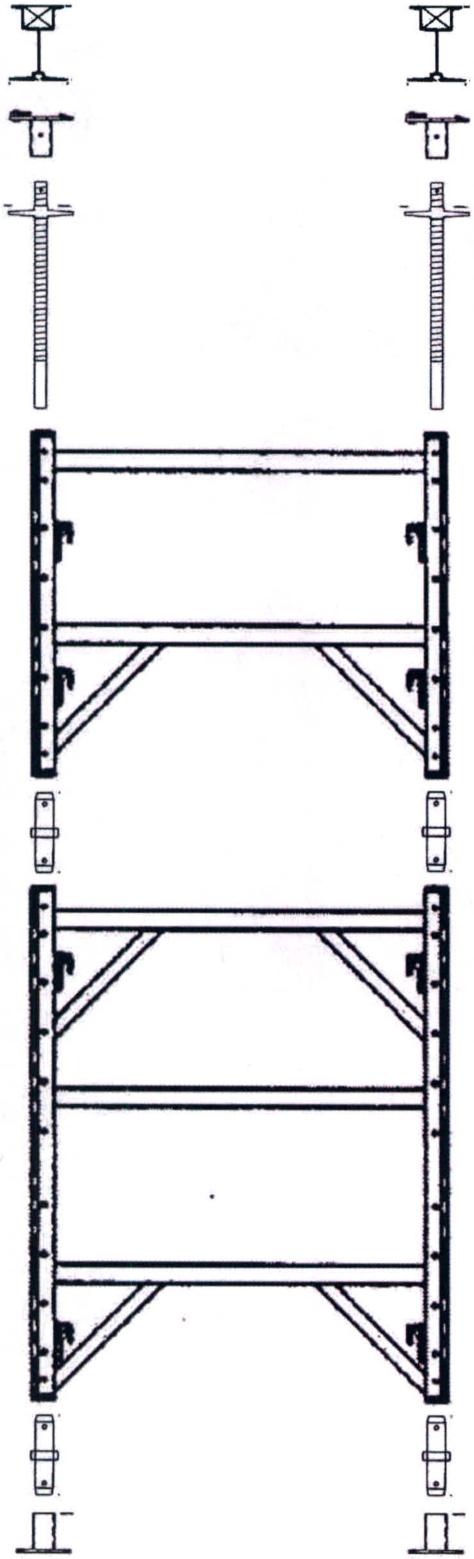
SH 40132
Clip
Zone 40G

Folder: 08-02-21; SH 40132

SH40132_1

SH40132_2

SH40132_3



SH 40133 Frame Piece Near SH 40124

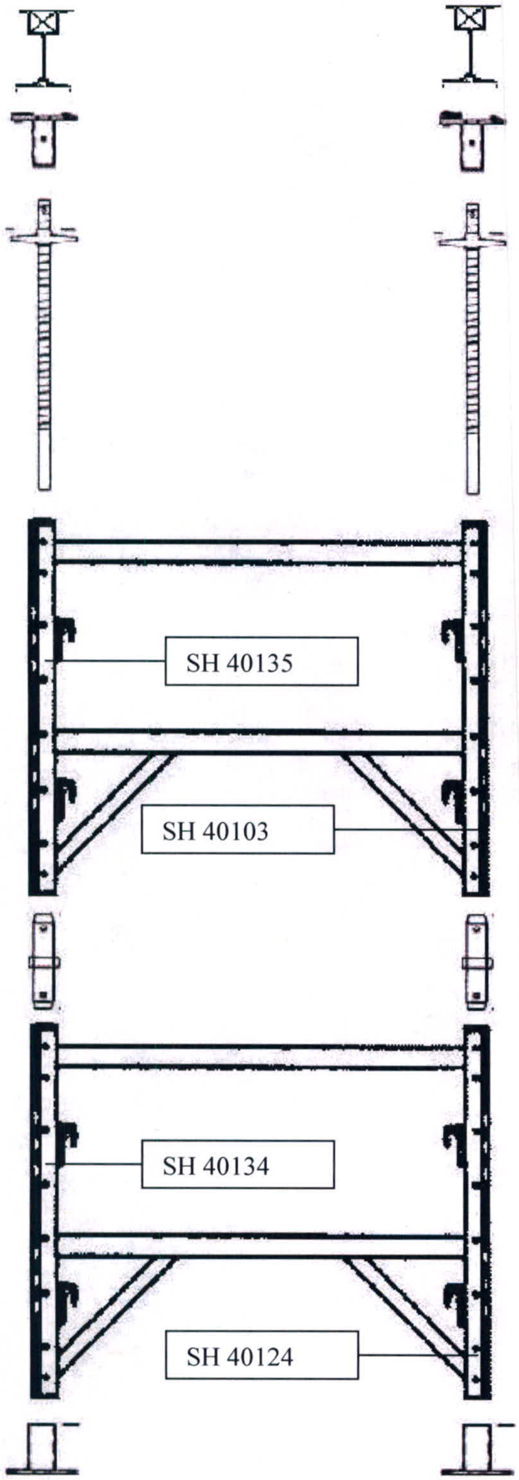
Folder: 08-02-21; SH 40133

SH40133_1

SH40133_3

SH 40134
Frame buried in Concrete
Bottom Frame of SH 135
Zone 40G

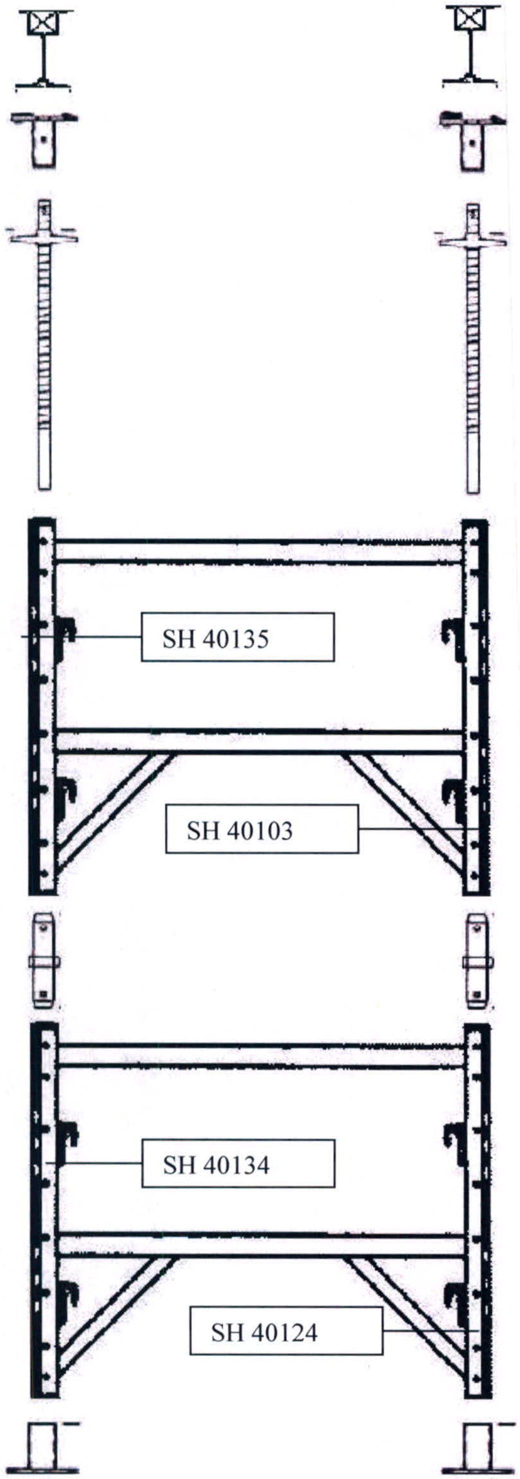
Folder: 08-02-25 SH 40134



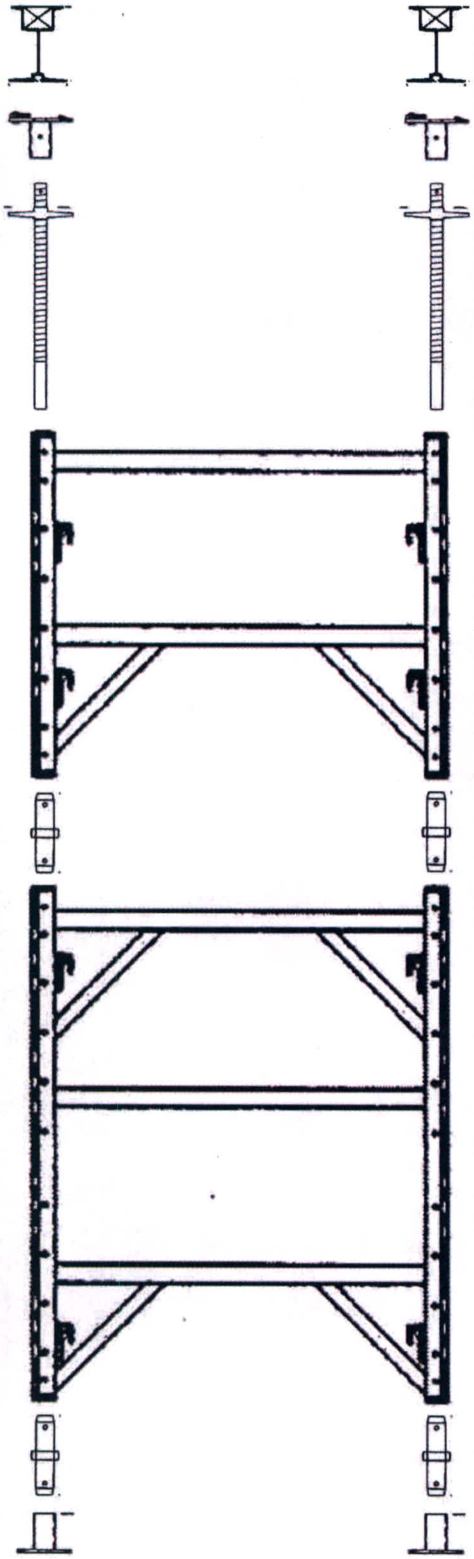
SH 40134_1

SH 40135
Frame buried in Concrete
Zone 40G

Folder: 08-02-25 SH 40135

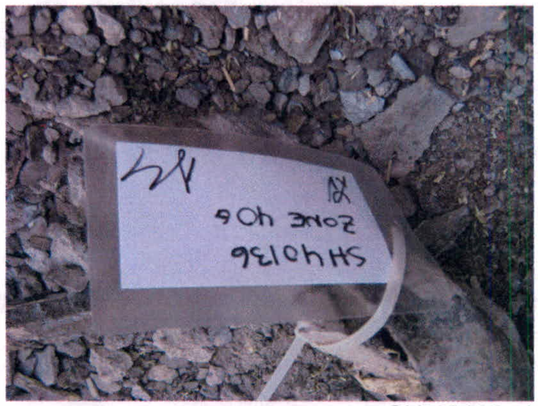


SH 40135_1



SH 40136
Brace
Zone 40G

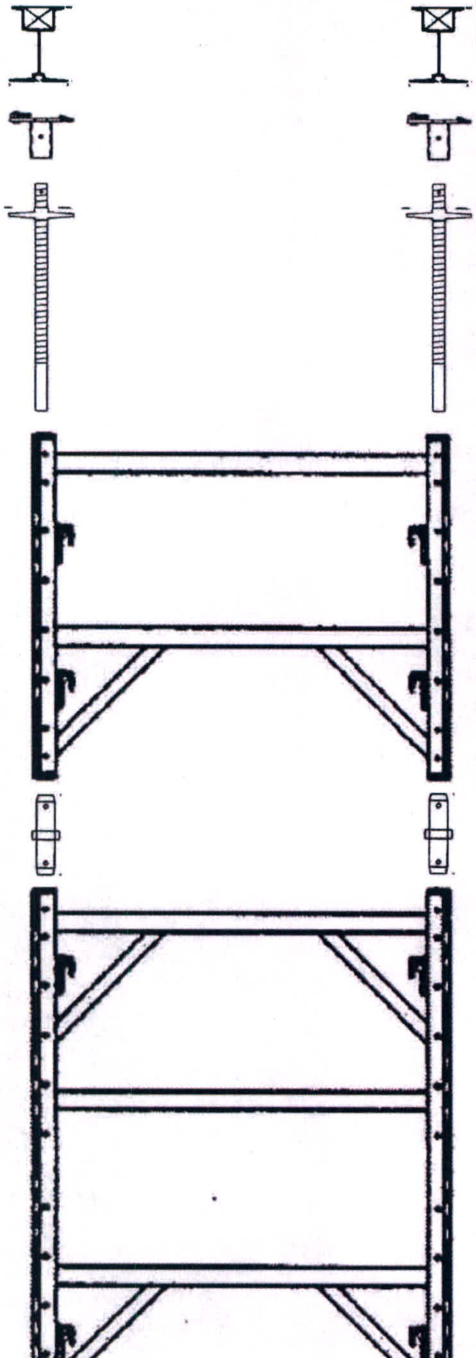
Folder: 08-02-25 SH 40136



SH 40136_1

SH 40136_2

SH 40136_4



SH 40137
X Brace
Zone 40G

Folder: 08-02-20; SH 40137

SH40137_1

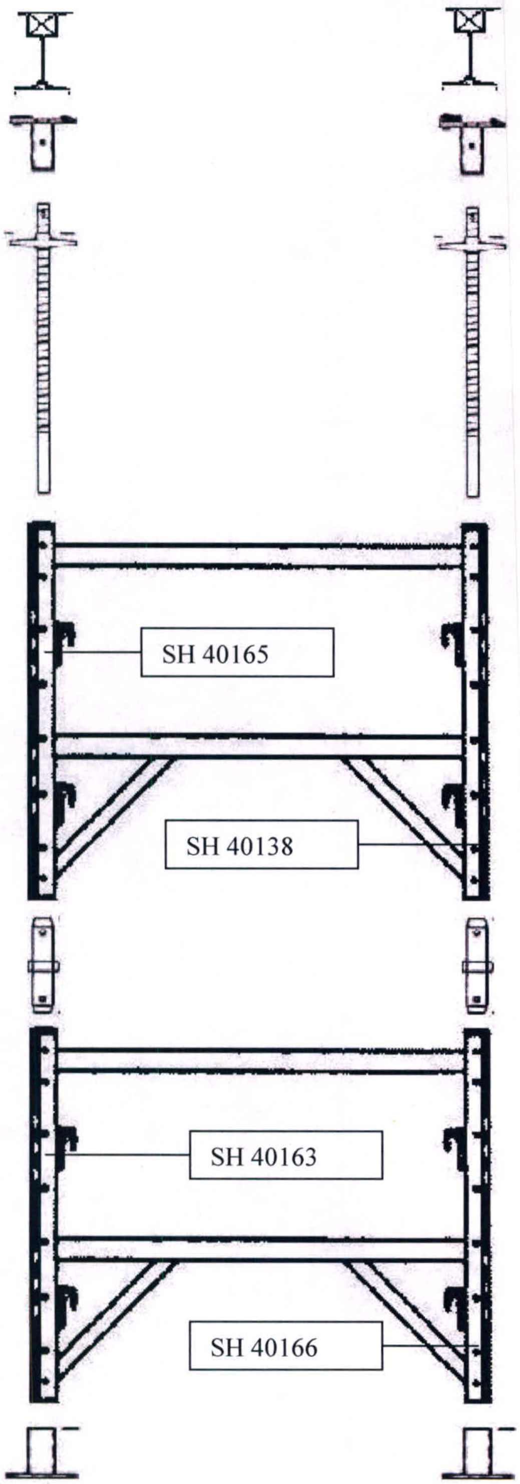
SH40137_2

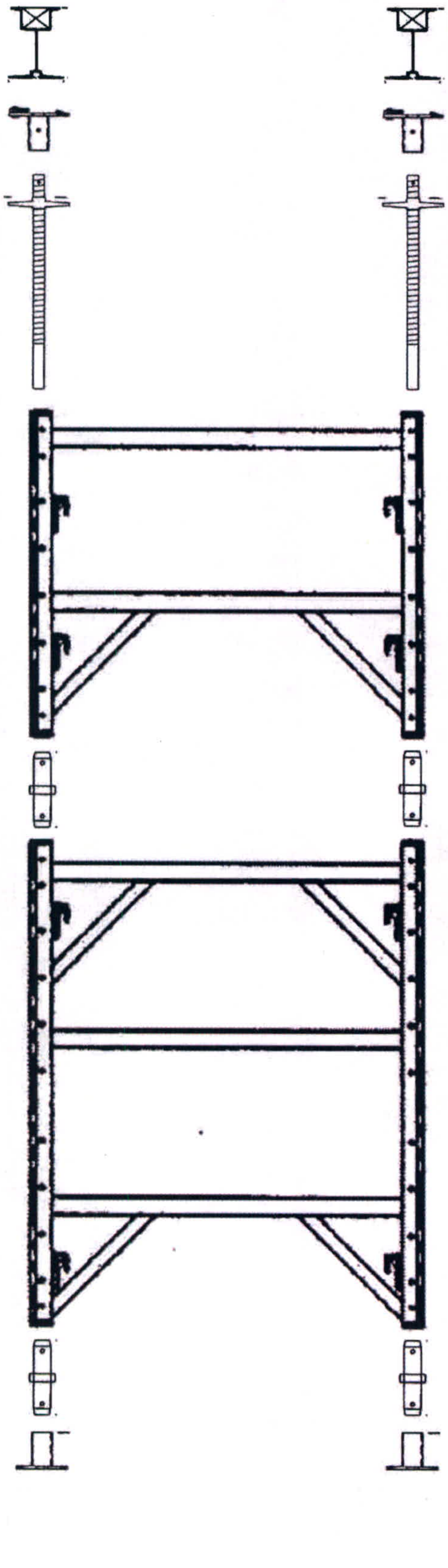
SH40137_3

SH 40138 on 41st floor
Broken top frame of SH 40166
Zone 40F

Folder: 08-02-20; SH 40138

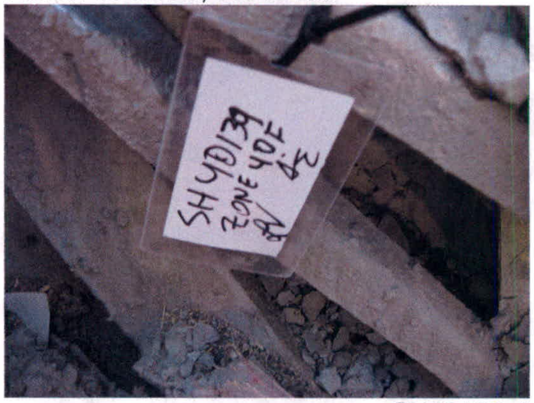
SH40138_1





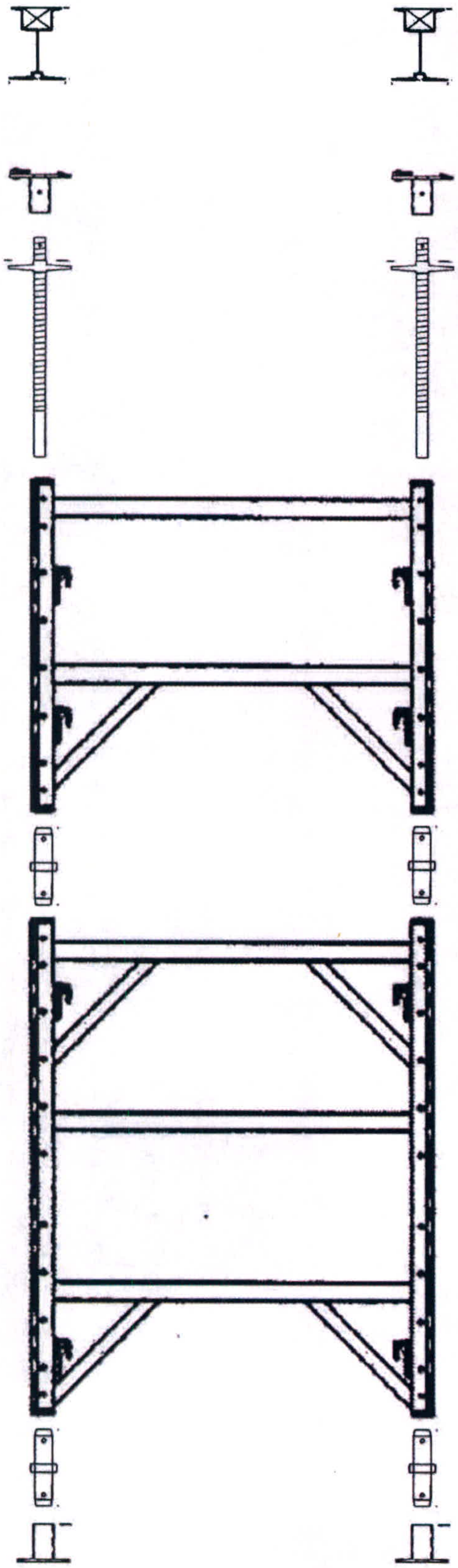
SH 40139
X Brace
Zone 40F

Folder: 08-02-20; SH 40139



SH40139_1

SH40139_2



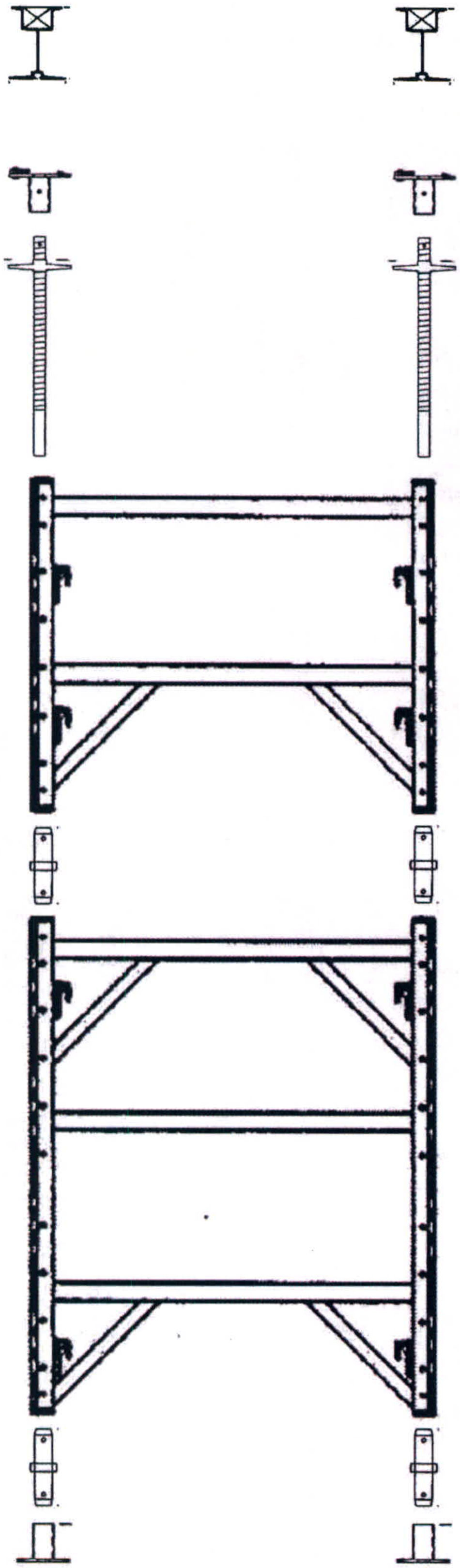
SH 40140
Top plate
Zone 40C

Folder: 08-02-25; 08-02-25; SH 40140

SH40140_3

SH40140_1

SH40140_2



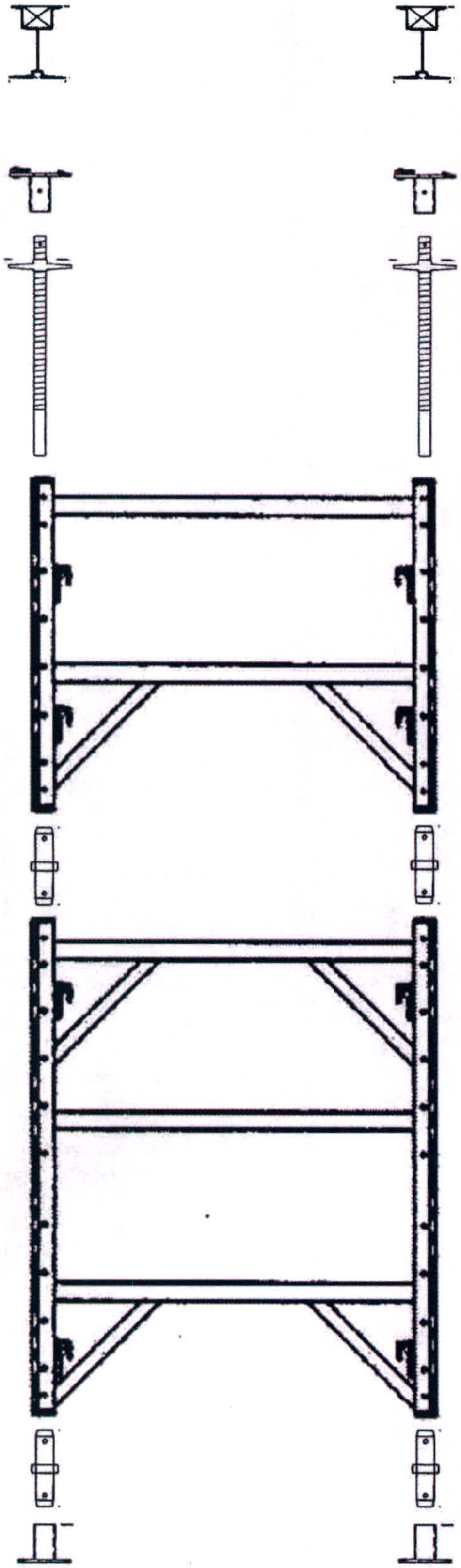
SH 40141
Brace
Zone 40G

Folder: 08-02-25; 08-02-25; SH 40141

SH40141_1

SH40140_2

SH40140_3

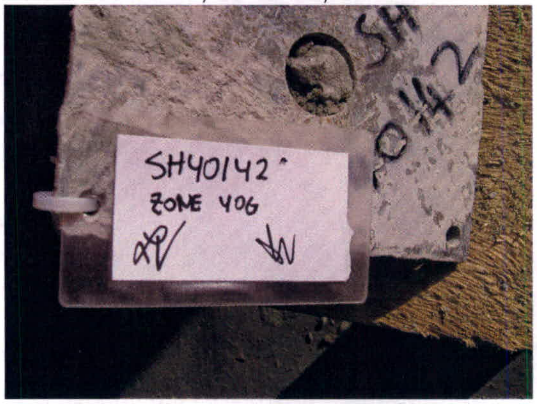


SH 40142
Bottom Plate
Moved from original position
Zone 40G-C

Folder: 08-02-26; SH 40142

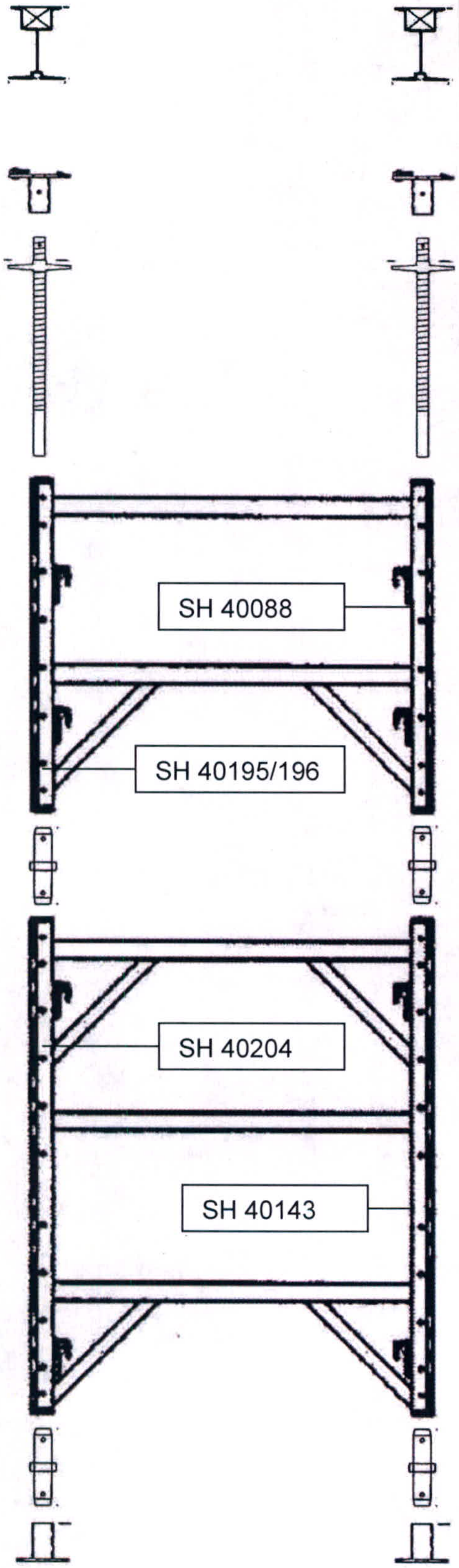
SH40142_1

Folder: 08-02-27; 08-02-27; SH



SH40142_2

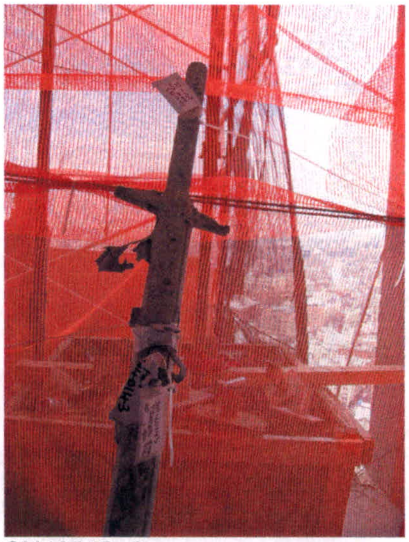
SH40142_4

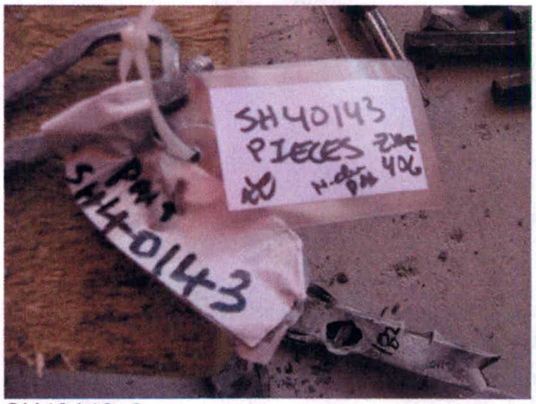


SH 40143
Screw jack and Broken Bottom Frame
Zone 40G-C

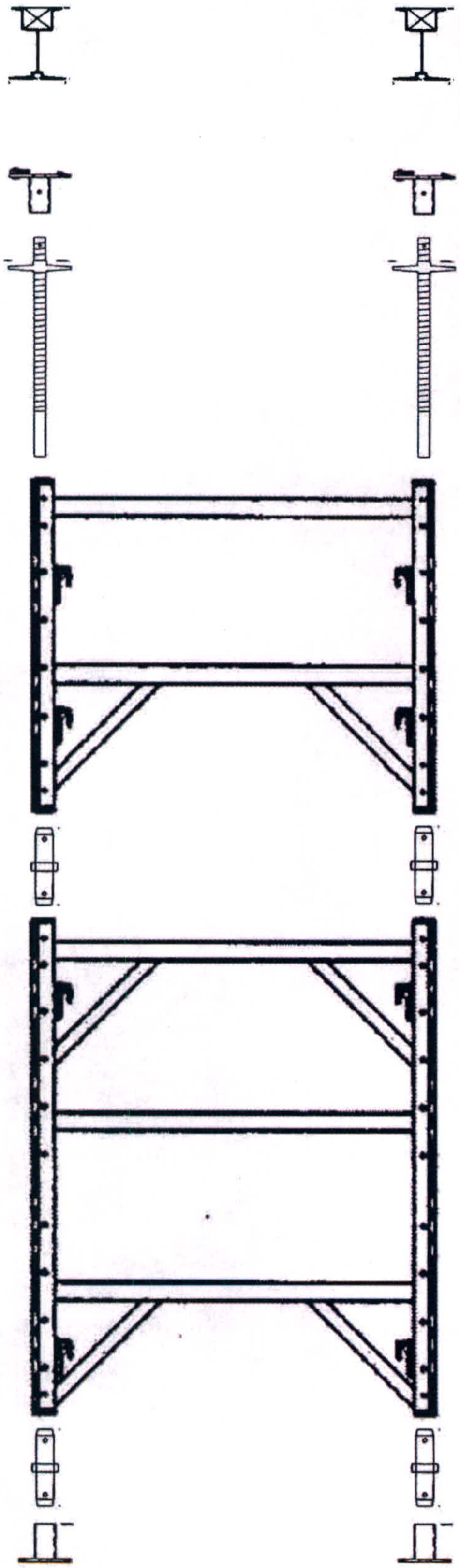
Folder: 08-02-26; 08-02-27 SH 40143


SH 40143_1


SH40143_6

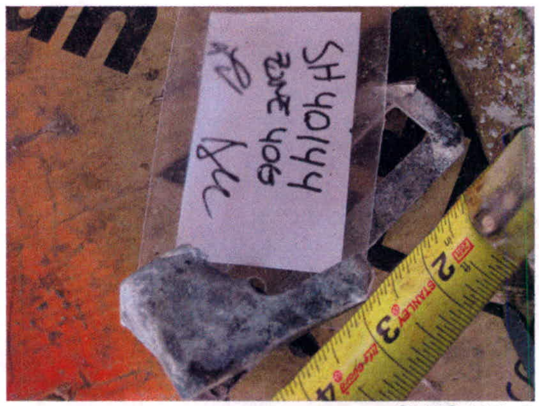


SH40143_3

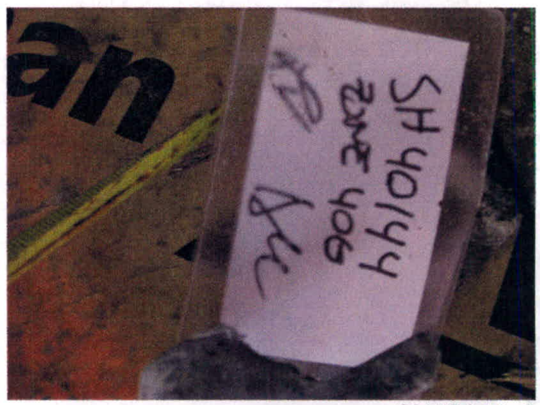


SH 40144
Clip
Zone 40G-C

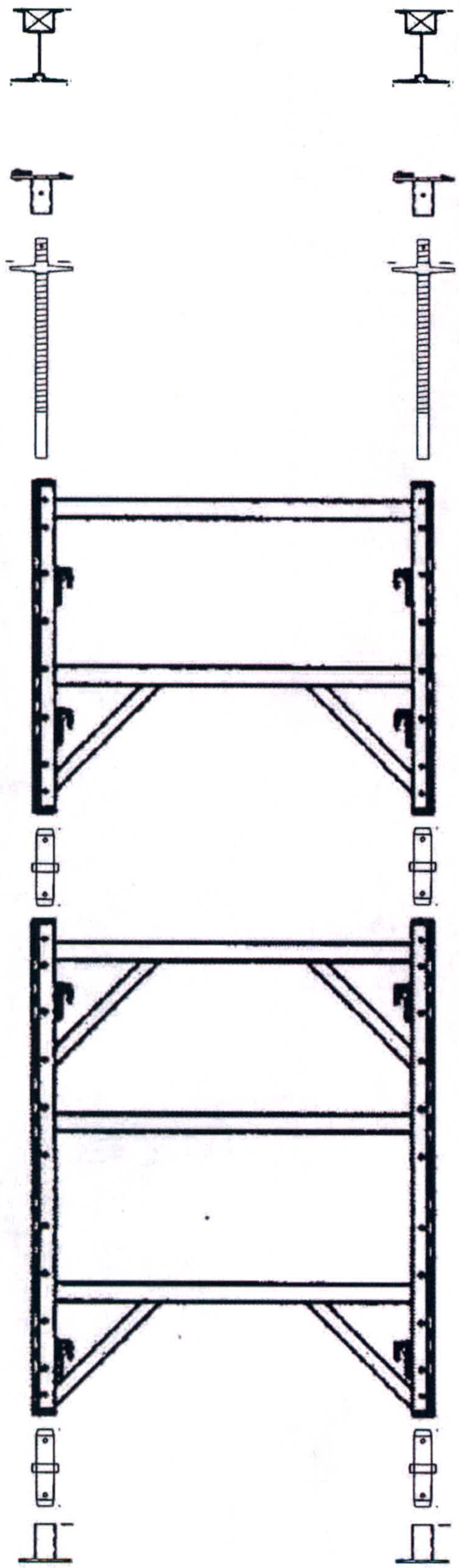
Folder: 08-02-26; 08-02-27; SH 40144



SH40144_2



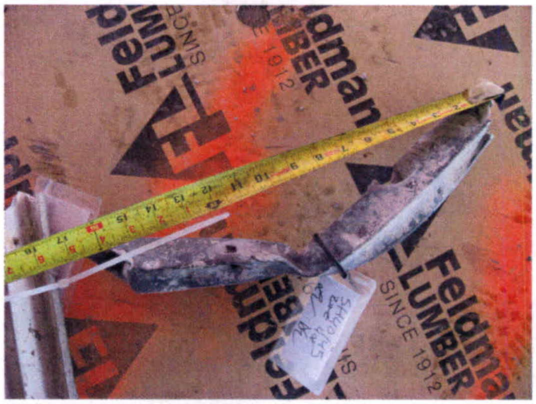
SH40144_3



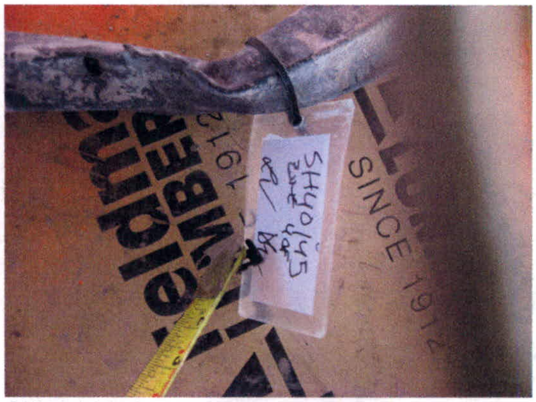
SH 40145
Broken piece of frame
Zone 40F

Folder: 08-02-26; 08-02-27; SH 40145

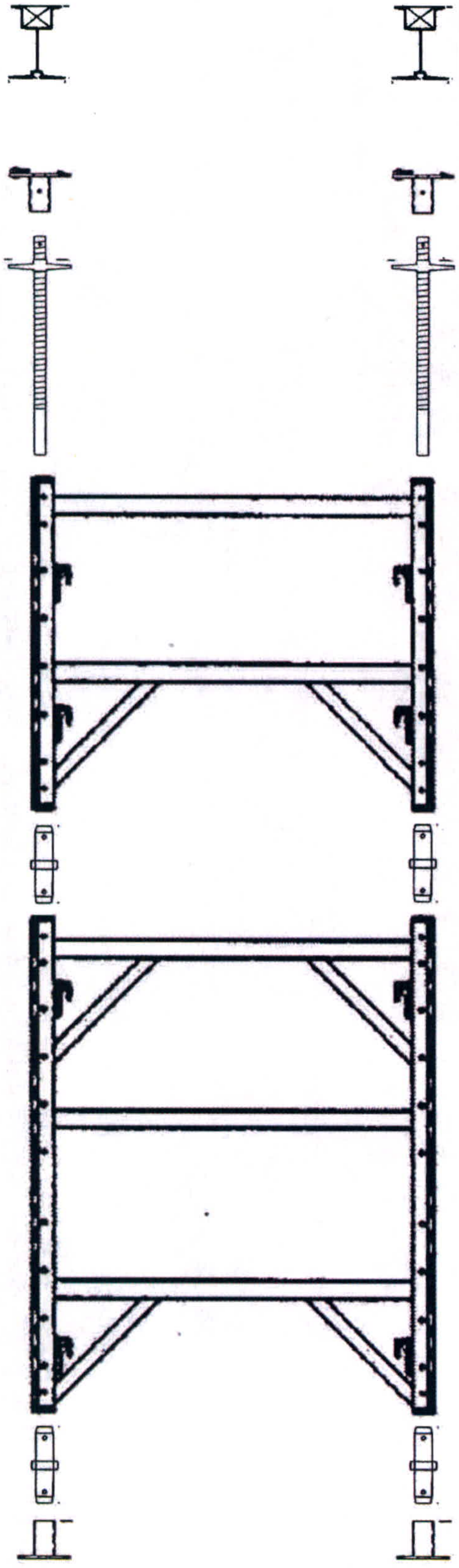
SH 40145_1



SH40145_2



SH40145_3

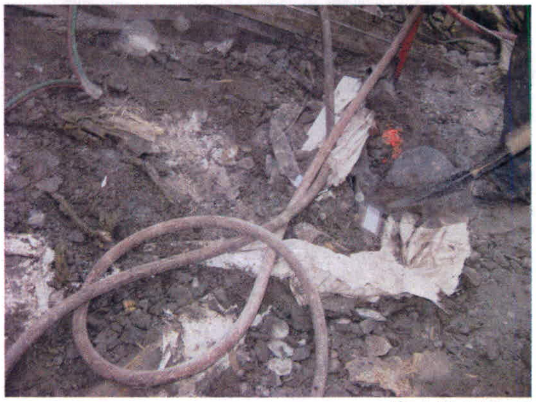


SH 40146
Broken piece of frame
Zone 40F

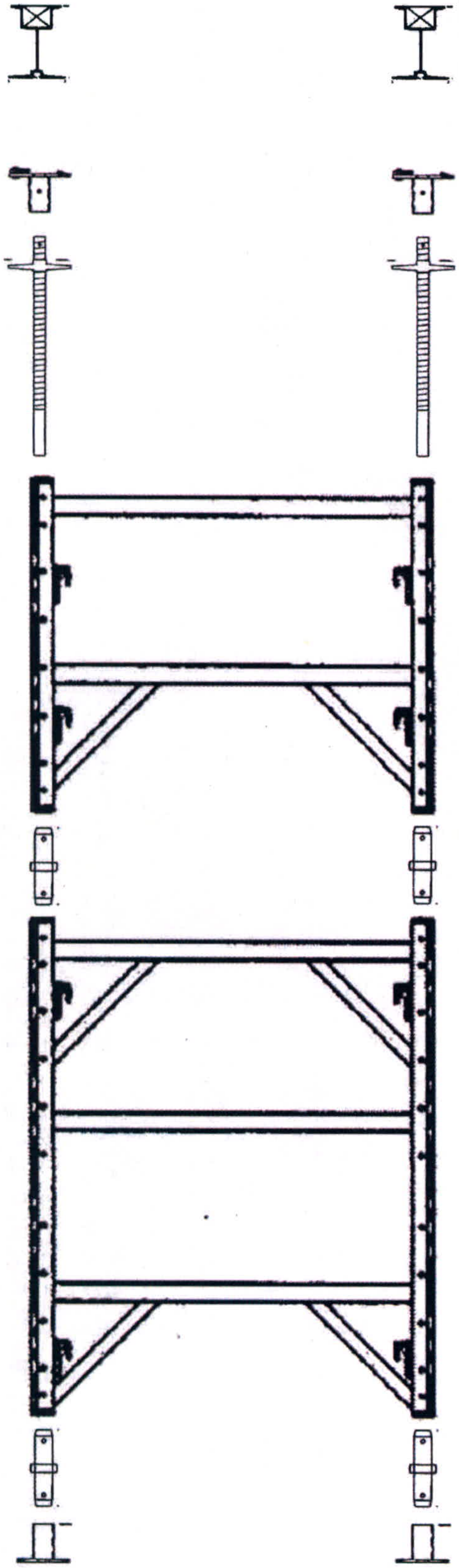
Folder: 08-02-26; 08-02-27; SH 40146

SH40146_1

SH40146_2



SH40146_3

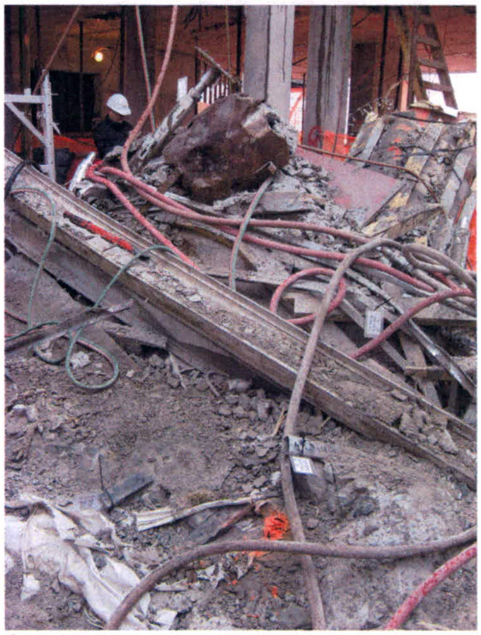


SH 40147
Piece of frame
Zone 40F

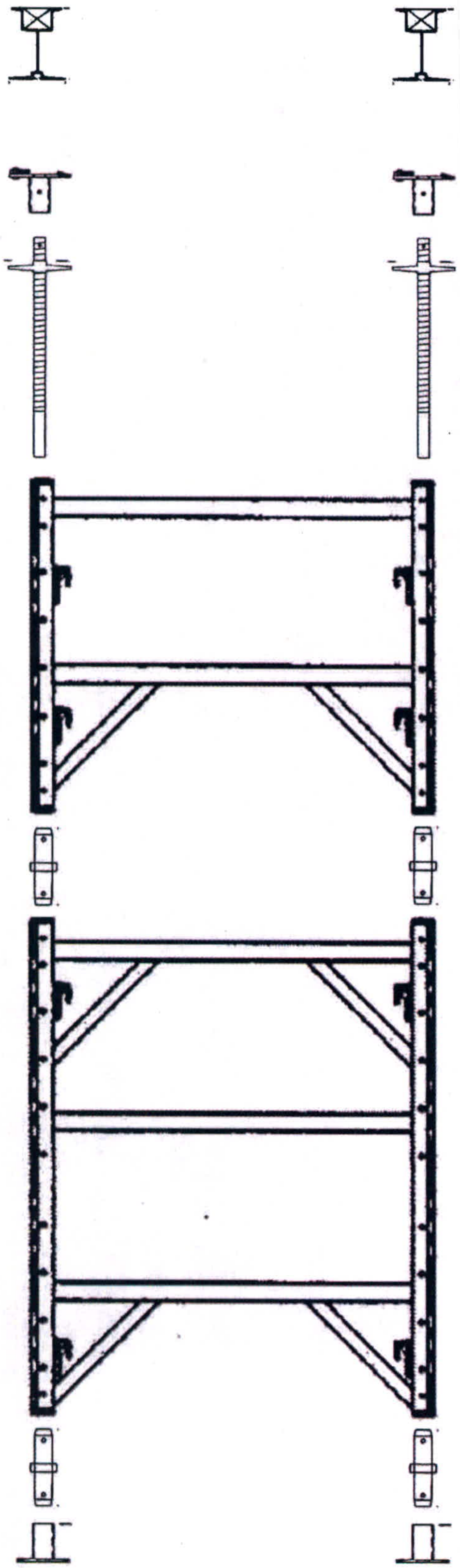
Folder: 08-02-26; 08-02-27; SH

SH40147_1

SH40147_2



SH40147_3

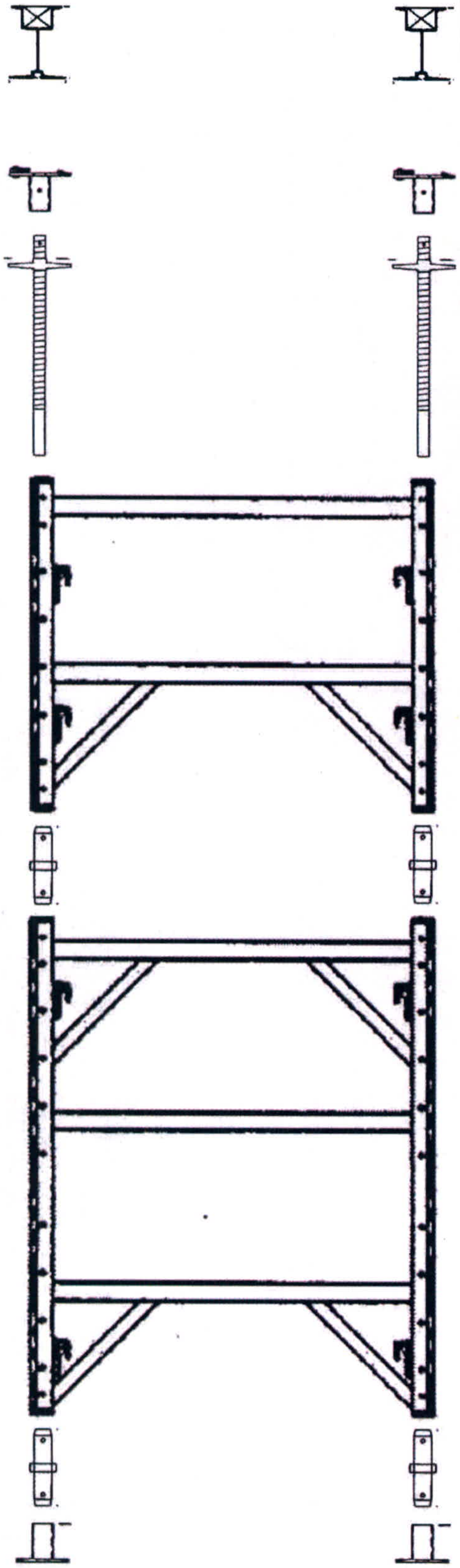


SH 40148
Piece of Frame
Zone 40F

Folder: 08-02-26; 08-02-27; SH 40148

SH40148_2

SH40148_1



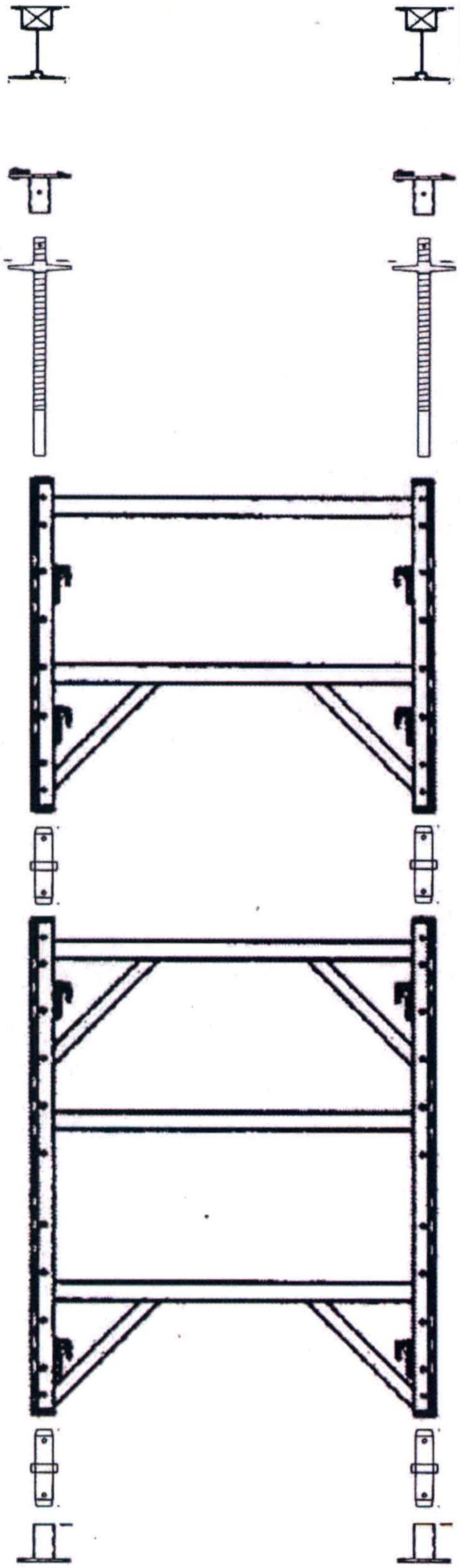
SH 40149
Broken piece of frame
Zone 40G

Folder: 08-02-26; 08-02-27; SH 40149

SH40149_1

SH40149_2

SH40149_5

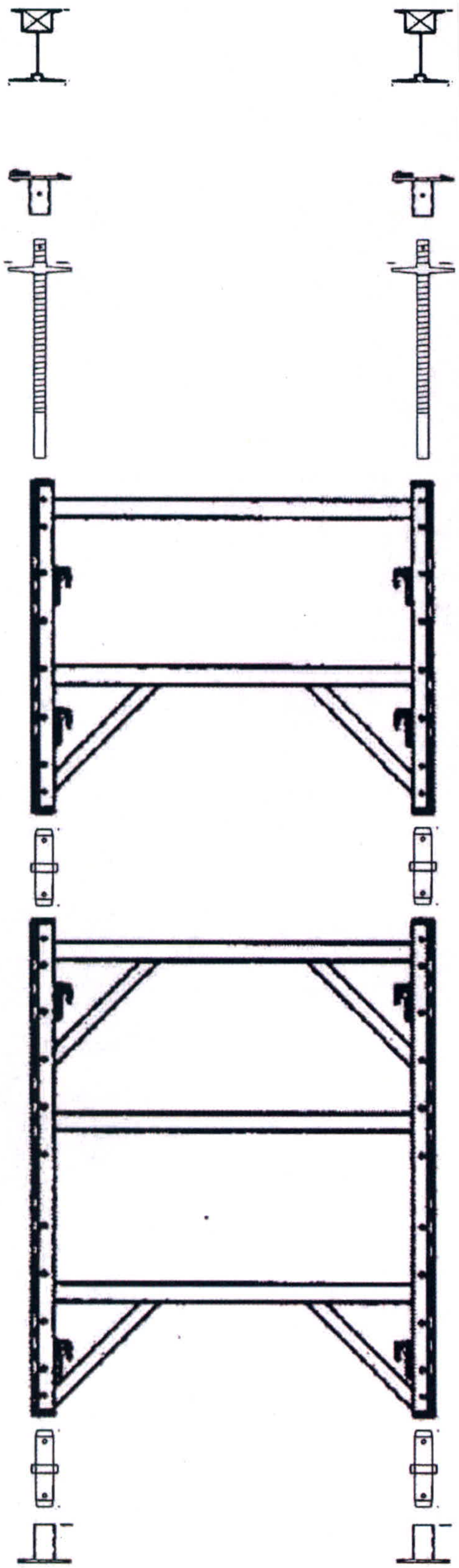


SH 40150
Screw Jack
Zone 40G

Folder: 08-02-26; 08-02-27; SH 40150

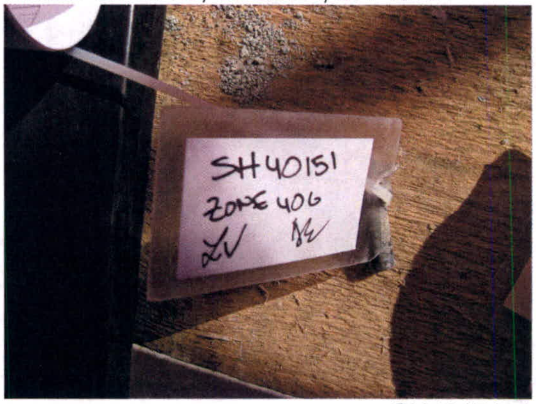
SH40150_1

SH40150_2

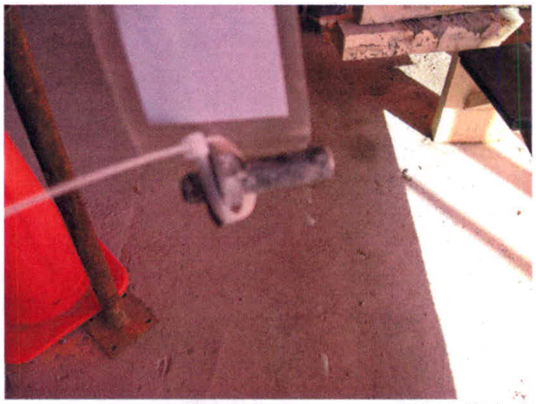


SH 40151
Screw for Brace
Zone 40G

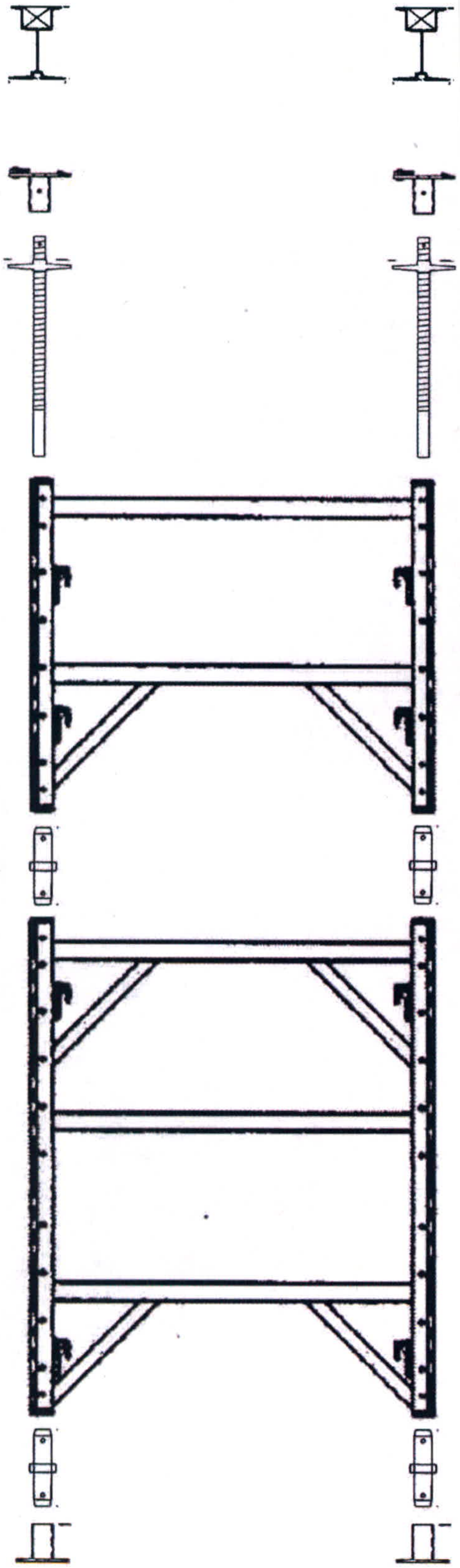
Folder: 08-02-26; 08-02-27; SH 40151



SH40151_1

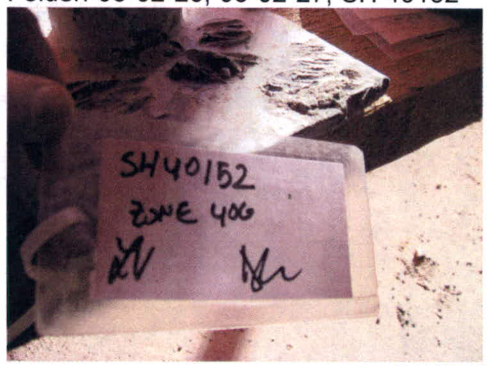


SH40151_2



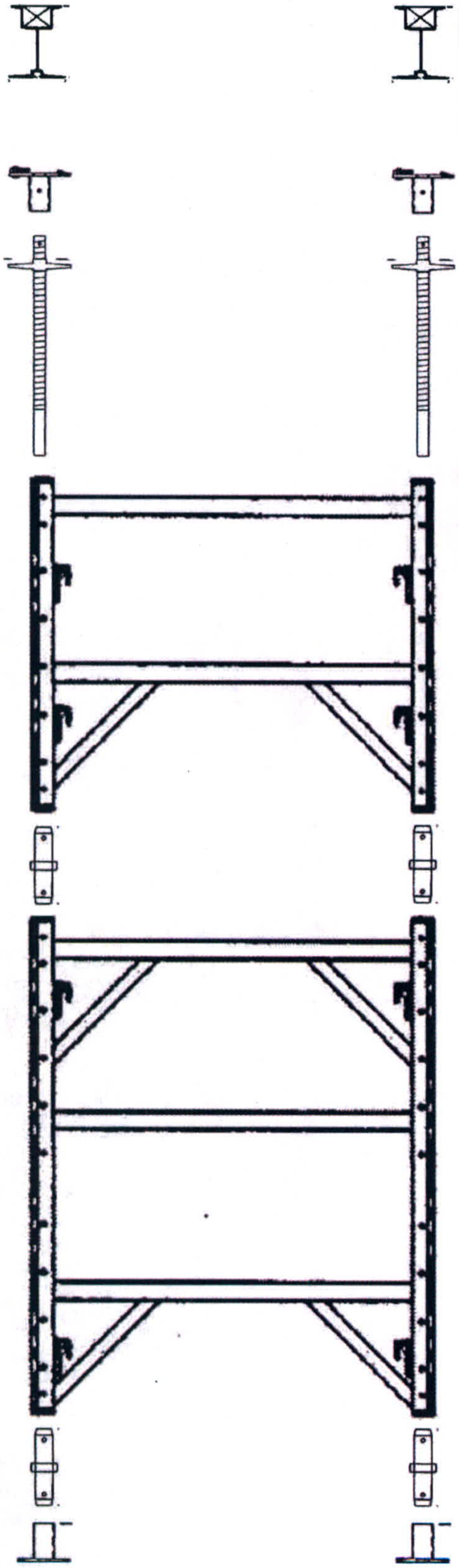
SH 40152
Bott Plate 3' East and 1' North of Col. 103
Zone 40G

Folder: 08-02-26; 08-02-27; SH 40152



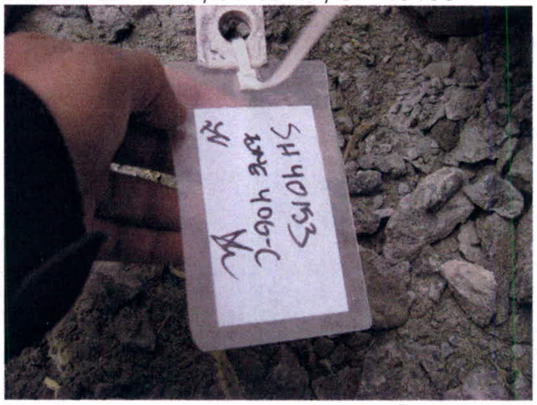
SH40152_1

SH40152_4



SH 40153
Brace
Zone 40G-C

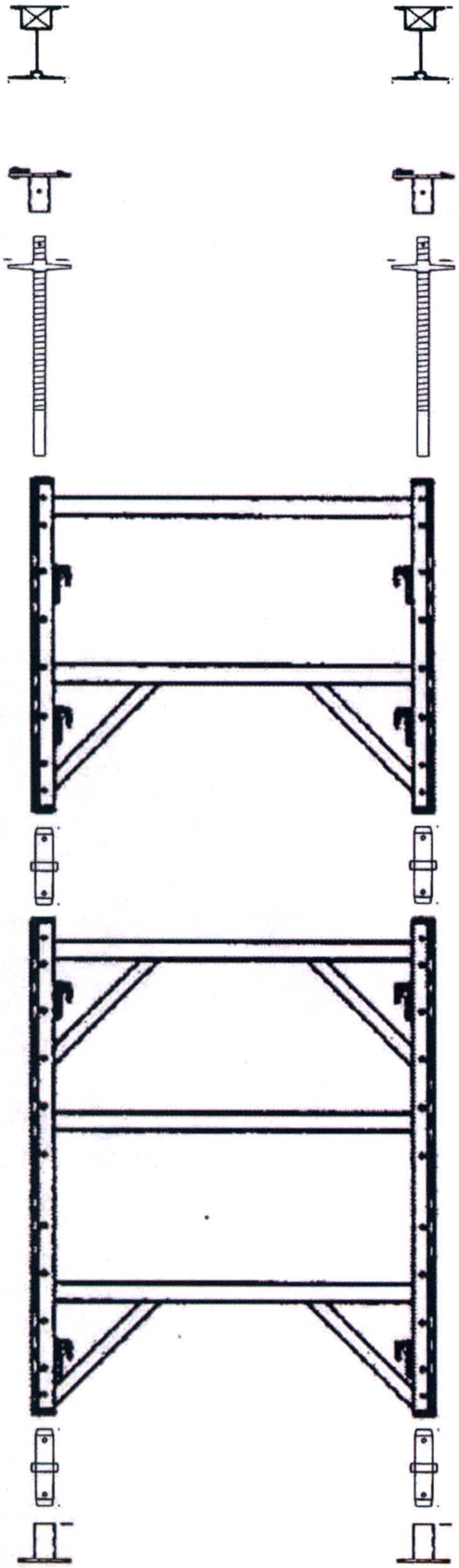
Folder: 08-02-26; 08-02-27; SH 40153



SH40153_3

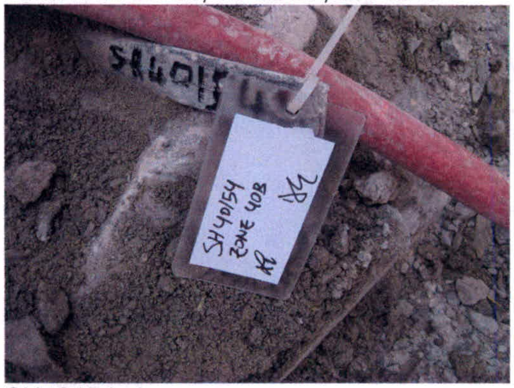
SH40153_1

SH40153_2



SH 40154
Bottom Brace
Zone 40B

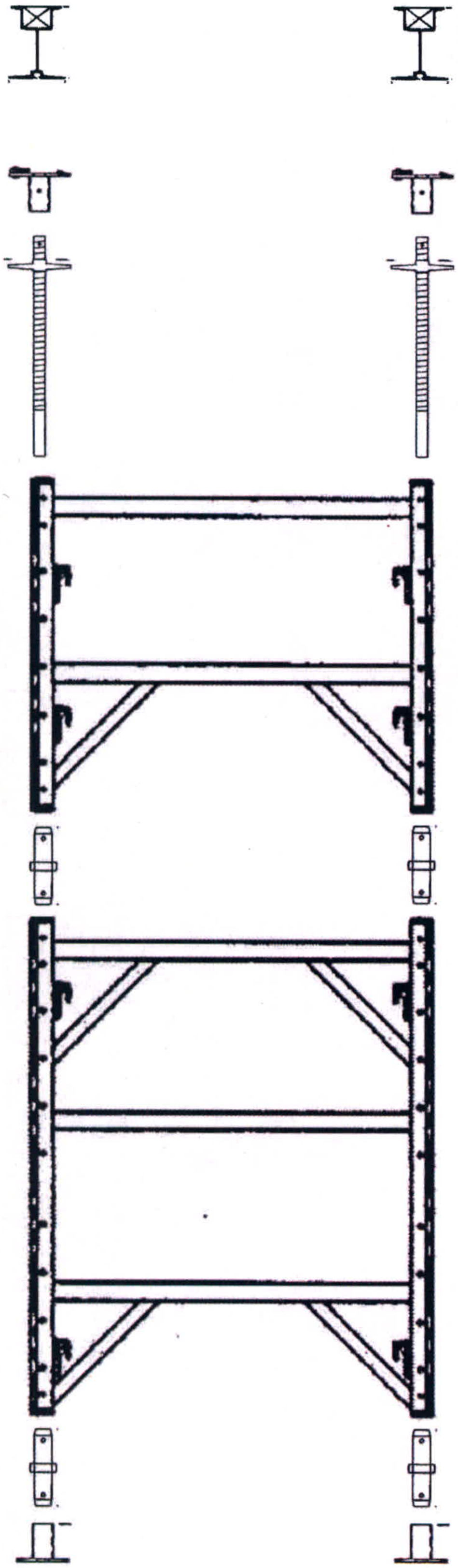
Folder: 08-02-25; 08-02-27; SH 40154



SH40154_1

SH40154_2

SH40154_3

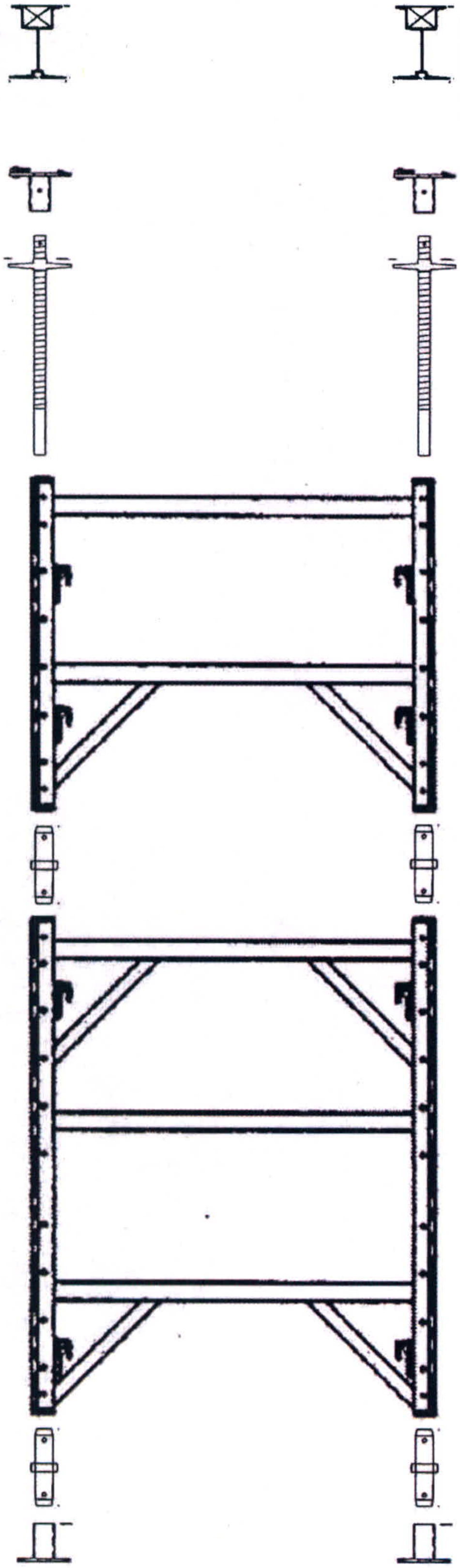


SH 40155
Bracket
Zone 40B

Folder: 08-02-25; 08-02-27; SH 40155

SH40155_1

SH40155_2

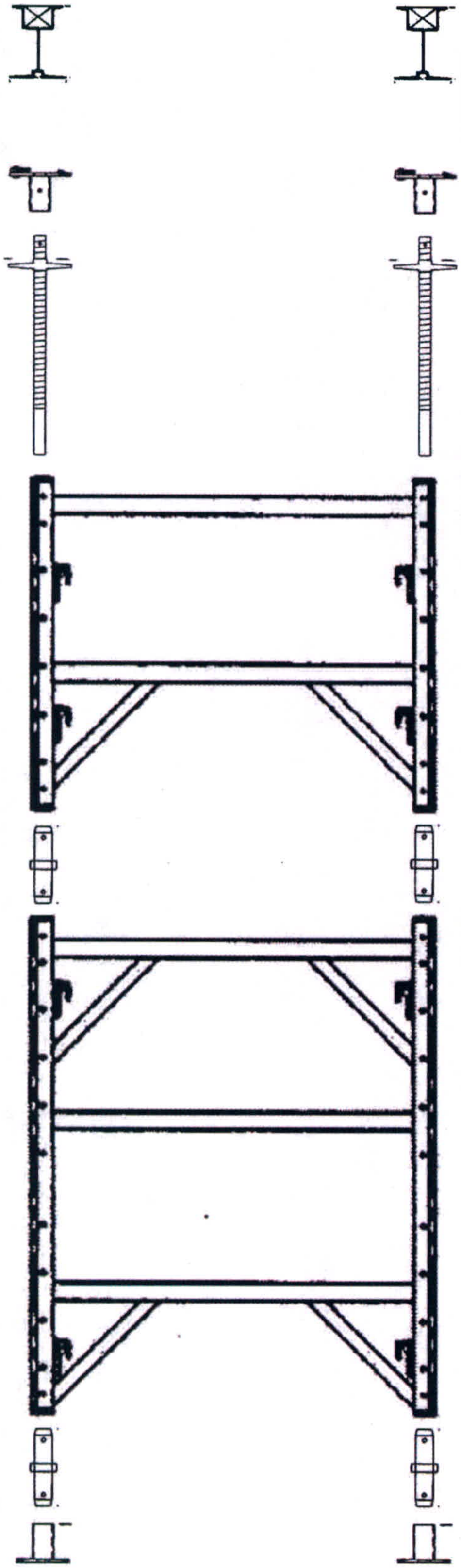


SH 40156
Screw for Brace
Zone 40B

Folder: 08-02-27; SH 40156

SH40156_1

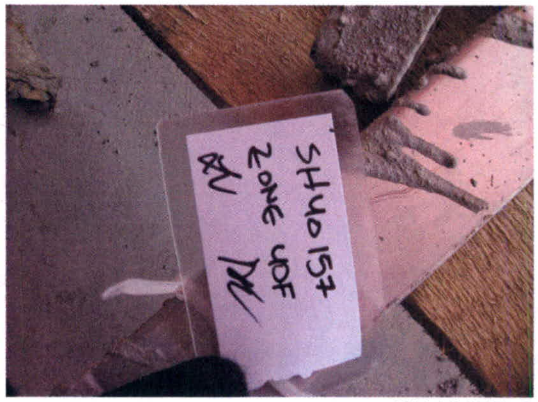
SH40156_2



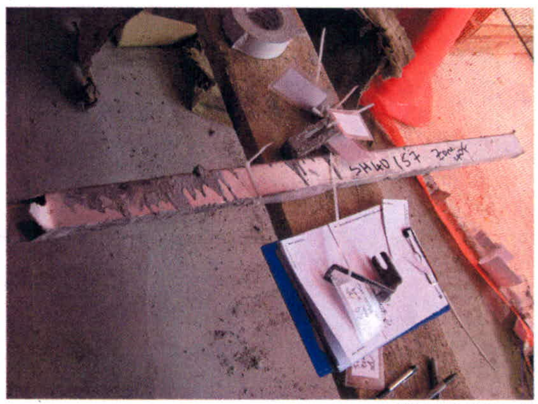
SH 40157
Broken piece of Frame
Zone 40G

Folder: 08-02-27; SH 40157

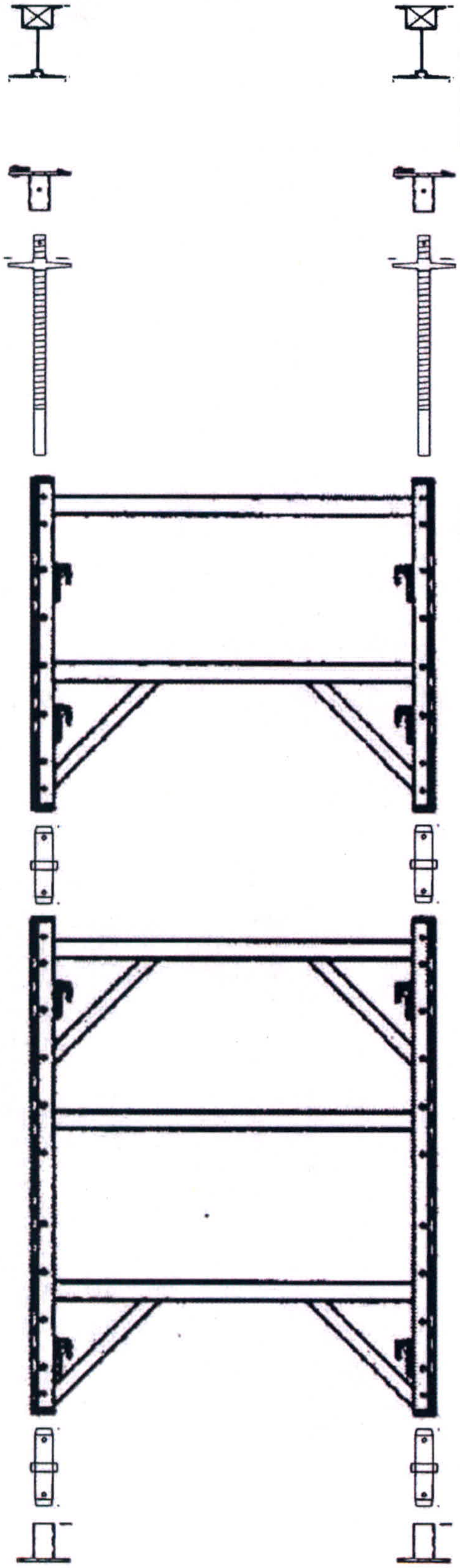
SH40157_1



SH40157_2



SH40157_3



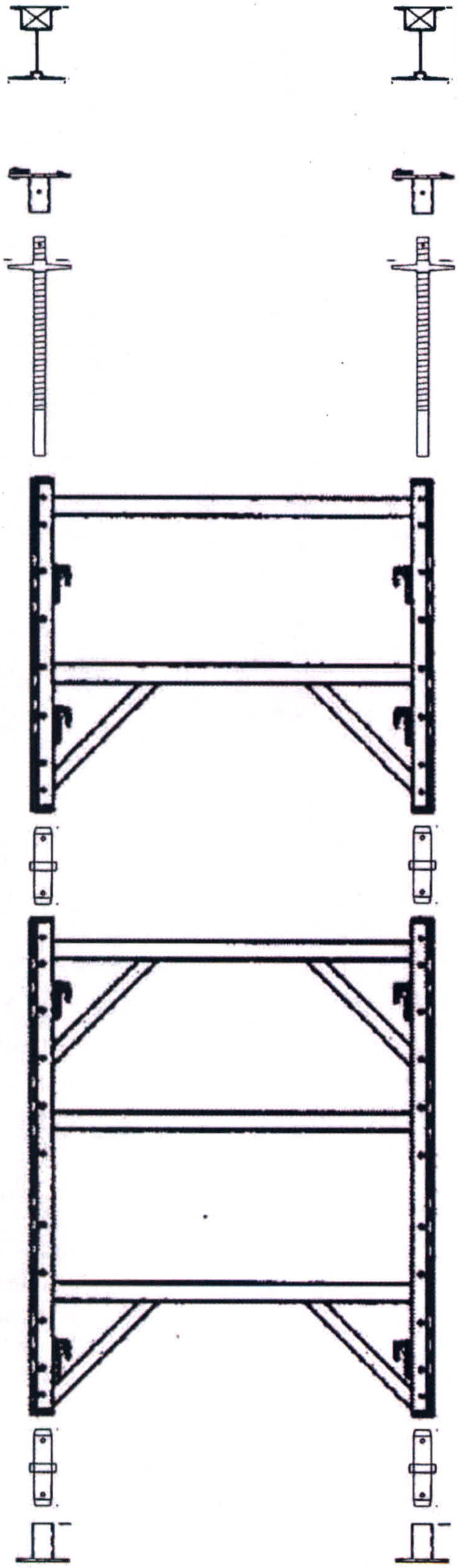
SH 40158
Screw for brace
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40158

SH40158_1

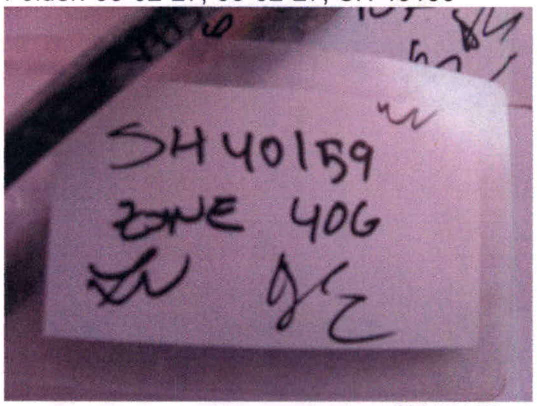
SH40158_2

SH40158_3

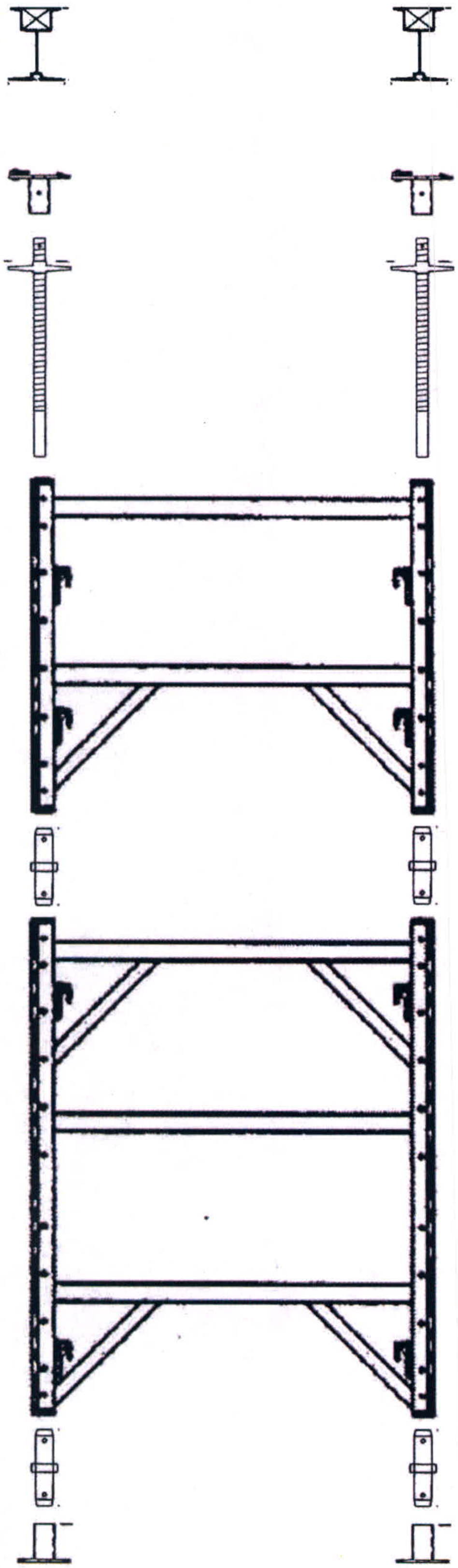


SH 40159
Bracket
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40159



SH40159_1



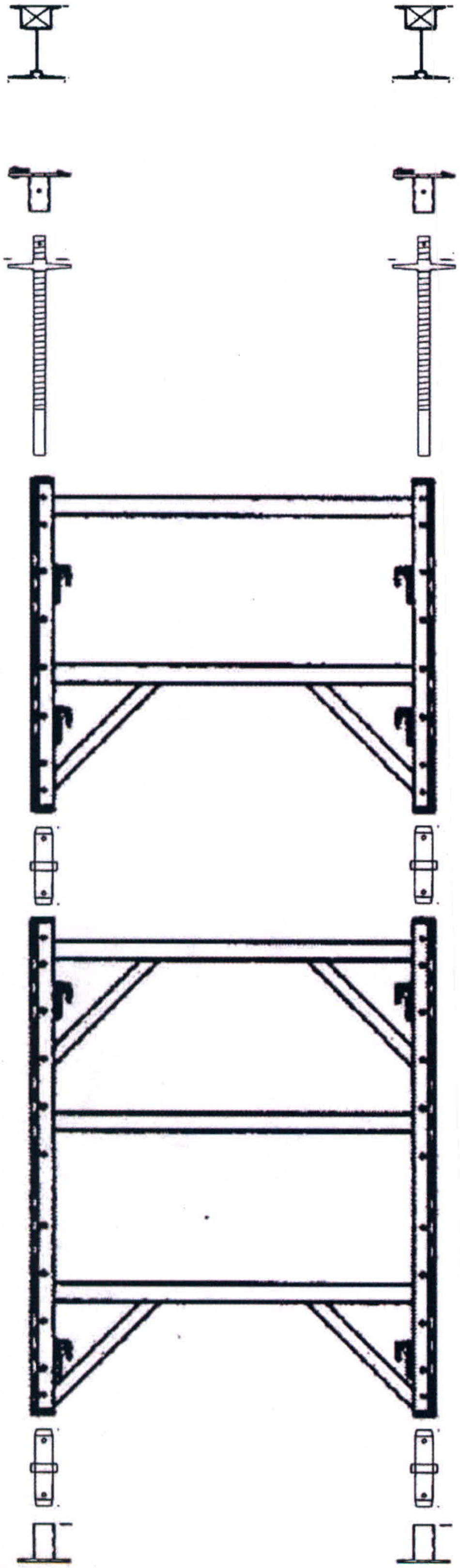
SH 40160
Brace
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40160

SH40160_1

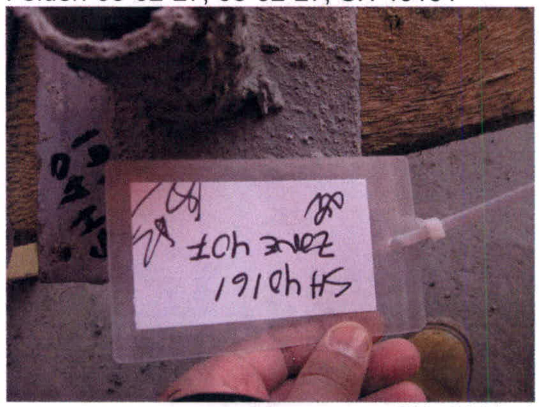
SH40160_2

SH40160_3



SH 40161
Top plate
Zone 40F

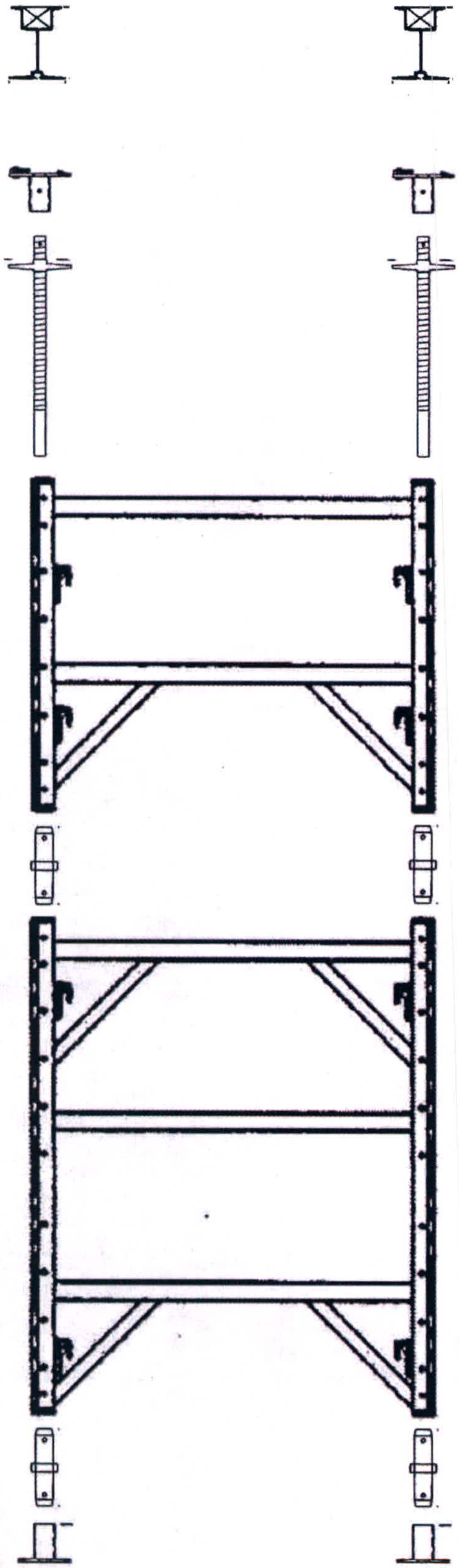
Folder: 08-02-27; 08-02-27; SH 40161



SH40161_1

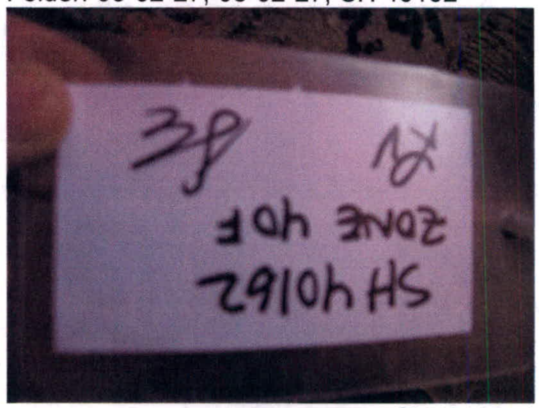
SH40161_2

SH40161_3

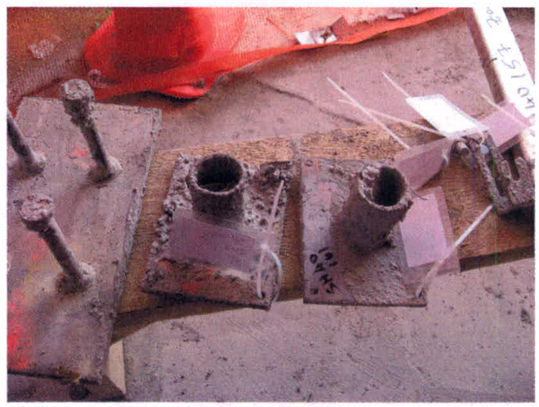


SH 40162
Top plate
Zone 40F

Folder: 08-02-27; 08-02-27; SH 40162



SH40162_1

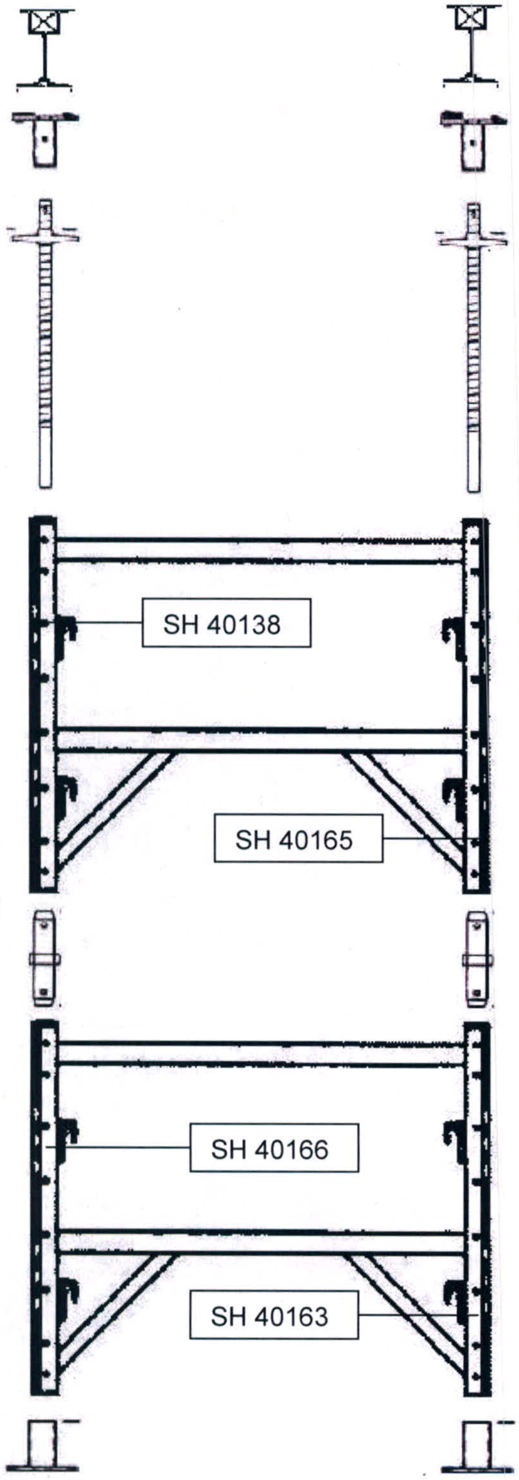


SH40162_2

SH40162_3

SH 40163 on 41st Floor
Bott Frame of SH 40165
Frame lying on Slab
Zone 40G-C

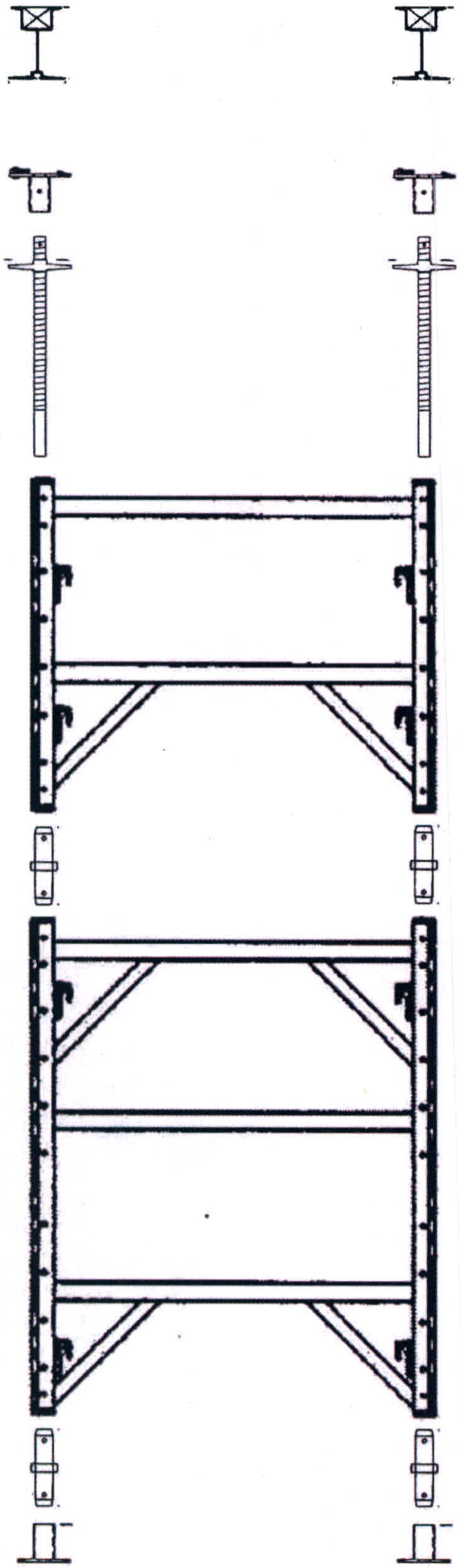
Folder: 08-02-27; 08-02-27; SH 40163



SH40163_1

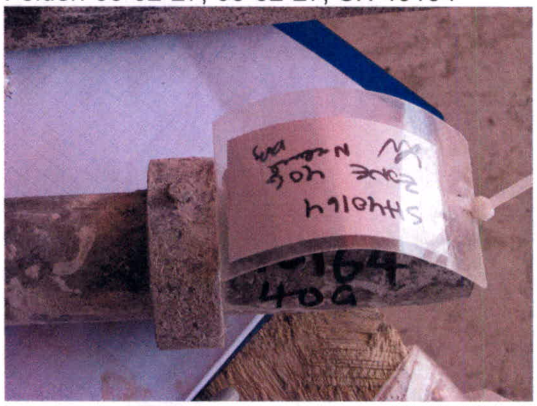
SH40163_2

SH40163_3



SH 40164
Coupler
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40164



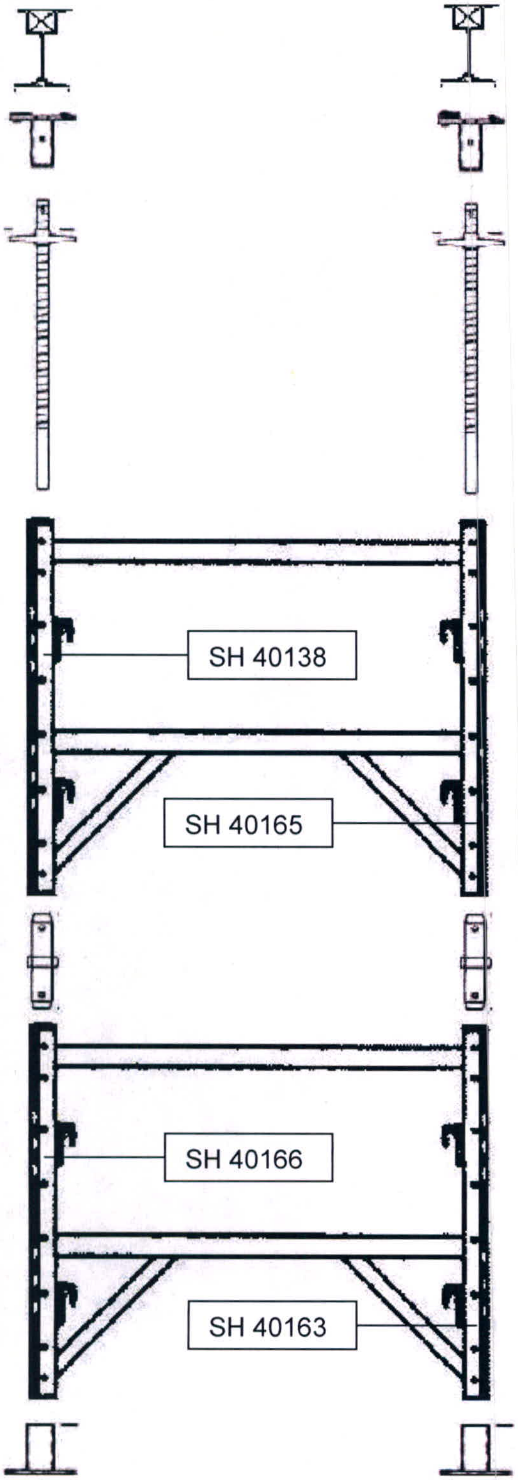
SH40164_1

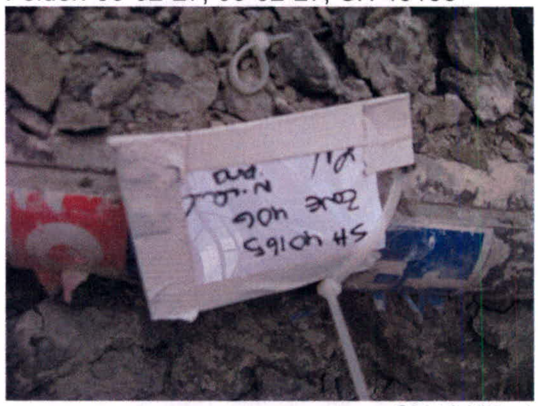
SH40164_2

SH40164_3

SH 40165 on 41st floor
Top Frame 3'-6"
Shoring tower frame is buried
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40165





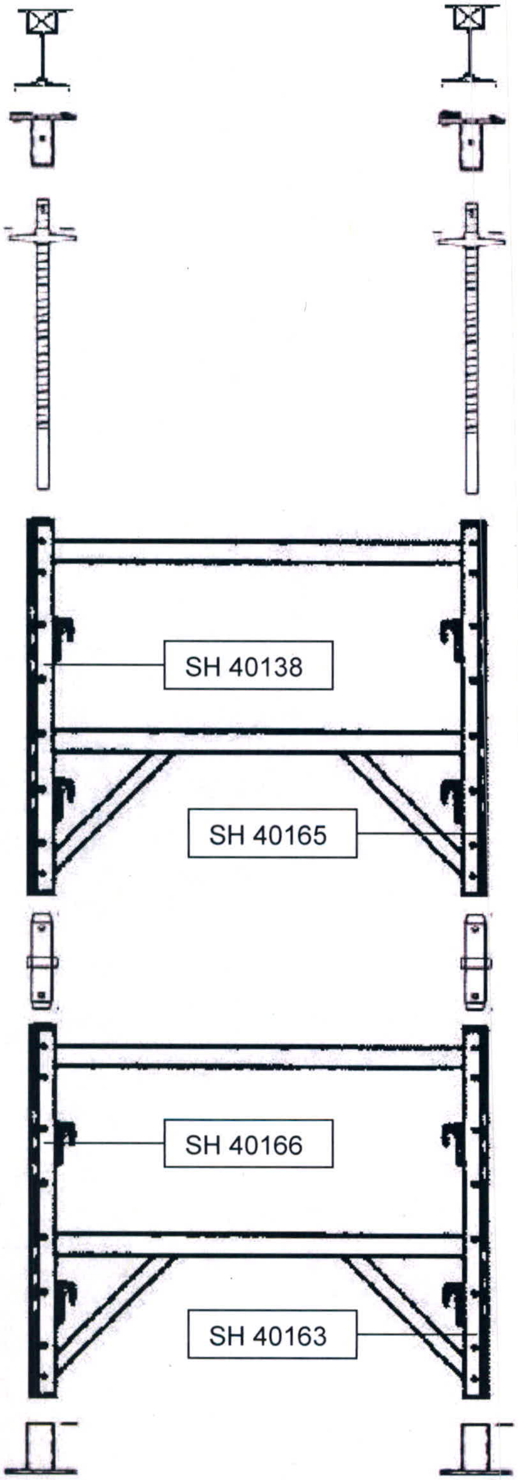
SH40165_1

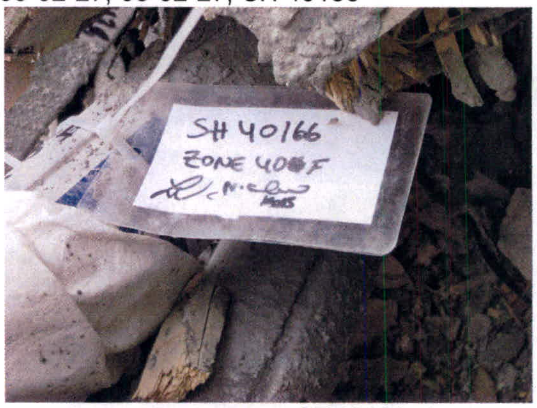
SH40165_2

SH40165_4

SH 40166
Bottom Frame lying on SH 40062
Zone 40F

Folder: 08-02-27; 08-02-27; SH 40166

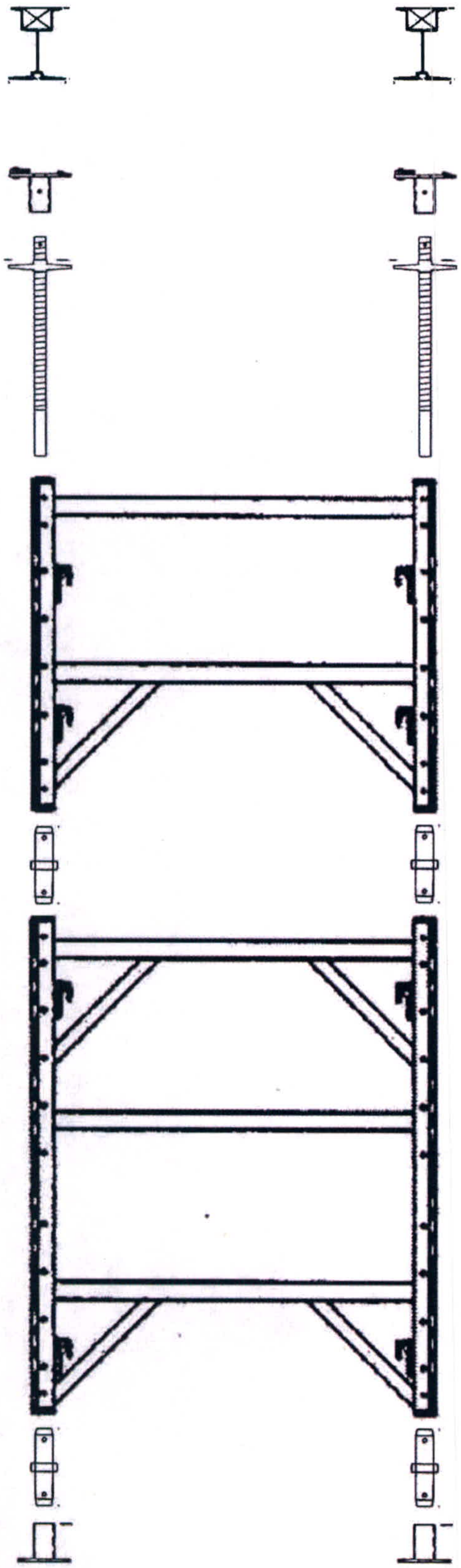




SH40166_1

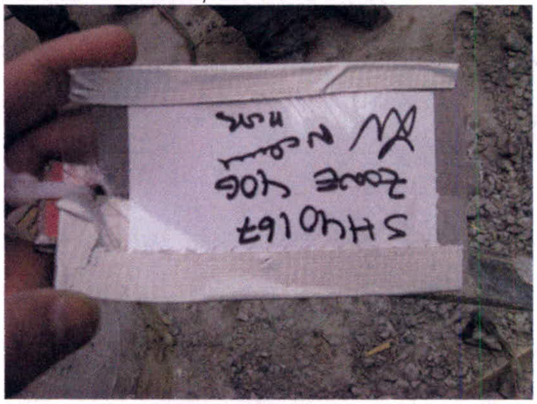

SH40166_2


SH40166_3



SH 40167
Brace
Zone 40G

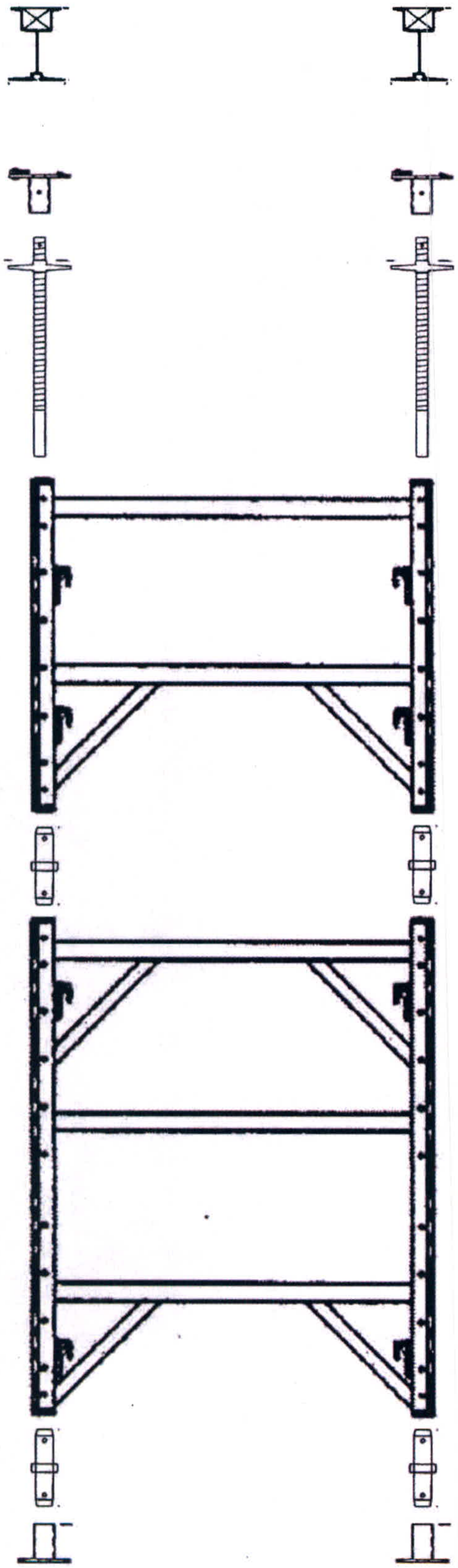
Folder: 08-02-27; SH 40167



SH40167_1

SH40167_2

SH40167_3



SH 40168
Brace
Zone 40G

Folder: 08-02-27; SH 40168

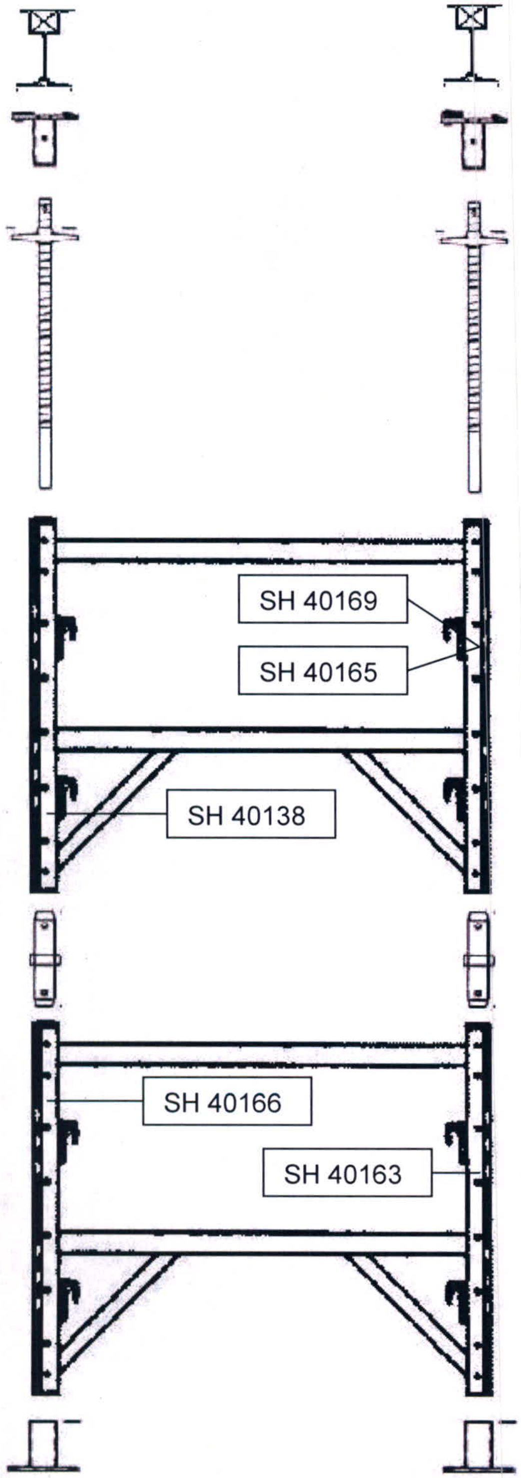
SH40168_3

SH40168_1

SH40168_2

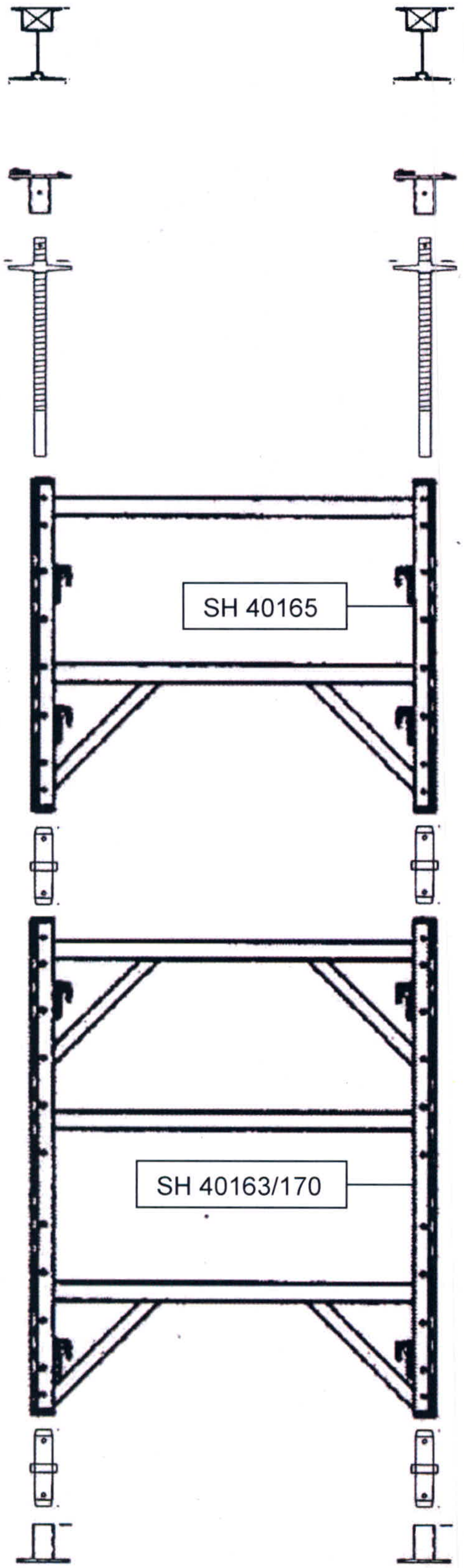
SH 40169
Broken frame part of SH 40165
Zone 40G

Folder: 08-02-27; SH 40169



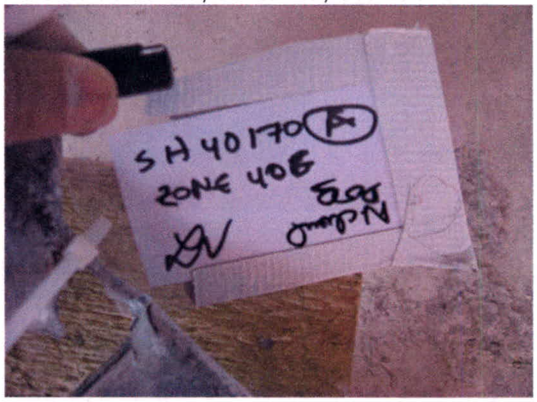
SH40169_1

SH40169_2

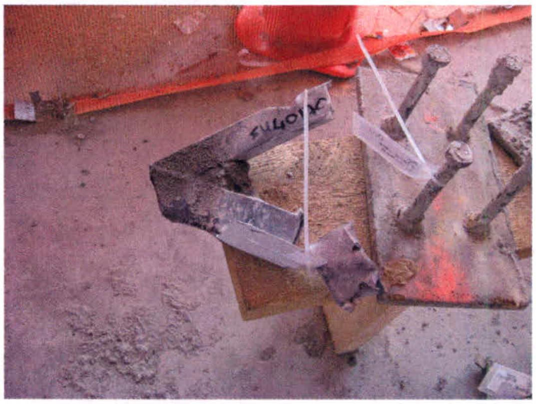


SH 40170
Piece of frame
Part of SH 40163
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40170

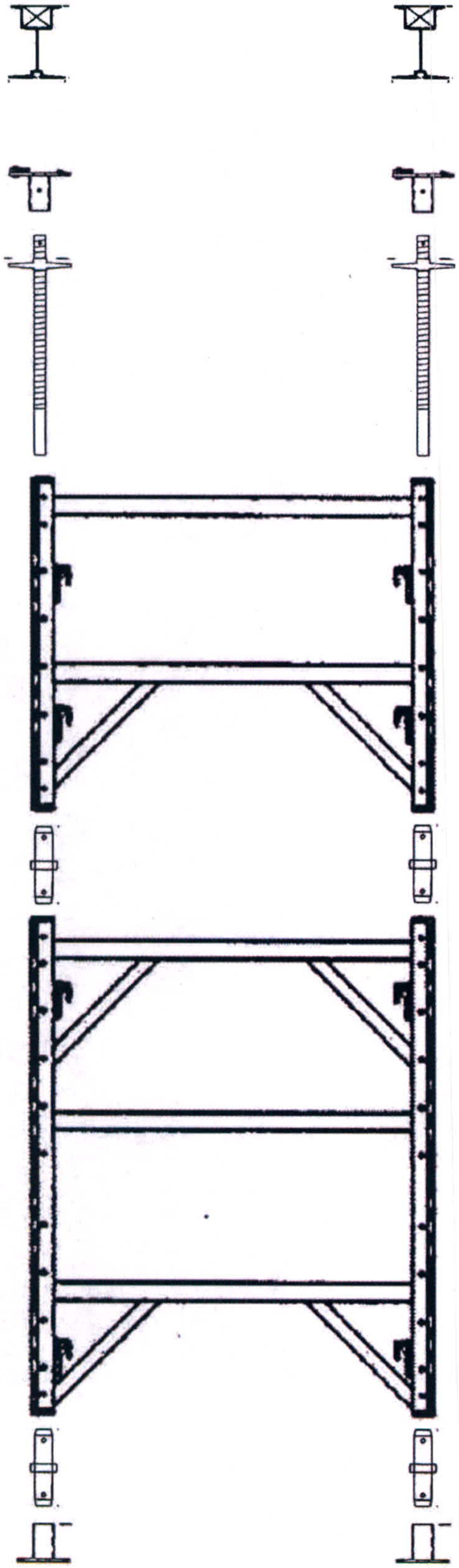


SH40170_1



SH40170_2

SH40170_3



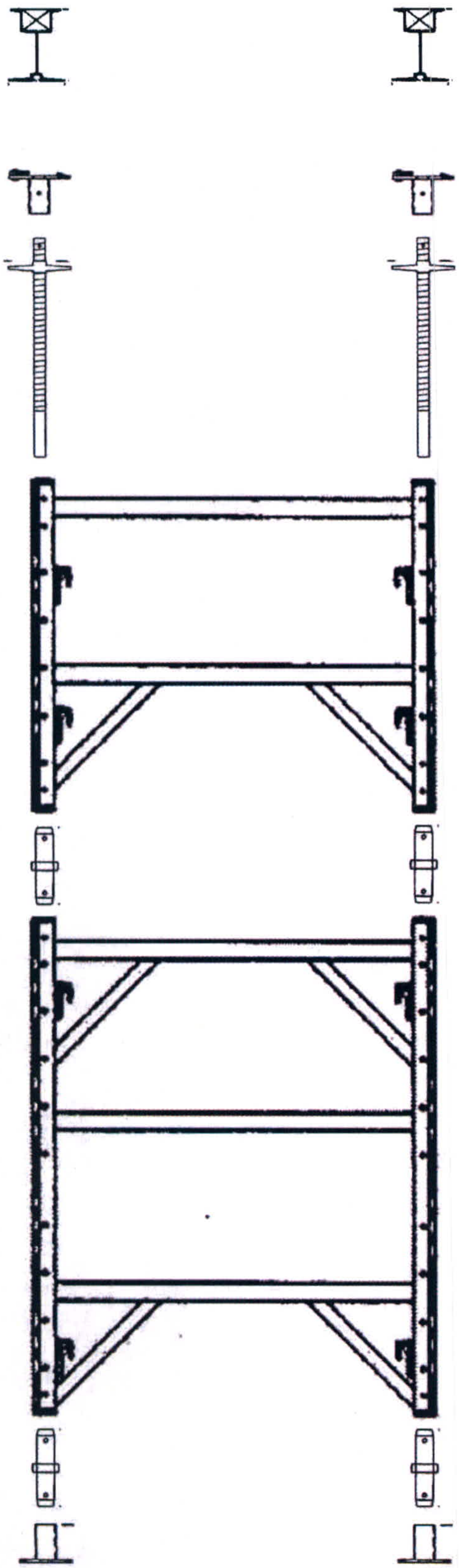
SH 40171
Top plate
Zone 40F

Folder: 08-02-27; 08-02-27; SH 40171

SH40171_1

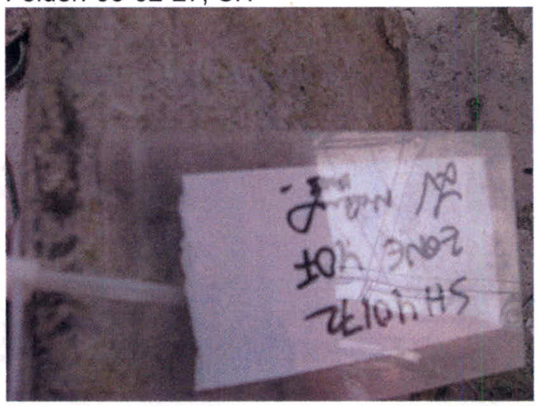
SH40171_2

SH40171_3



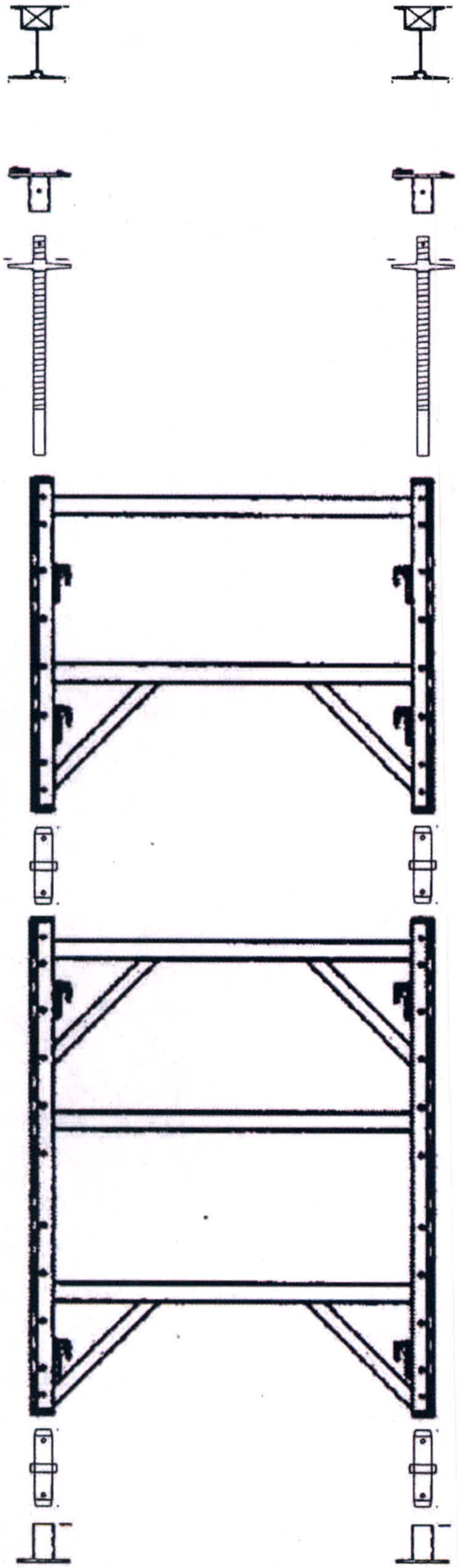
SH 40172
Top plate
Zone 40F

Folder: 08-02-27; SH



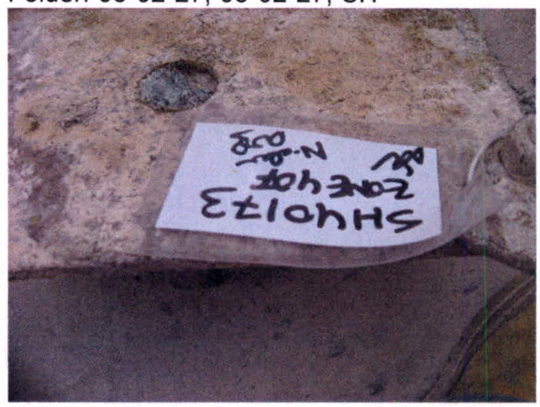
SH40172_1

SH40172_2



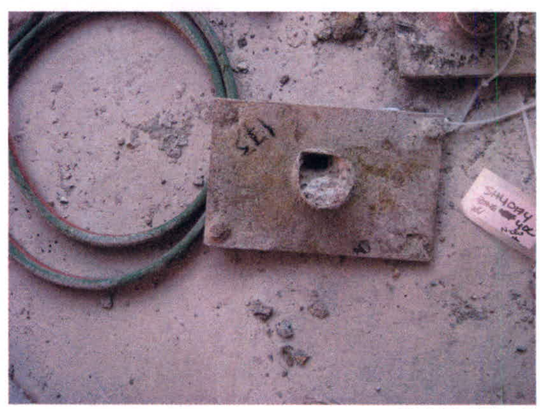
SH 40173
Top plate
Zone 40F

Folder: 08-02-27; 08-02-27; SH

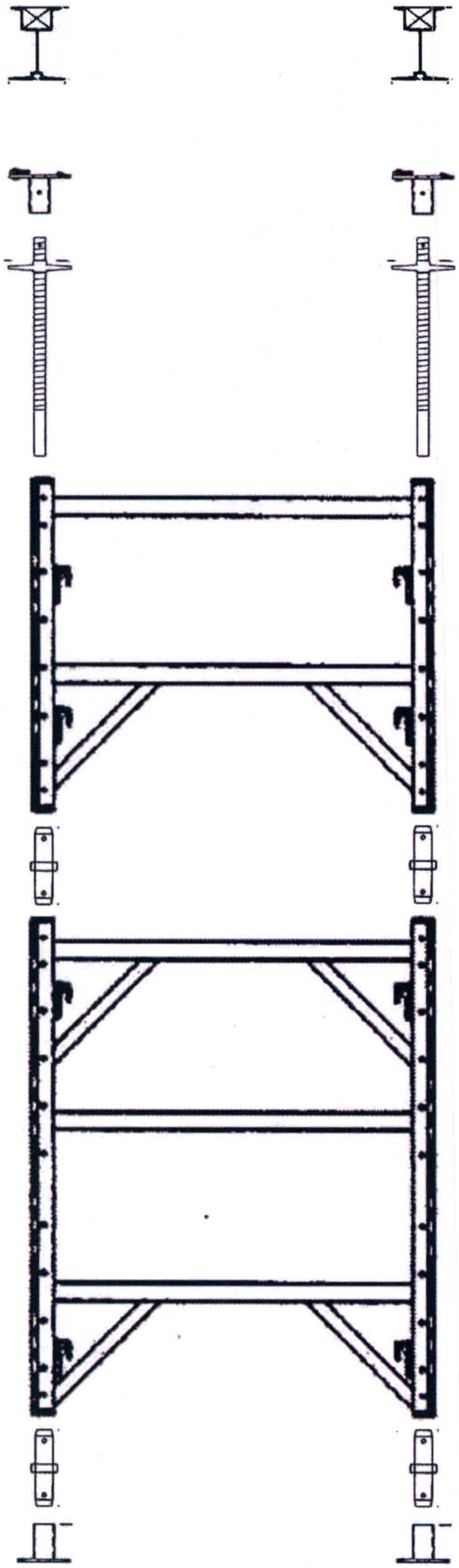


SH40173_1

SH40173_2

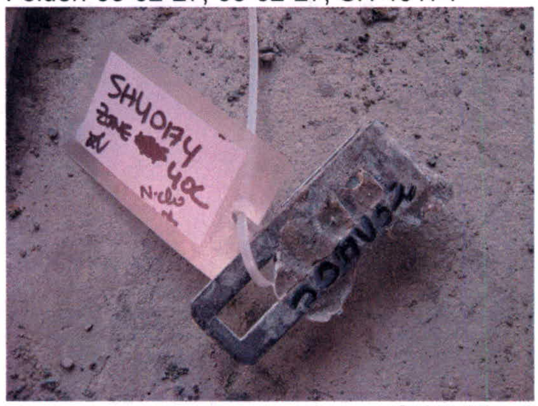


SH40173_3

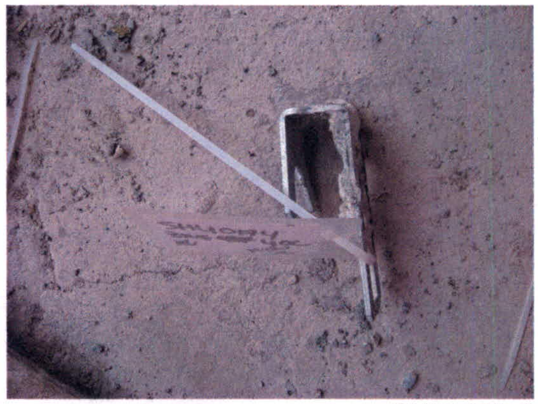


SH 40174
Bracket
Zone 40C

Folder: 08-02-27; 08-02-27; SH 40174

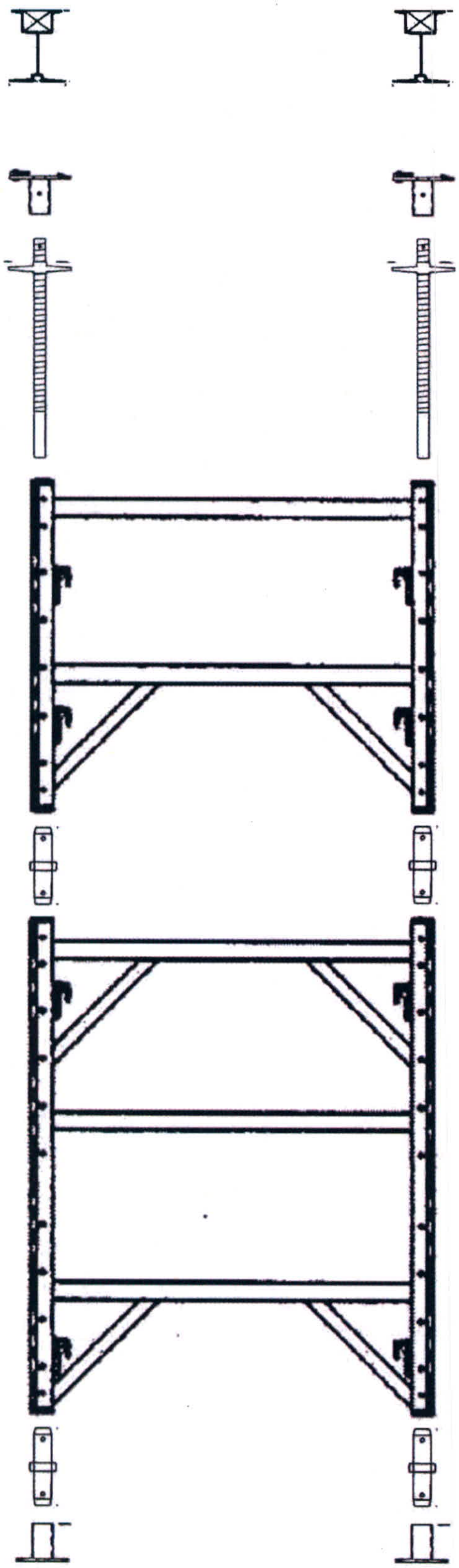


SH40174_1



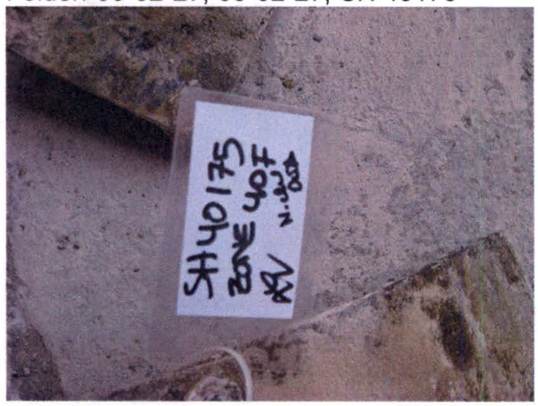
SH40174_2

SH40174_3



SH 40175
Top plate
Zone 40F

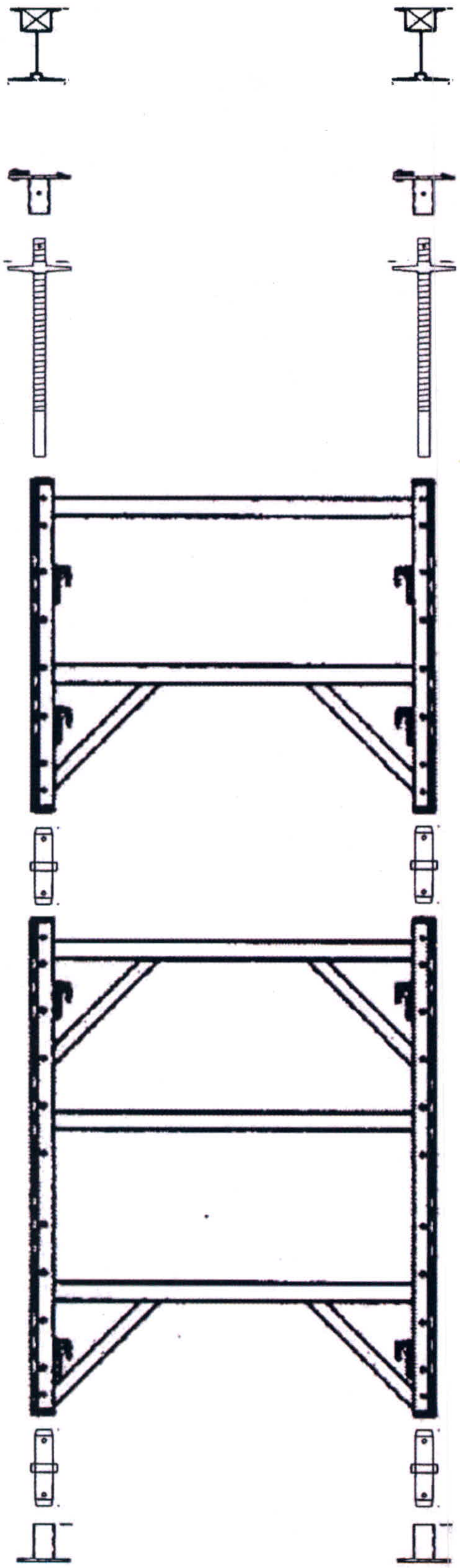
Folder: 08-02-27; 08-02-27; SH 40175



SH40175_1

SH40175_2

SH40175_3



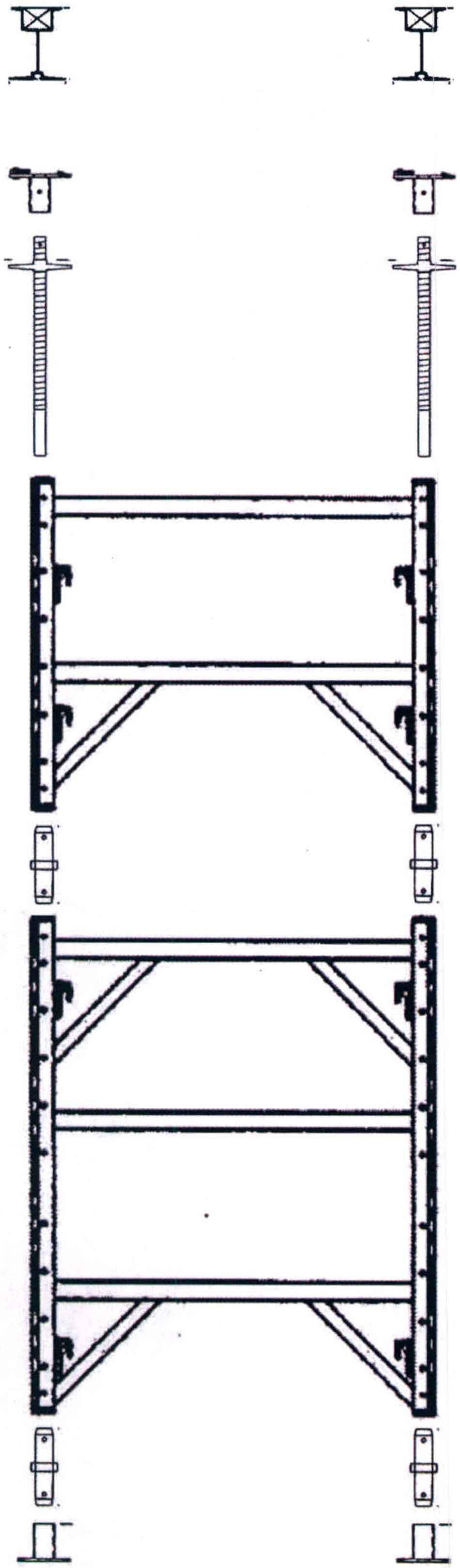
SH 40176
Piece of frame, part of frame SH 40180
Zone 40G

Folder: 08-02-27; 08-02-27; SH

SH40176_1

SH40176_3

SH40176_4



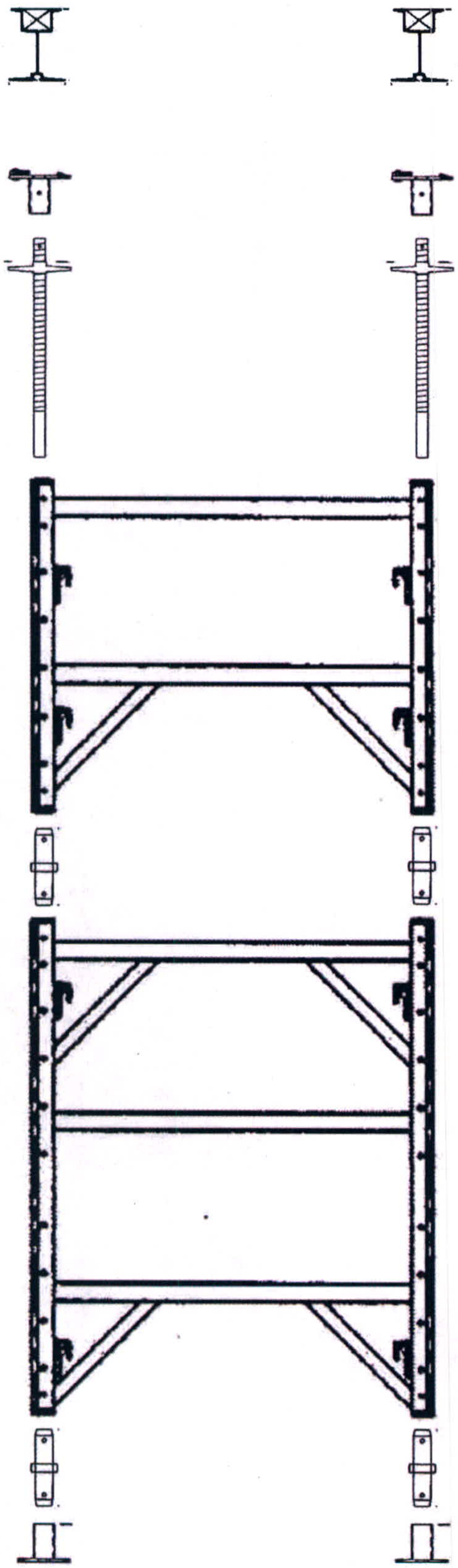
SH 40177
Piece of frame
Part of SH 40180
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40177

SH40177_1

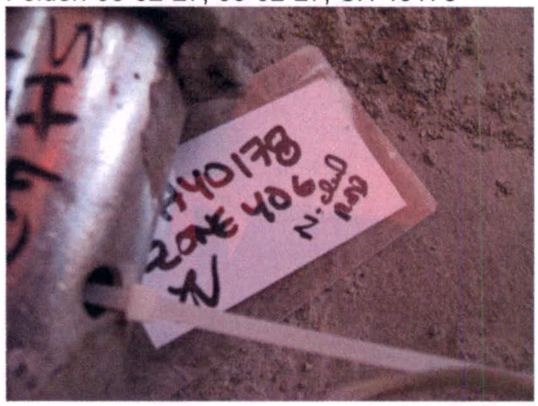
SH40177_2

SH40177_3



SH 40178
Coupler
Zone 40G

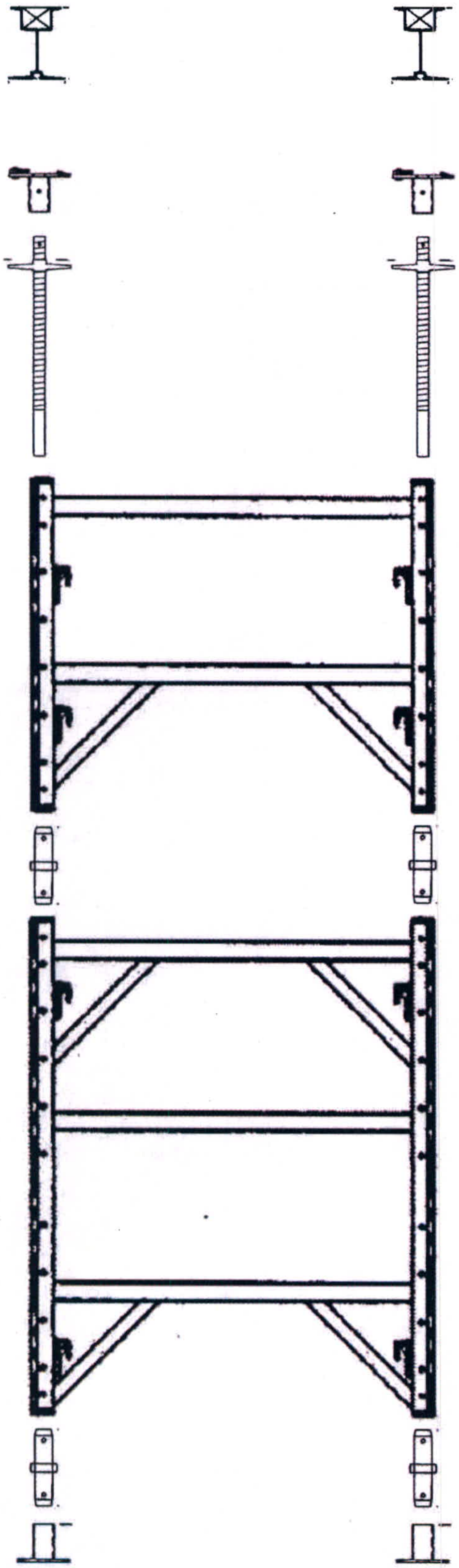
Folder: 08-02-27; 08-02-27; SH 40178



SH40178_3

SH40178_1

SH40178_2

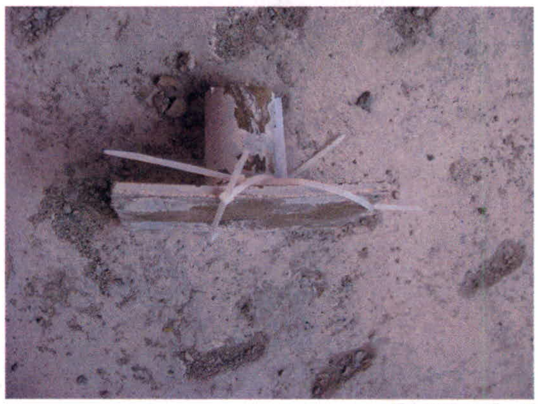


SH 40179
Bottom plate
Zone 40G

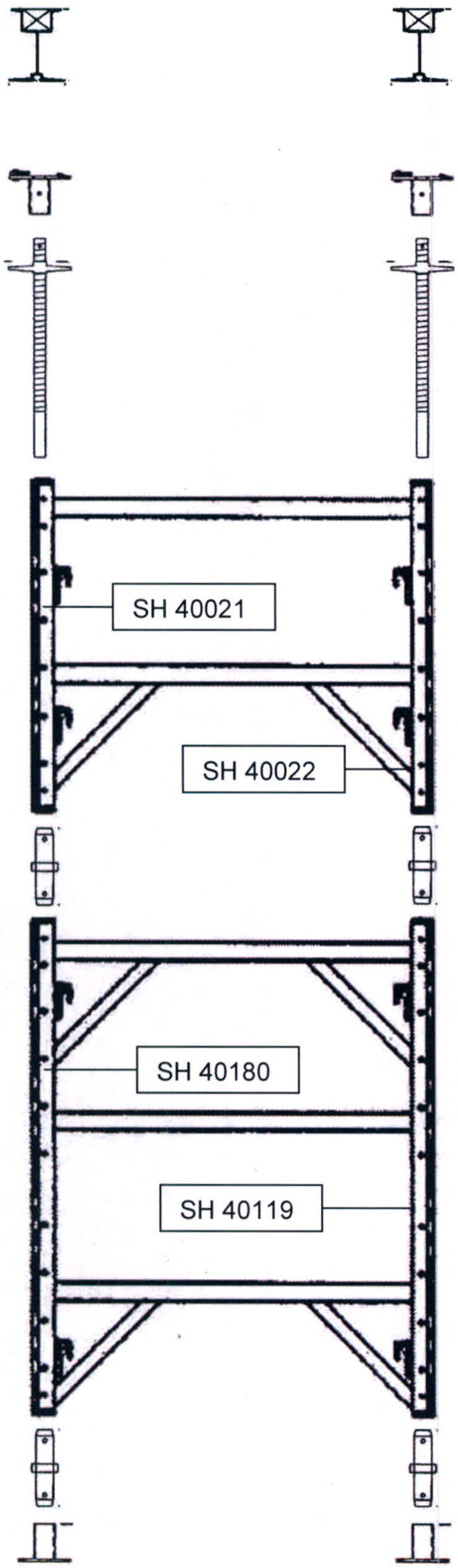
Folder: 08-02-27; 08-02-27; SH 40179

SH40179_1

SH40179_2



SH40179_3



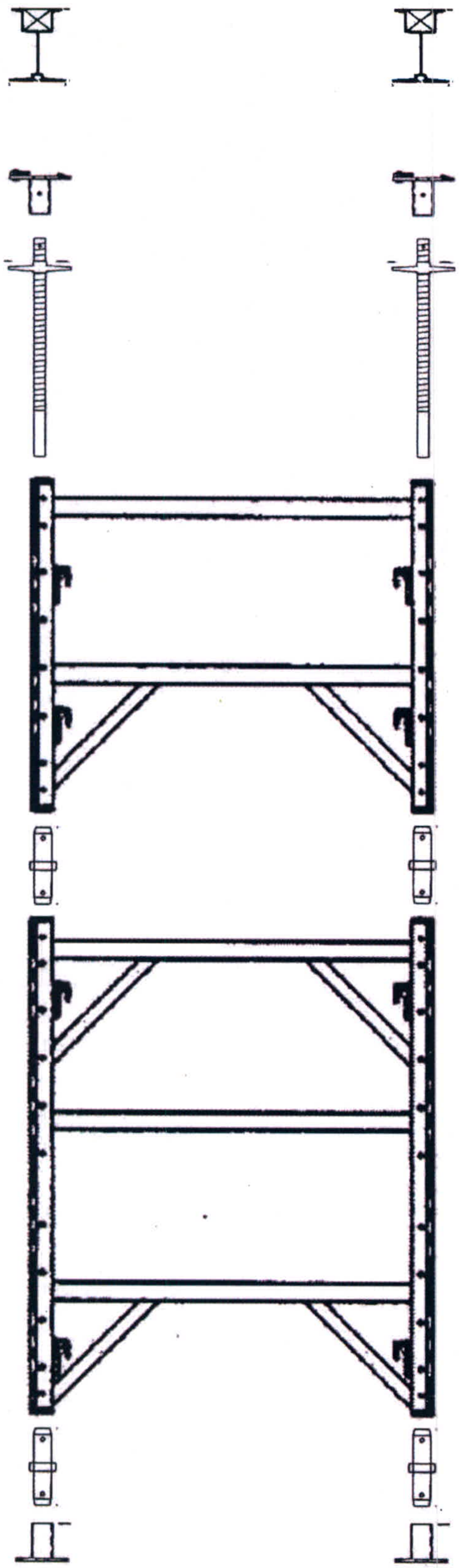
SH 40180
Bottom Frame (5'-3") bottom screw jack
located near SH 40195/196
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40180

SH40180_1

SH40180_2

SH40180_3



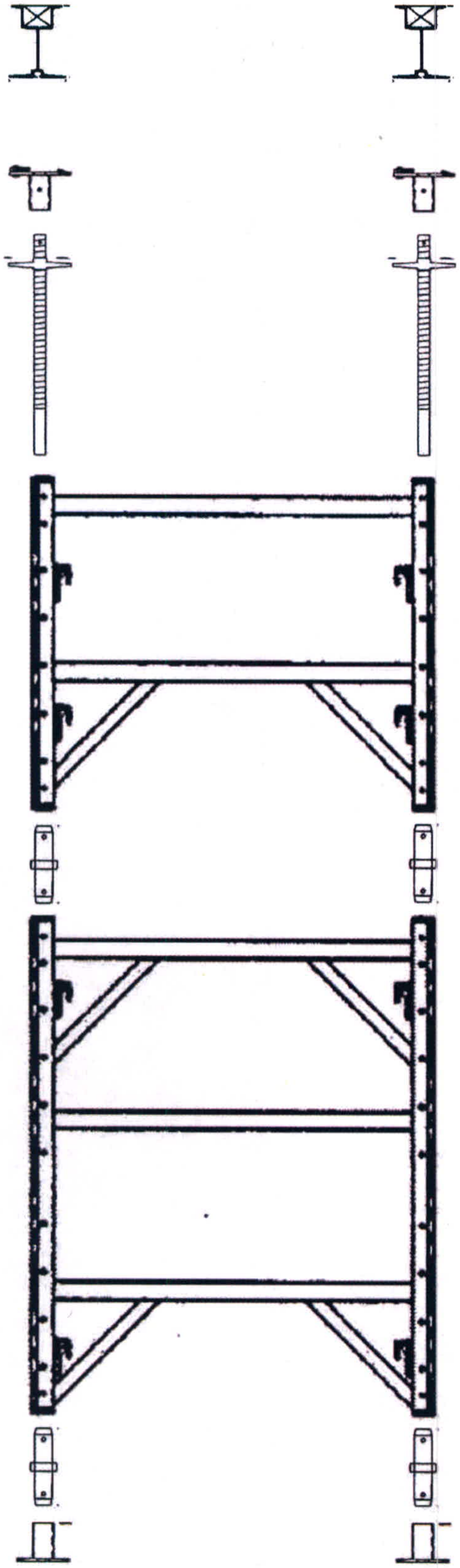
SH 40181
Brace
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40181

SH40181_1

SH40181_2

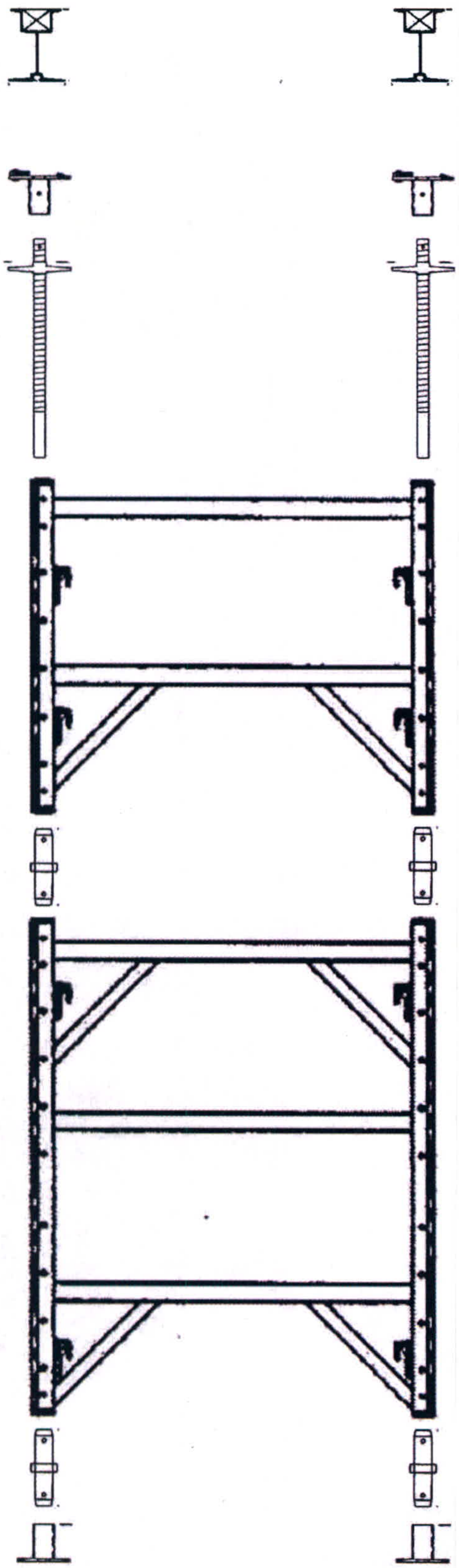
SH40181_3



SH 40182
Piece of SH 40163 Bottom diagonal
Zone 40G

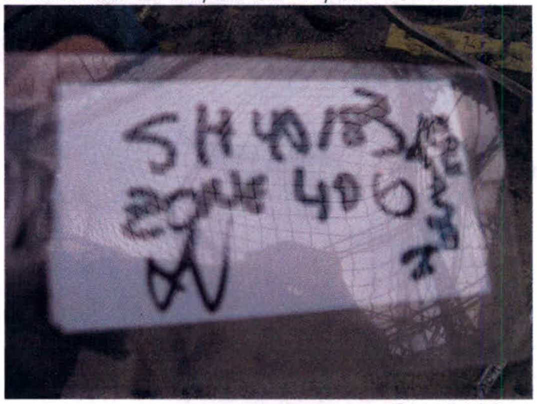
Folder: 08-02-27; 08-02-27; SH 40182

SH40182_1



SH 40183
Piece of diagonal
Zone 40G

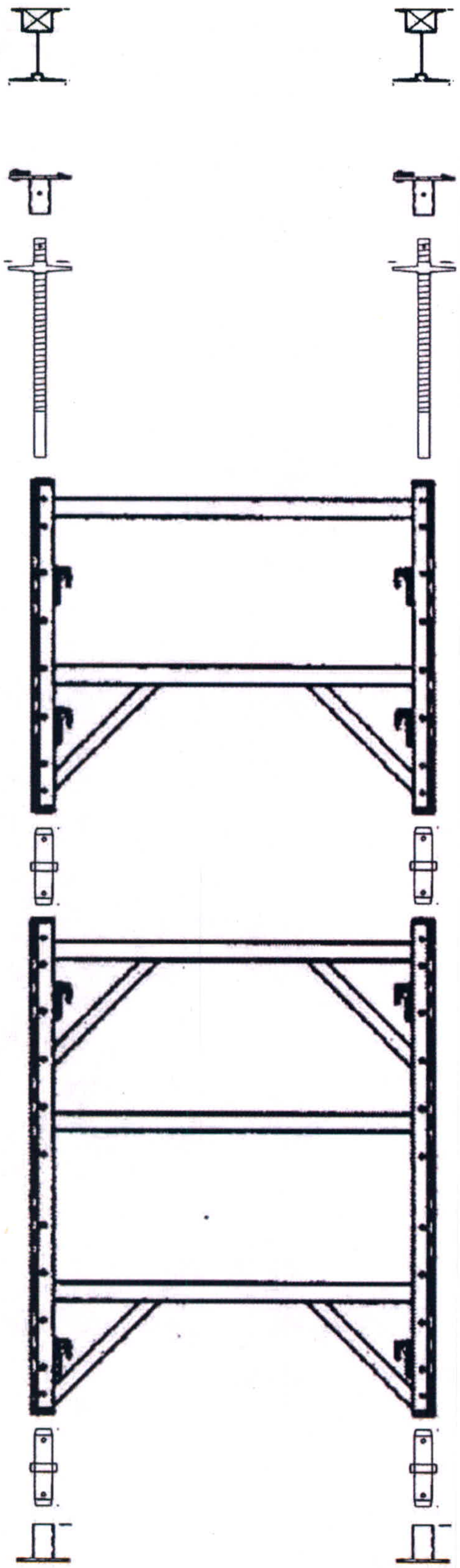
Folder: 08-02-27; 08-02-27; SH 40183



SH40183_1

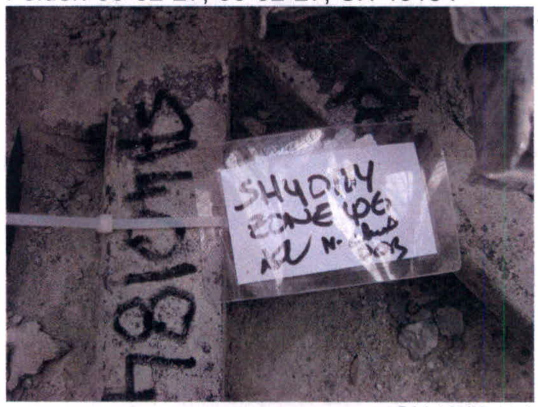
SH40183_2

SH40183_3

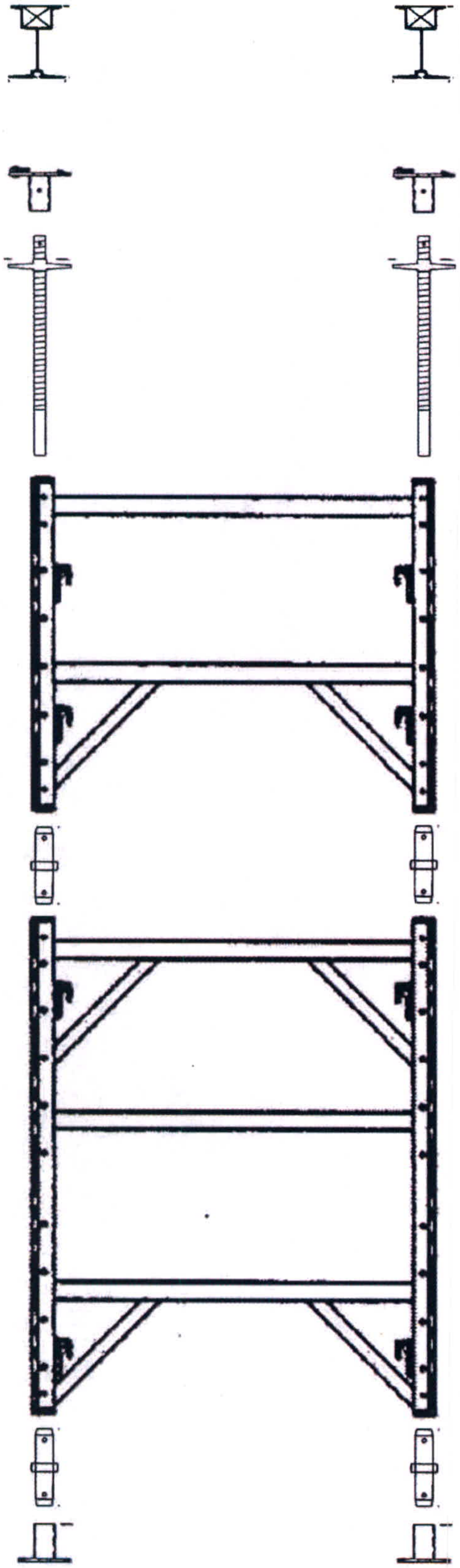


SH 40184
Horizontal and diagonal section of Tower
part of Frame SH 40180
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40184

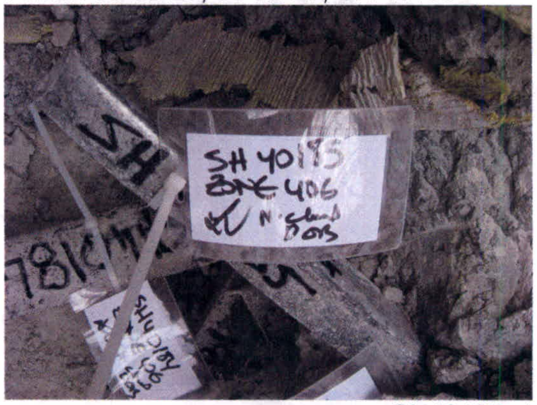


SH40184_1

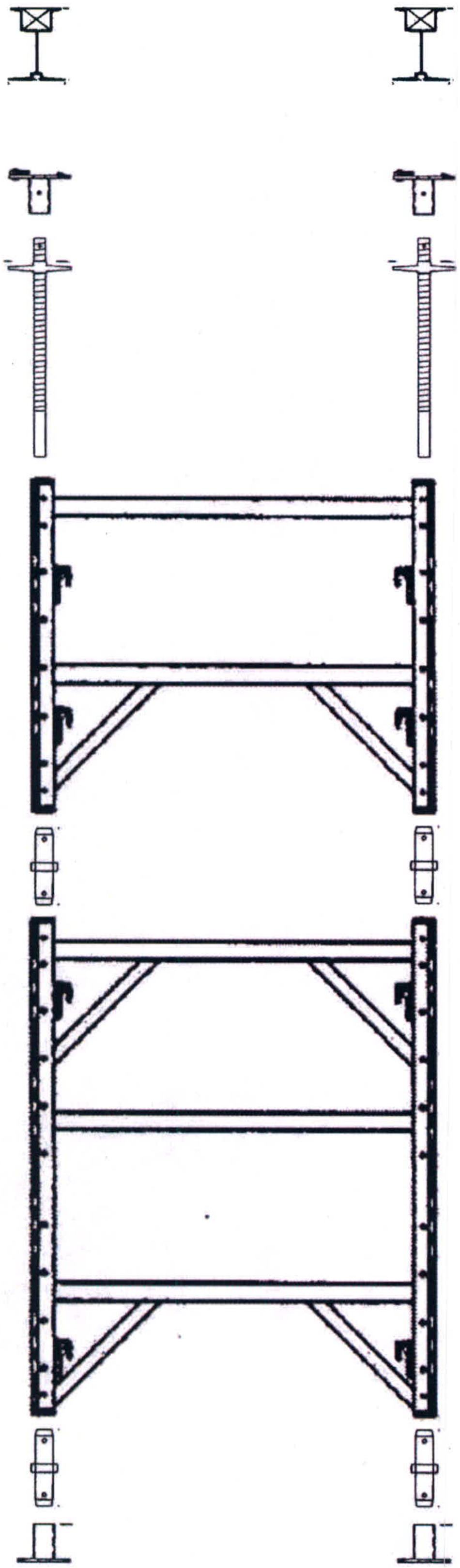


SH 40185
Brace
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40185

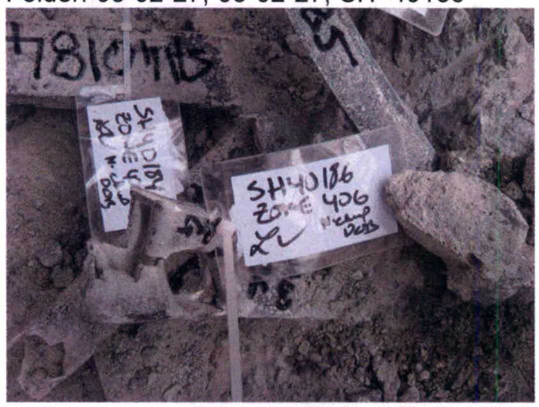


SH40185_1

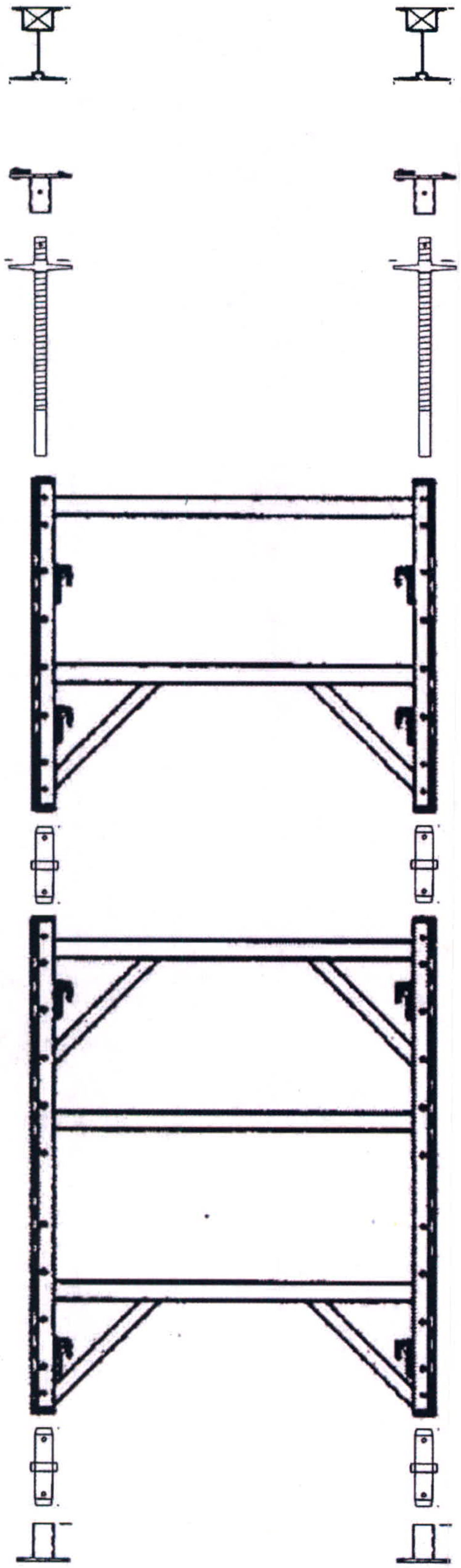


SH 40186
Horizontal and Diagonal section of Tower
Part of SH 40180
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40186



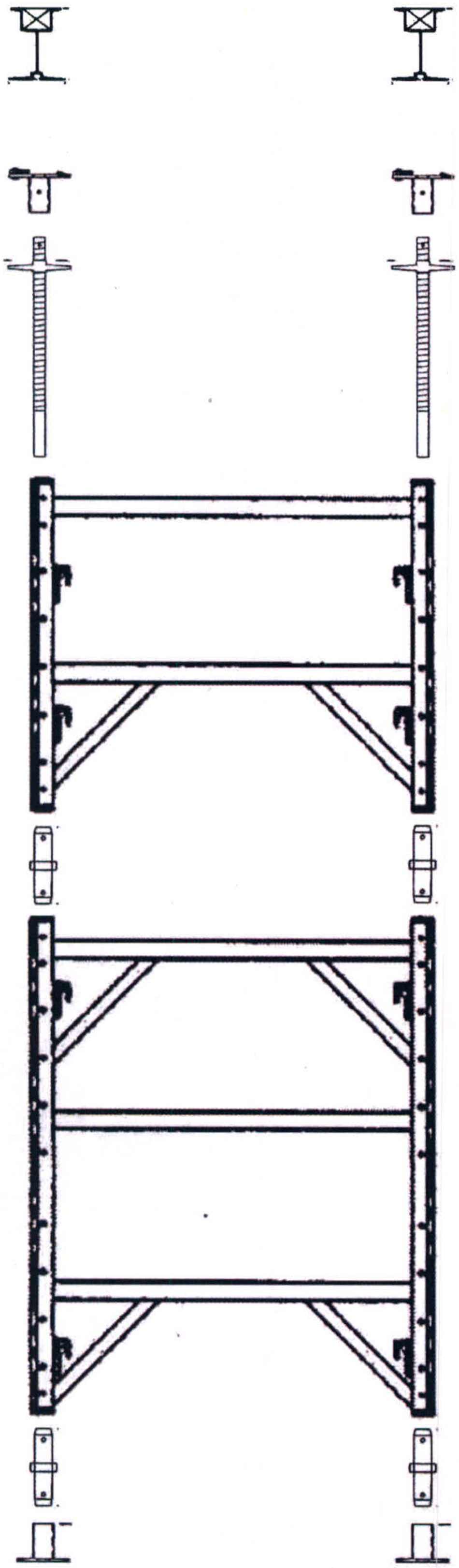
SH40186_1



SH 40187
Horizontal and Diagonal section of tower
Part of SH 40135
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40187

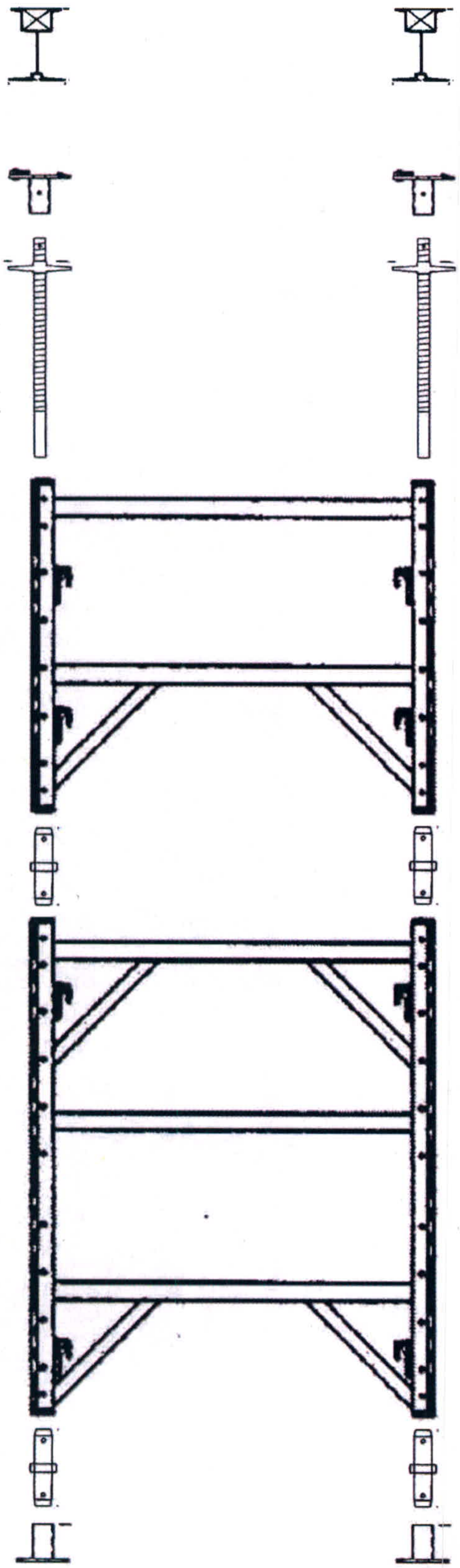
SH40187_1



SH 40188
Brace connect to SH 40135
Zone 40G

Folder: 08-02-27; 08-02-27; SH 40188

SH40188_1



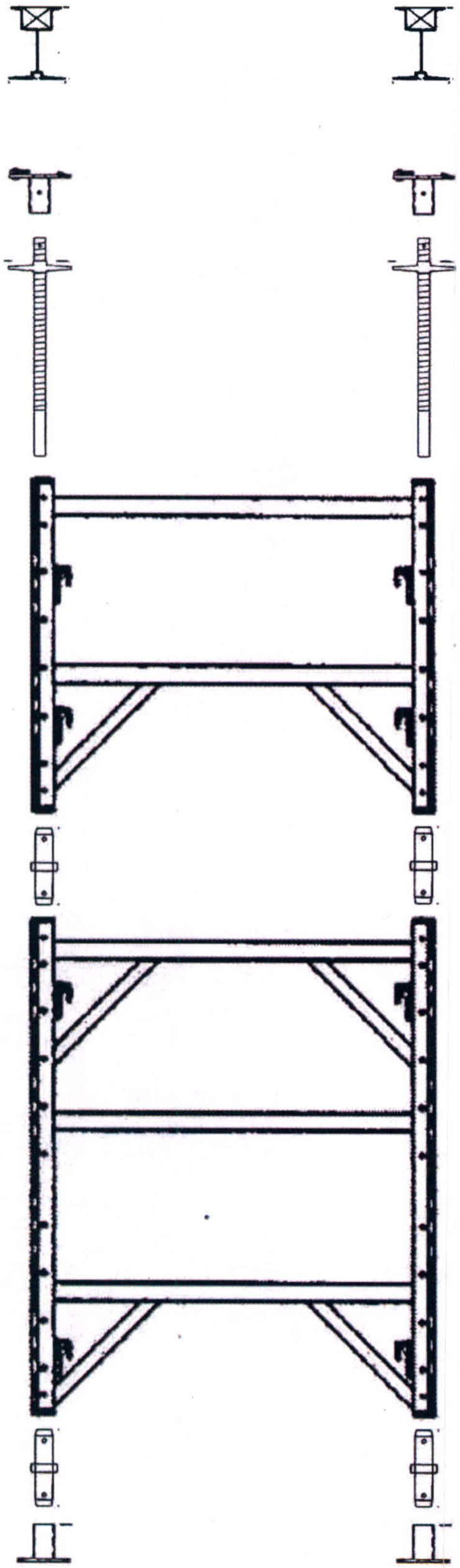
SH 40189
Bottom plate
Zone 40G

Folder: 08-02-28; 08-02-28; SH 40189

SH40189_1

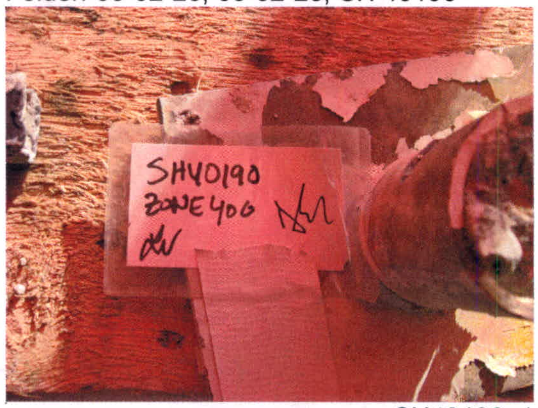
SH40189_2

SH40189_3



SH 40190
Bottom plate
Zone 40G

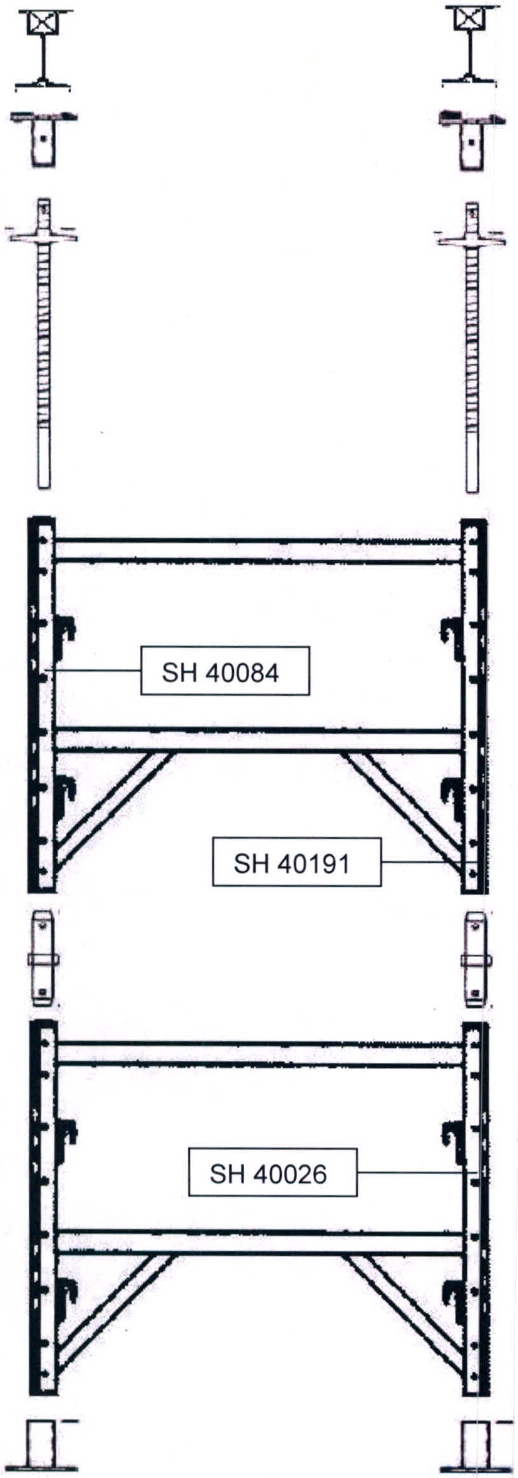
Folder: 08-02-28; 08-02-28; SH 40190



SH40190_1

SH40190_2

SH40190_3



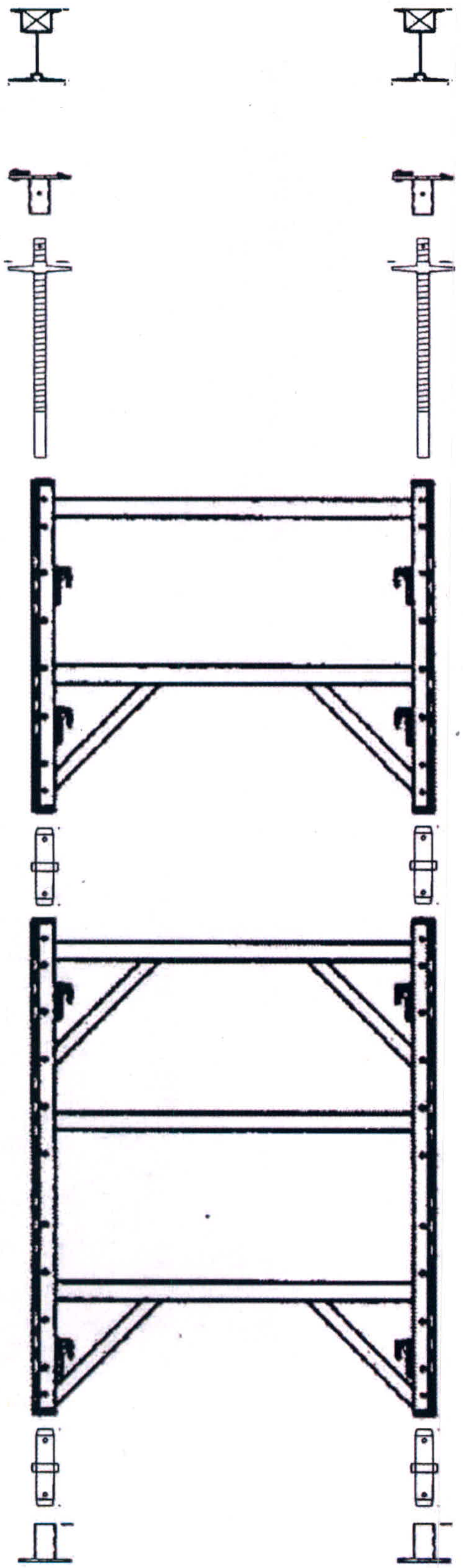
SH 40191
Top Frame connected to SH 40135 on 41st floor
Zone 40G-F

Folder: 08-02-28; 08-02-28; SH 40191

SH40191_1

SH40191_2

SH40191_3



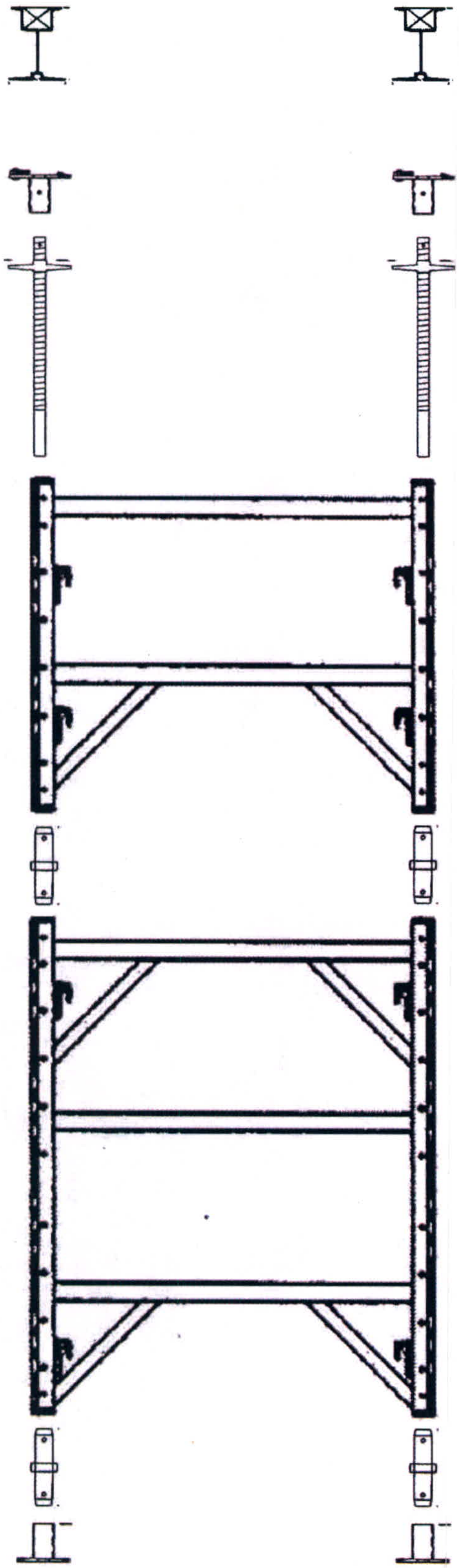
SH 40192
Piece of frame
Part of SH 40180
Zone 40G

Folder: 08-02-28; 08-02-28; SH 40192

SH40192_1

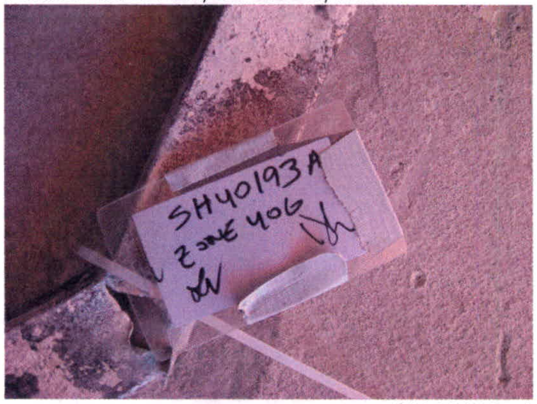
SH40192_2

SH40192_3



SH 40193
Piece of frame
Zone 40G

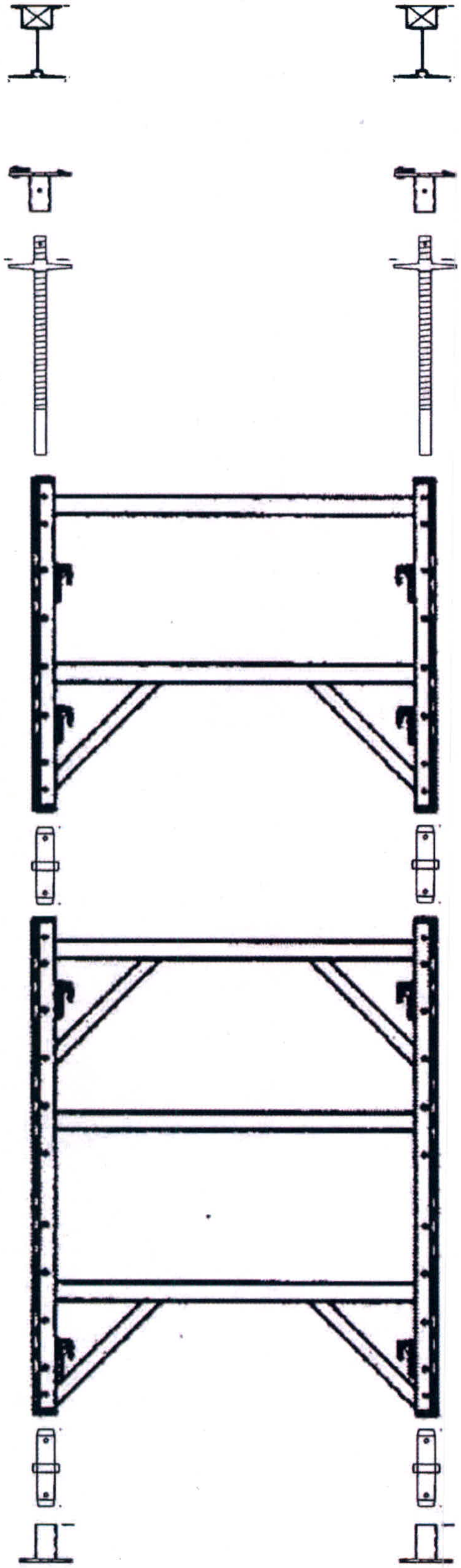
Folder: 08-02-28; 08-02-28; SH



SH40193_1

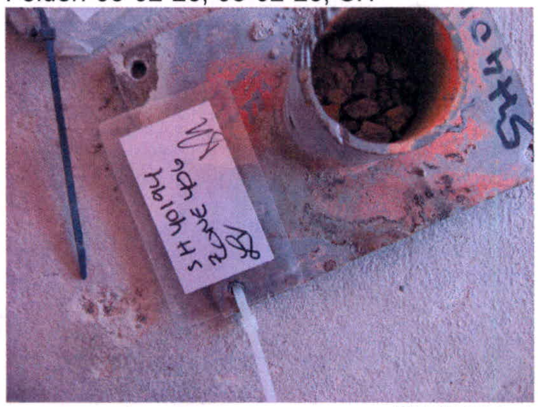

SH40193_2


SH40193_3



SH 40194
Bottom plate
Zone 40G

Folder: 08-02-28; 08-02-28; SH

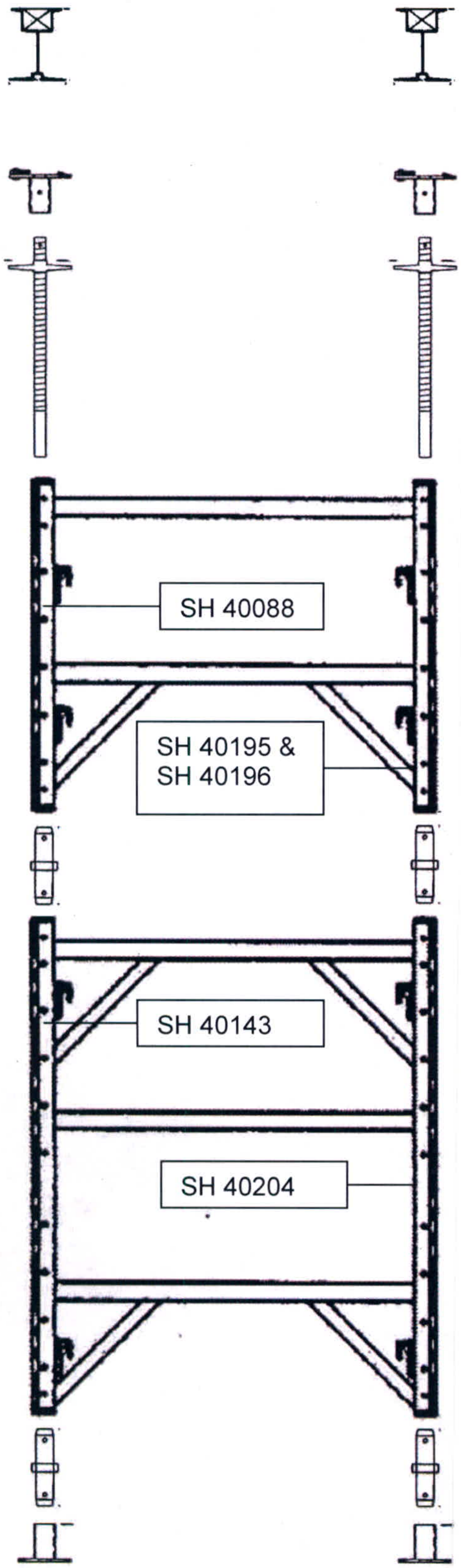


SH40194_1

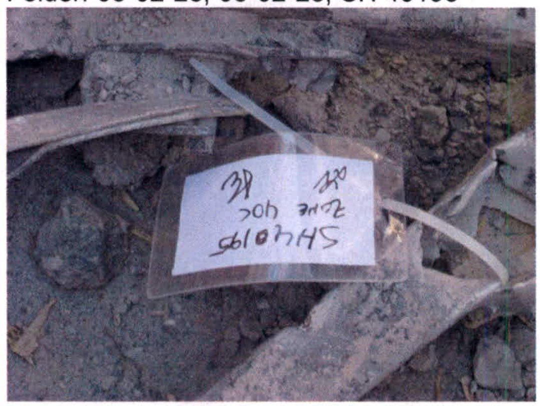
SH40194_2

SH40194_3

SH 40195
Top frame fractured off from SH 40196
SH 40185 and SH 40195 were connected
Zone 40C



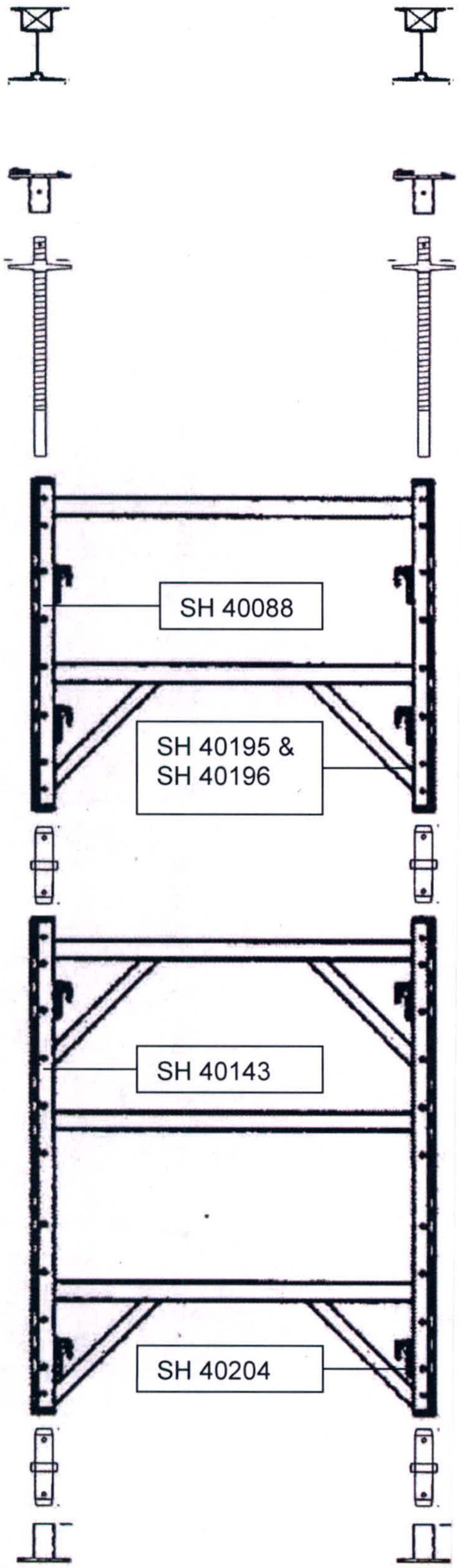
Folder: 08-02-28; 08-02-28; SH 40195



SH40195_1

SH40195_2

SH40195_3



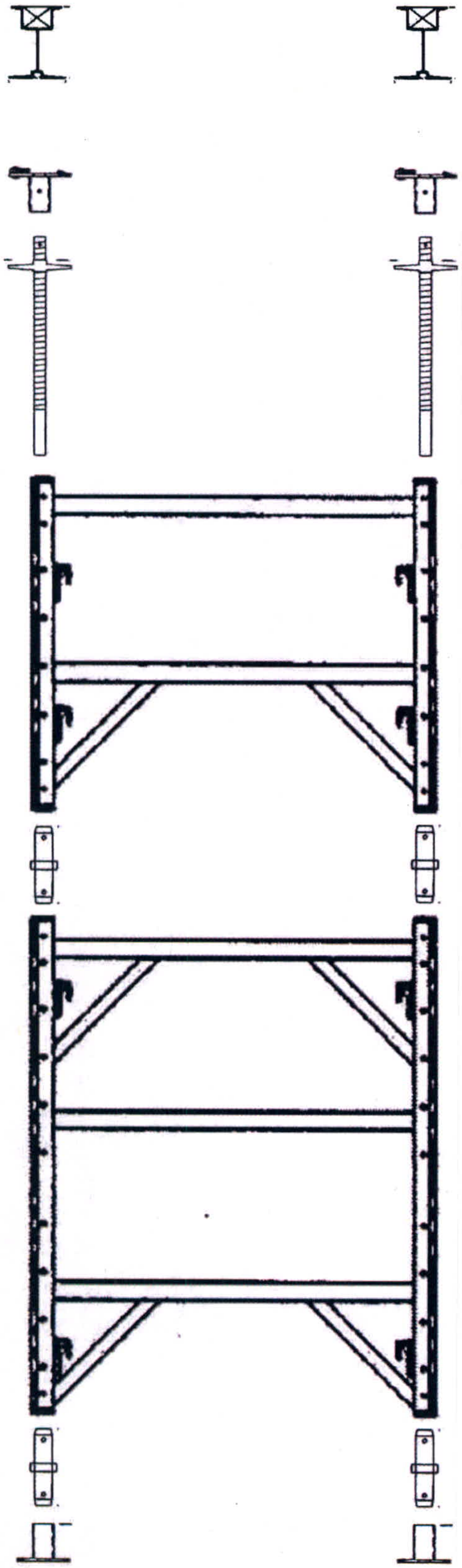
SH 40196
Remaining fractured part of SH 40195 frame buried in concrete
Zone 40C

Folder: 08-02-28; 08-02-28; SH 40196

SH40196_1

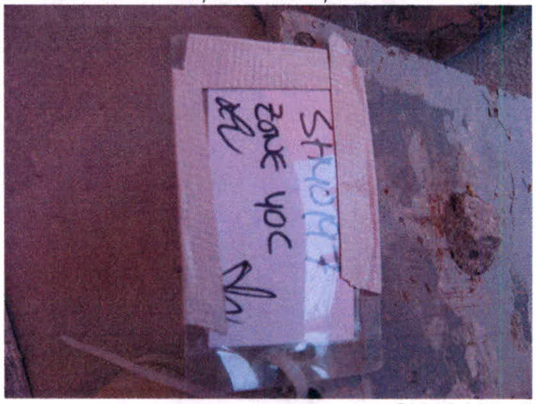
SH40196_2

SH40196_3



SH 40197
Bottom plate
Zone 40C

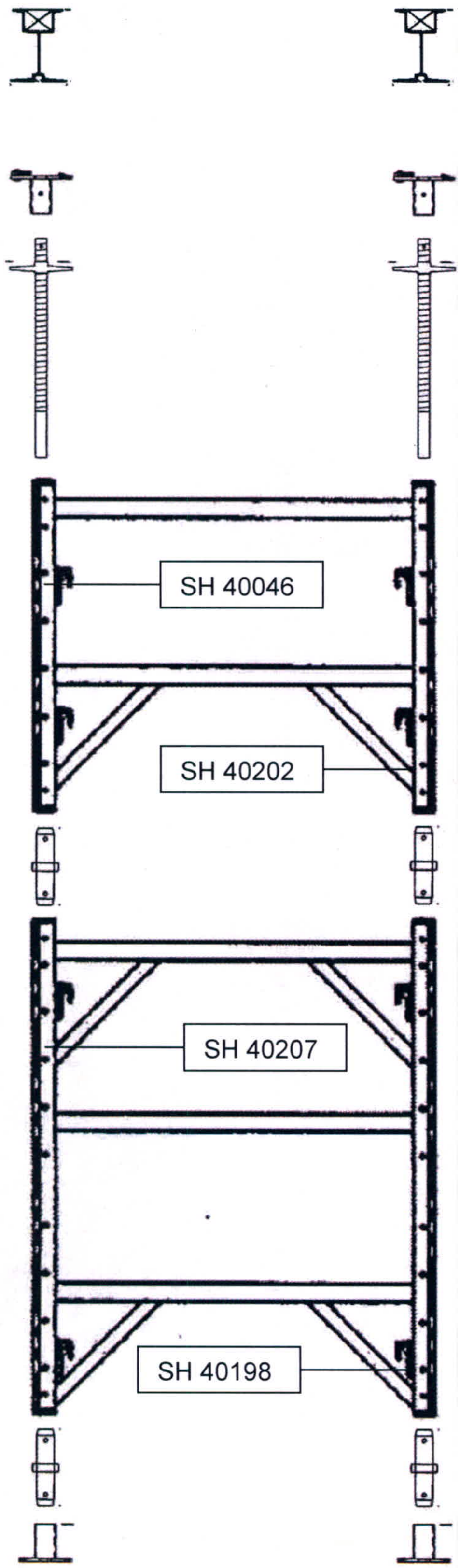
Folder: 08-02-28; 08-02-28; SH 40197



SH40197_1

SH40197_2

SH40197_4



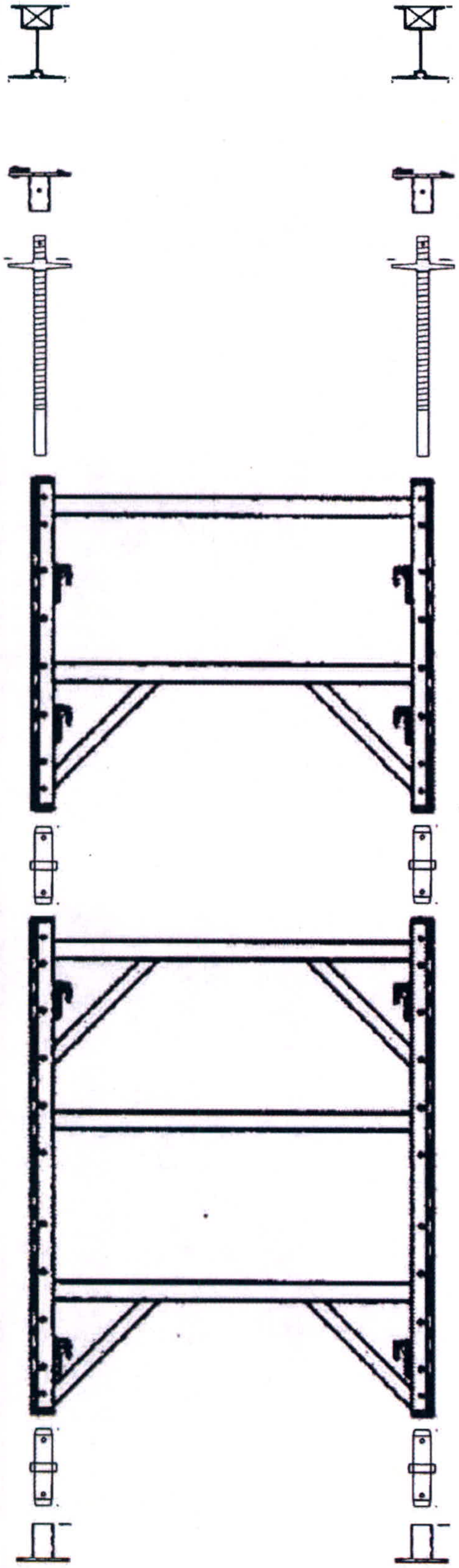
SH 40198
Frame buried in concrete
Zone 40C

Folder: 08-02-28; 08-02-28; SH 40198

SH40198_1

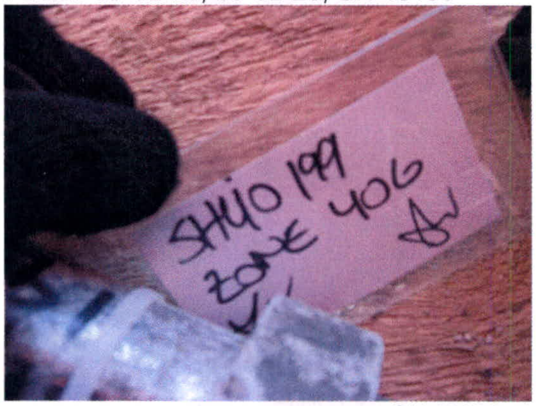
SH40198_2

SH40198_4



SH 40199
Coupler buried in Concrete
Zone 40C

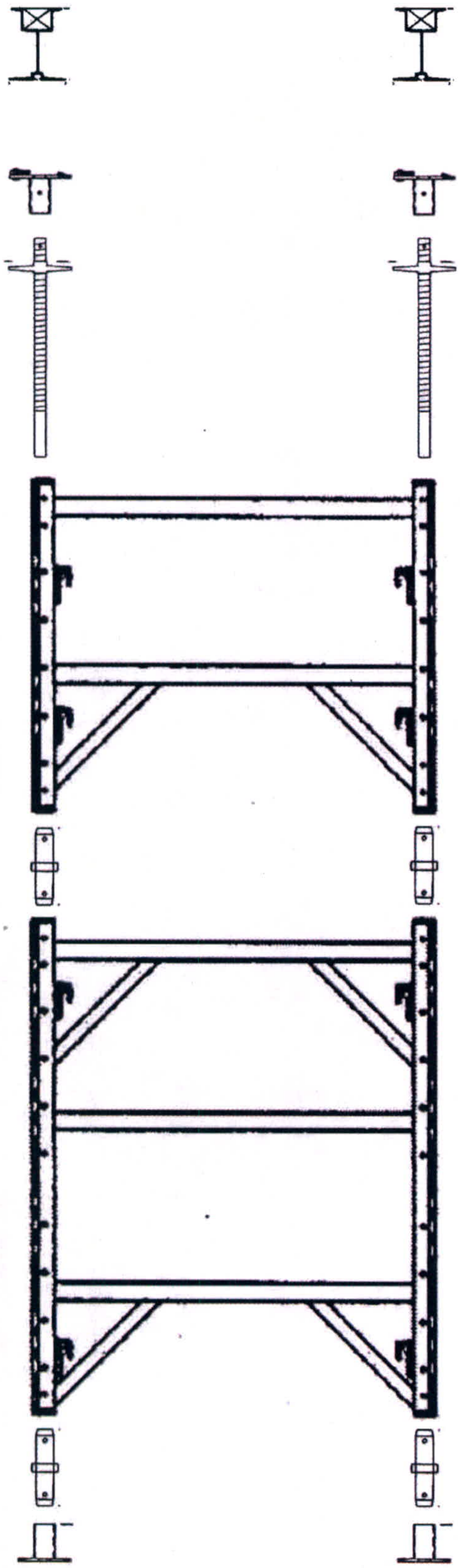
Folder: 08-02-28; 08-02-28; SH 40199



SH40199_1

SH40199_2

SH40199_3



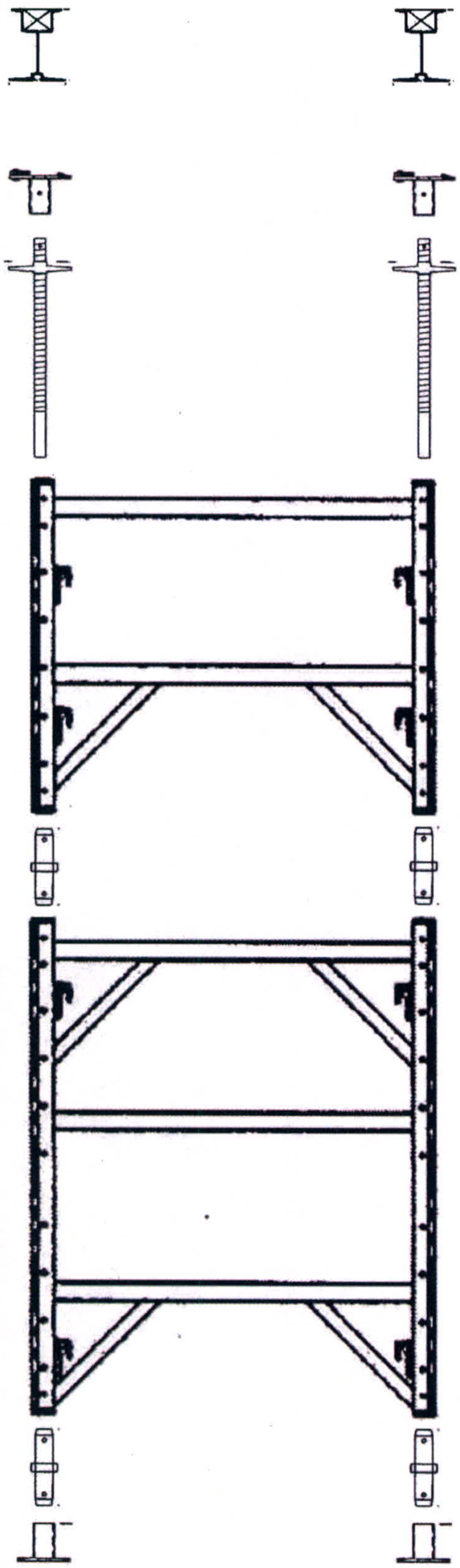
SH 40200
Brace buried in concrete
Zone 40C

Folder: 08-02-28; 08-02-28; SH 40200

SH40200_1

SH40200_2

SH40200_3

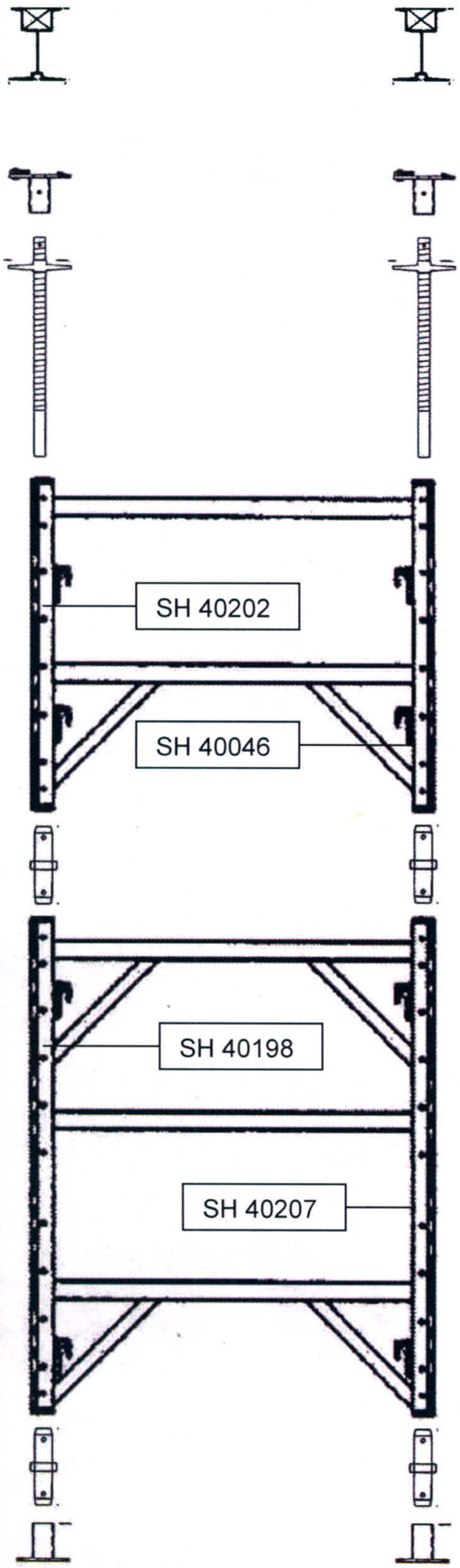


SH 40201
Brace
Zone 40C

Folder: 08-02-28; 08-02-28; SH 40201

SH40201_1

SH40201_2



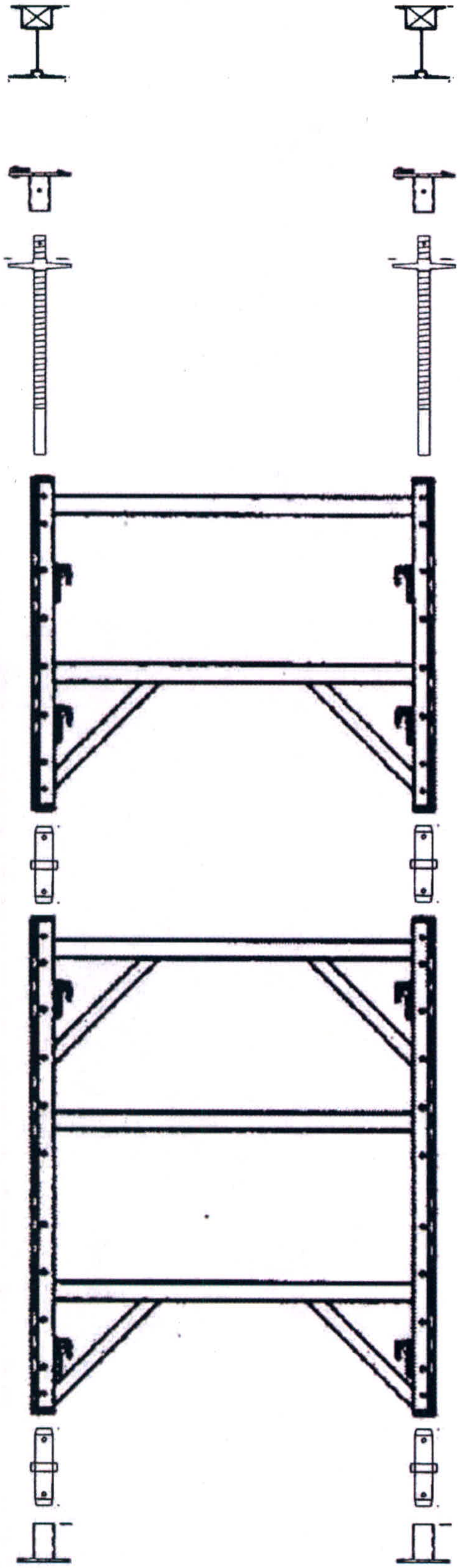
SH 40202
Shoring tower
Zone 40C

Folder: 08-02-28; 08-02-28; SH 40202

SH40202_1

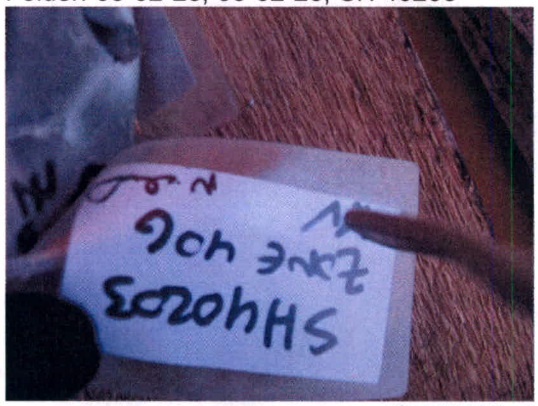
SH40202_2

SH40202_3



SH 40203
Piece of Frame
Zone 40G

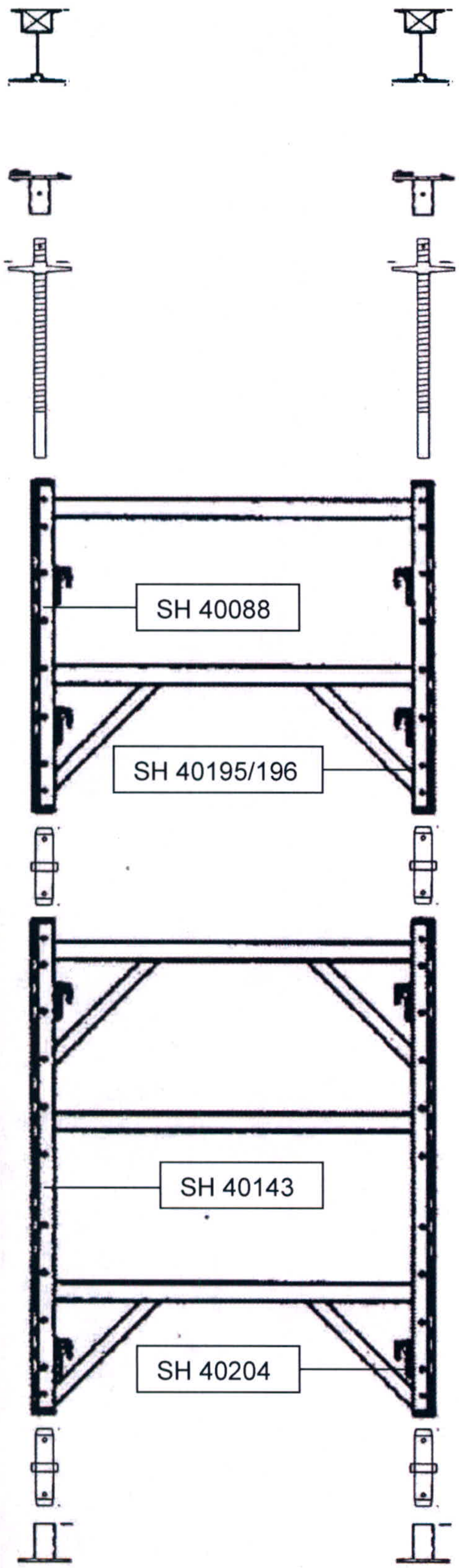
Folder: 08-02-28; 08-02-28; SH 40203



SH40203_2

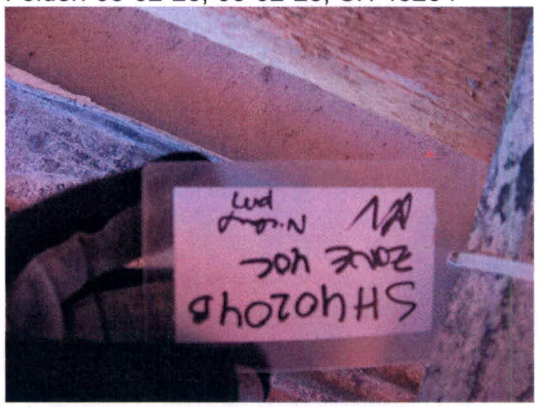
SH40203_3


SH40203_4



SH 40204
Bottom Frame buried
Zone 40C

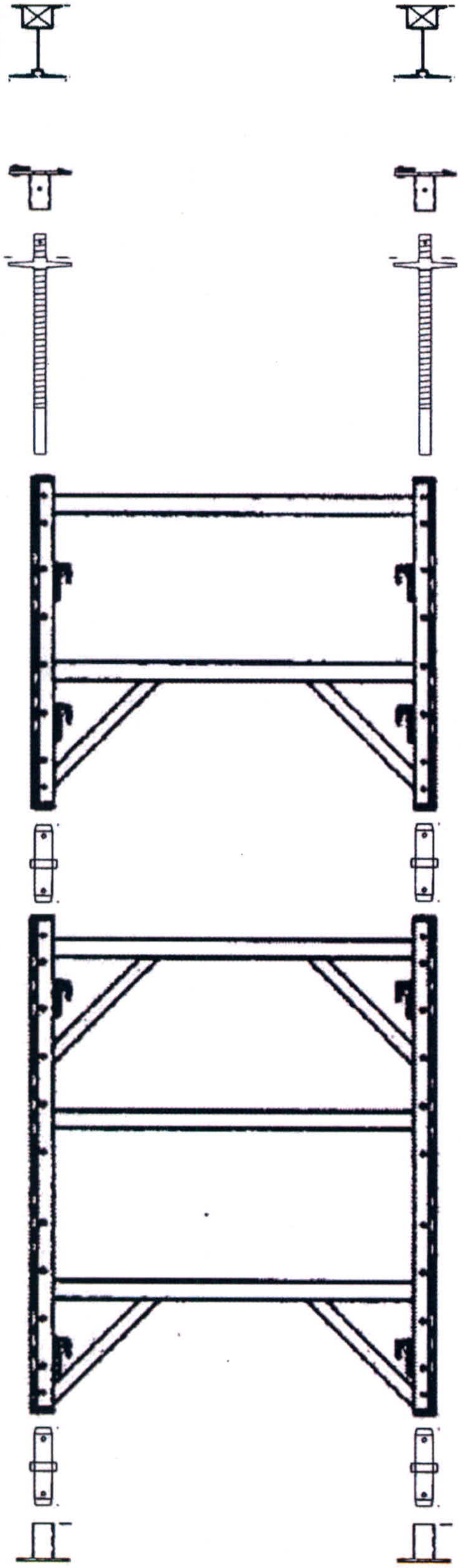
Folder: 08-02-28; 08-02-28; SH 40204



SH40204_1

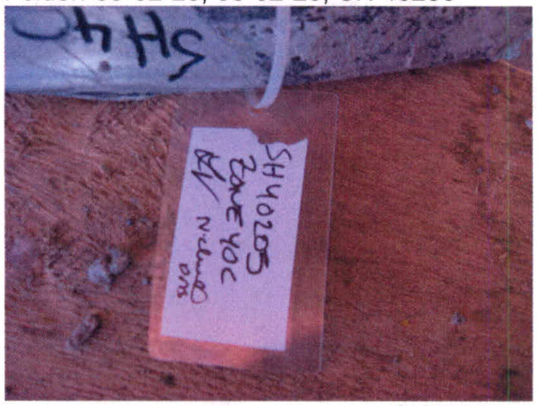
SH40204_2

SH40204_3



SH 40205
Piece of frame
Zone 40C

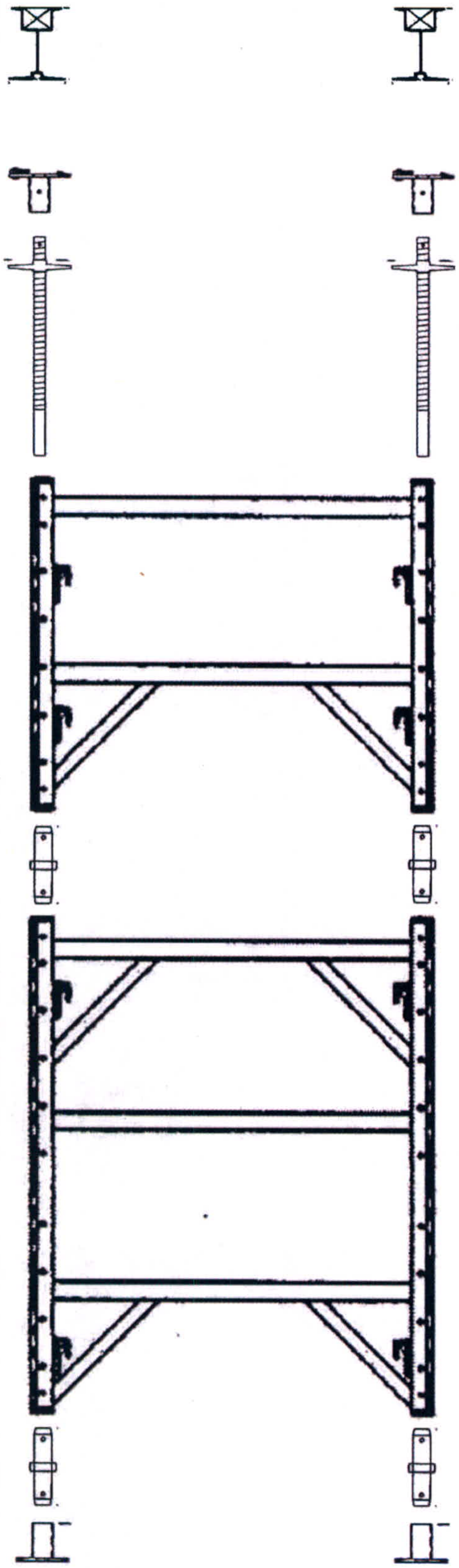
Folder: 08-02-28; 08-02-28; SH 40205



SH40205_1

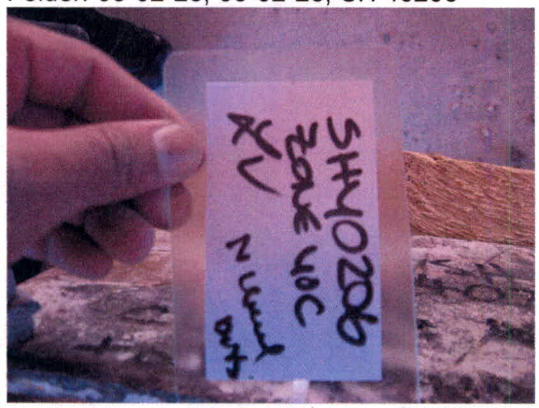
SH40205_2

SH40205_3



SH 40206
Top piece connect to SH 40196(Top plate)
Zone 40C

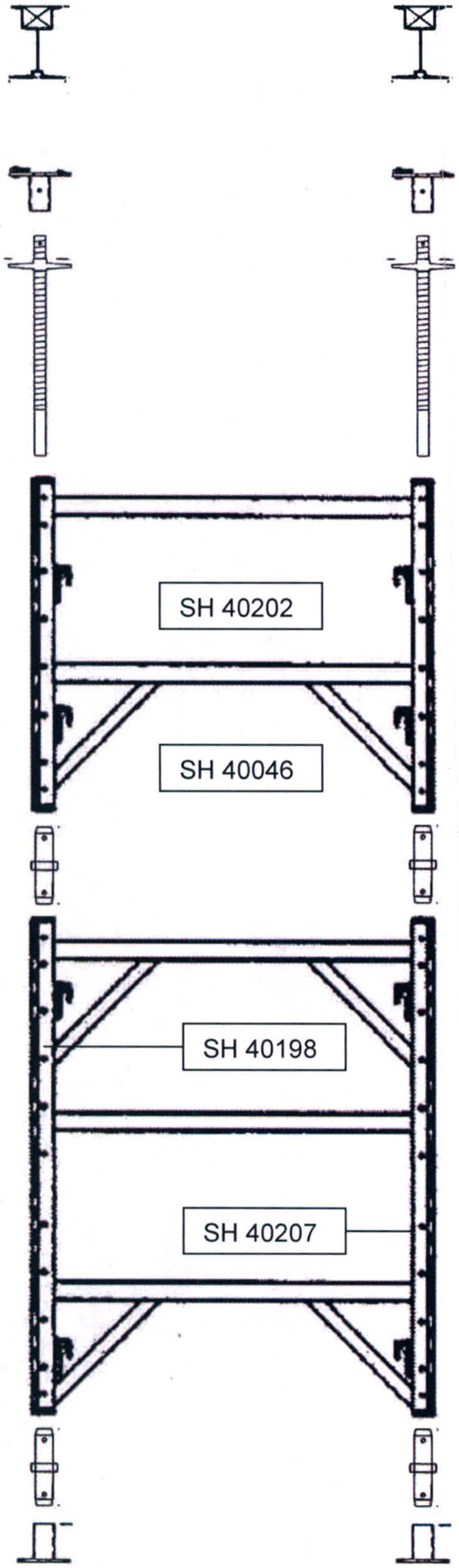
Folder: 08-02-28; 08-02-28; SH 40206



SH40206_1

SH40206_2

SH40206_3



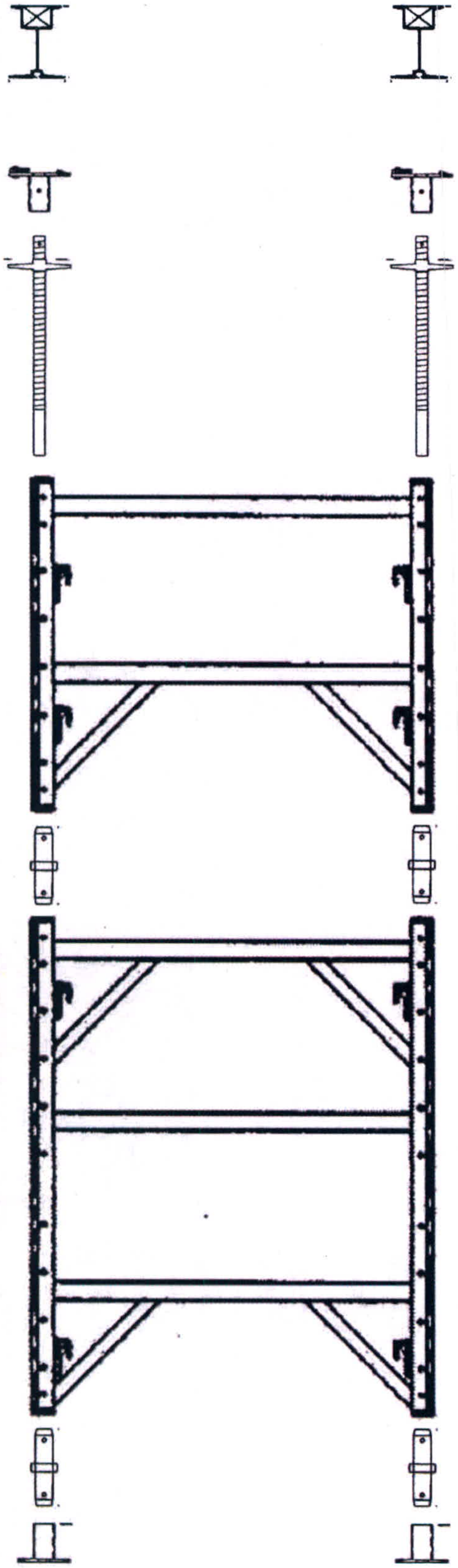
SH 40207
Bottom Frame Broken (torched)
Zone 40C

Folder: 08-02-29; SH 40207

SH40207_3

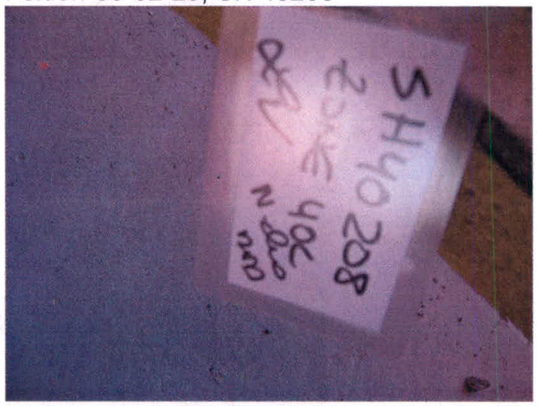
SH40207_4

SH40207_2

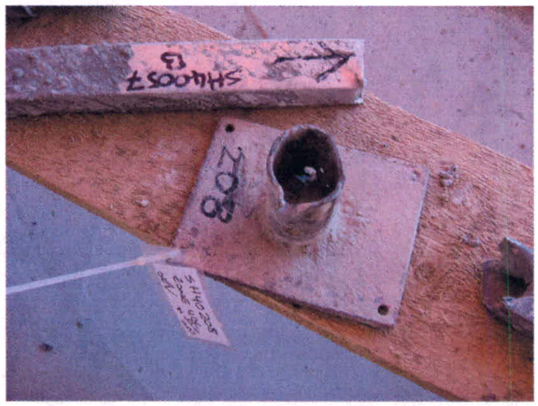


SH 40208
Top plate
Zone 40C

Folder: 08-02-29; SH 40208



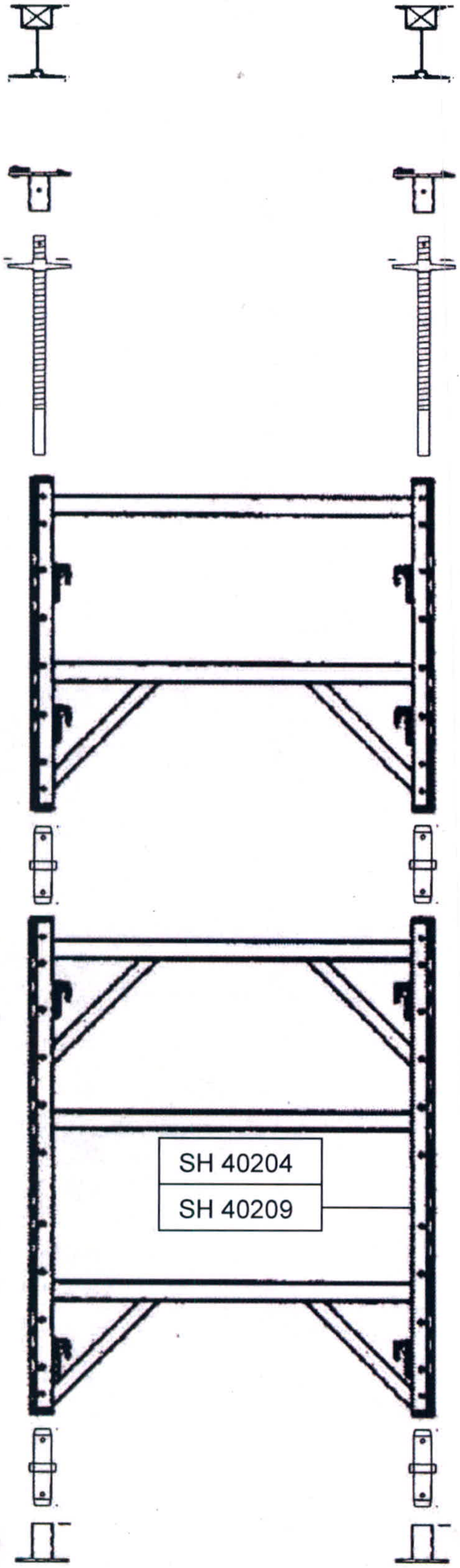
SH40208_2



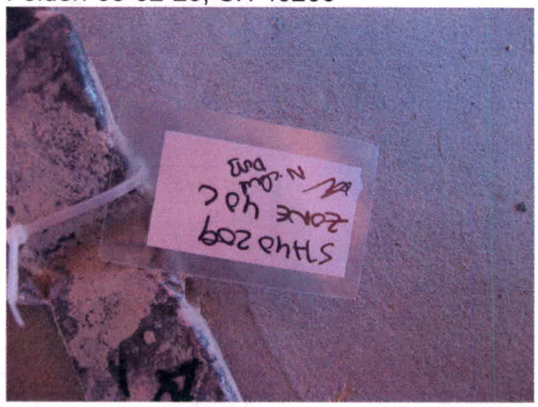
SH40208_4

SH40208_1

SH 40209
Piece of frame part of SH 40204
Zone 40C



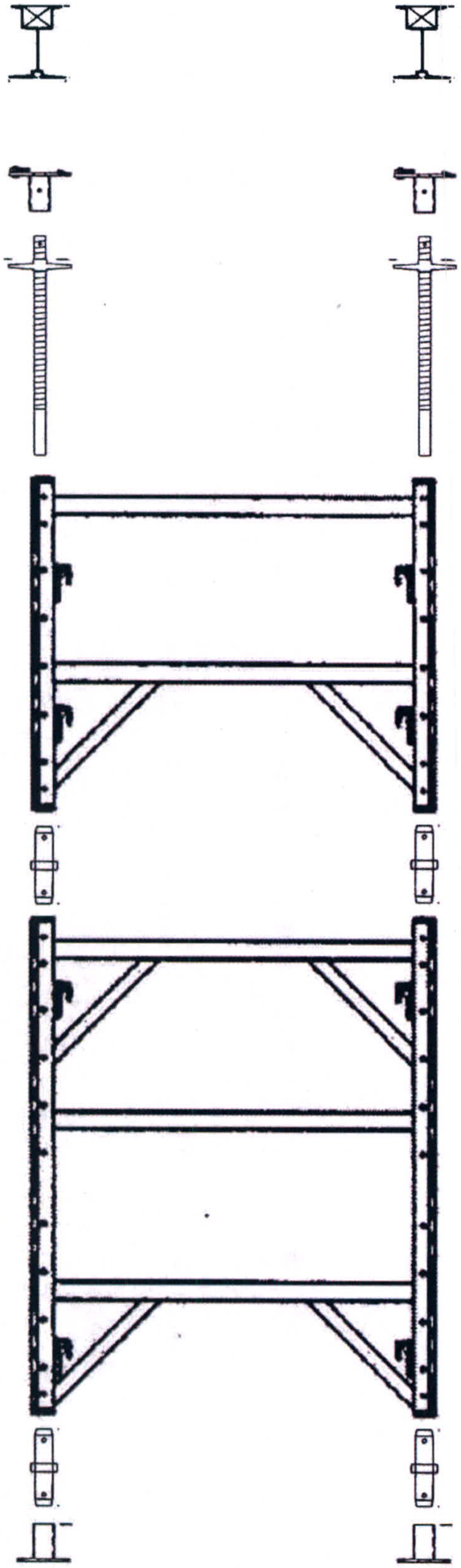
Folder: 08-02-29; SH 40209



SH40209_3

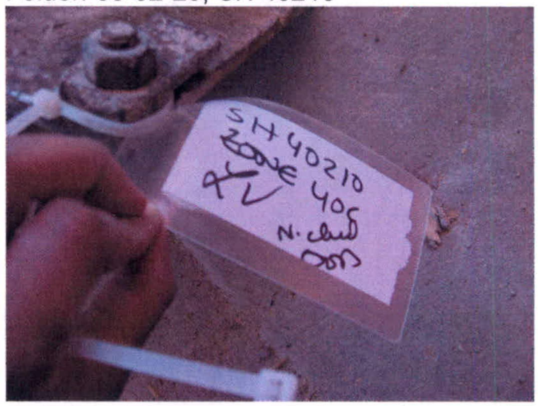
SH40209_4

SH40209_1

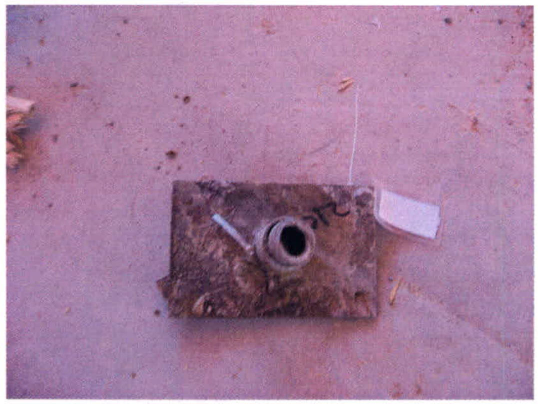


SH 40210
Top plate
Zone 40B

Folder: 08-02-29; SH 40210

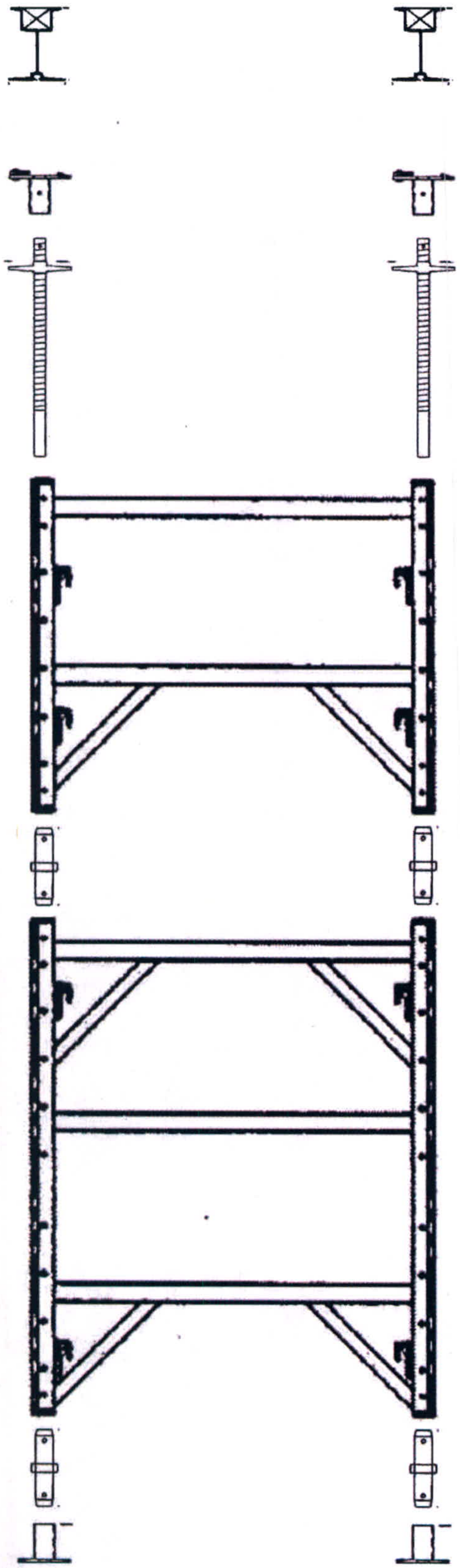


SH40210_2



SH40210_3

SH40210_1



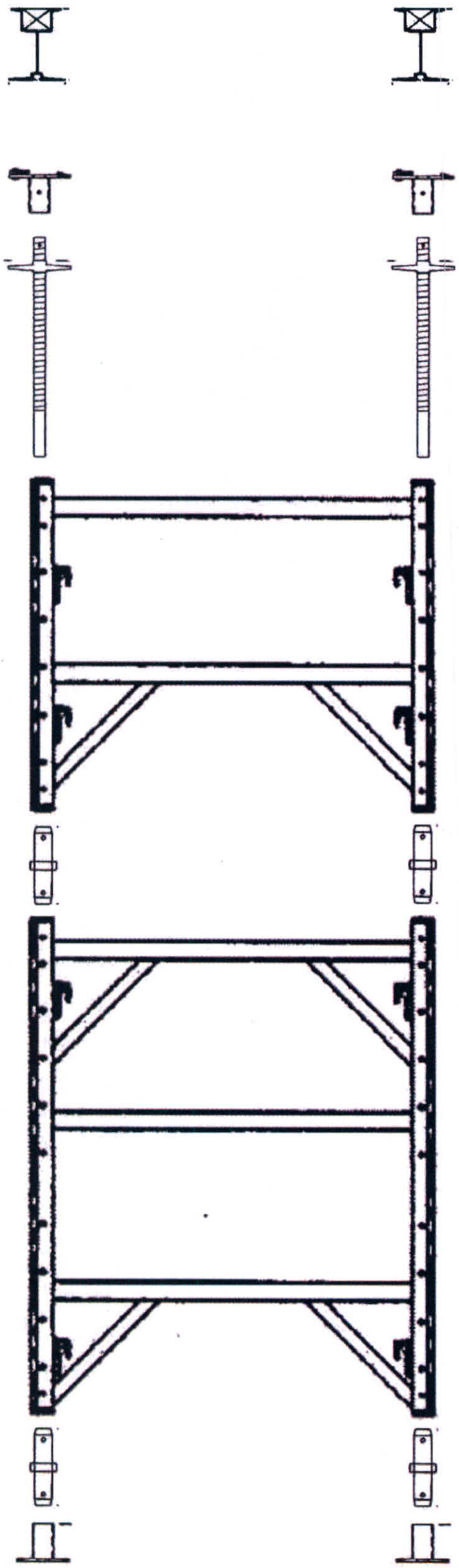
SH 40211
Screw Jack and Bottom plates for SH 40056
Zone 40B

Folder: 08-02-29; SH 40211

SH40211_1

SH40211_2

SH40211_3



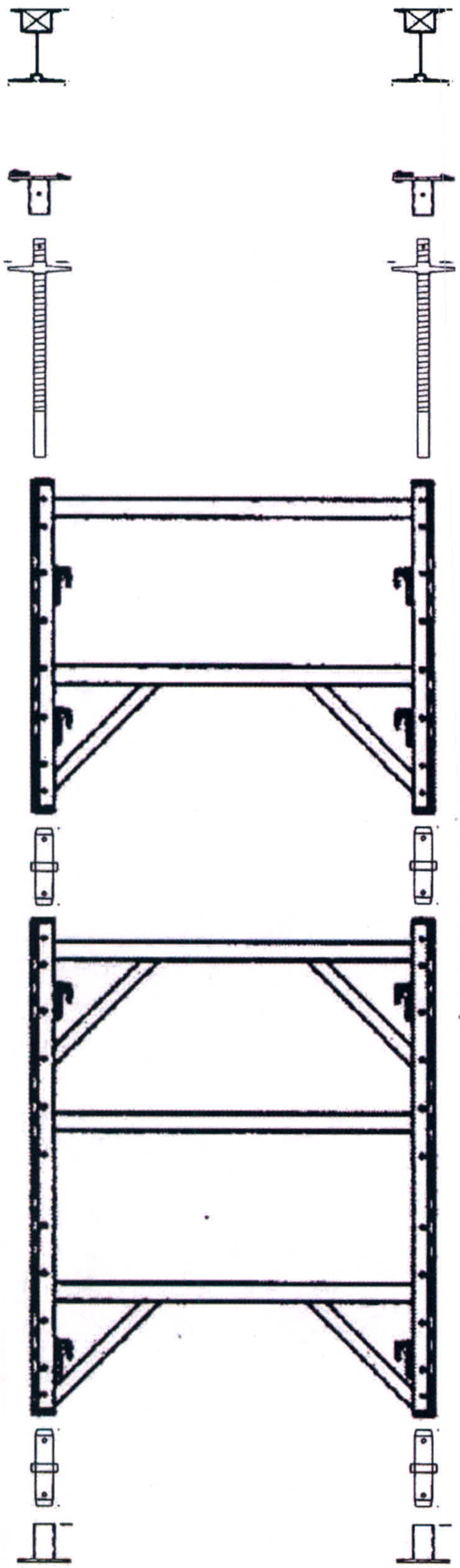
SH 40212
Screw Jack and Bottom plates of SH 40056
Zone 40B

Folder: 08-02-29; SH 40212

SH40212_1

SH40212_2

SH40213_1



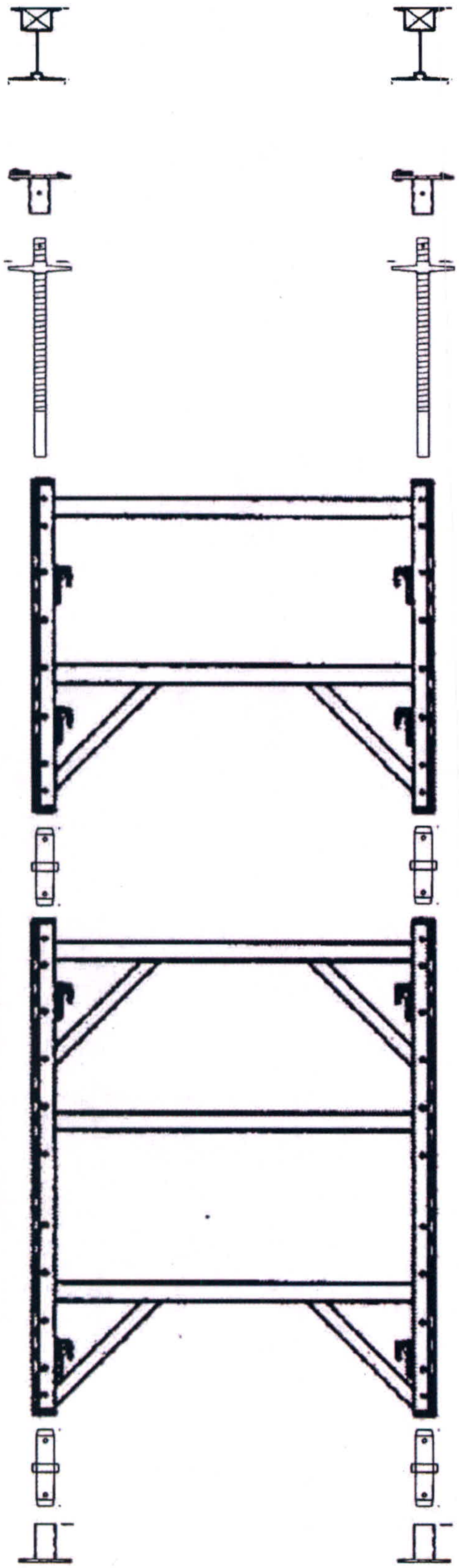
SH 40213
Screw jack and Bottoms
Zone 40B

Folder: 08-02-29; SH 40213

SH40213_1

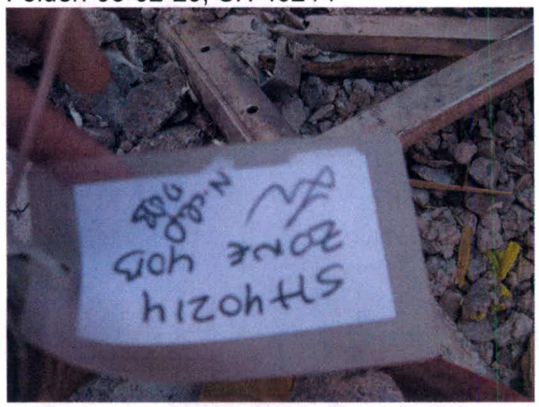
SH40213_2

SH40213_3



SH 40214
Screw jack and Bottom plates
Zone 40B

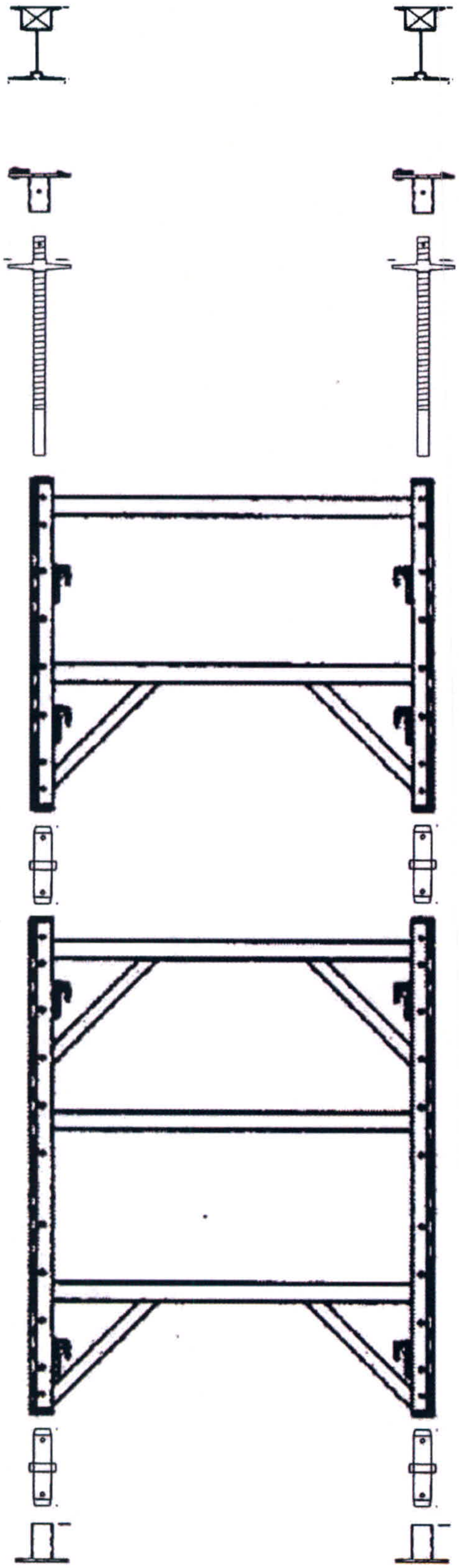
Folder: 08-02-29; SH 40214



SH40214_1

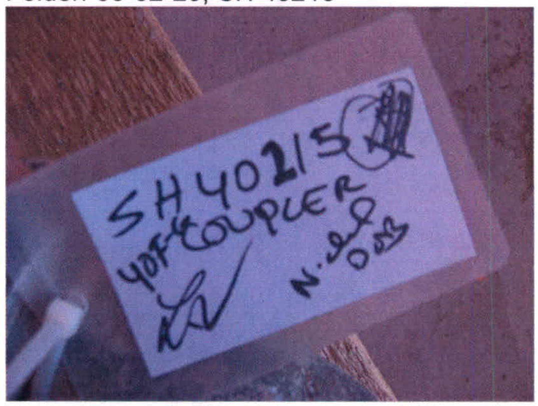
SH40214_2

SH40214_3



SH 40215
Coupler
Zone 40F-G

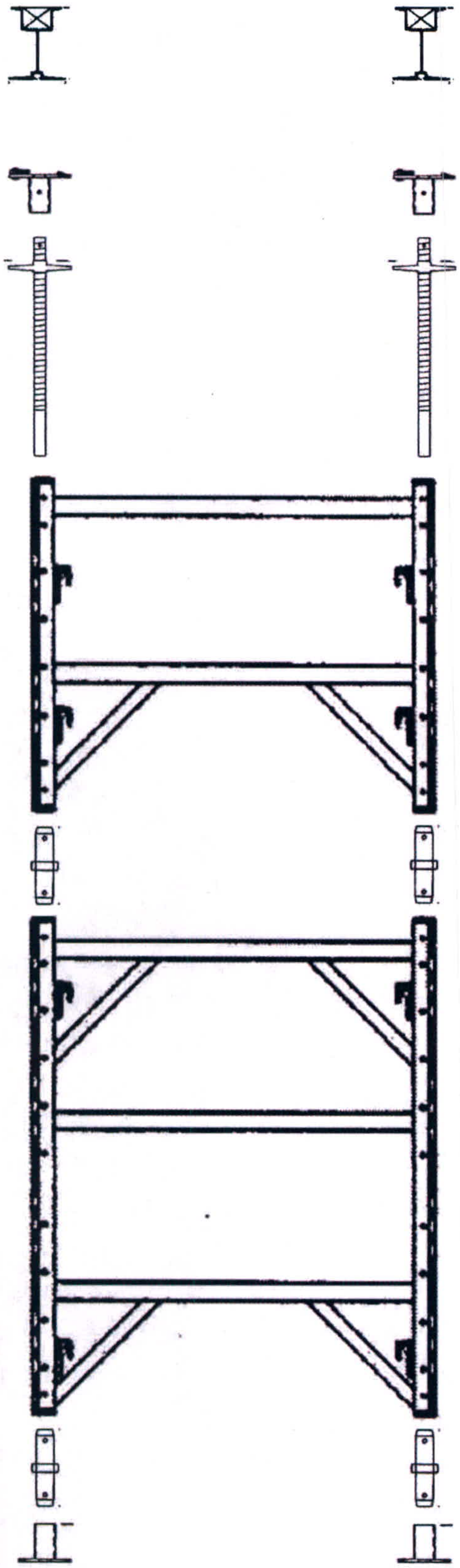
Folder: 08-02-29; SH 40215



SH40215_3

SH40215_1

SH40215_2



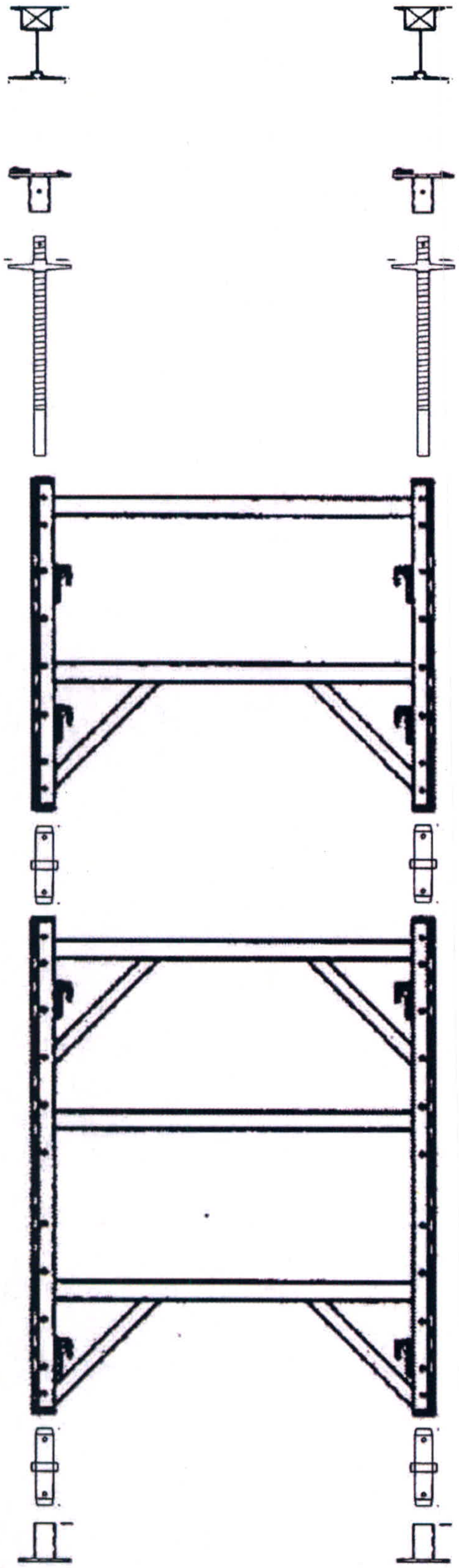
SH 40216
Bottom plate
Zone 40B

Folder: 08-02-29; SH 40216

SH40216_3

SH40216_1

SH40216_2



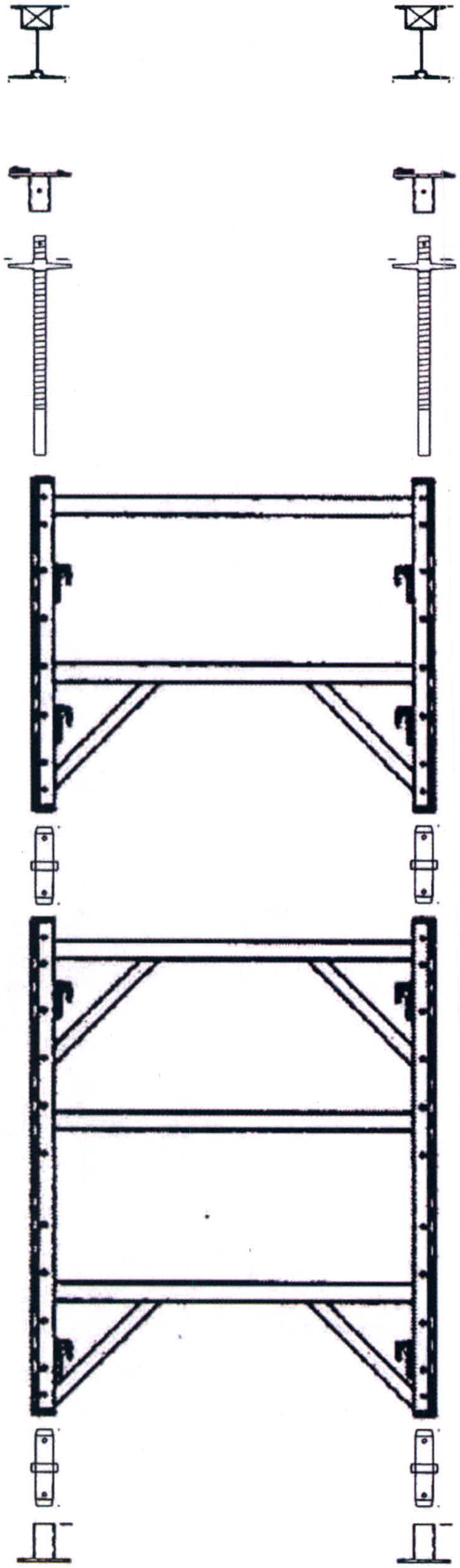
SH 40217
Top plate
Zone 40B

Folder: 08-03-03; SH 40217

SH40217_1

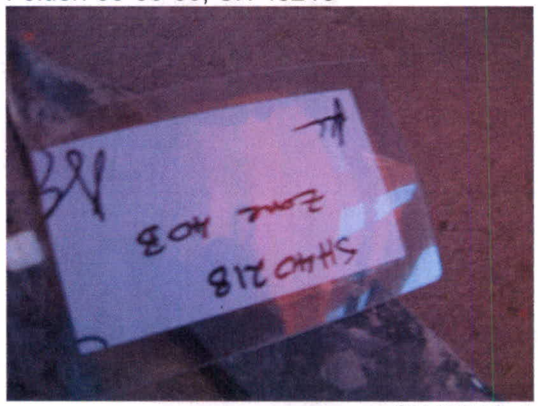
SH40217_3

SH40217_4



SH 40218
Piece of frame SH 40198
Zone 40B

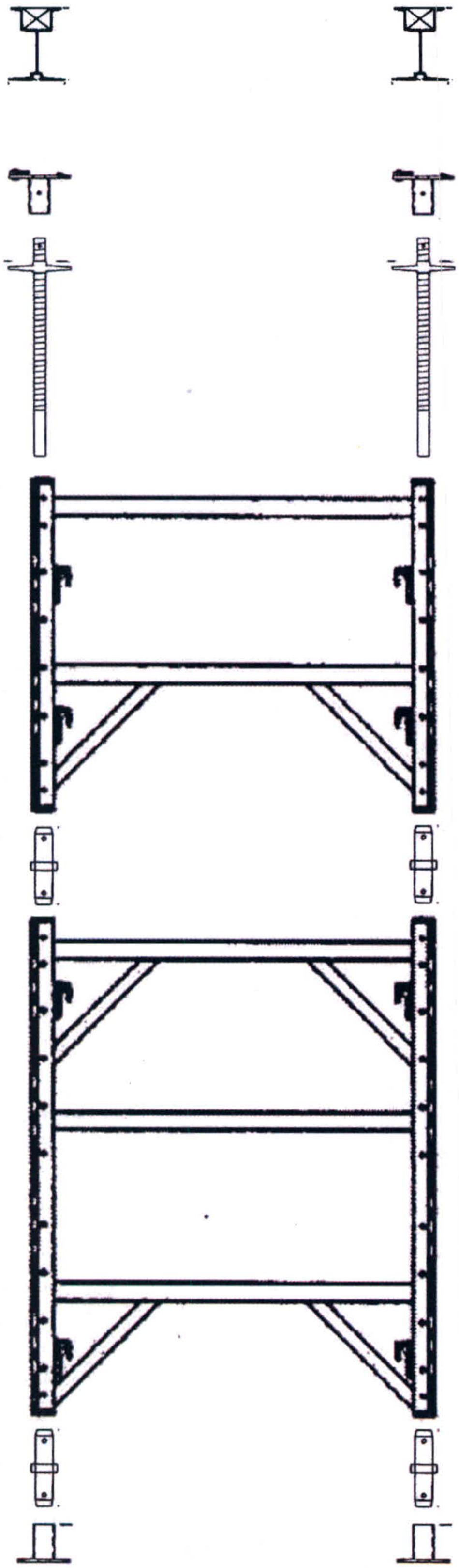
Folder: 08-03-03; SH 40218



SH40218_3

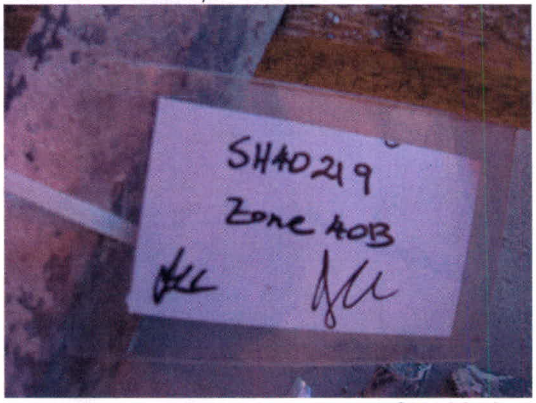
SH40218_4

SH40218_1



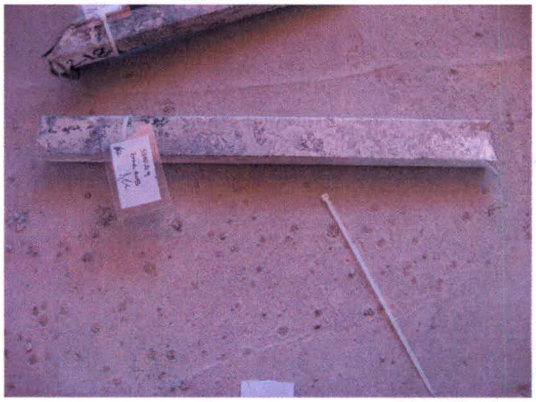
SH 40219
Piece of frame
Zone 40B

Folder: 08-03-03; SH 40219

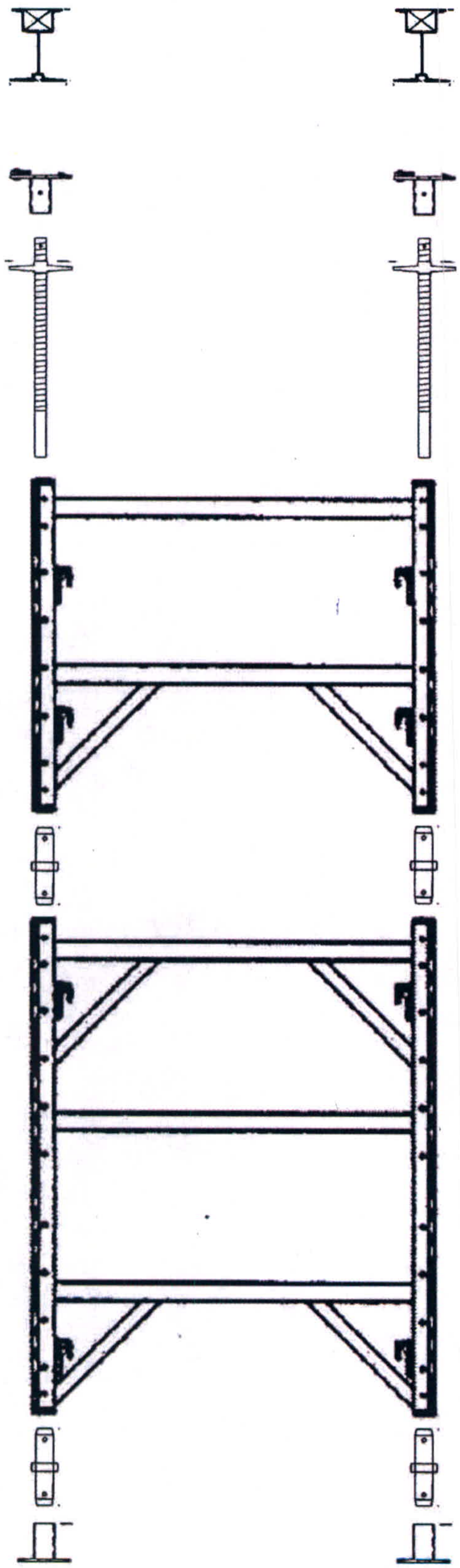


SH40219_5

SH40219_3

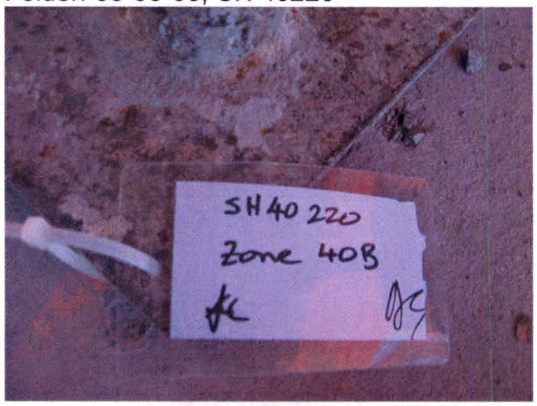


SH40219_4

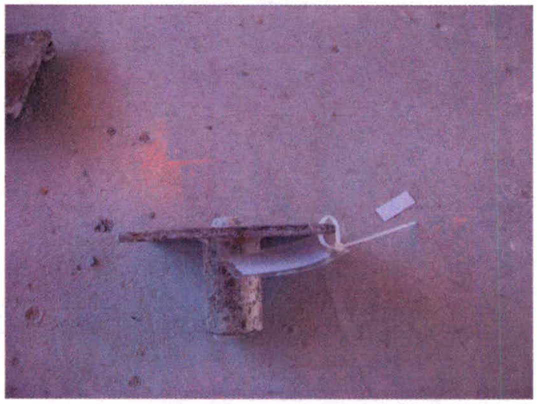


SH 40220
Bottom plate
Zone 40B

Folder: 08-03-03; SH 40220

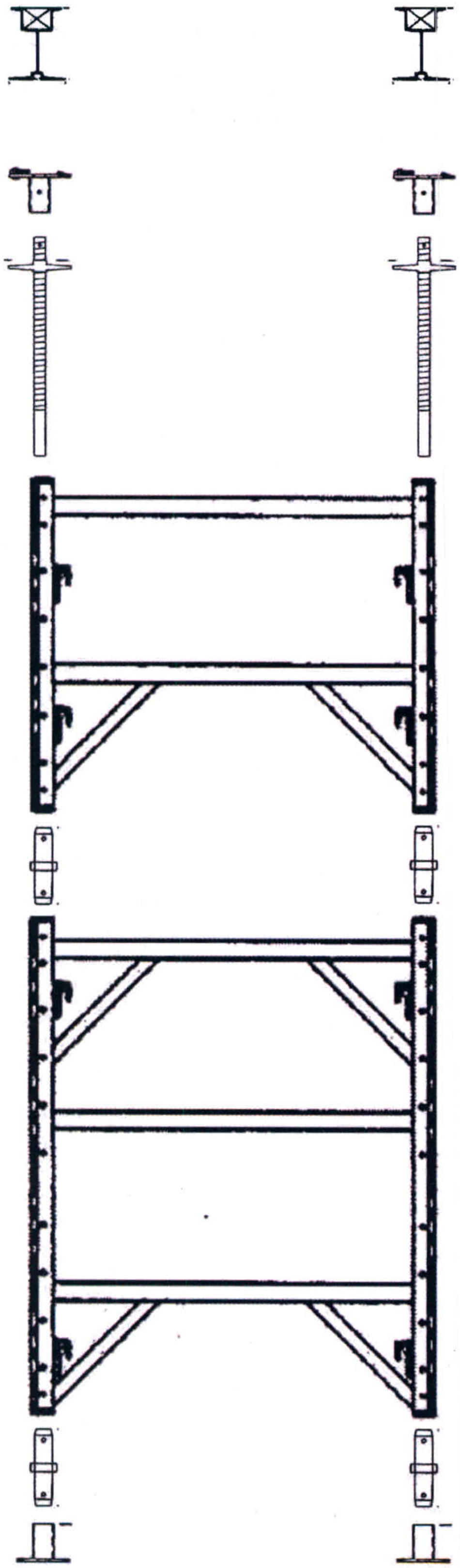


SH40220_6



SH40220_3

SH40220_1

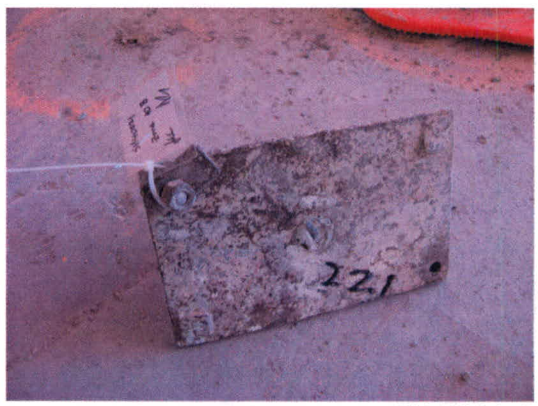


SH 40221
To plate
Zone 40B

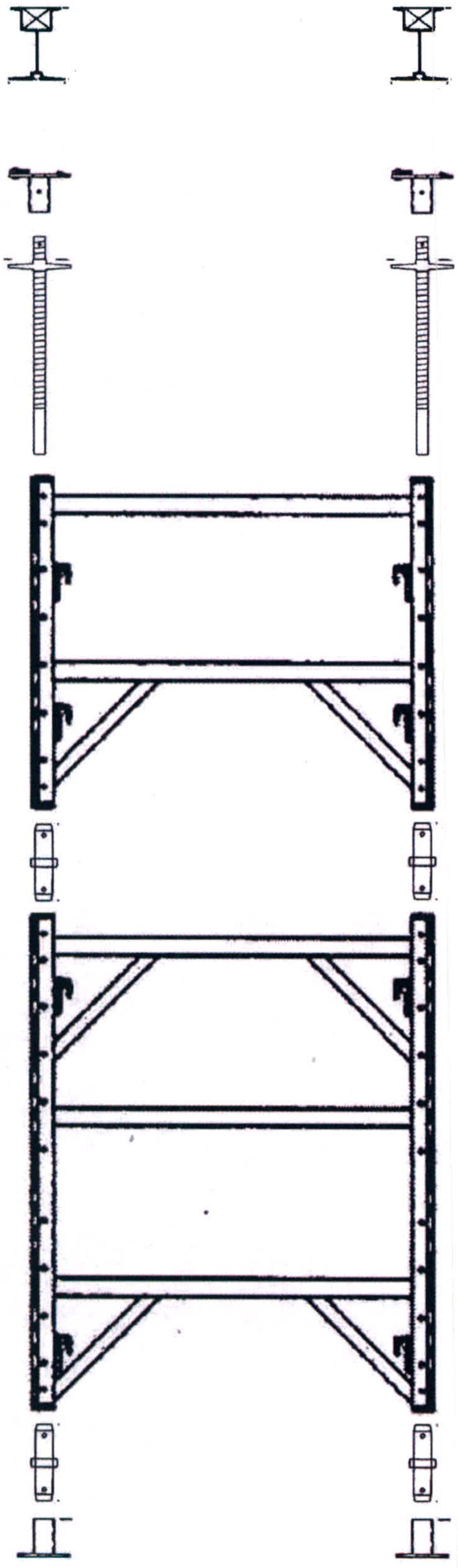
Folder: 08-03-03; SH 40221

SH40221_4

SH40221_2

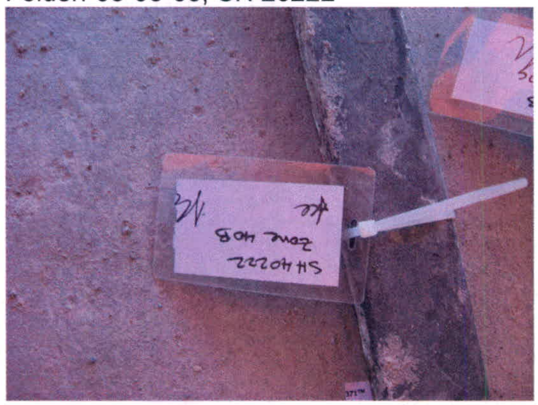


SH40221_3



SH 40222
Piece of frame
Zone 40B

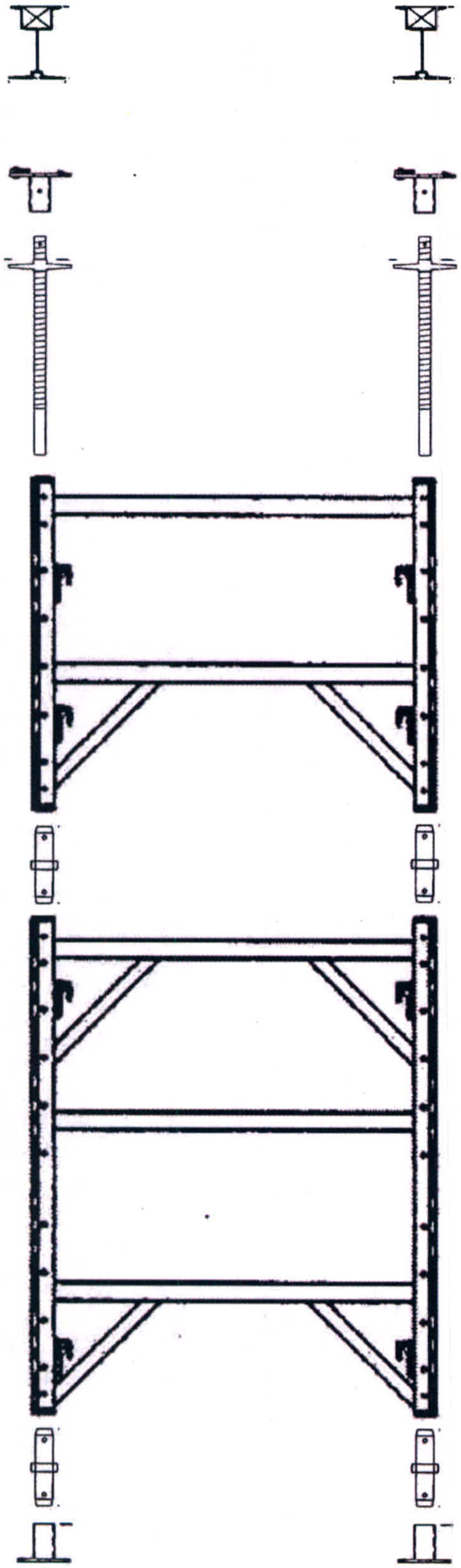
Folder: 08-03-03; SH 20222



SH40222_4

SH40222_3

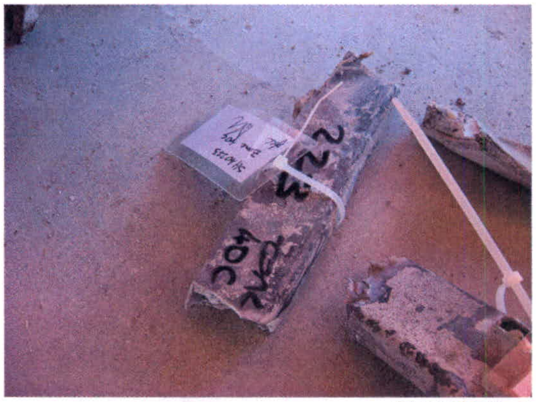
SH40222_1



SH 40223
Piece of frame
Zone 40C

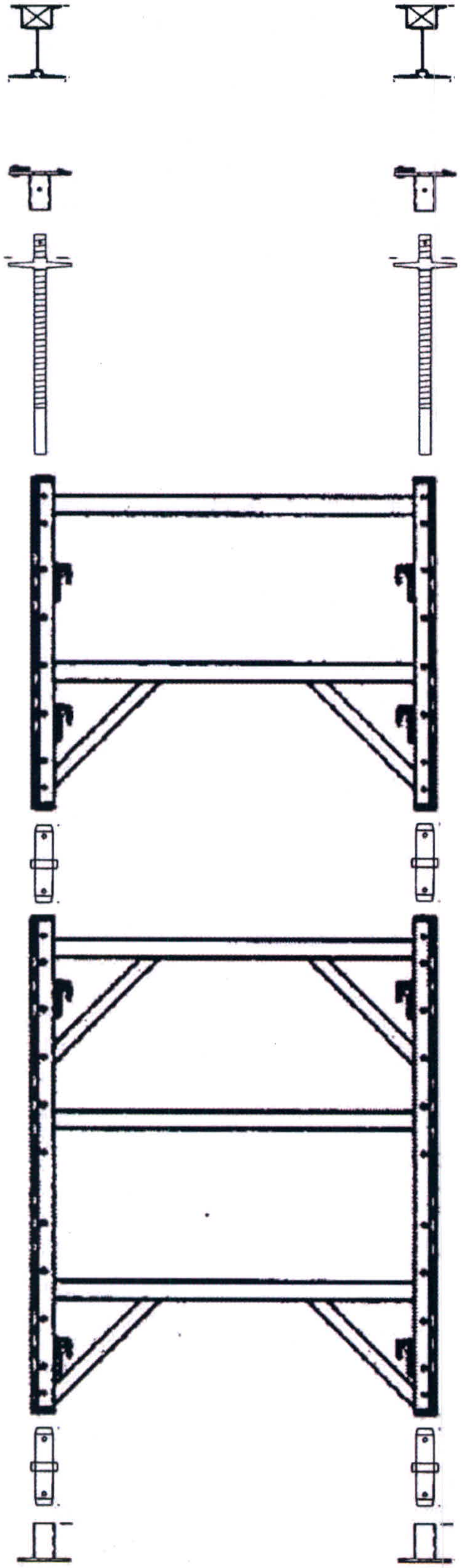
Folder: 08-03-03; SH 40223

SH40223_1



SH40223_2

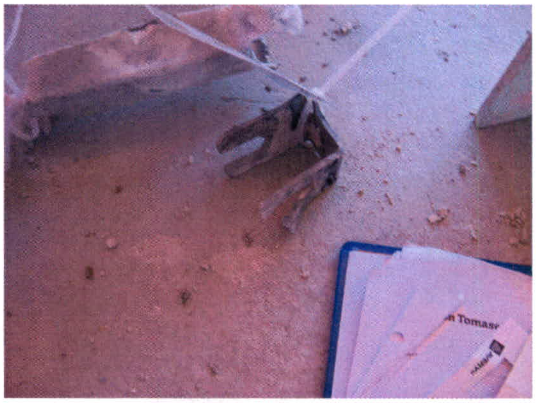
SH40223_3



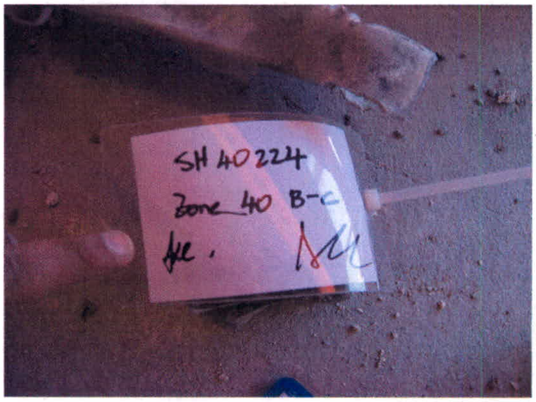
SH 40224
Piece of frame
Zone 40B

Folder: 08-03-03; SH 40224

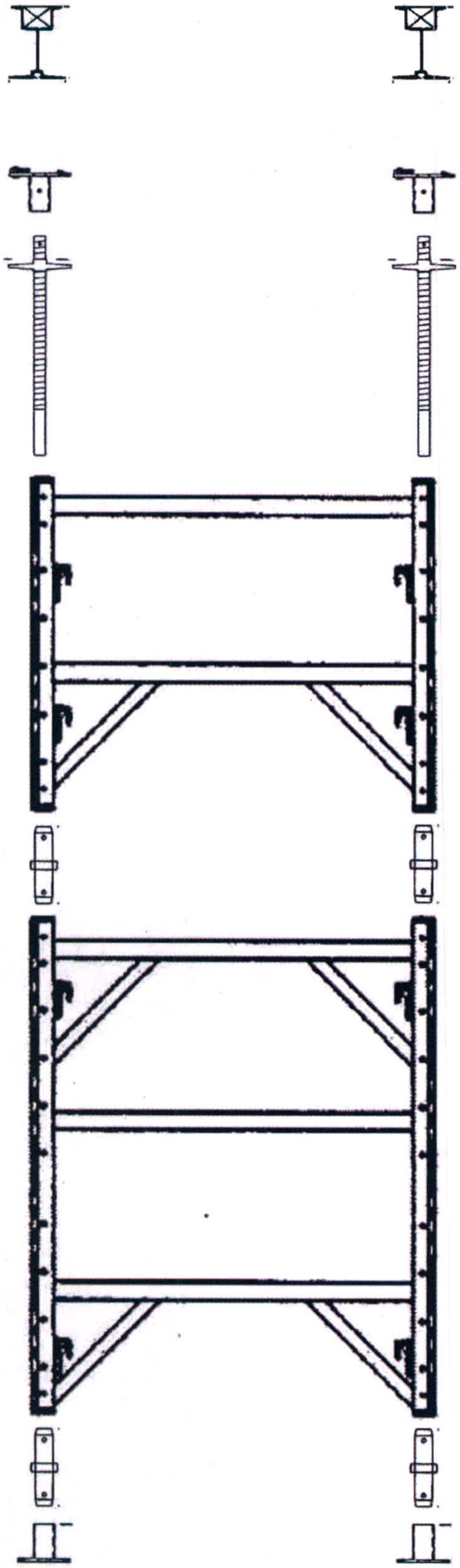
SH40224_1



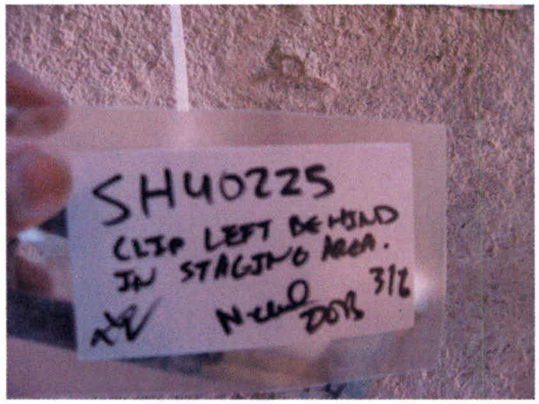
SH40224_2



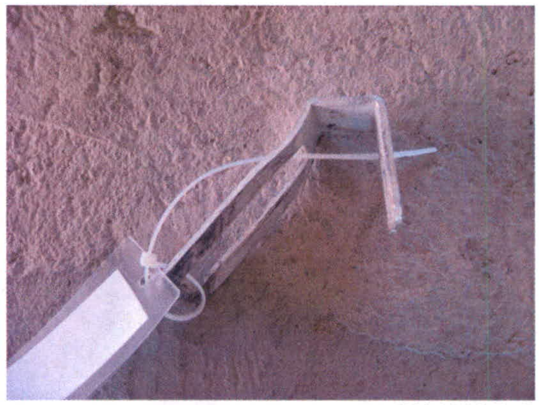
SH40224_3



SH 40225
Clip left behind in staging area

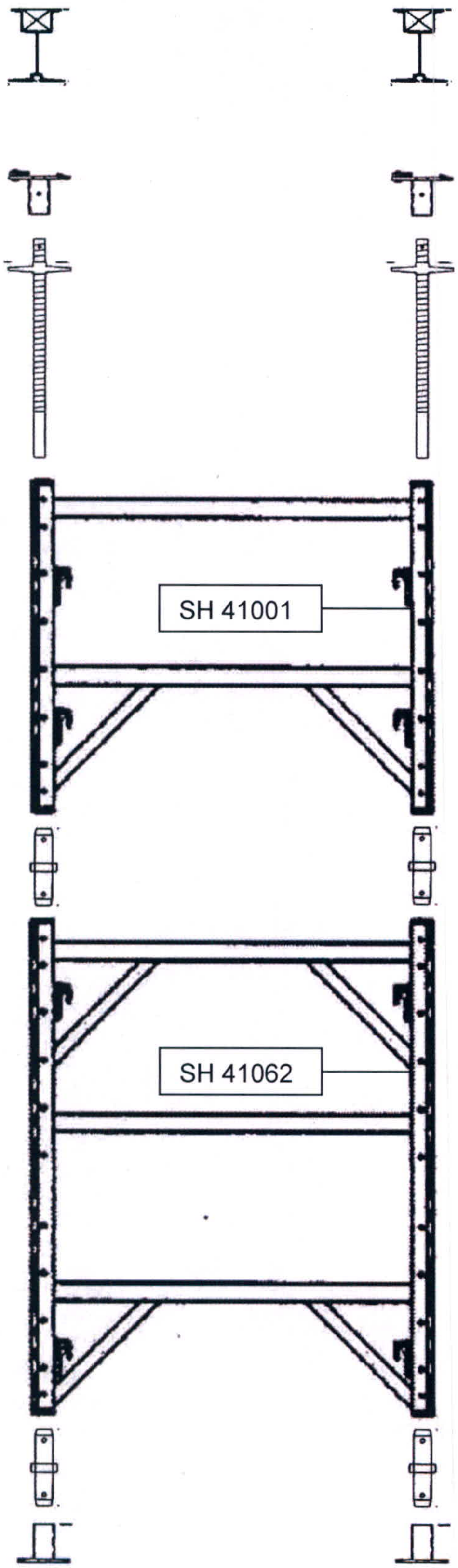


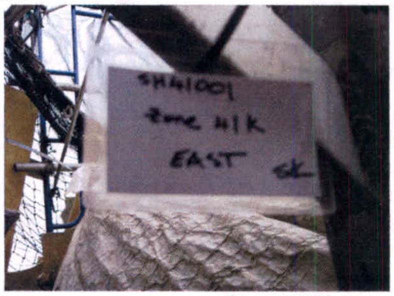
SH40225_1


SH40225_2

SH 41001
Top Frame
Bottom Frame SH 41062
Connect to SH 41002
Zone 41K

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH

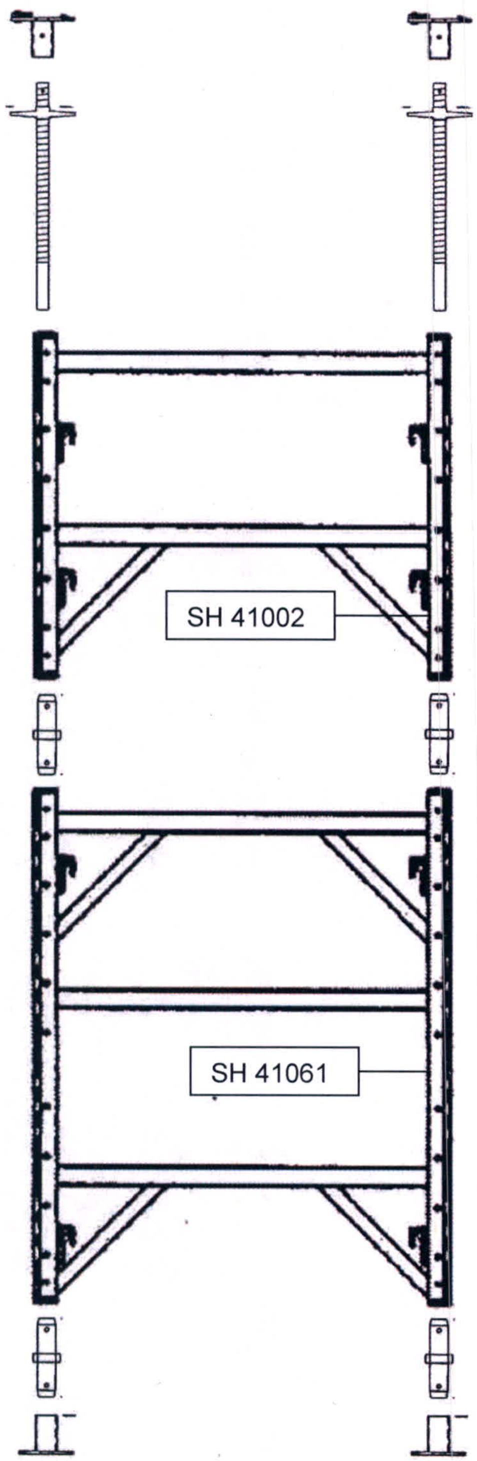




SH41001_1

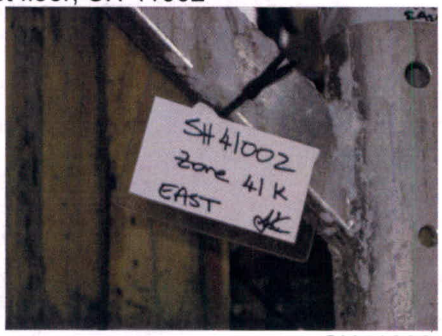

SH41001_3


SH41001_4



SH 41002
Top Frame
Bottom Frame SH 41061
Connect to SH 41001(Tower)
Zone 41K

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41002



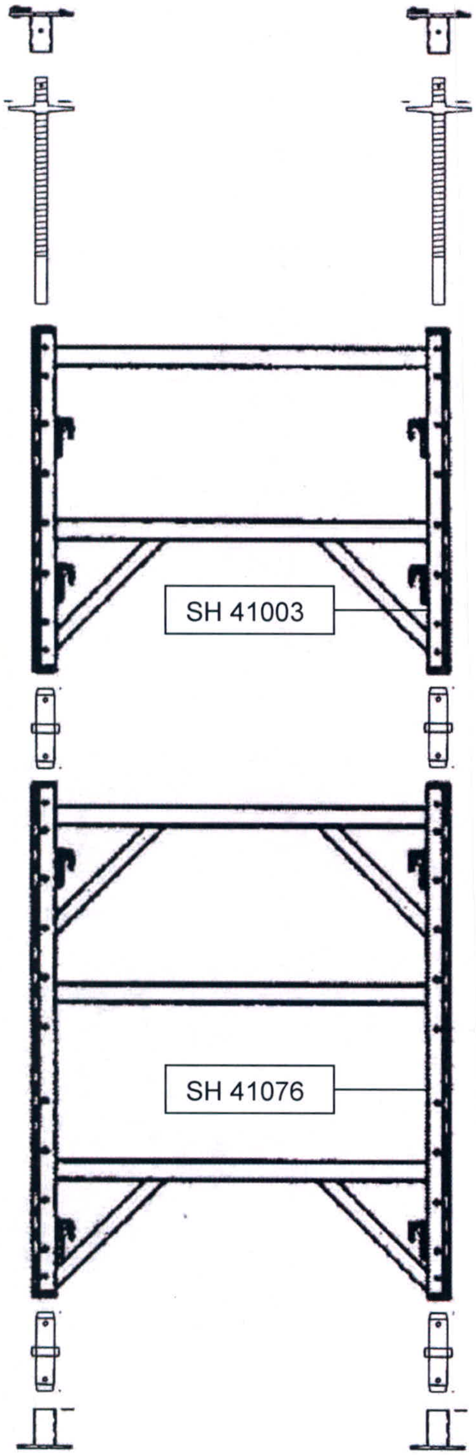
SH41002_1

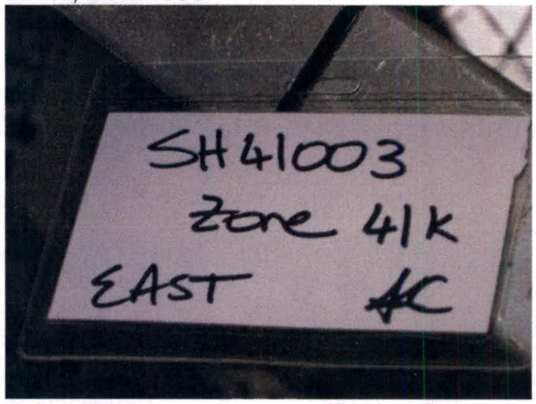
SH41002_3

SH41002_4

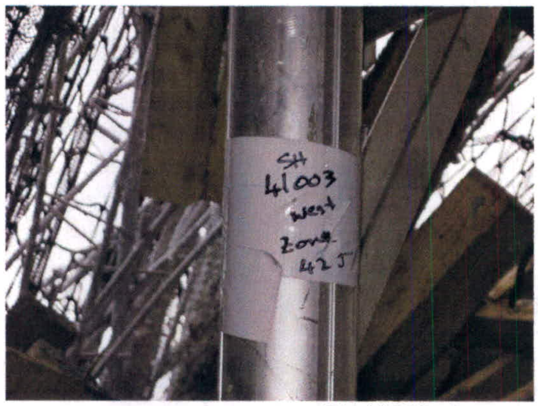
SH 41003
Top Frame
Bottom Frame SH 41026
Connect to top SH 41004(Tower)
Zone 41K-J

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41003

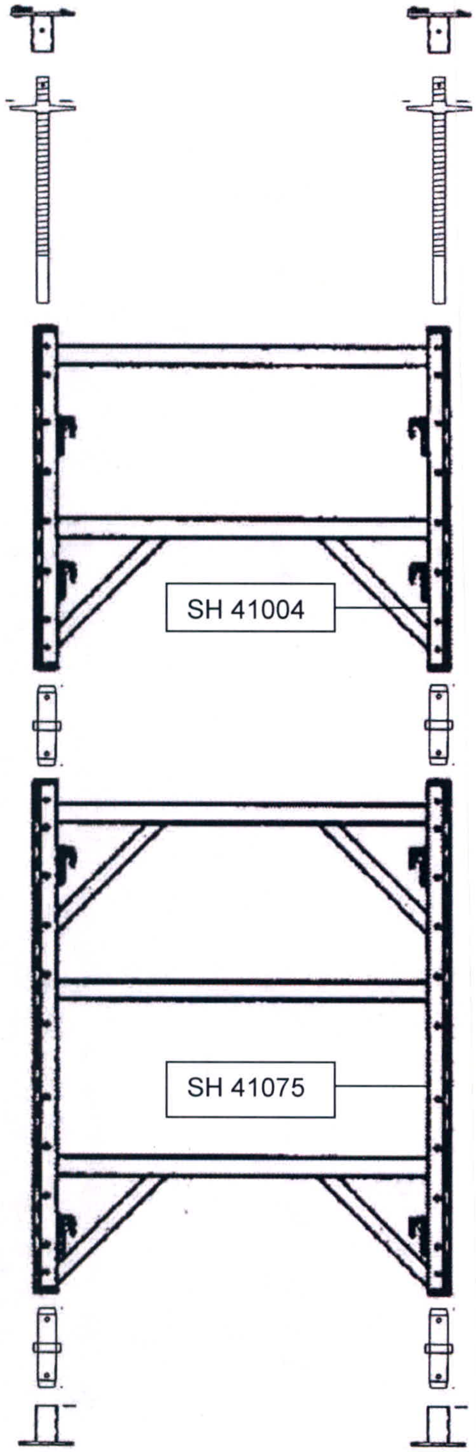




SH41003_1

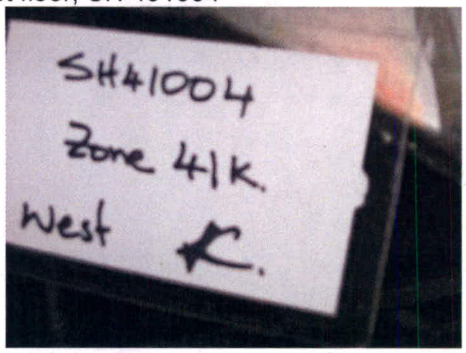

SH41003_3


SH41003_5



SH 41004
Top Frame
Bottom Frame SH 41075
Connect to top SH 41003
Zone 41K

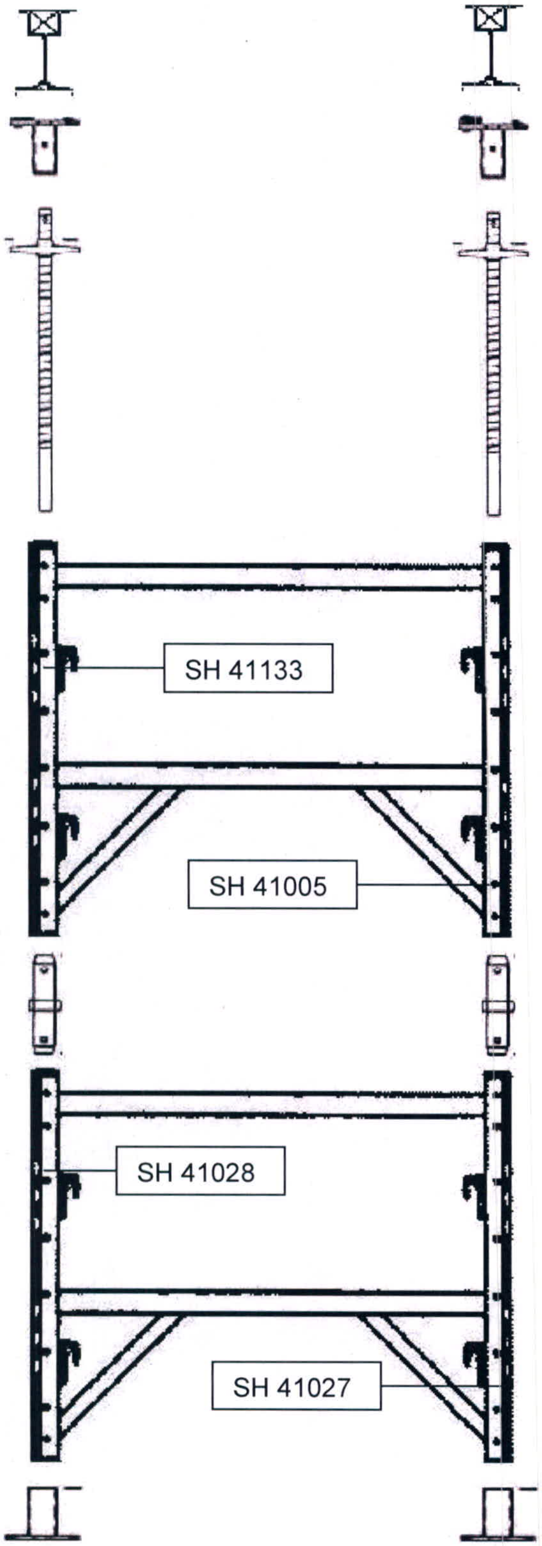
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 401004



SH41004_2

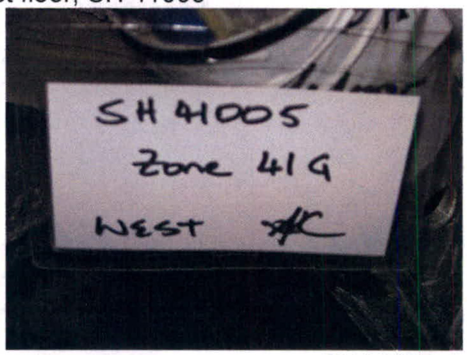

SH41003_4


SH41004_3



SH 41005
Broken Top Frame
Bottom Frame SH 41027 and SH 41028
Supports AB 41004
Zone 41G

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41005



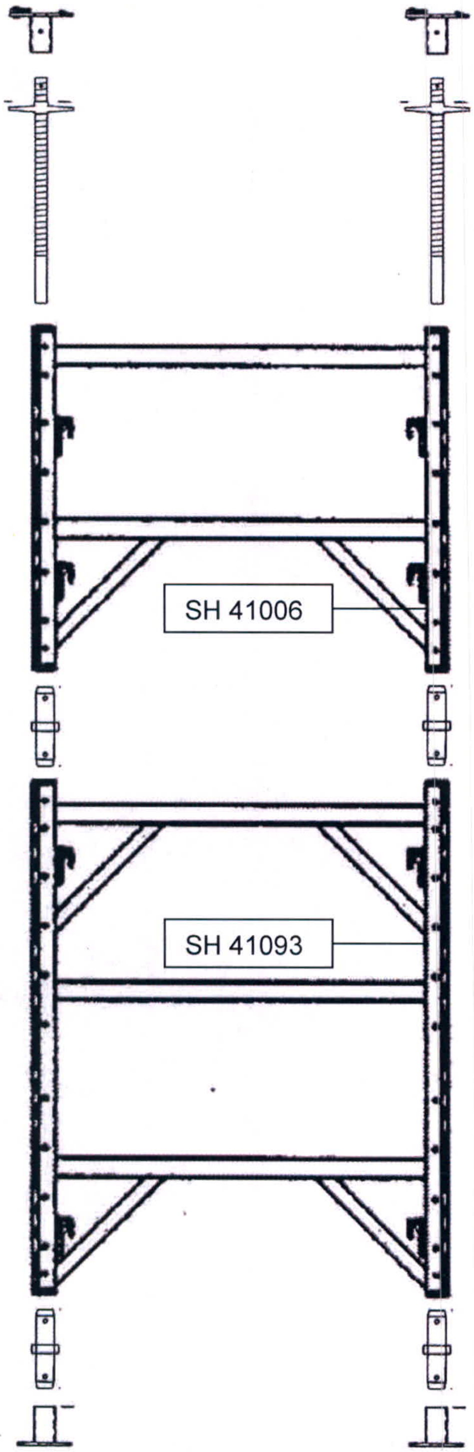
SH41005_1

SH41005_2

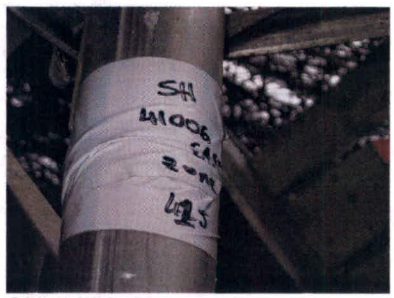
SH41005_4

SH 41006
Top Frame
Bottom Frame SH 41093
Zone 41J

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41006



SH41006_1

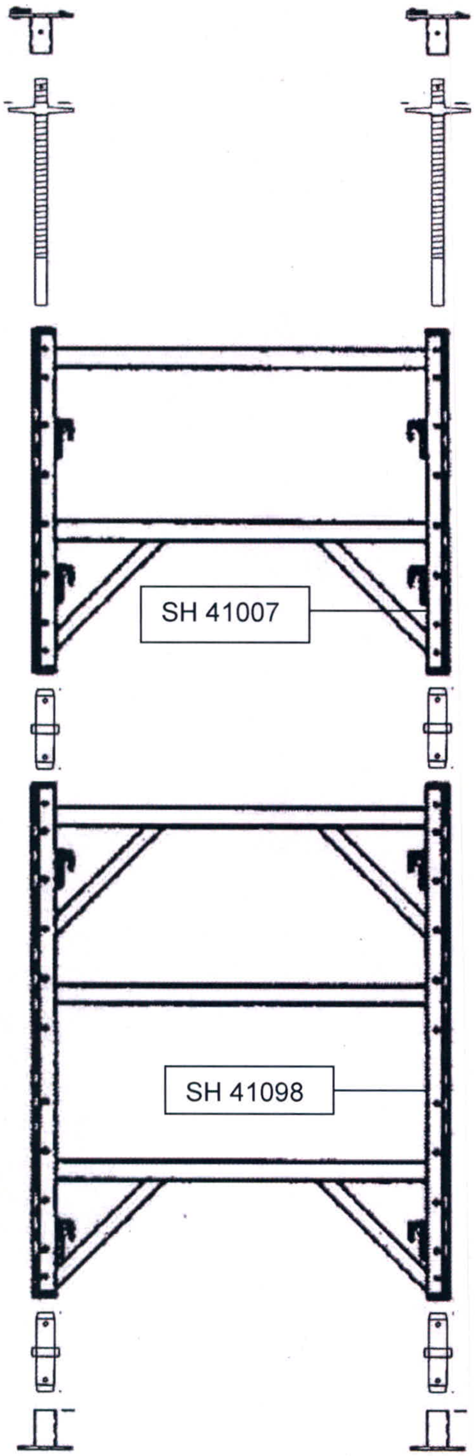


SH41006_3

SH41006_5

SH 41007
Top Frame
Bottom Frame SH 41098
Zone 41J

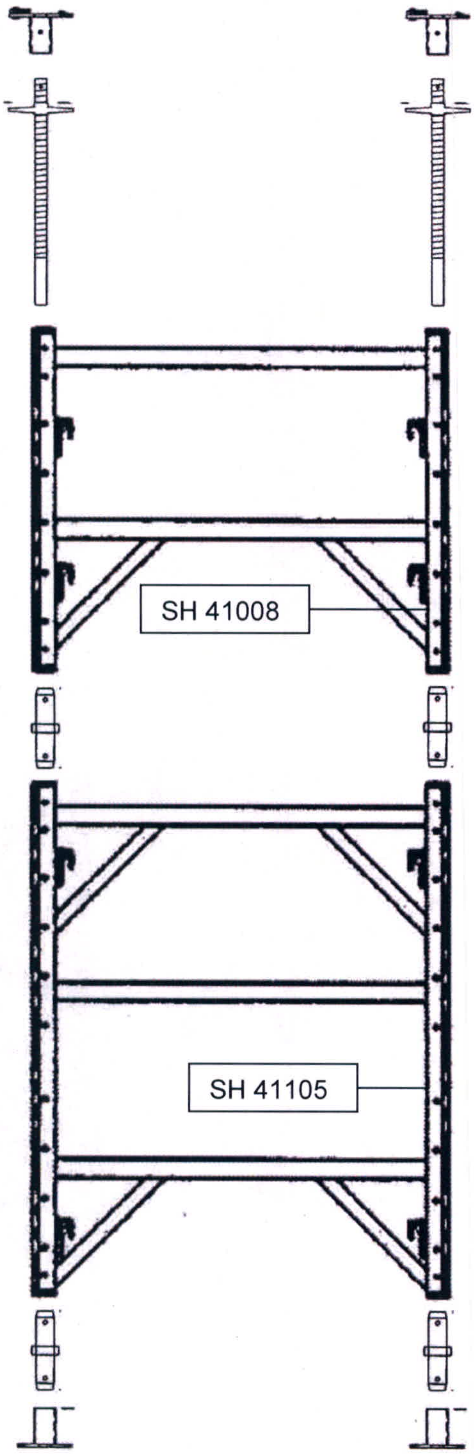
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41007



SH41007_2

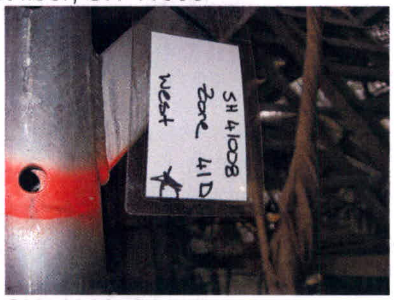
SH41007_4

SH41007_5

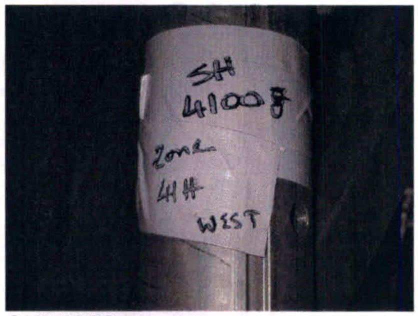


SH 41008
Top Frame
Bottom Frame SH 41105
Zone 41D-H

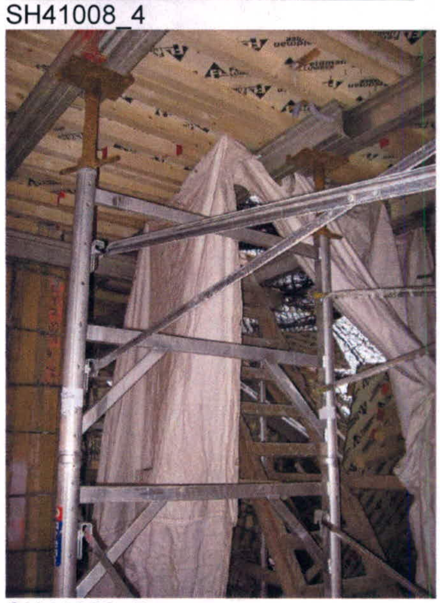
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41008



SH41008_2

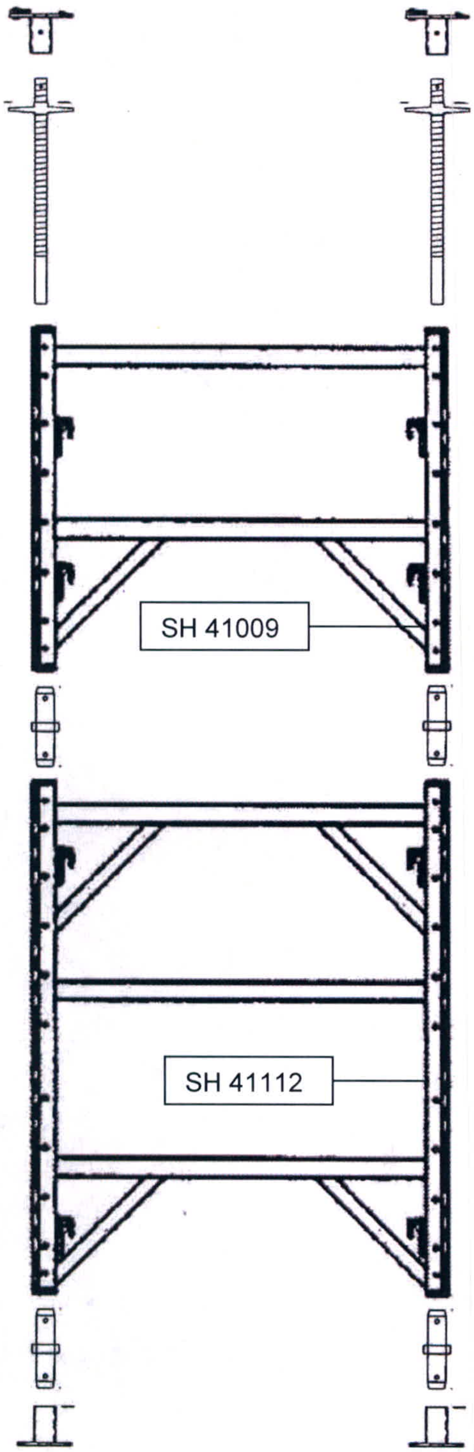


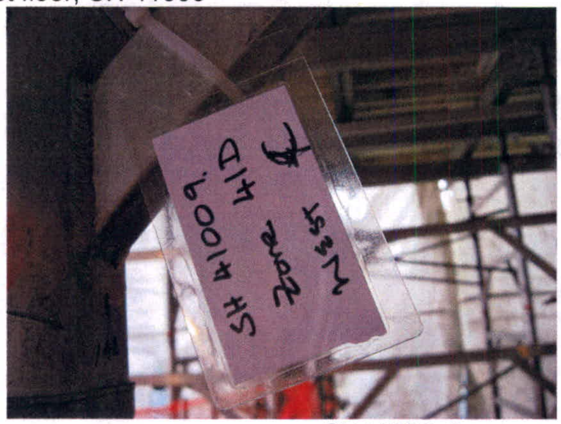
SH41008_4


SH41008_5

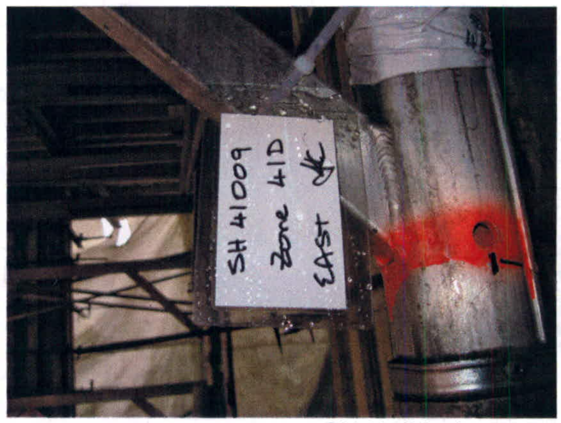
SH 41009
Top Frame
Bottom Frame SH 41112
Zone 41D

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41009



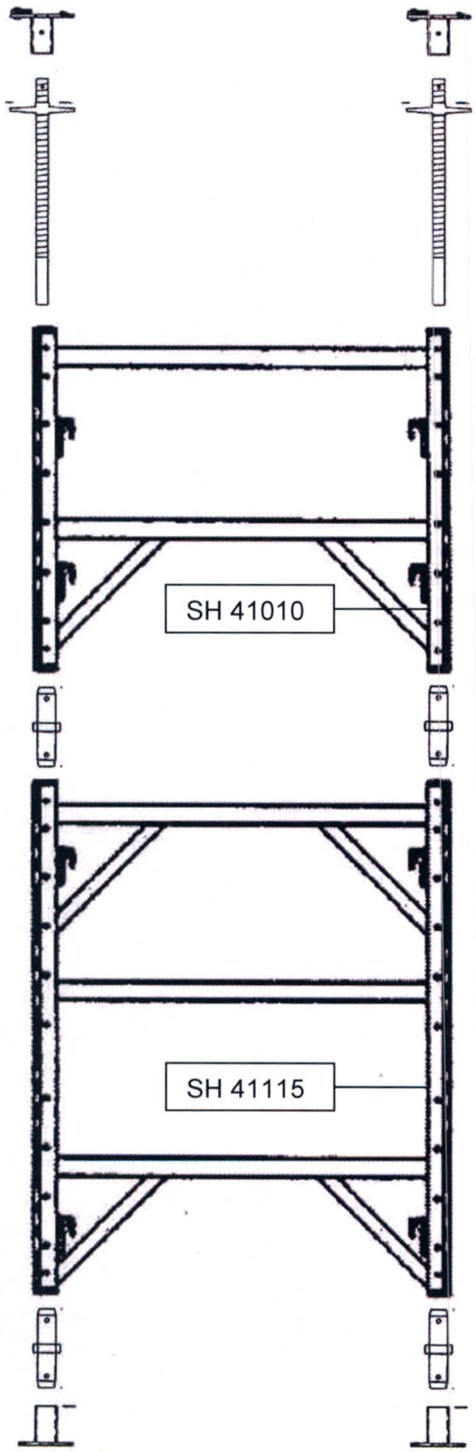


SH41009_2



SH41009_1


SH41009_6



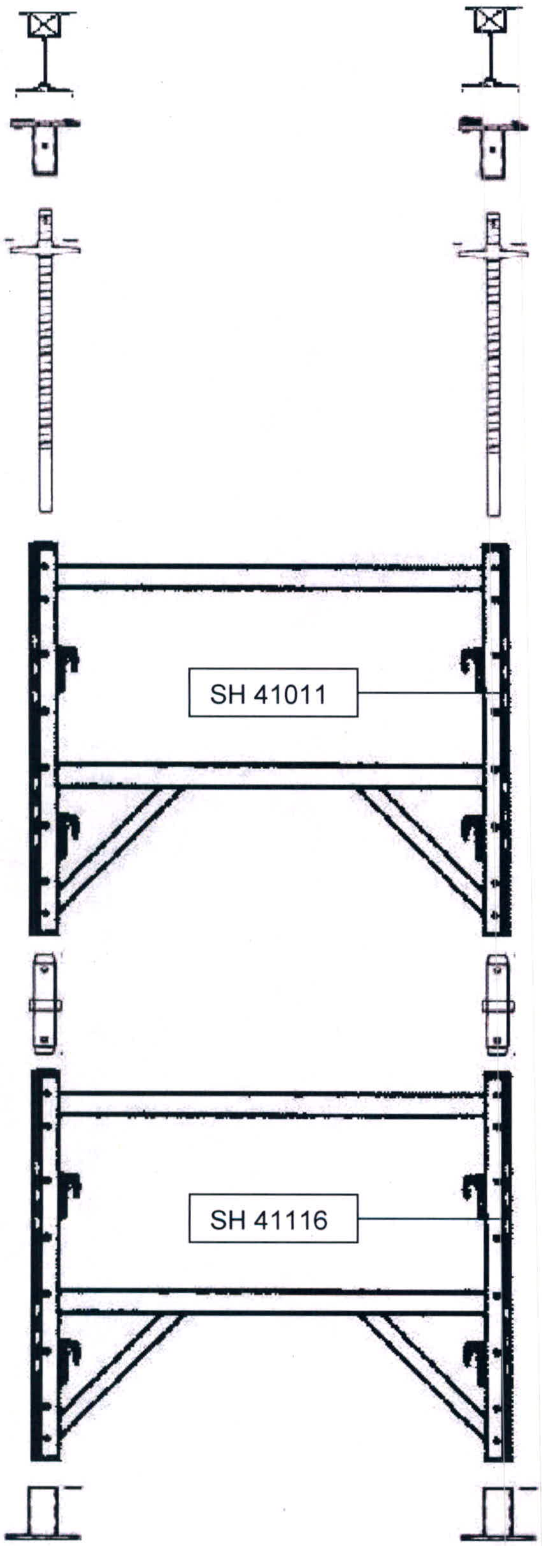
SH 41010
Top Frame
Bottom Frame SH 41115
Connect to Top SH 41009(Tower)
Zone 41D

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41010

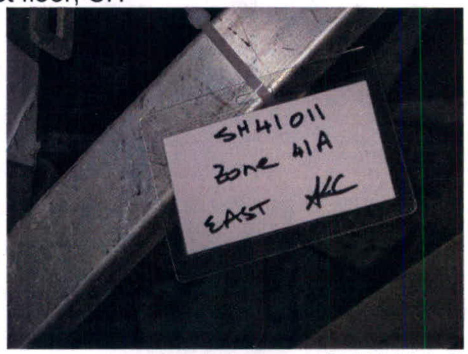
SH41010_1


SH41010_3

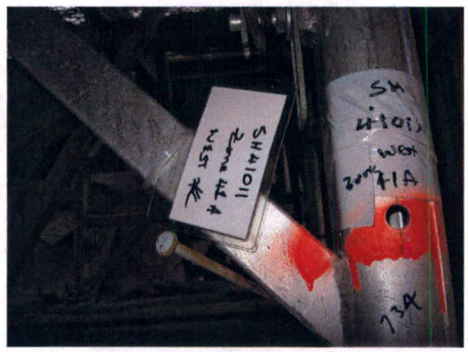

SH41010_6



SH 41011
Top Frame
Bottom Frame SH 41116
Zone 41A
Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH

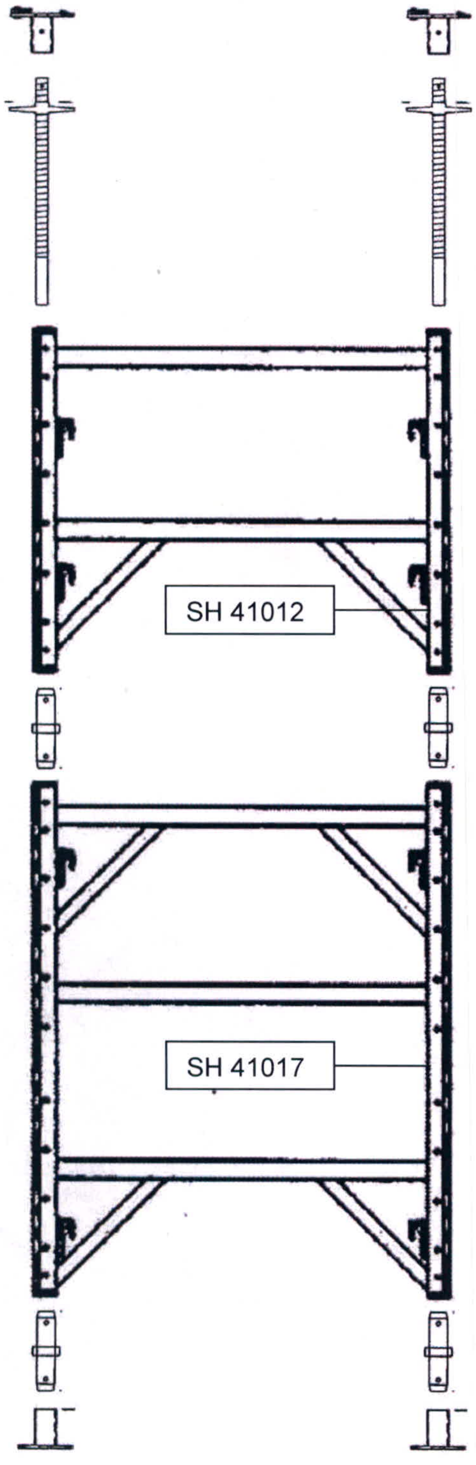


SH41011_2



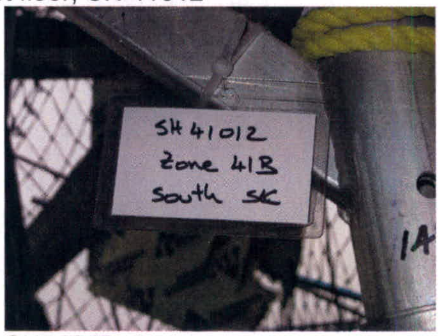
SH41011_1

SH41011_5



SH 41012
Top Frame
Bottom Frame SH 41017
Zone 41B

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41012



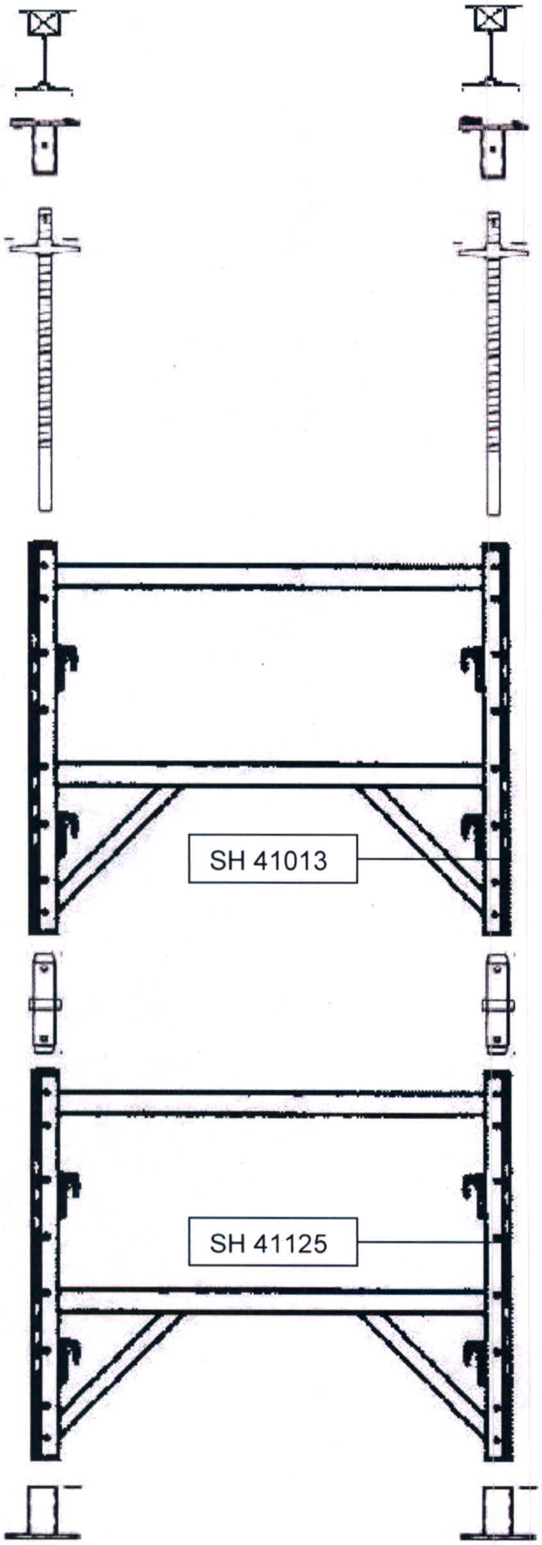
SH41012_1

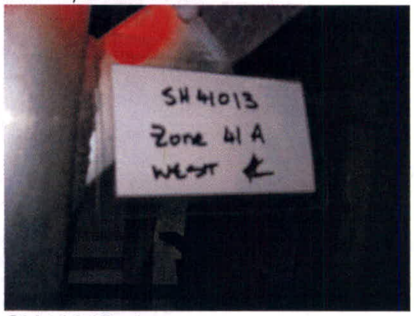
SH41012_7

SH41012_9

SH 41013
Top Frame
Bottom Frame SH 41125
Zone 41A-B

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41013

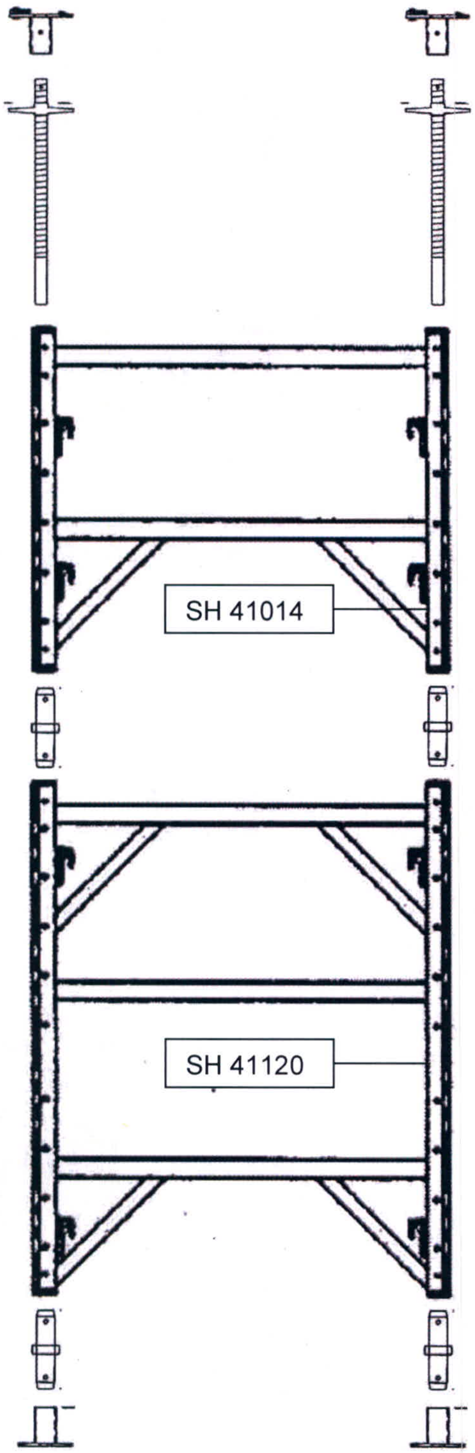




SH41013_1

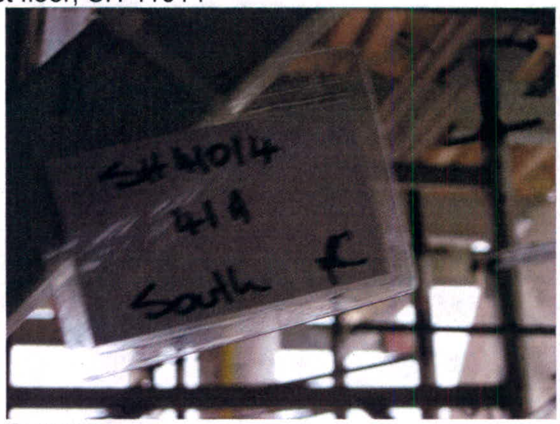

SH41013_5


SH41013_7



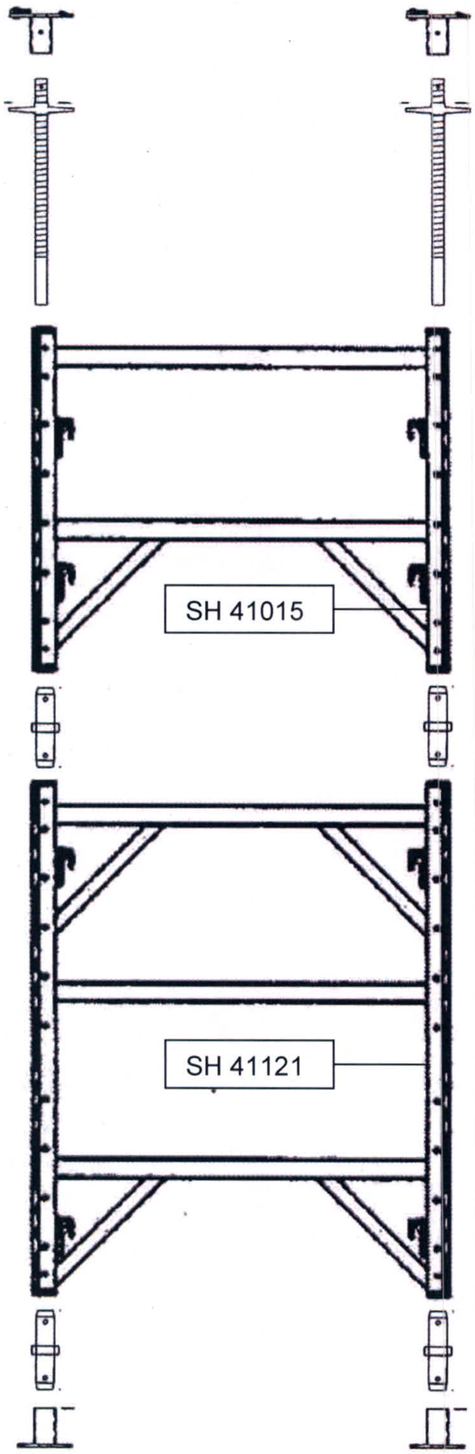
SH 41014
Top Frame
Bottom Frame SH 41120
Zone 41A

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41014



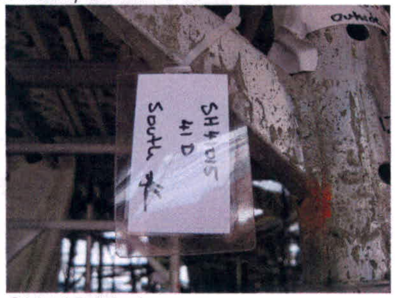
SH41014_1

SH41014_5



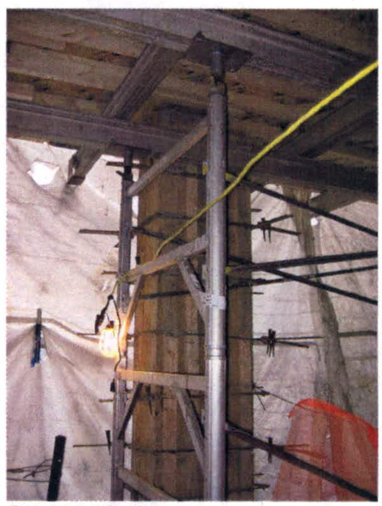
SH 41015
Top Frame
Bottom Frame SH 41121
Zone 41D

Folder: 08-01-29 (updated 08-02-04 & 08-02-05); 41st floor; SH 41015

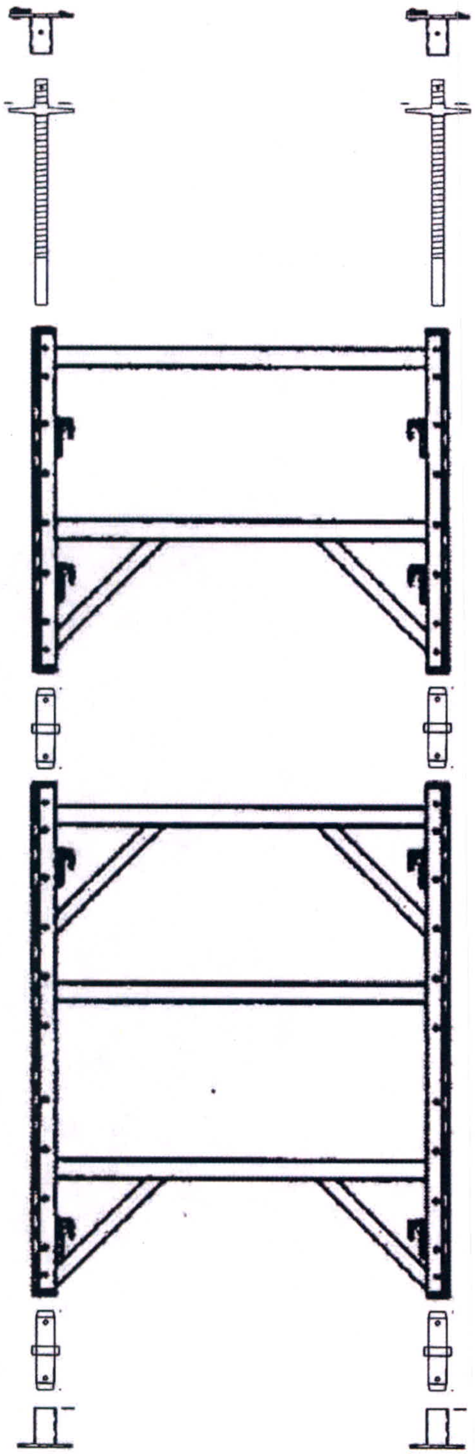


SH41015_1

SH41015_5

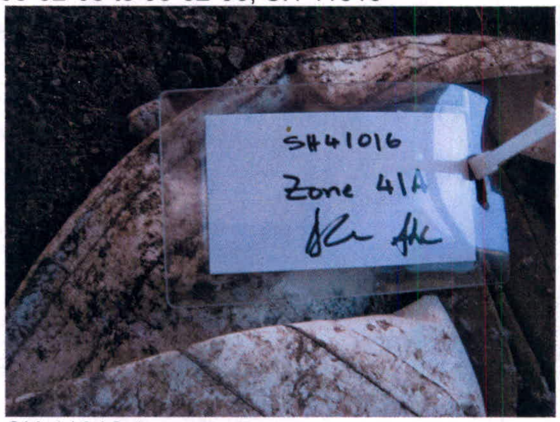


SH41015_6



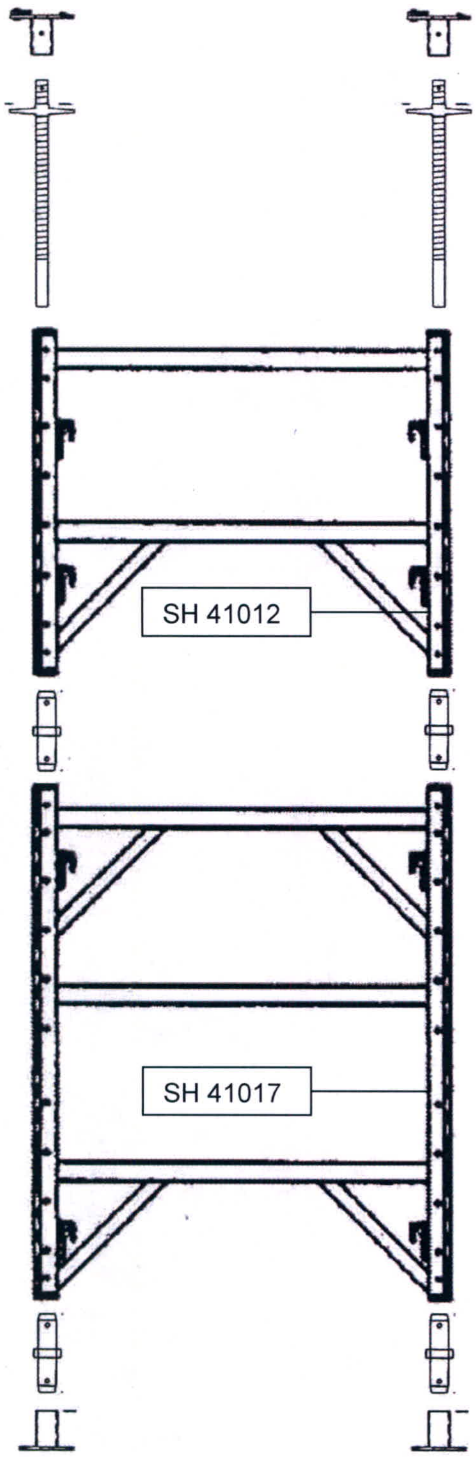
SH 41016
X Brace
Zone 41A

Folder: 08-02-06 to 08-02-08; SH 41016



SH 41016 1


SH 41016 2

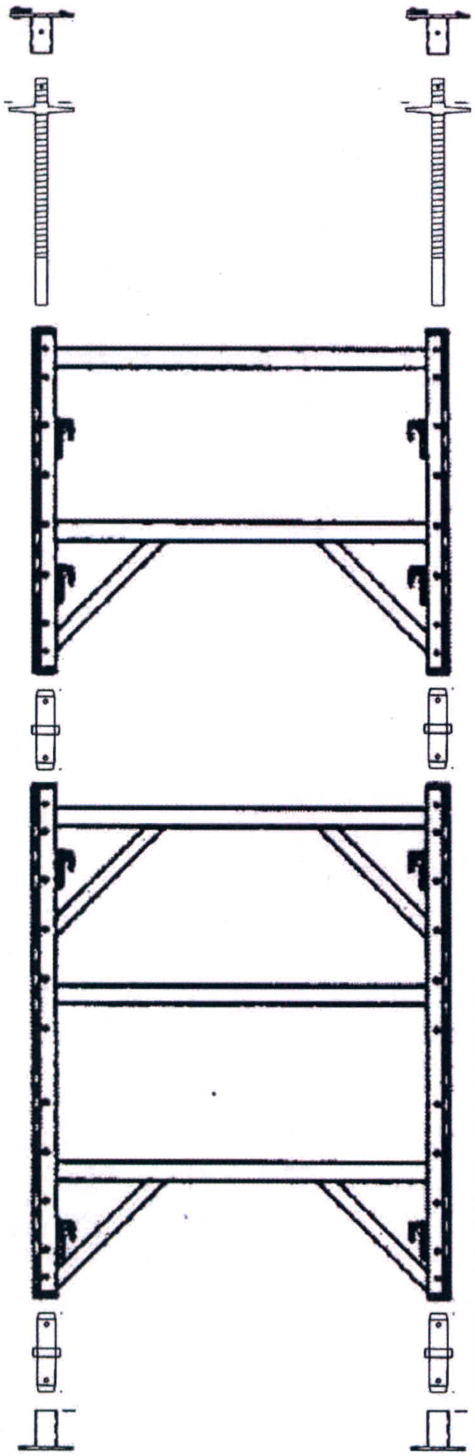


SH 41017
Bottom of SH 41012
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41017

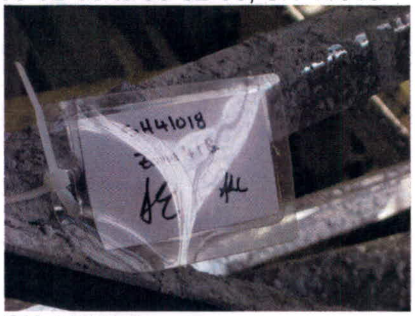
SH 41017 1

SH 41017 2



SH 41018
Brace for SH 41017
Zone 41B

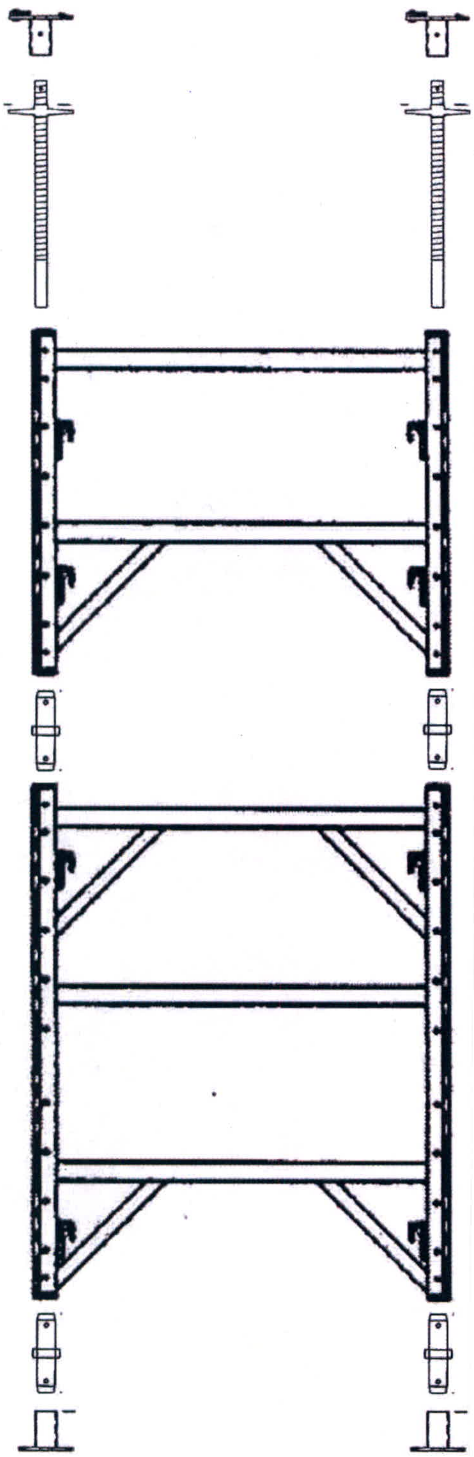
Folder: 08-02-06 to 08-02-08; SH 41018



SH 41018 1

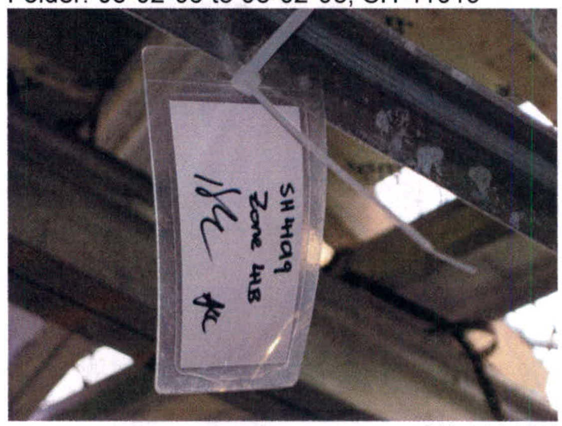
SH 41018 3

SH 41018 2



SH 41019
Top brace
Connect to SH 41012
Zone 41B

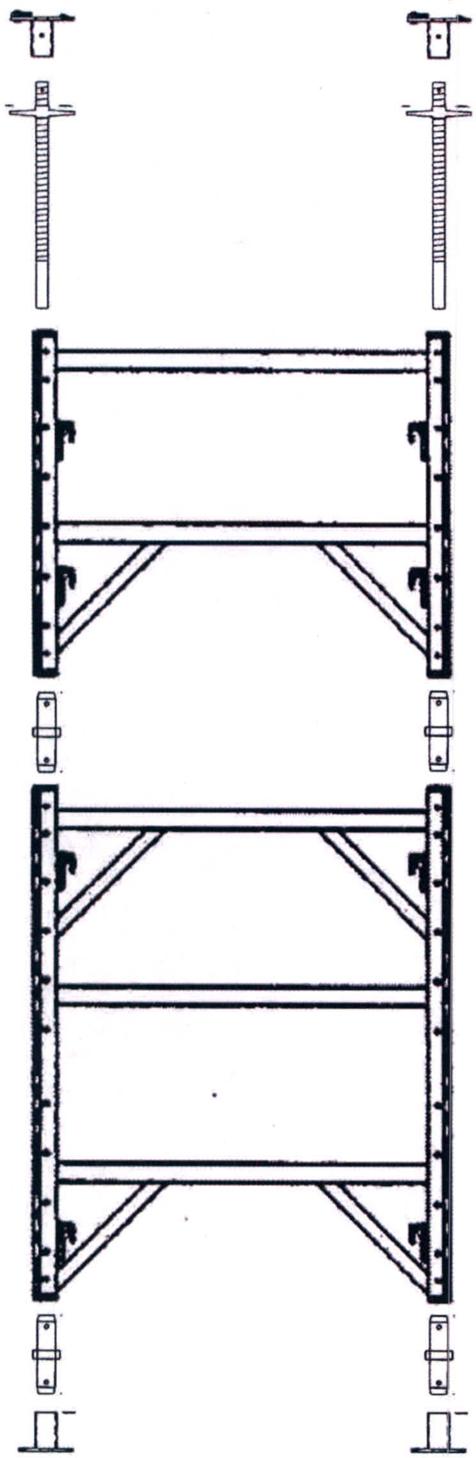
Folder: 08-02-06 to 08-02-08; SH 41019



SH 41019 1

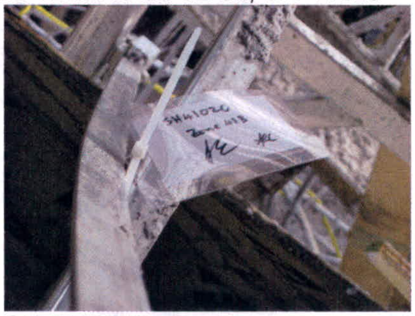
SH 41019 2

SH 41019 3



SH 41020
Bottom Brace
Attached to SH 41017 & SH 41039
Zone 41B

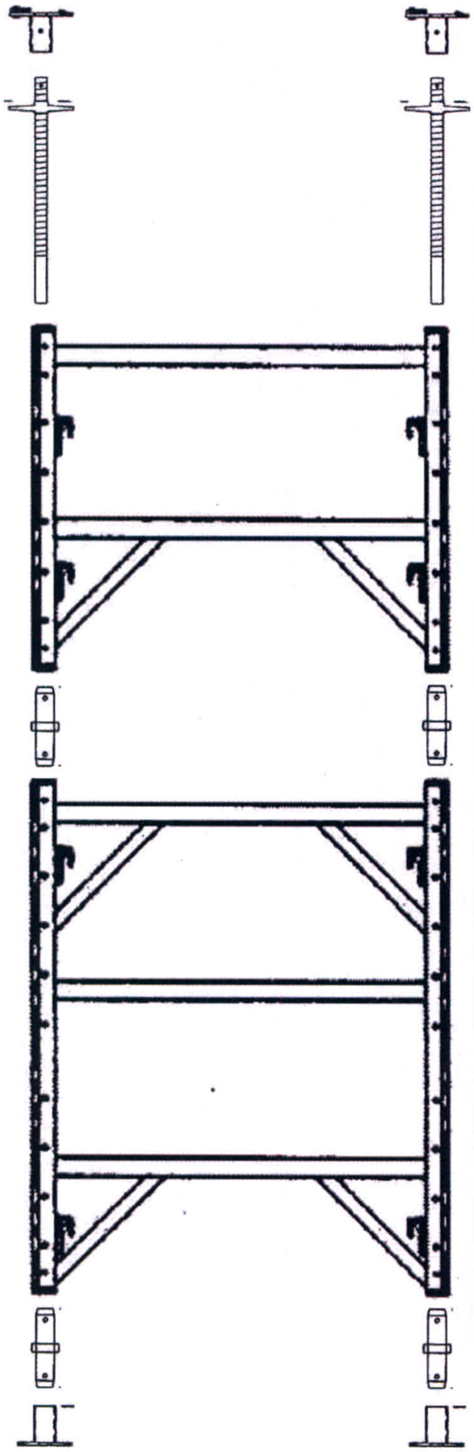
Folder: 08-02-06 to 08-02-08; SH 41020



SH 41020 1

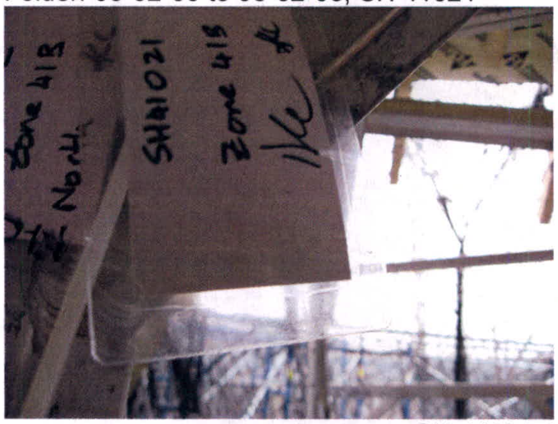
SH 41020 2

SH 41020 3



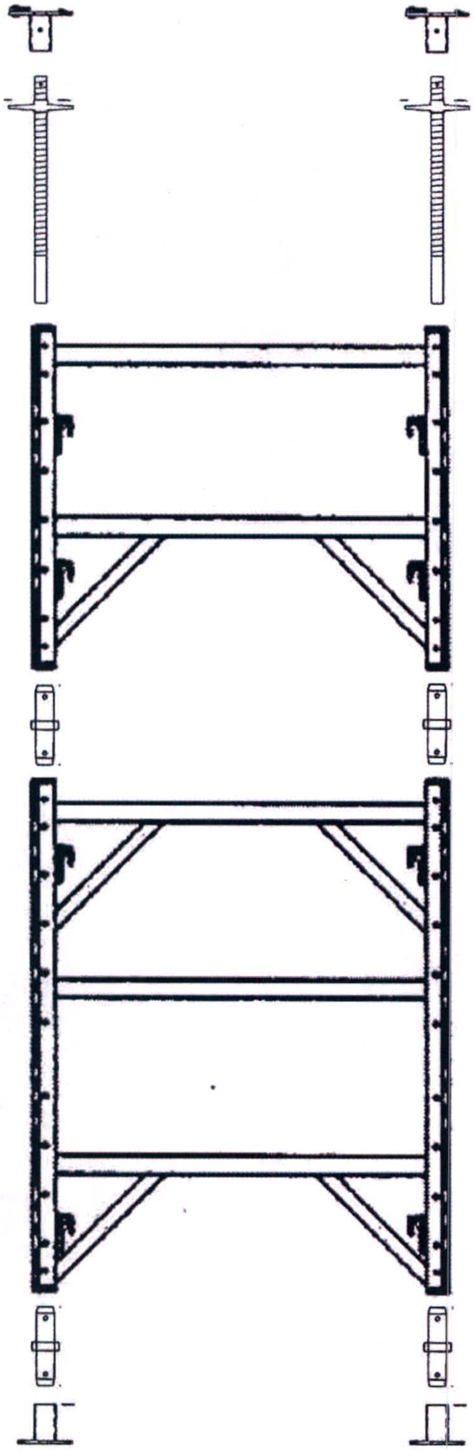
SH 41021
Top brace
Attached to SH 41012
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41021



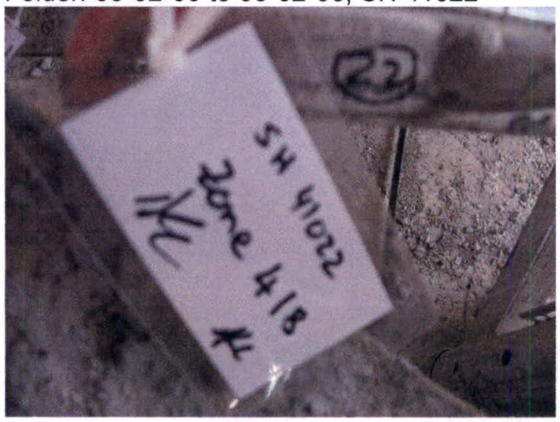
SH 41021 1

SH 41021 2



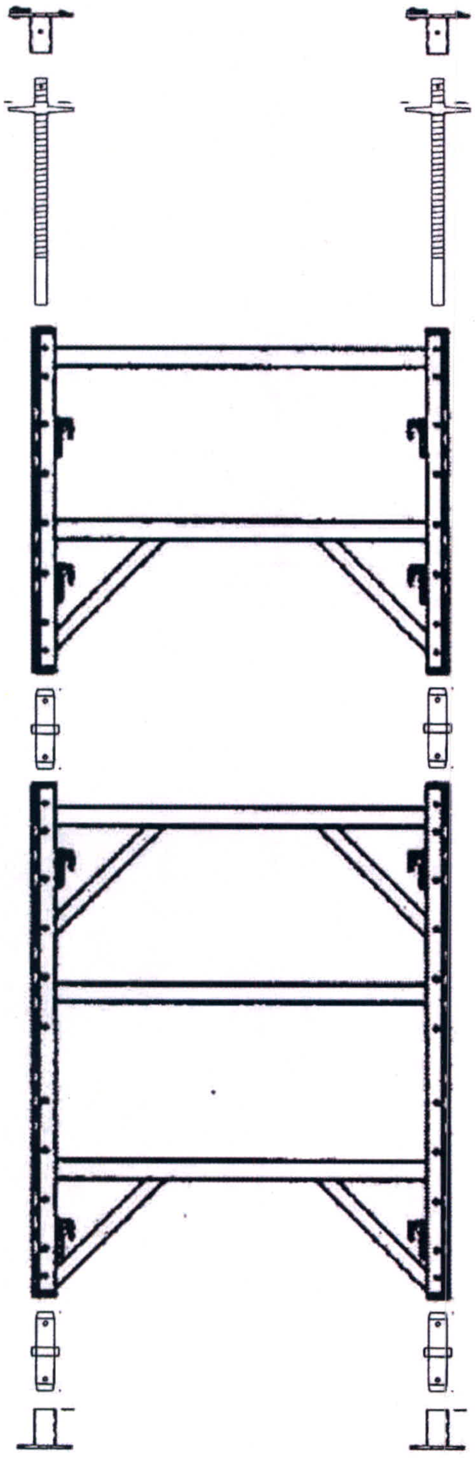
SH 41022
Brace
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41022



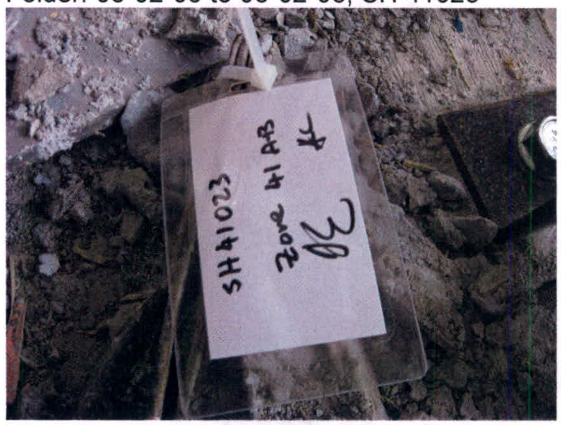
SH 41022 1

SH 41022 2



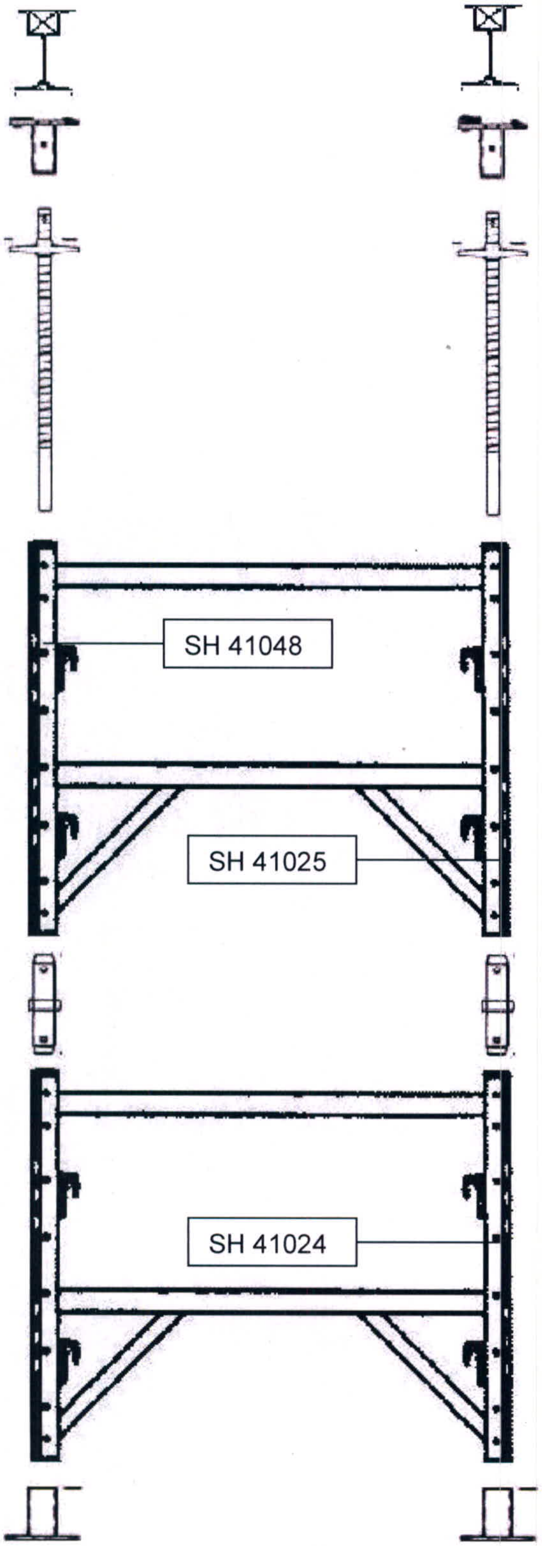
SH 41023
Brace
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41023



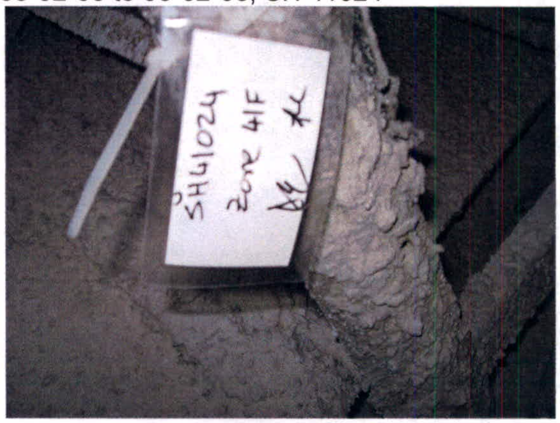
SH 41023 1

SH 41023 2



SH 41024
Bottom Shore of SH 41025 and SH 41048
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41024

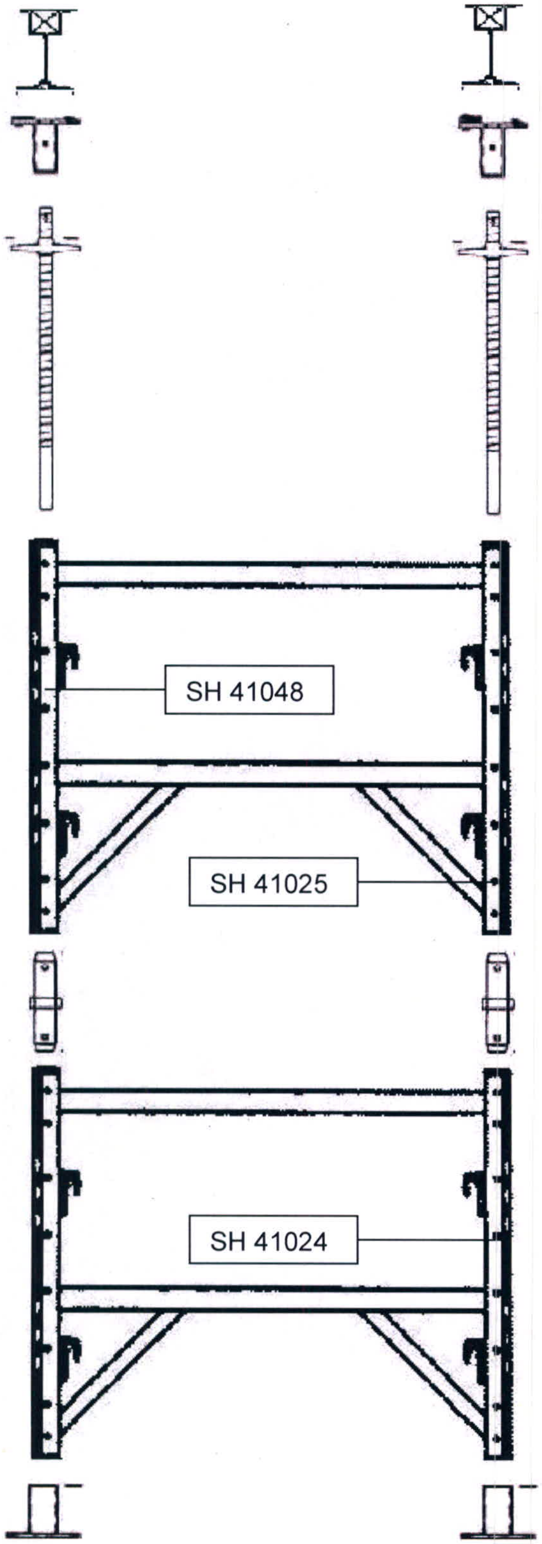


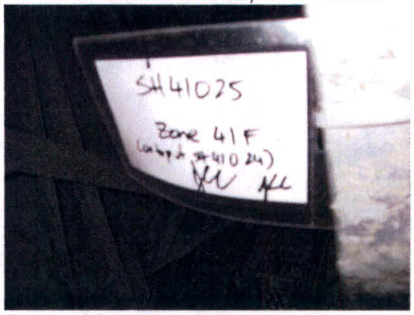
SH 41024 1

SH 41024 2

SH 41025
Broken Top Shore of SH 41024
Connect to top plate SH 41026
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41025

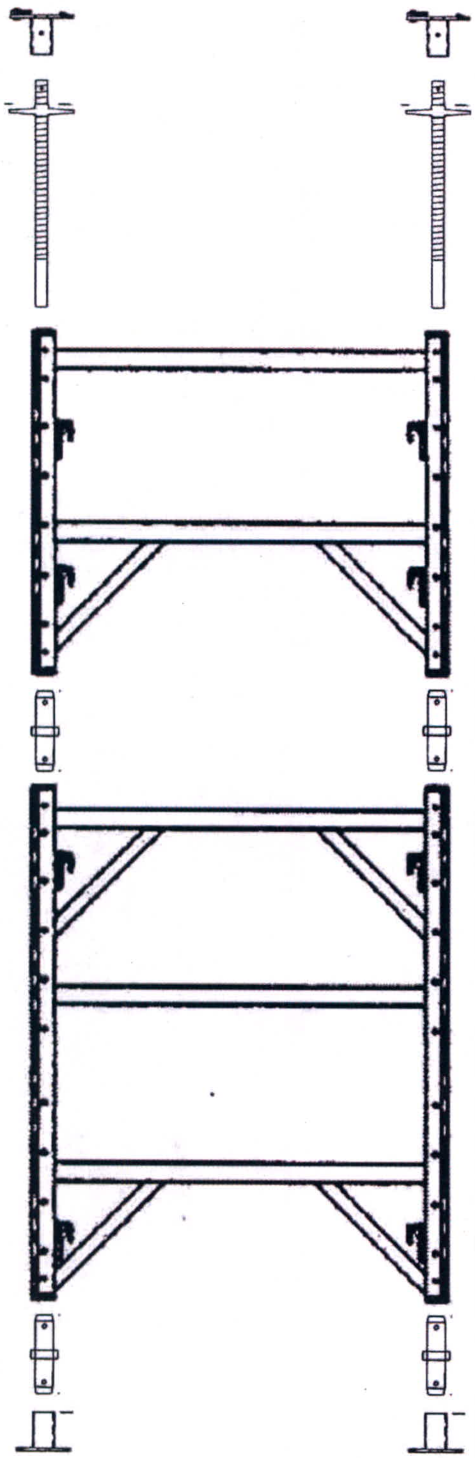




SH 41025 1

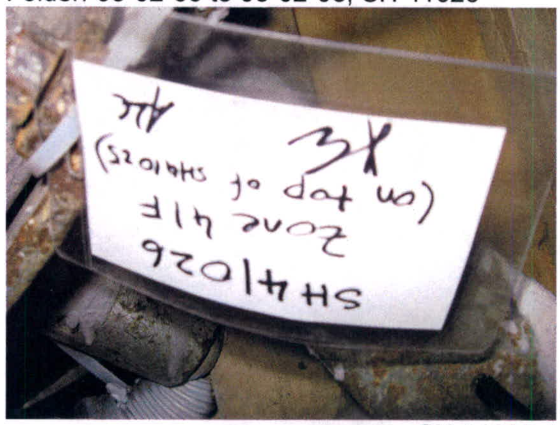

SH 41024 2


SH 41025 2



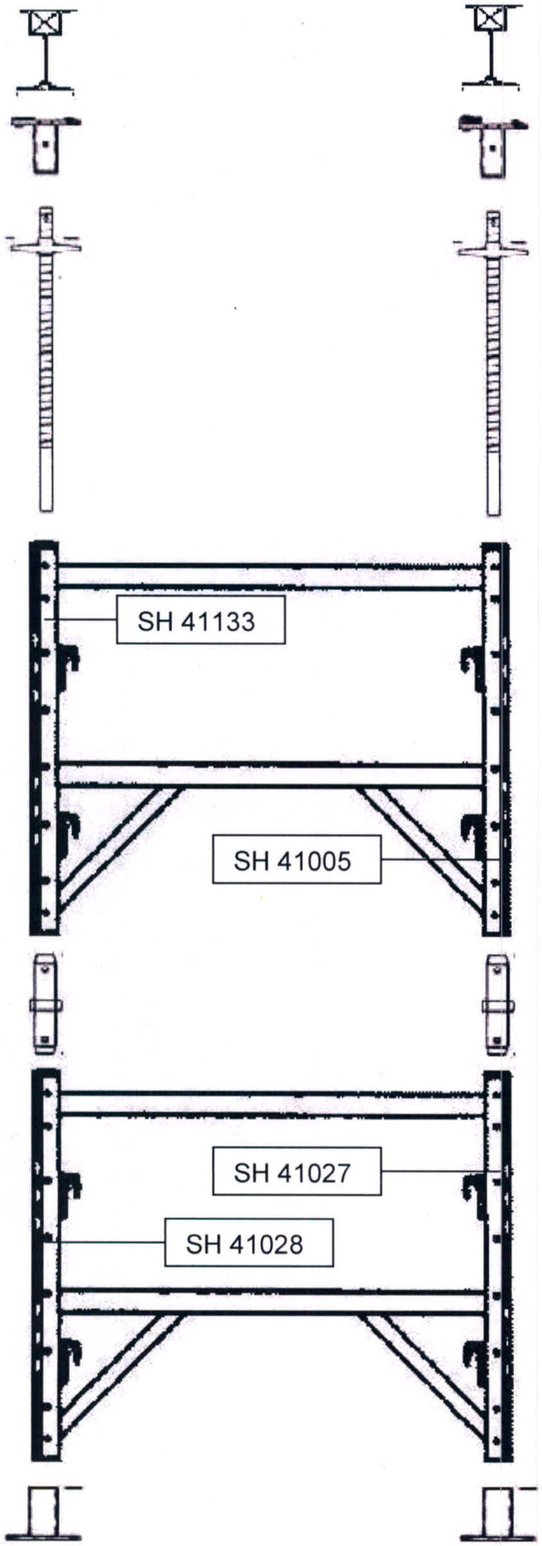
SH 41026
Top plate of SH 41025
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41026



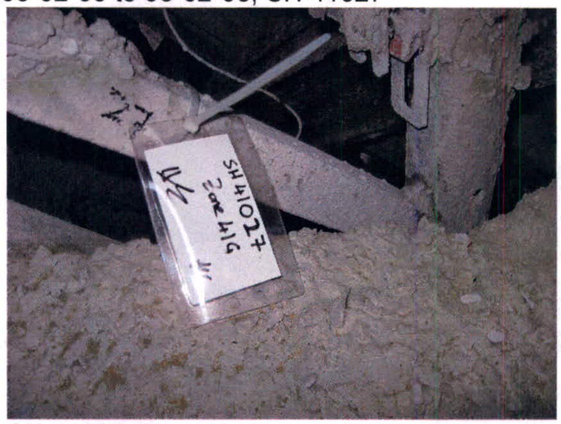
SH 41026 1

SH 41025 2



SH 41027
Broken
Bottom Frame of SH 41005
Zone 41G

Folder: 08-02-06 to 08-02-08; SH 41027

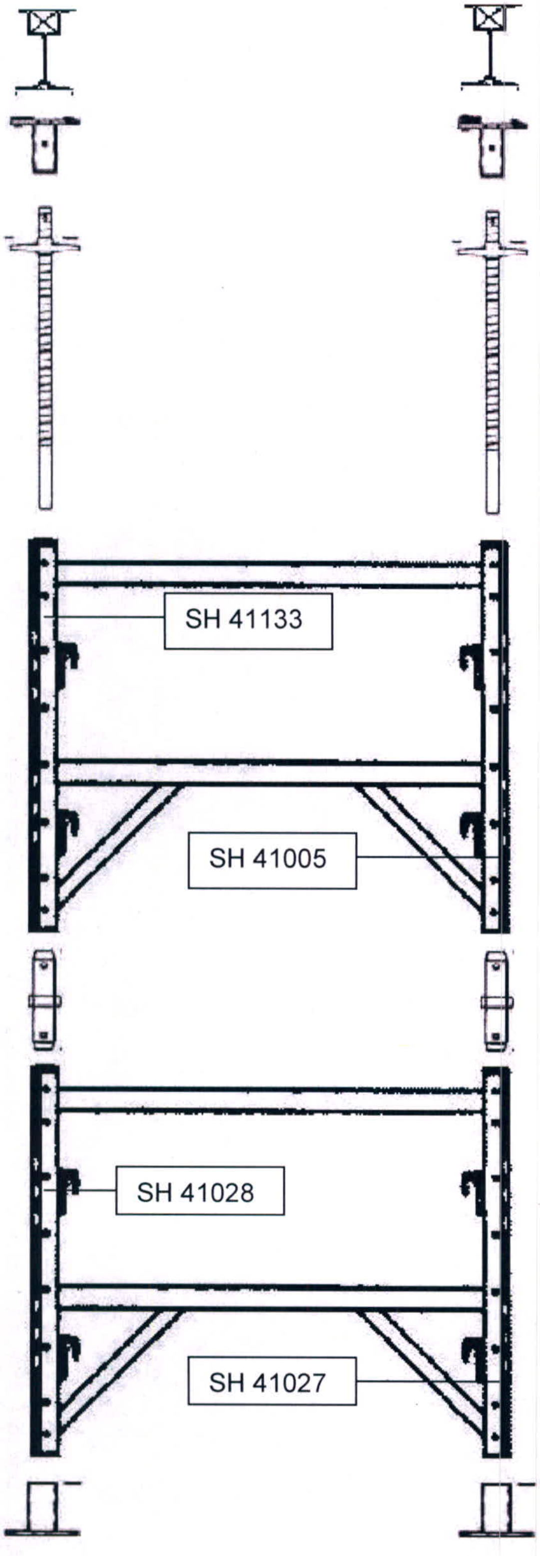


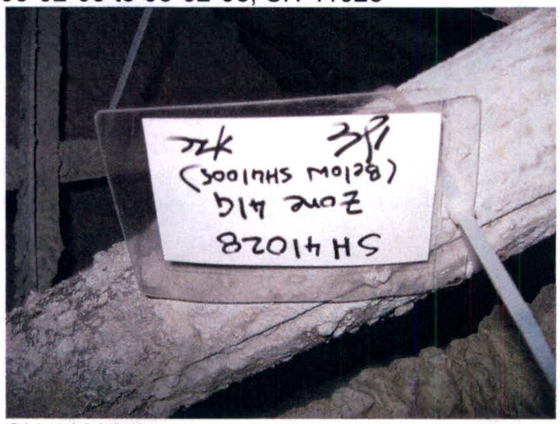
SH 41027 1

SH 41027 2

SH 41028
Broken
Bottom Frame 3'6"
Bottom Shore of SH 41005
Connected to brace SH 41030
Zone 41G

Folder: 08-02-06 to 08-02-08; SH 41028

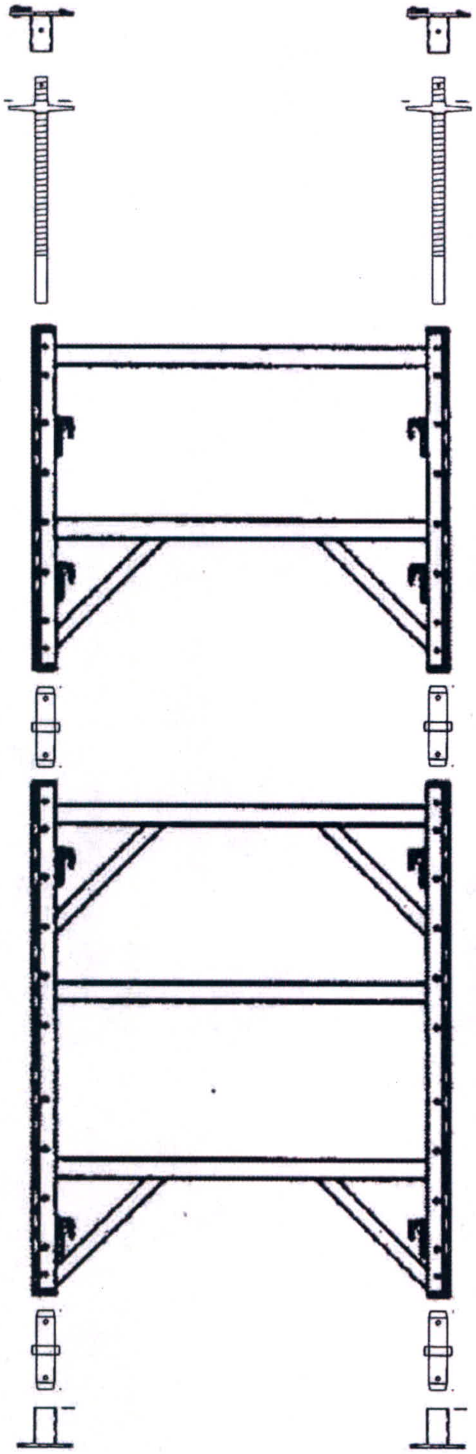




SH 41028 1

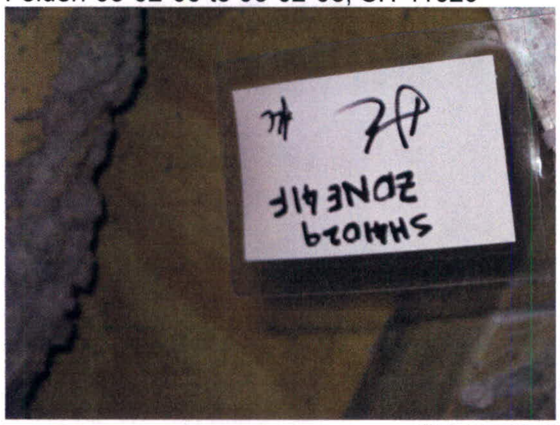

SH 41028 2


SH 41028_3



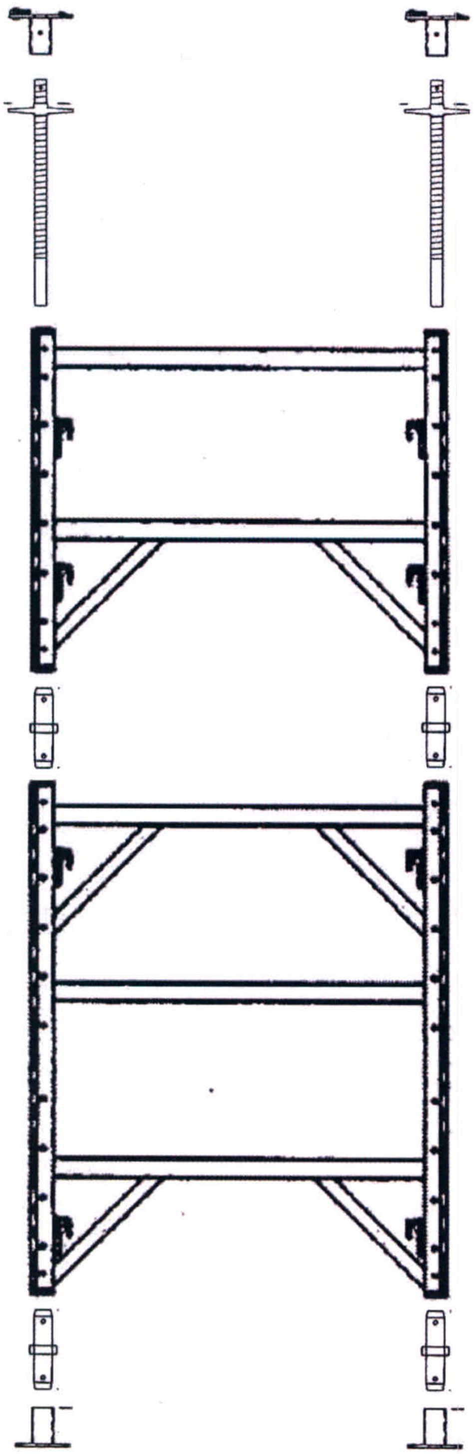
SH 41029
Top brace
Attached to SH 410005
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41029



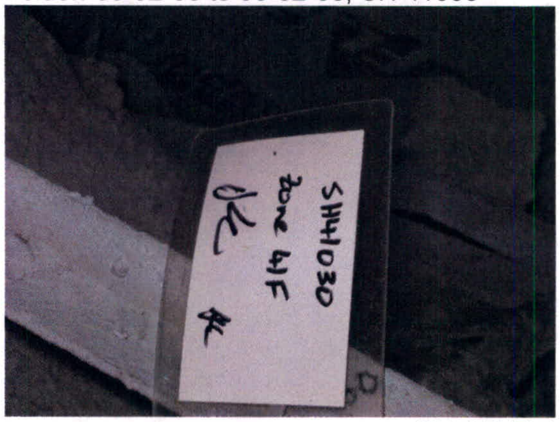
SH 41029 1

SH 41029 2



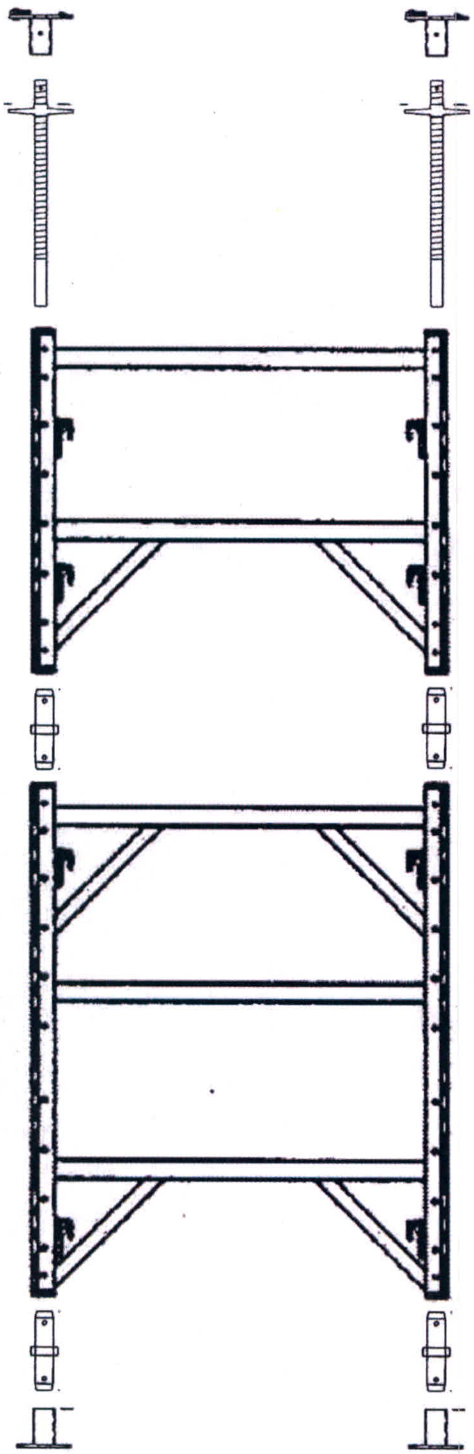
SH 41030
Bottom Brace
Attached to SH 41028
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41030



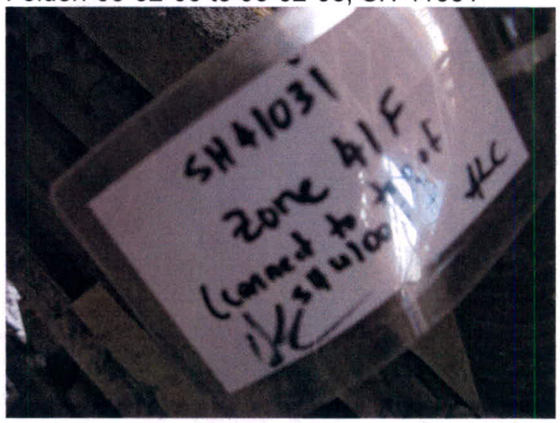
SH 41030 1

SH 41030 3



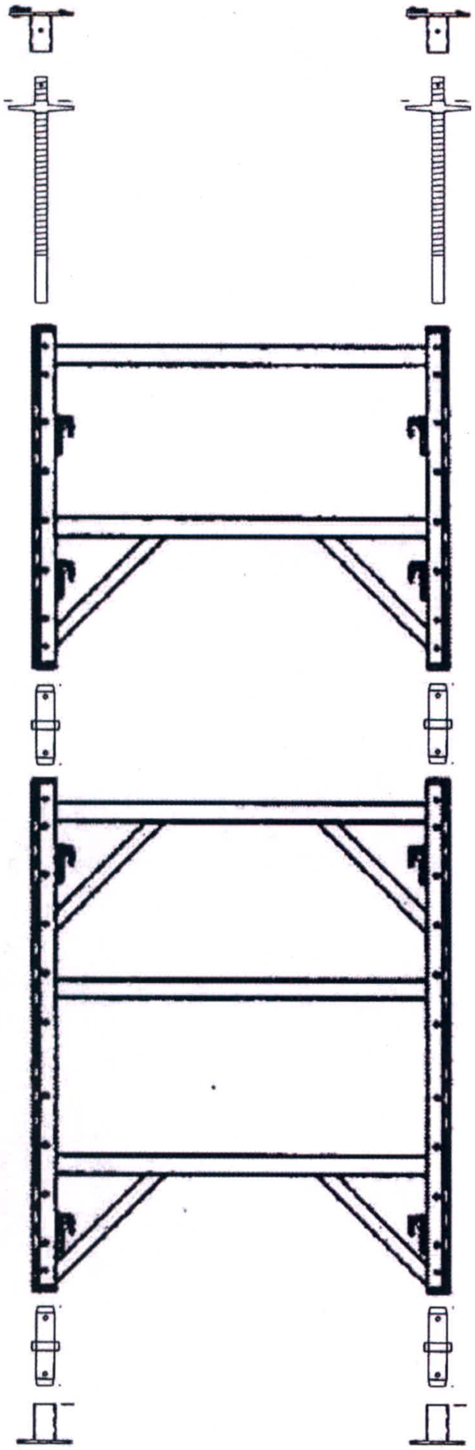
SH 41031
Top plate of SH 41005 and AB 41004
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41031



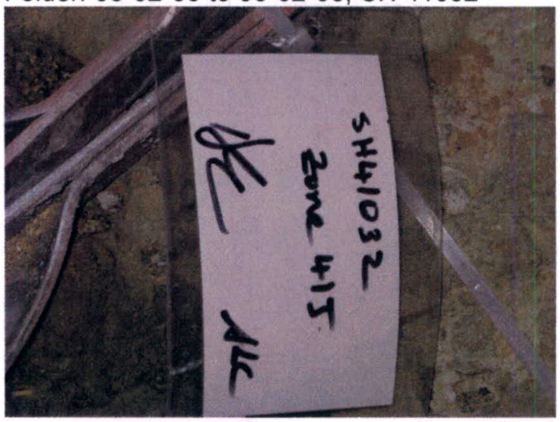
SH 41031 1

SH 41031 2



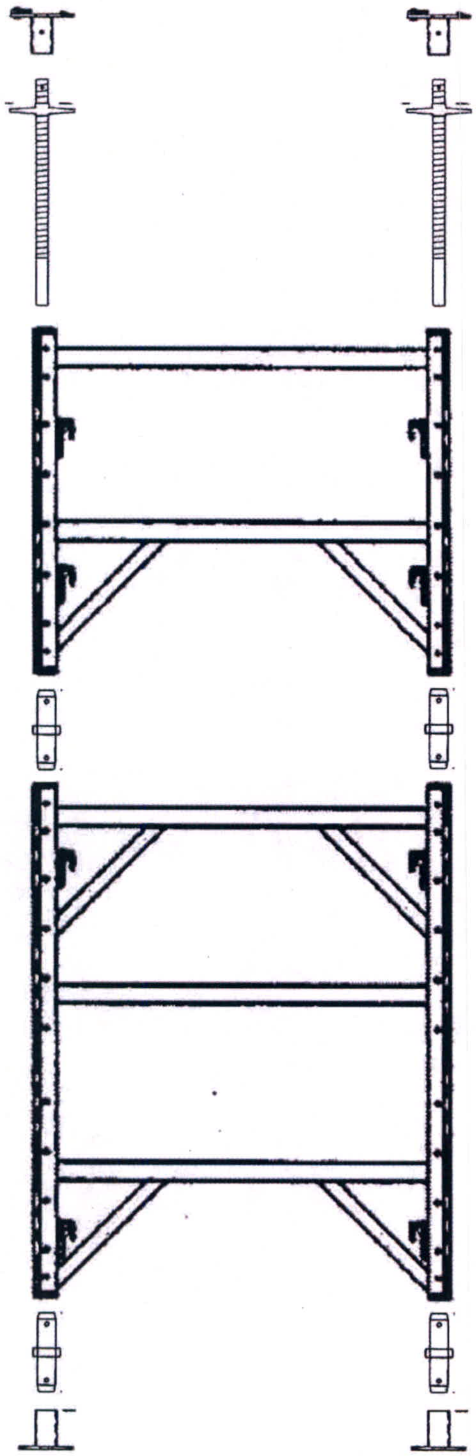
SH 41032
Brace
Zone 41J

Folder: 08-02-06 to 08-02-08; SH 41032



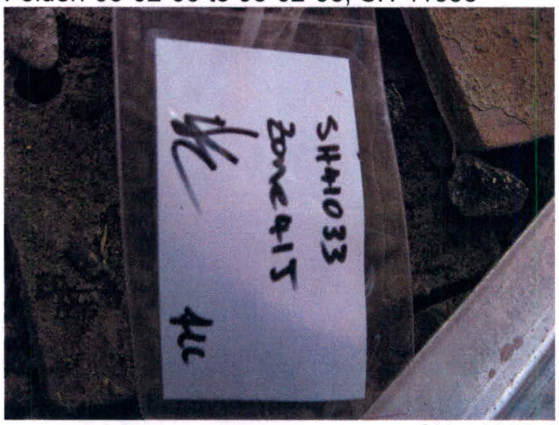
SH 41032 1

SH 41032 2



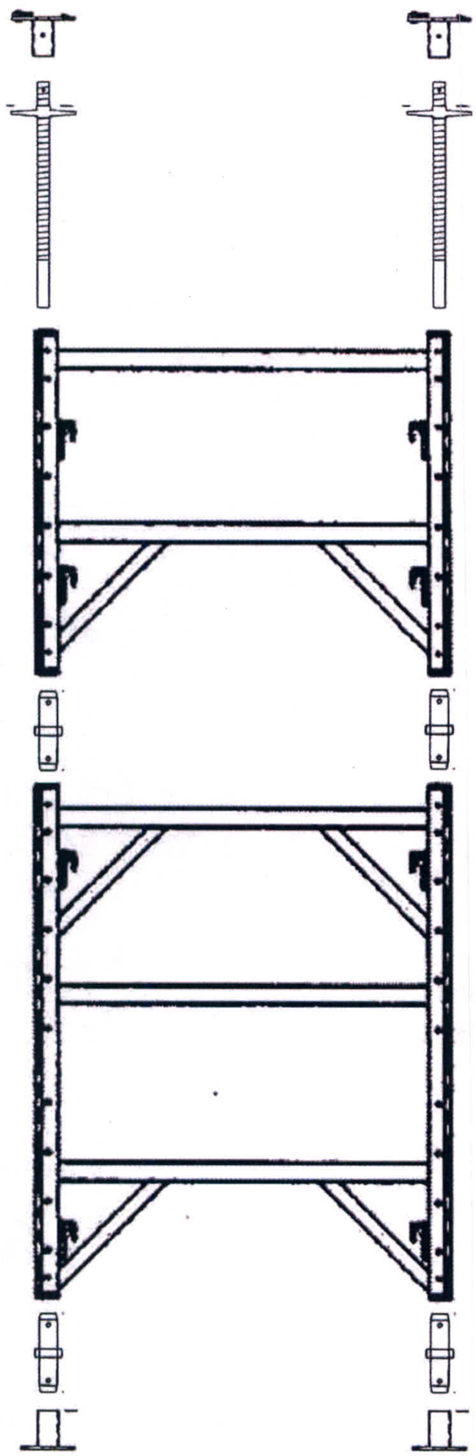
SH 41033
End plate
Zone 41J

Folder: 08-02-06 to 08-02-08; SH 41033



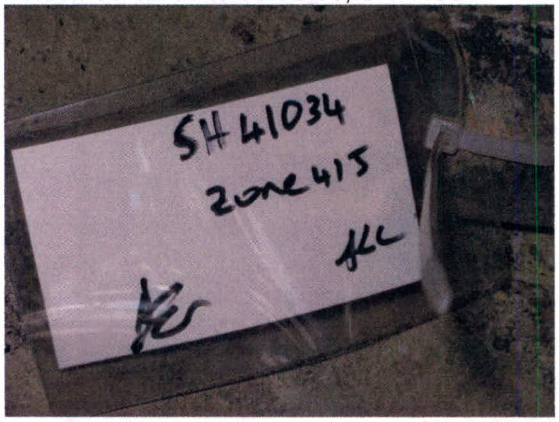
SH 41033 1

SH 41033 2



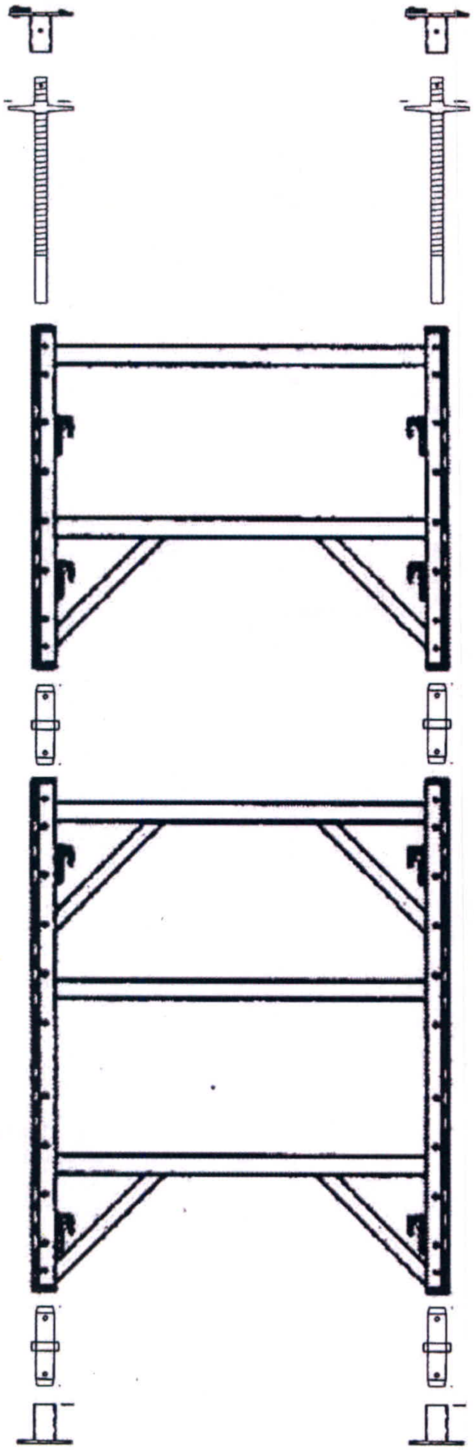
SH 41034
Coupler
Zone 41J

Folder: 08-02-06 to 08-02-08; SH 41034



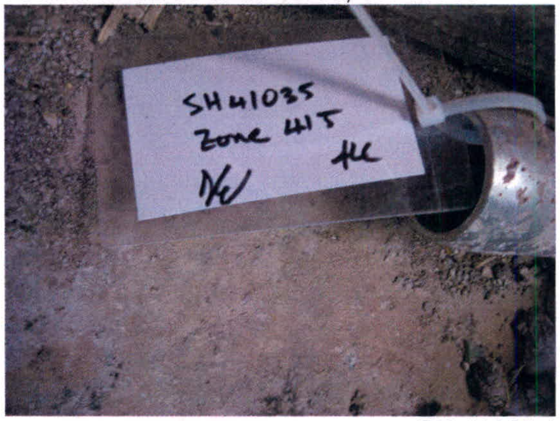
SH 41034 1

SH 41034 2



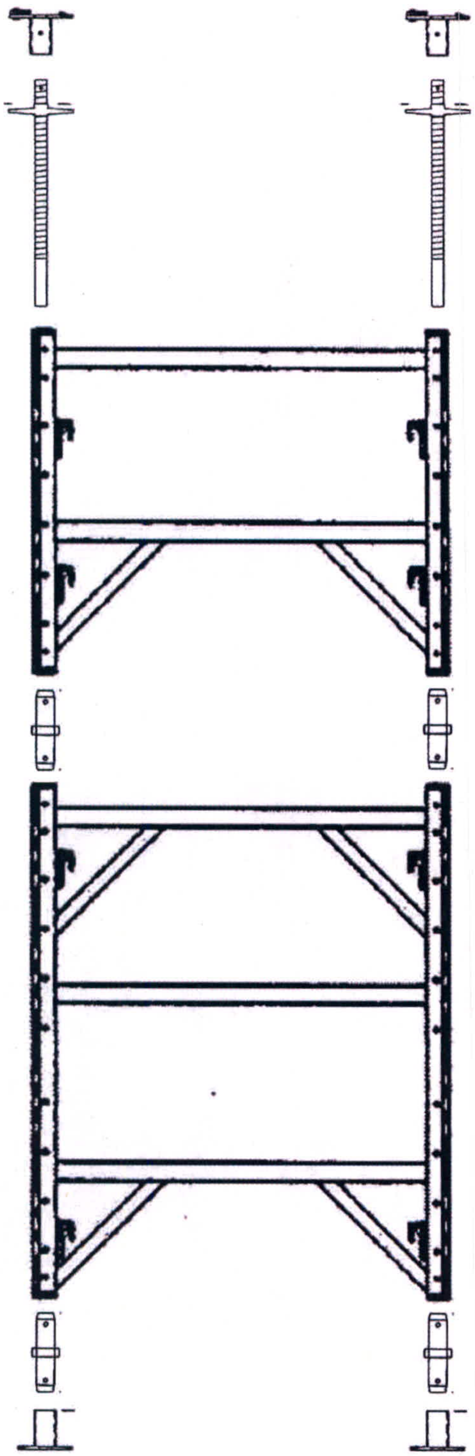
SH 41035
Coupler

Folder: 08-02-06 to 08-02-08; SH 41035



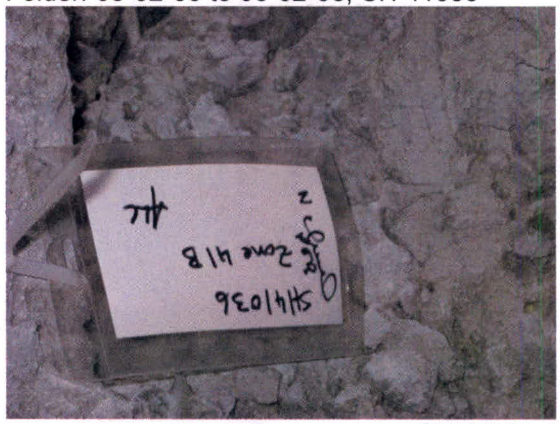
SH 41035 1

SH 41035 2



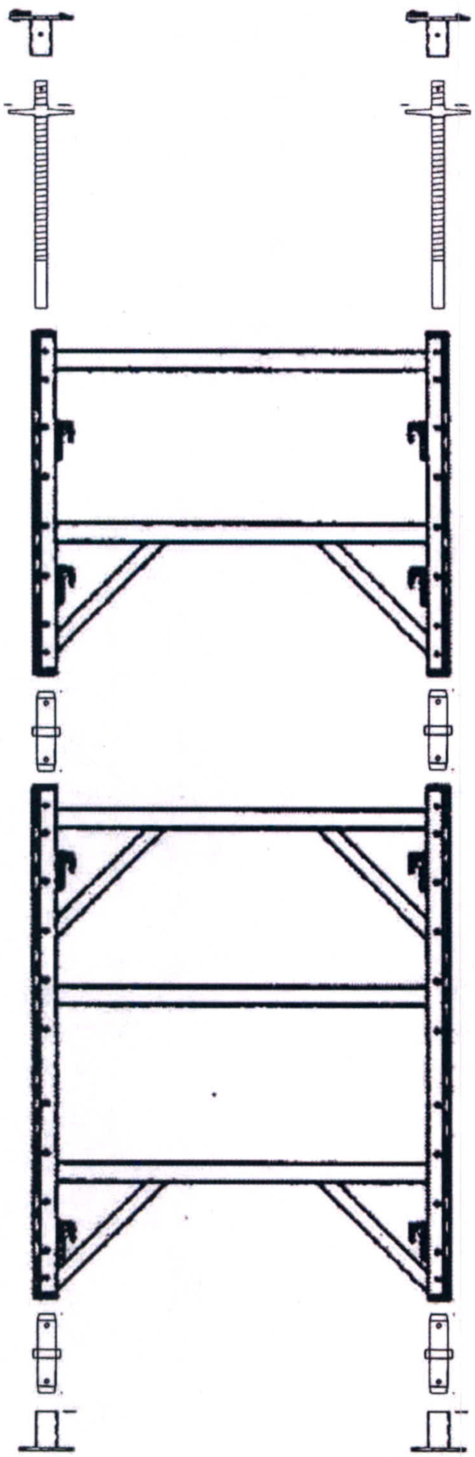
SH 41036
Brace
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41036



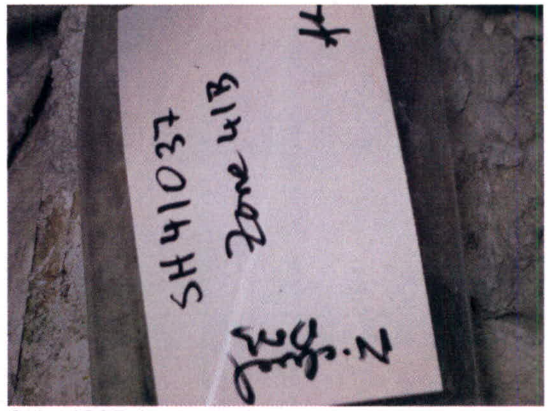
SH 41036 1

SH 41036 2



SH 41037
Brace
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41037

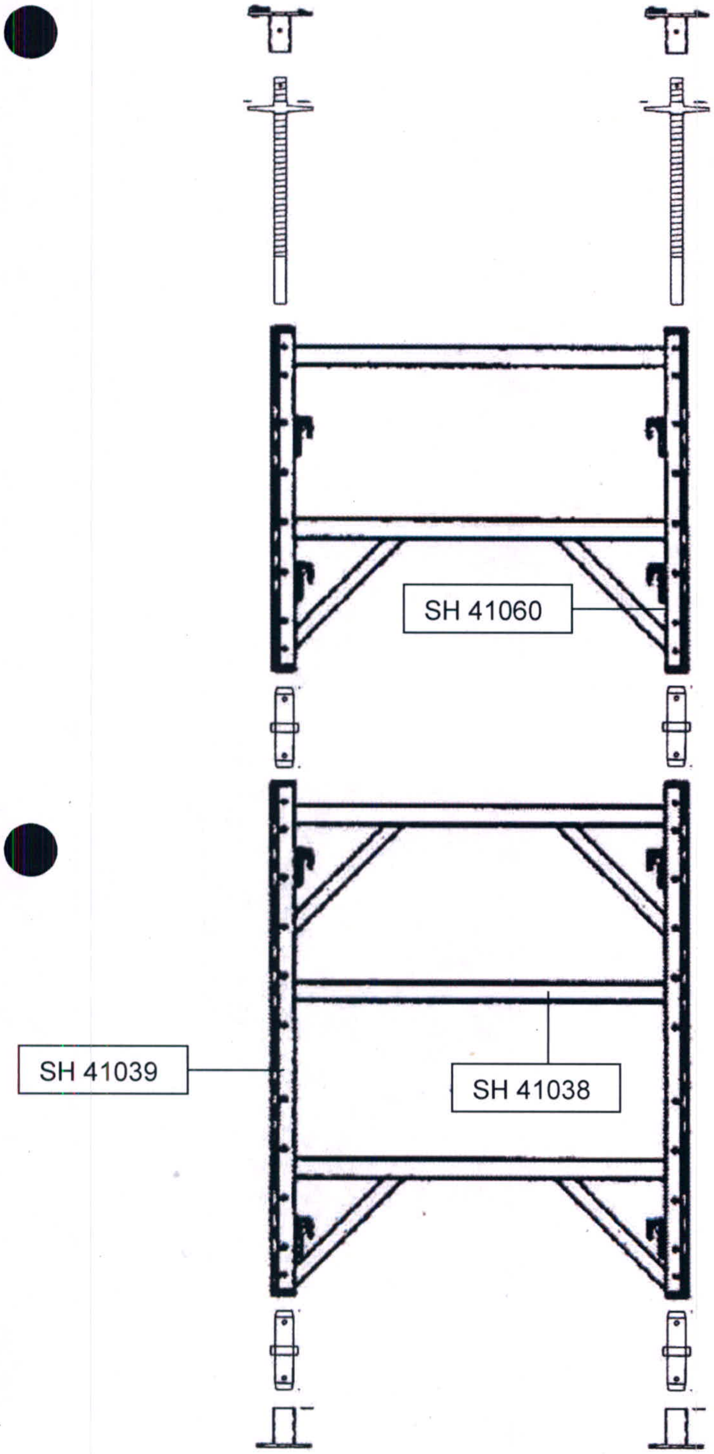


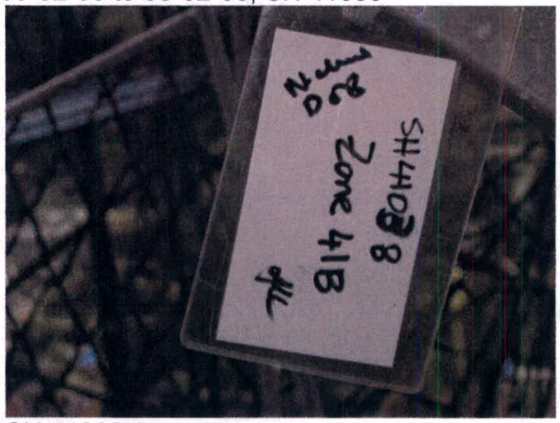
SH 41037 1


SH 41037 2

SH 41038
Bottom Piece of Shoring frame
Attached to SH 41060
Zone 41B

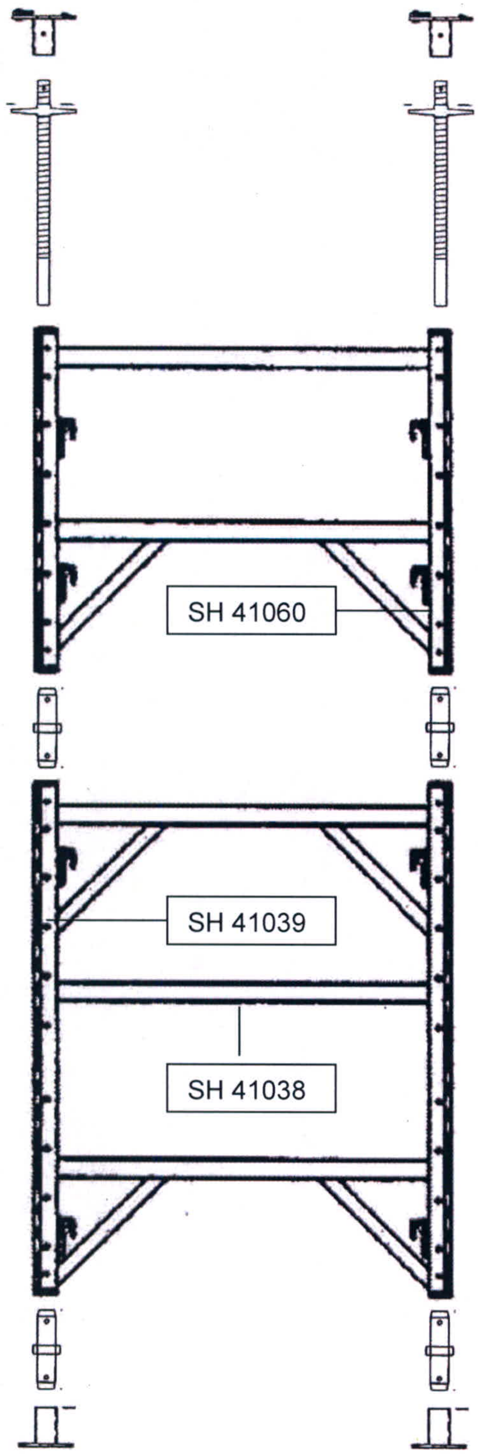
Folder: 08-02-06 to 08-02-08; SH 41038





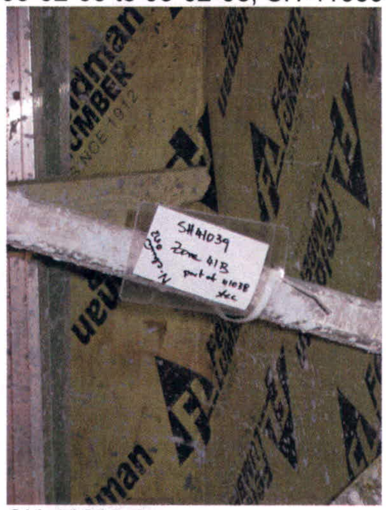
SH 41038 1


SH 41038 2



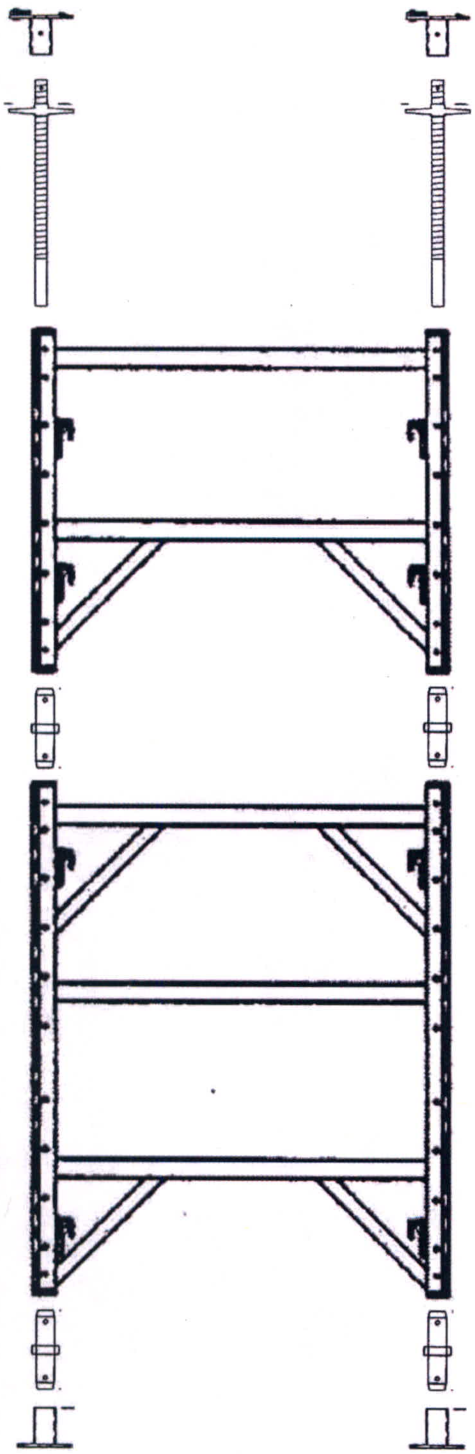
SH 41039
Bottom Frame of SH 41060
Part of SH 41038
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41039

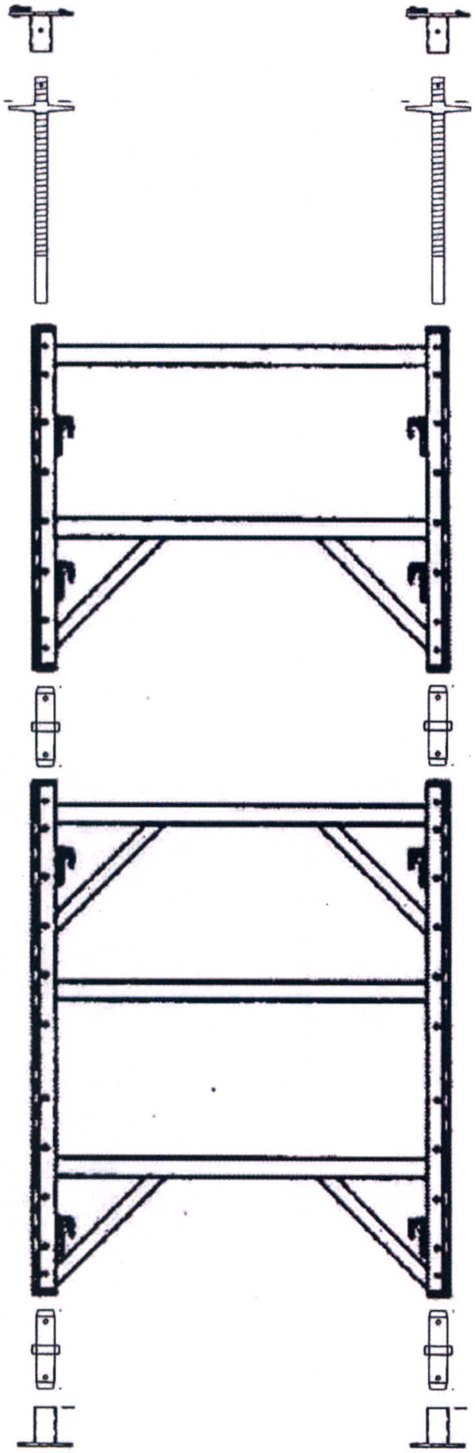


SH 41039 1

SH 41039 2

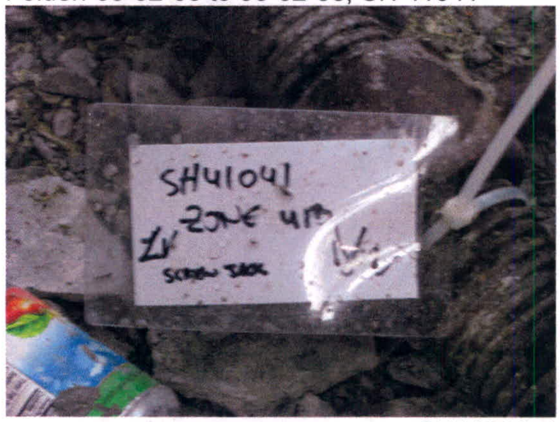


SH 41040
Cross Brace SH 41024
No photo
Zone 41B



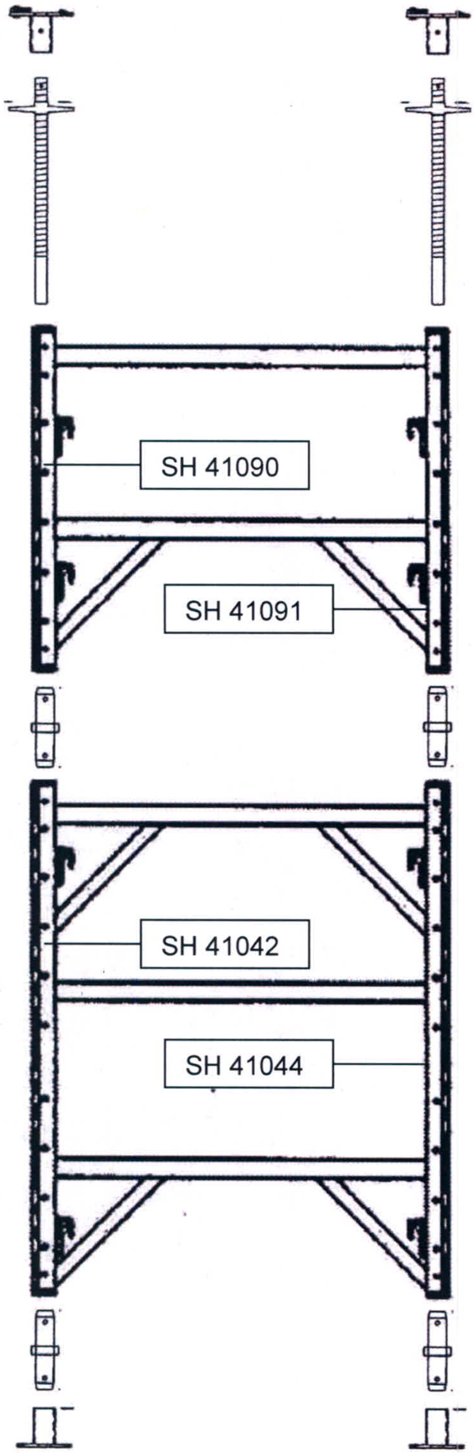
SH 41041
Screw Jack
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41041



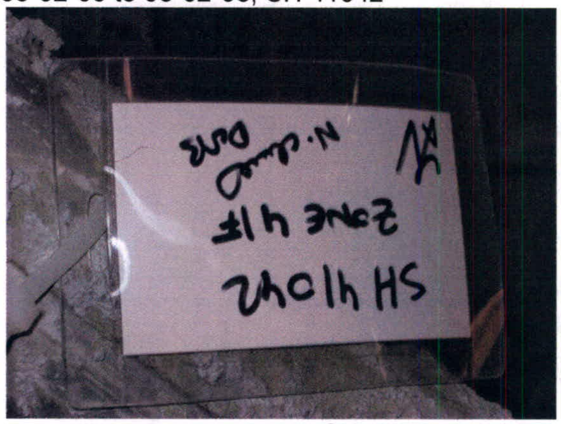
SH41041_1

SH41041_2



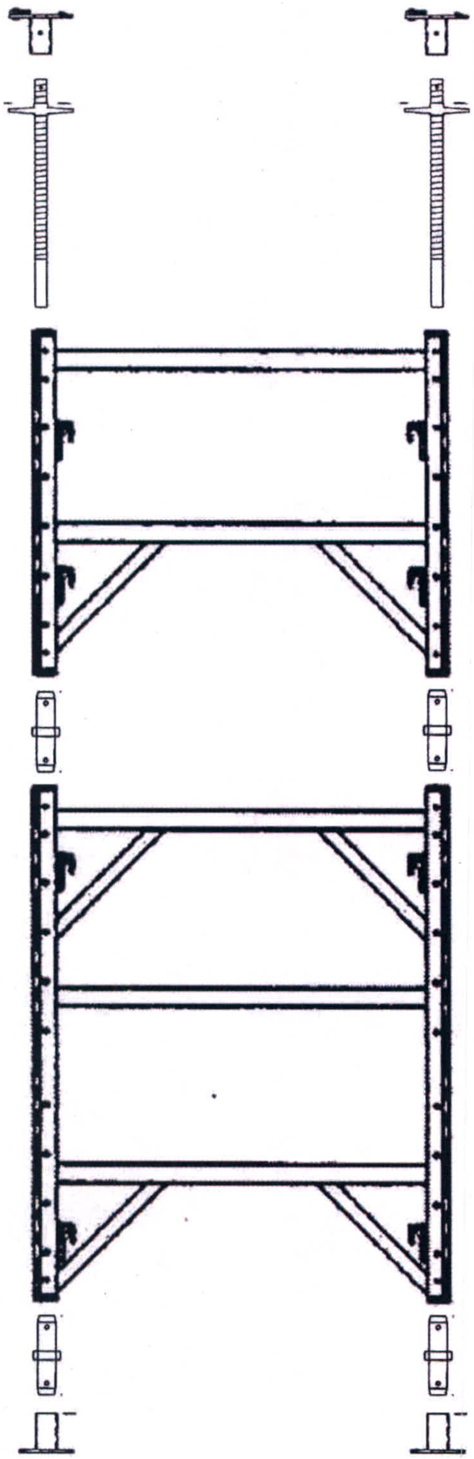
SH 41042
Bottom frame of SH 41091 & SH 41090
Attached to SH 41043
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41042



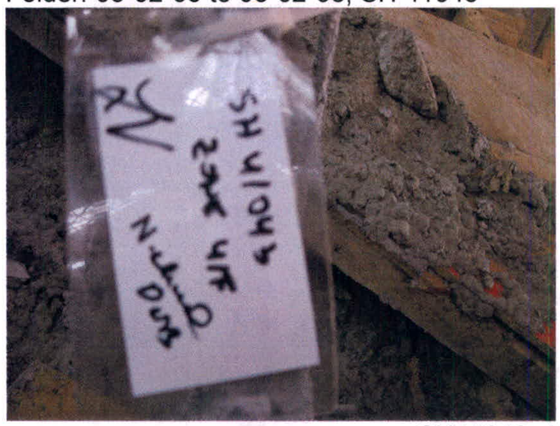
SH41042_1

SH41042_2

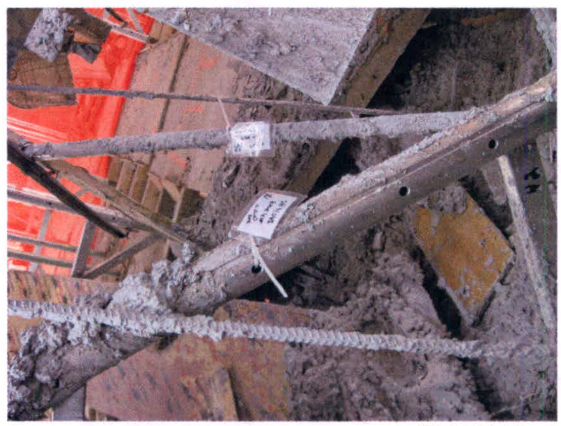


SH 41043
Brace
Attached to SH 41042
Zone 41F

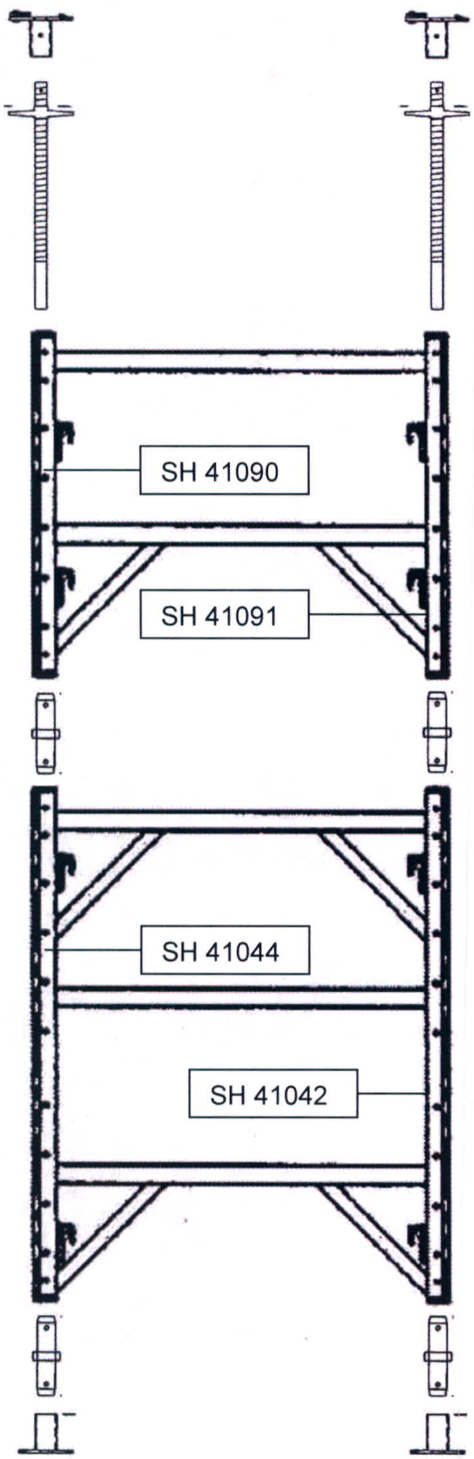
Folder: 08-02-06 to 08-02-08; SH 41043



SH41043_1

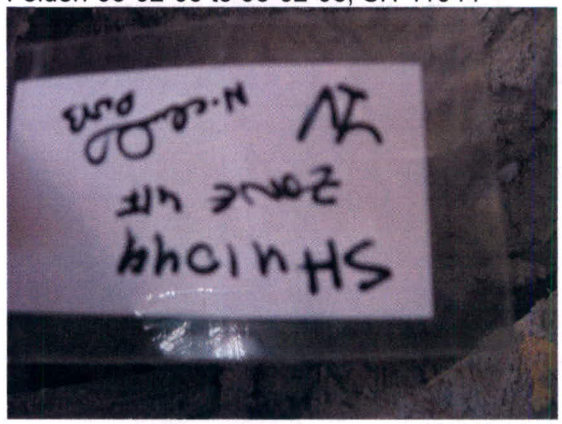


SH41043_2



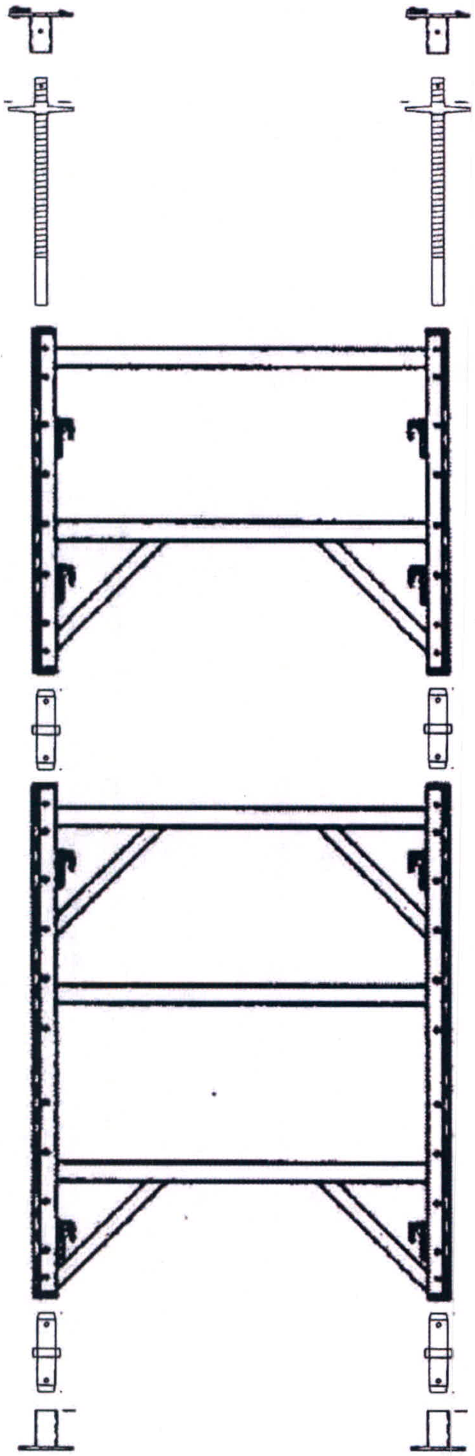
SH 41044
Broken With SH 41042
Bottom Shore of SH 41090 & SH 41091
Attached to SH 41042
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41044



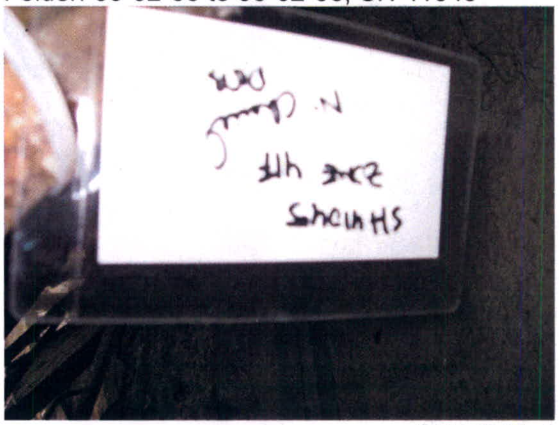
SH41044_1

SH41044_2

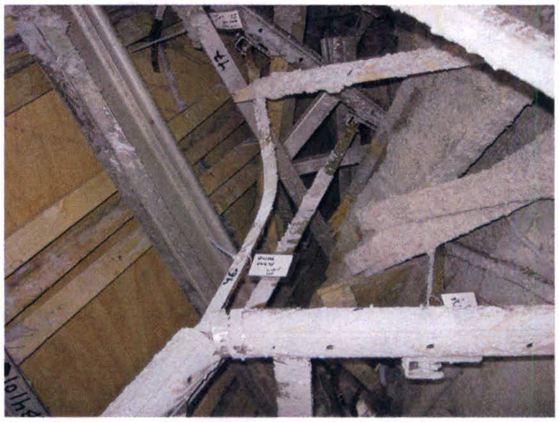


SH 41045
Bottom Brace of SH 41024
Zone 41F

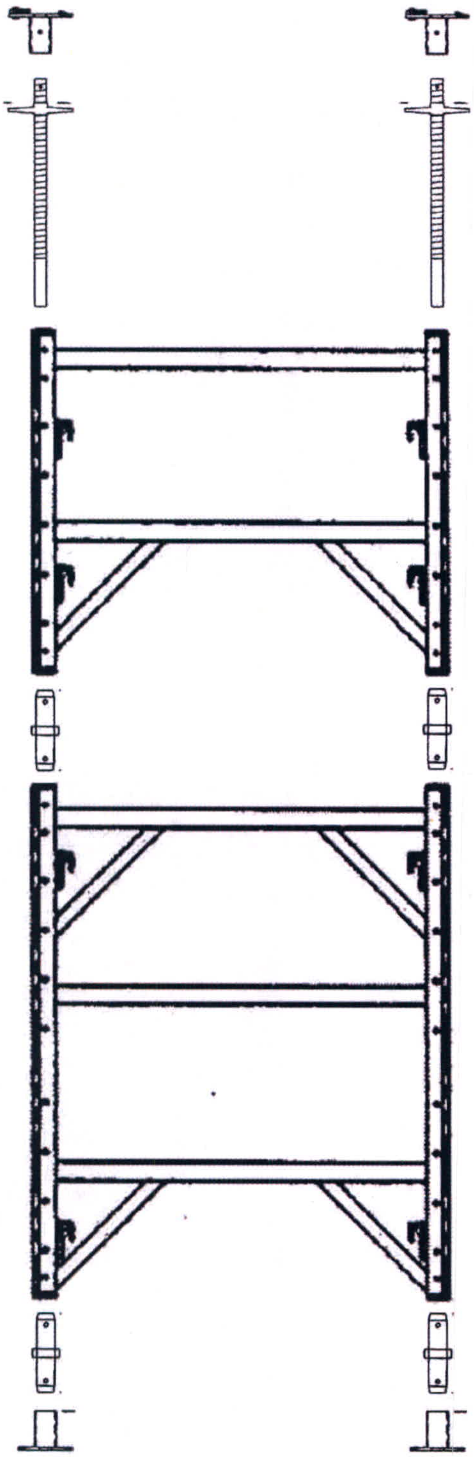
Folder: 08-02-06 to 08-02-08; SH 41045



SH41045_1



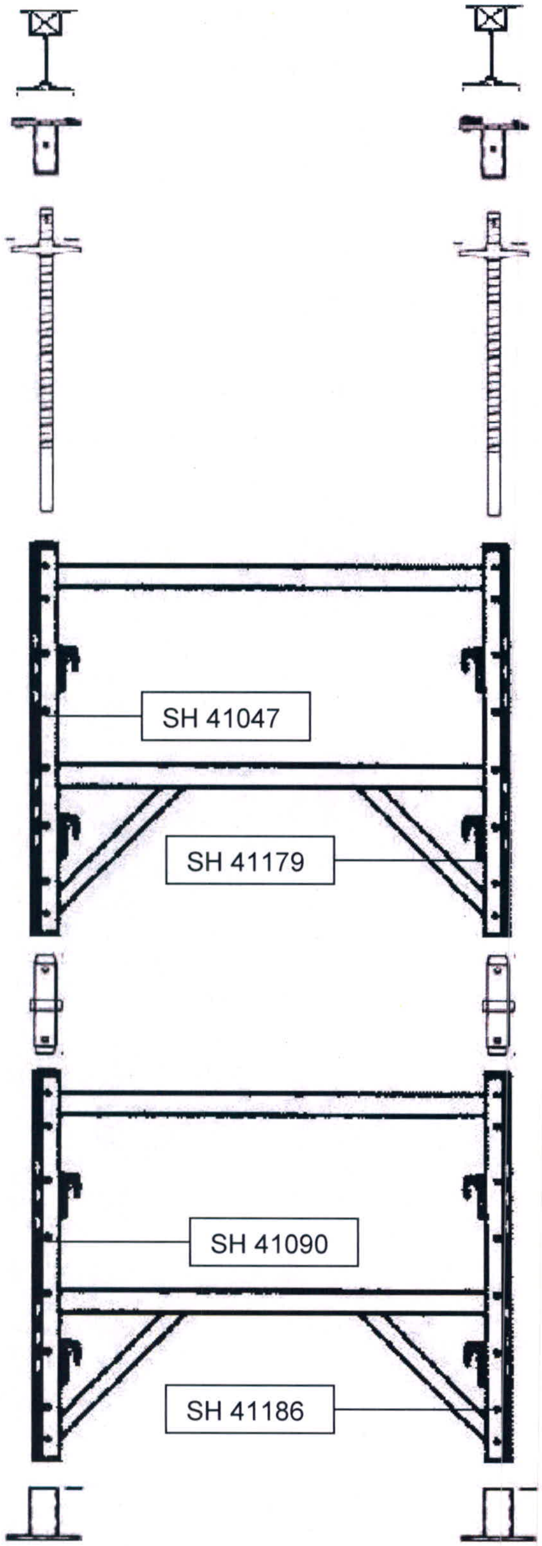
SH41045_2



SH 41046
Brace
Connected to SH 41047
Zone 41F

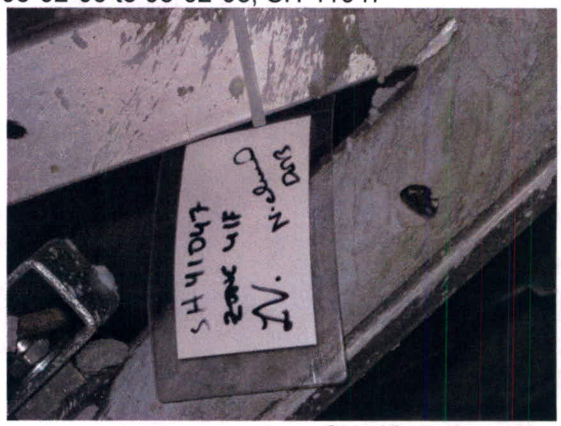
Folder: 08-02-06 to 08-02-08; SH 41046

SH41047_2



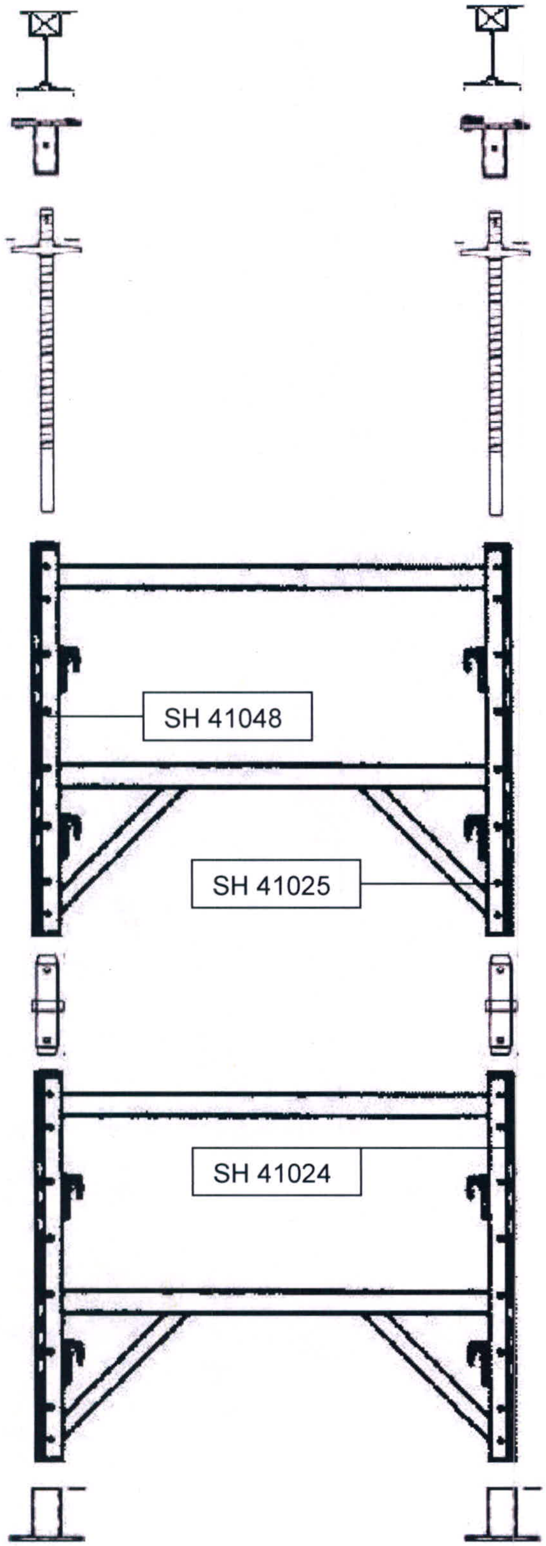
SH 41047
Broken With SH 41179
Top Shore of SH 41186 & SH 41190
Attached to brace SH 41046
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41047



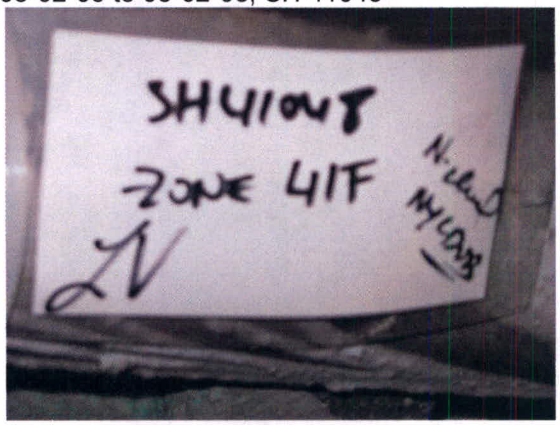
SH41047_1

SH41047_2



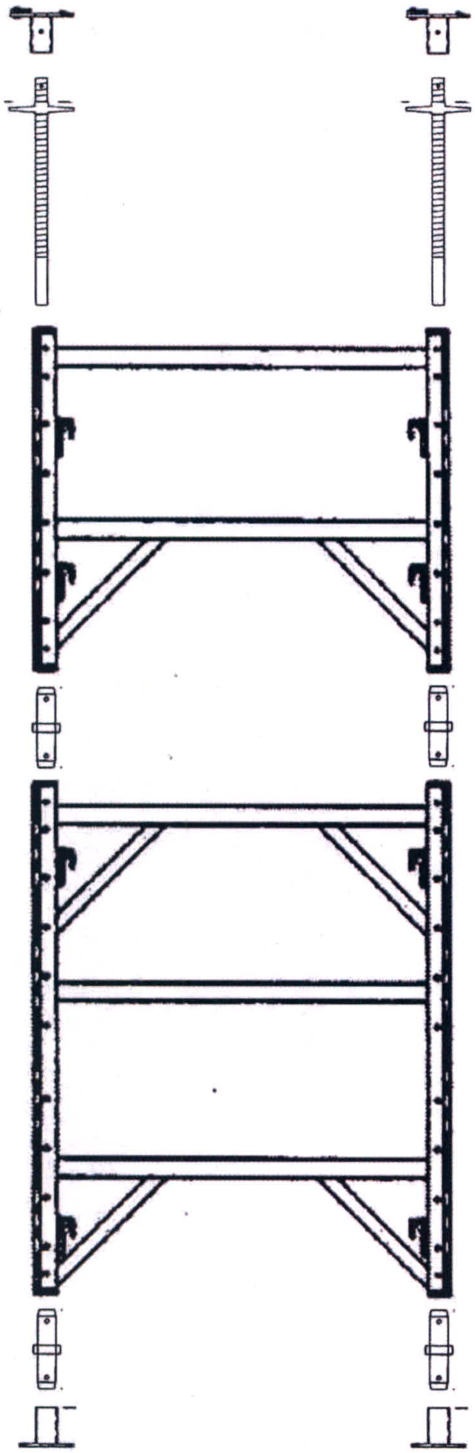
SH 41048
Broken
Top Shore
Attached to SH 41046 on top SH 41024
Piece of Frame SH 41049
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41048



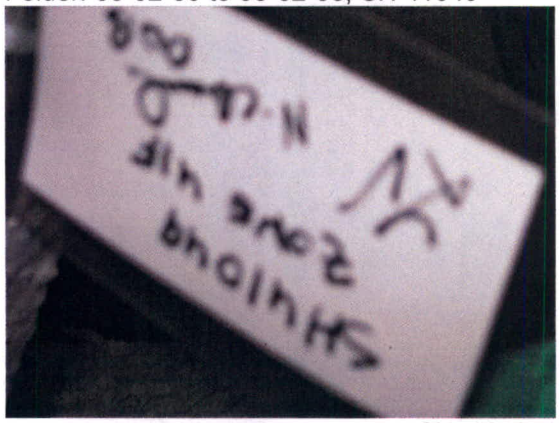
SH41048_1

SH41048_2



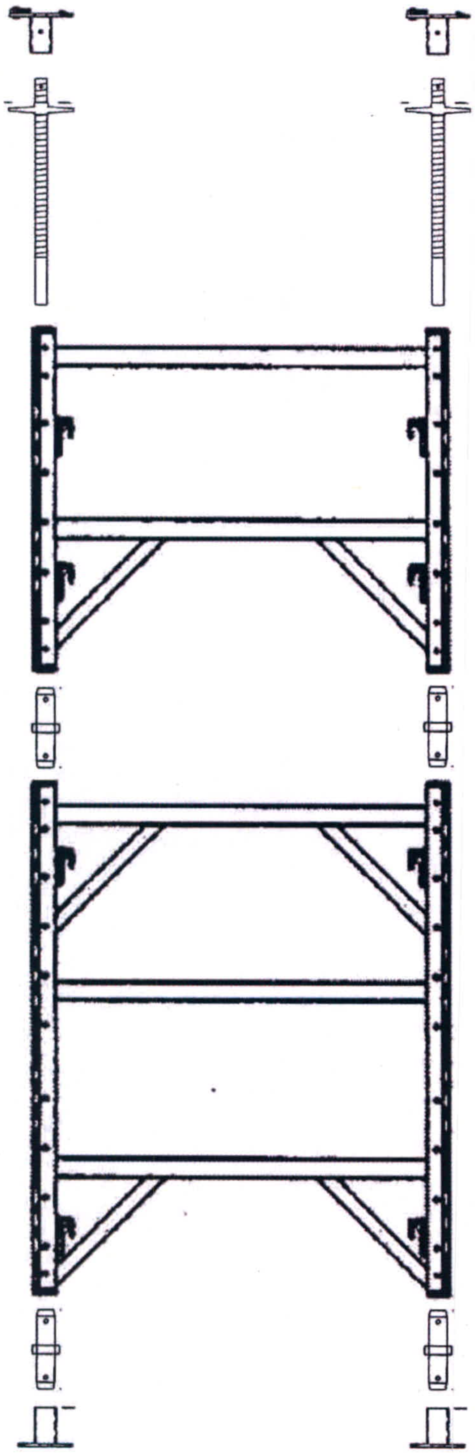
SH 41049
Broken piece from SH 41048
Zone 41 F

Folder: 08-02-06 to 08-02-08; SH 41049



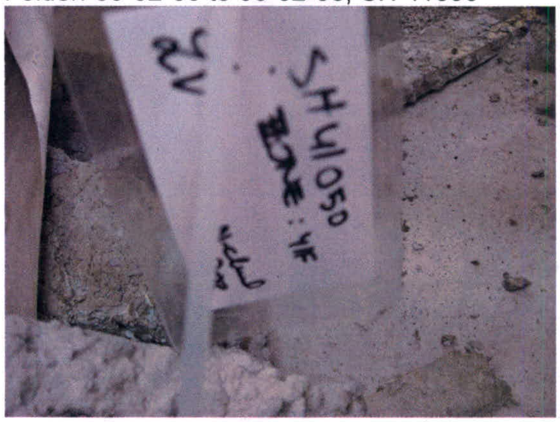
SH41049_1

SH41049_2



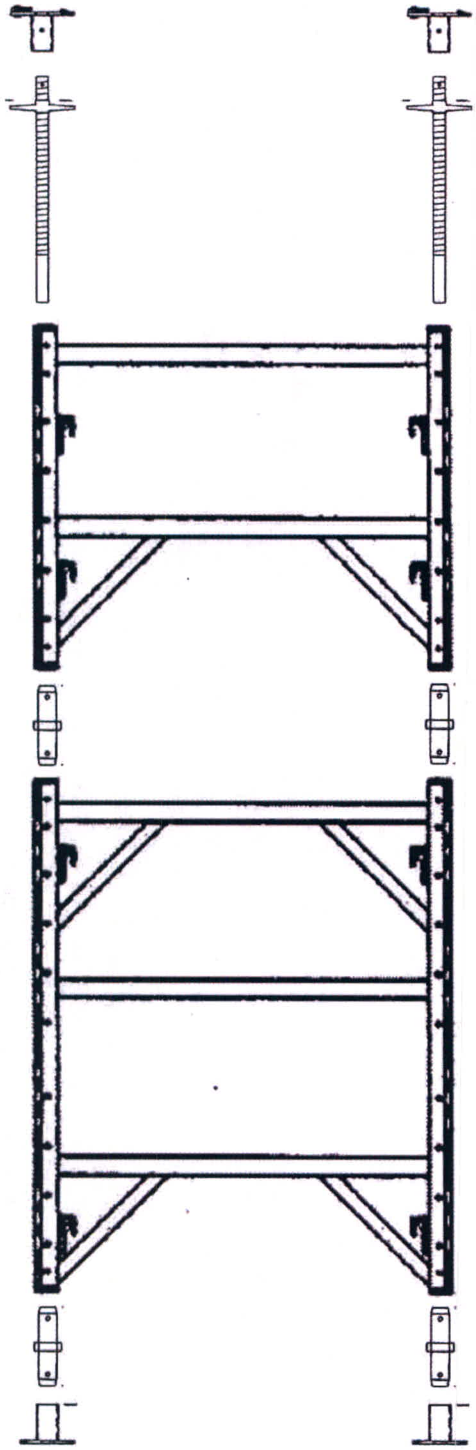
SH 41050
Brace
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41050



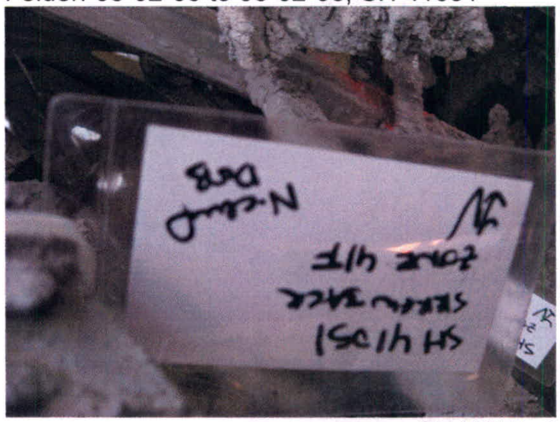
SH41050_1

SH41050_2



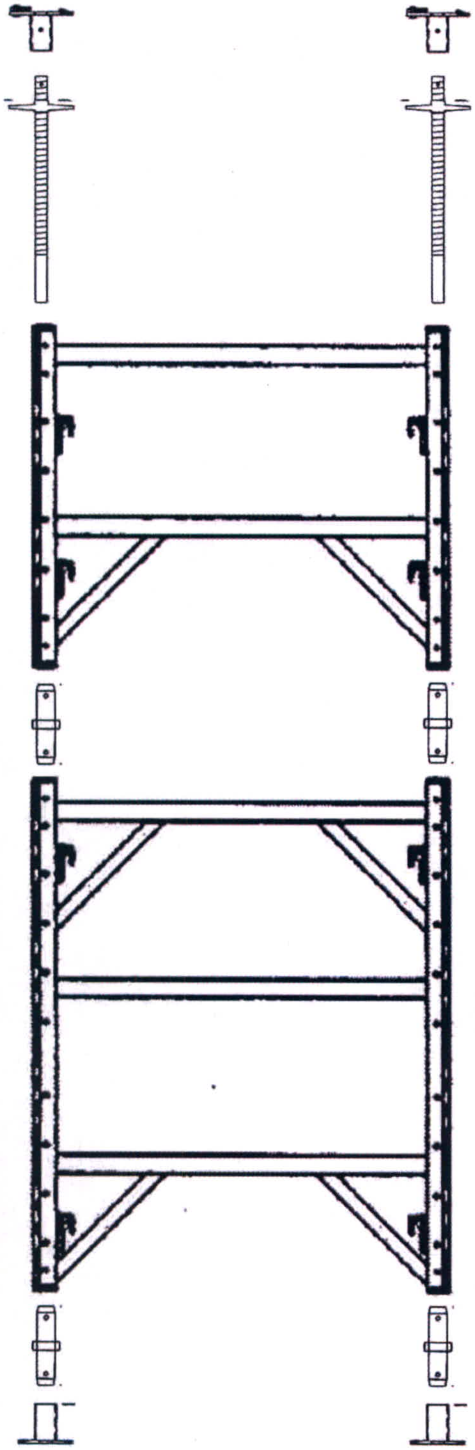
SH 41051
Broken piece of frame
Attached to bottom SH 41044
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41051



SH41051_1

SH41051_2

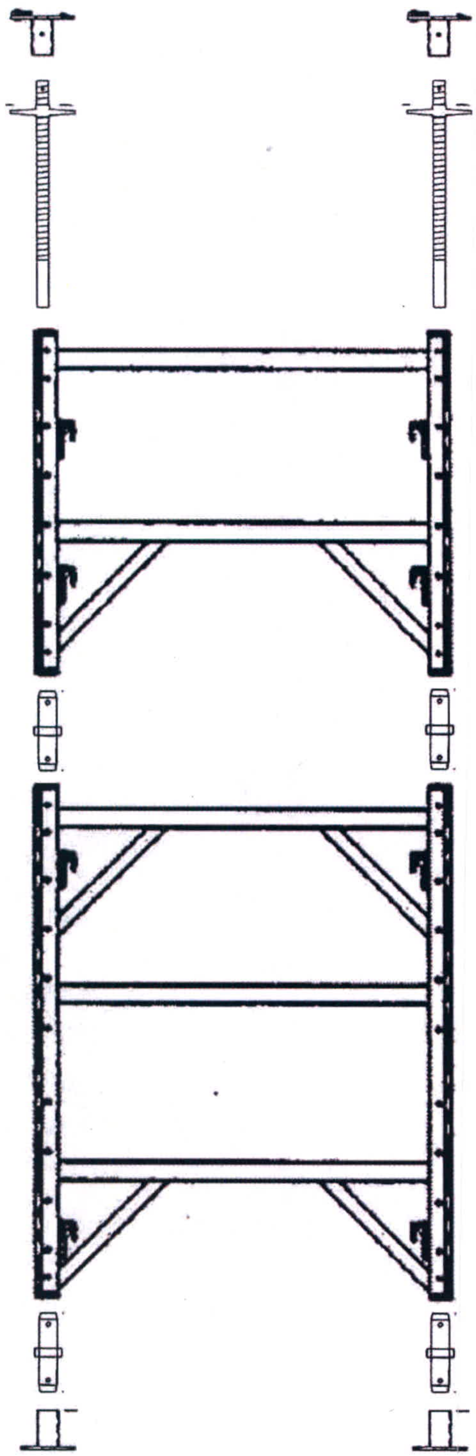


SH 41052
Brace
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41052

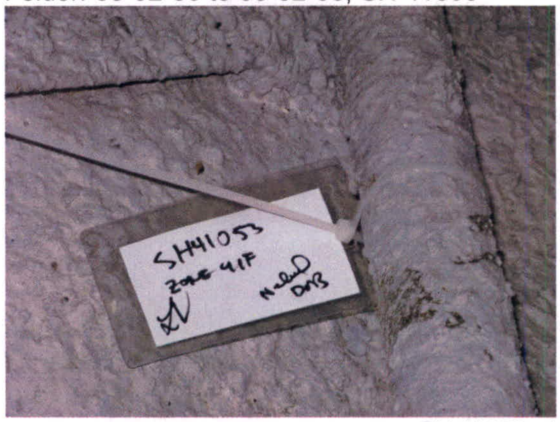
SH41052_1

SH41052_2



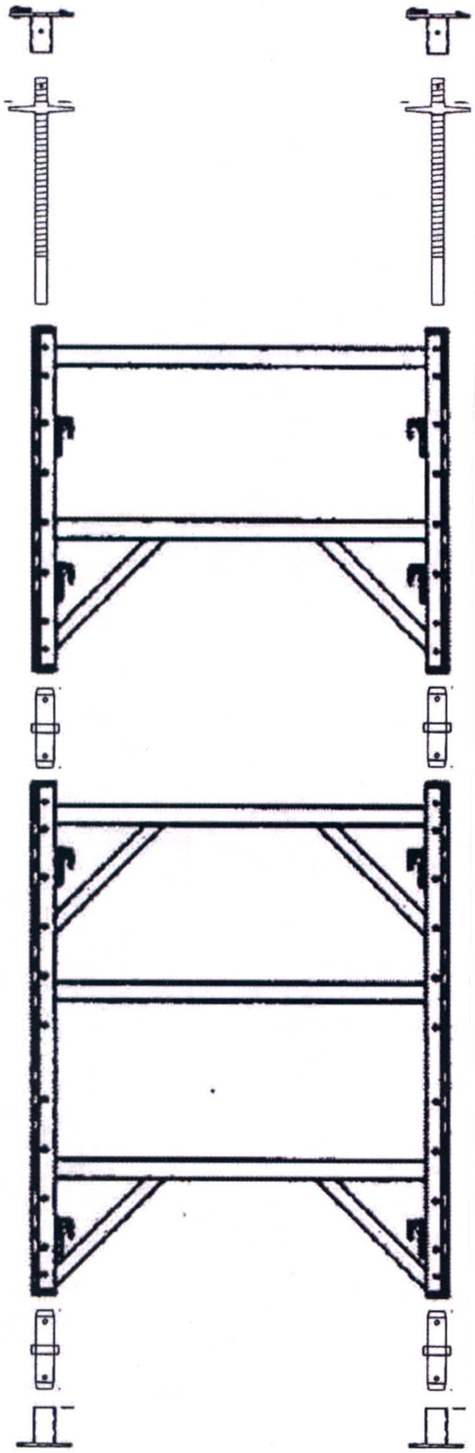
SH 41053
Screw Jack
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41053



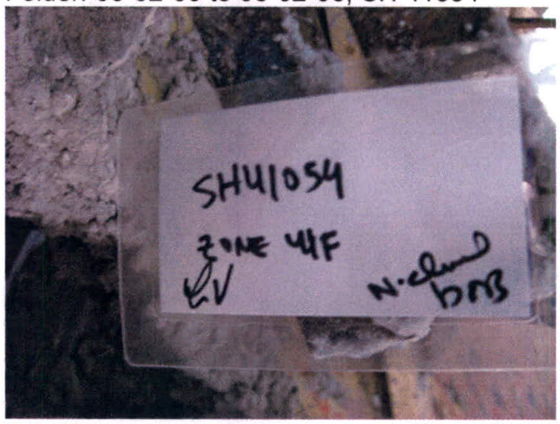
SH41053_1

SH41053_2



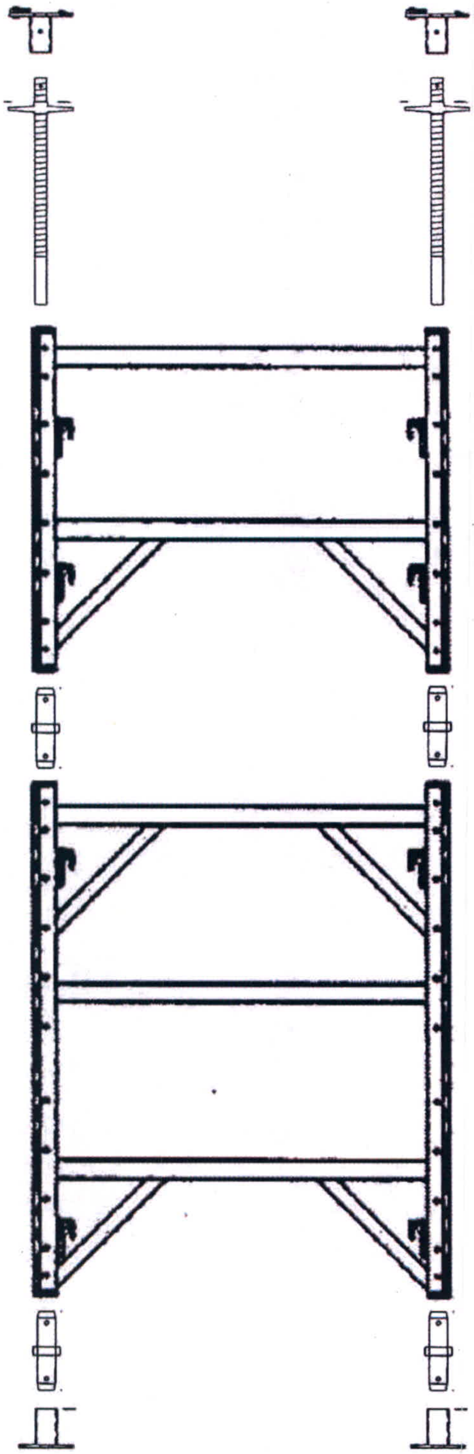
SH 41054
Brace
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41054



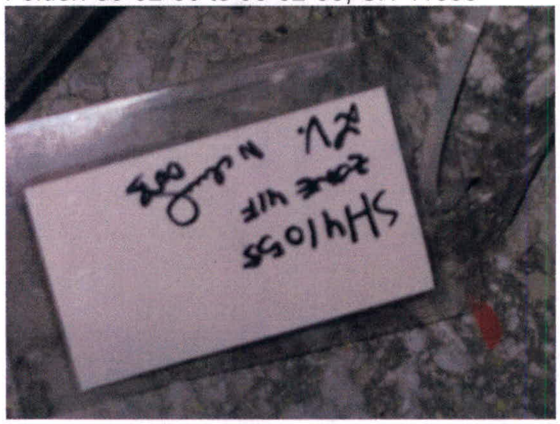
SH41054_1

SH41054_2



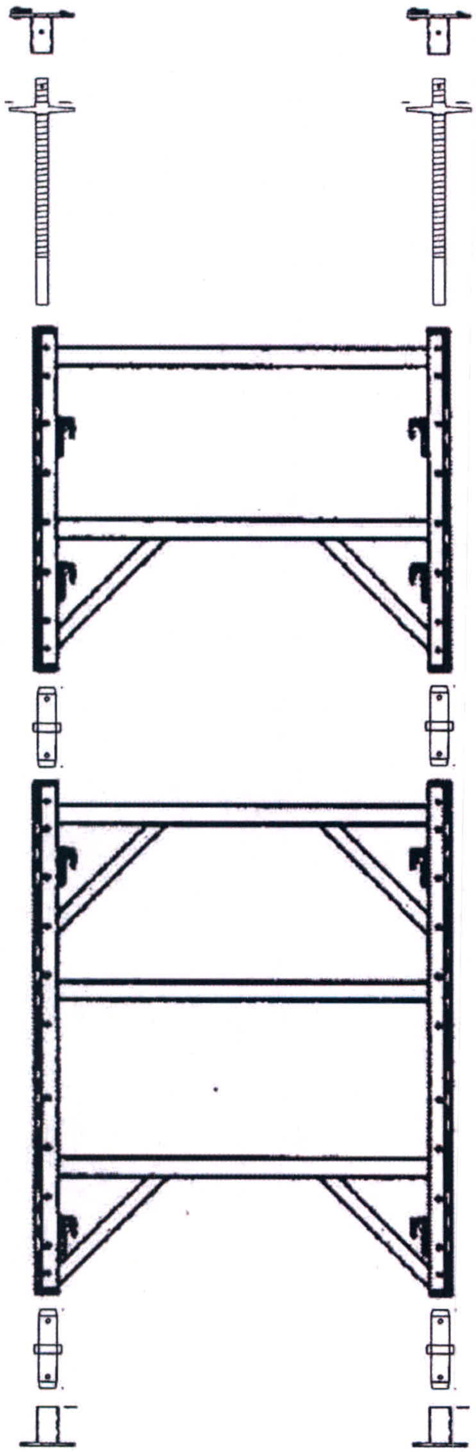
SH 41055
Bottom Plate
Zone 41F

Folder: 08-02-06 to 08-02-08; SH 41055



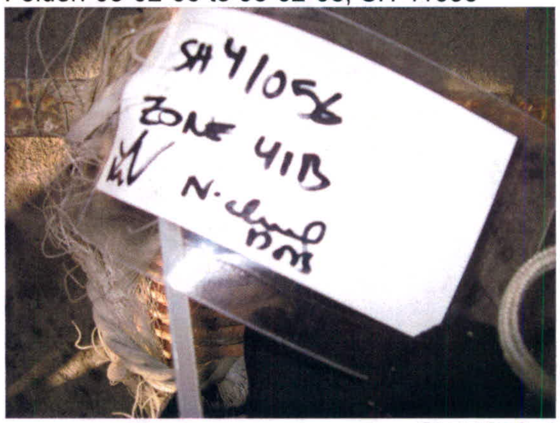
SH41055_2

SH41055_1



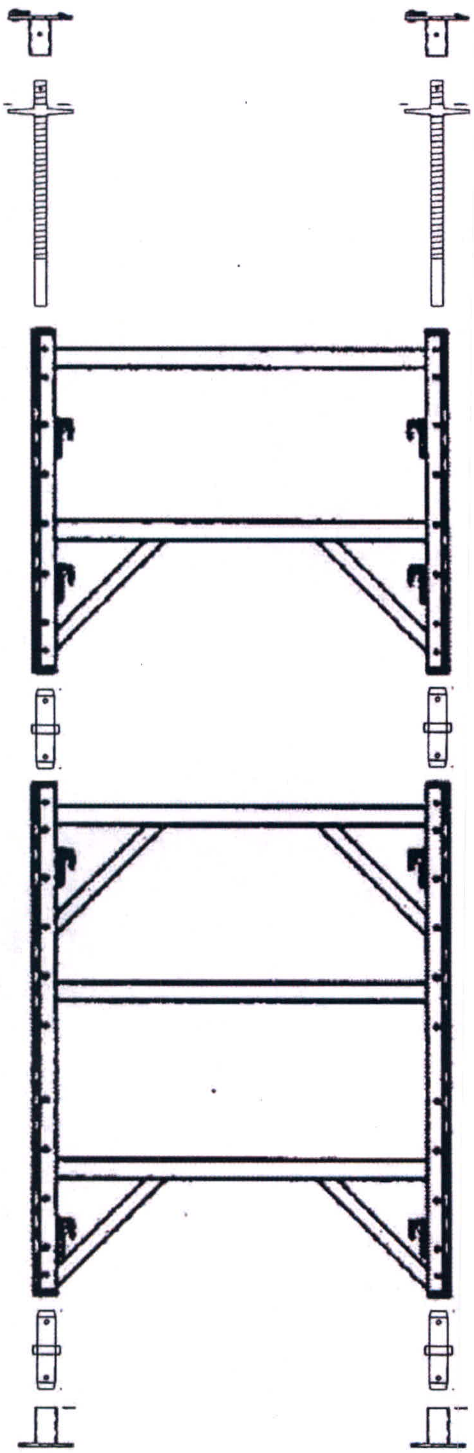
SH 41056
Top plate and Screw Jack
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41056



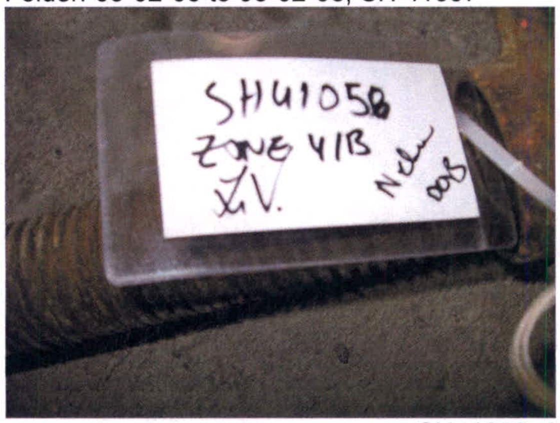
SH41056_1

SH41056_2



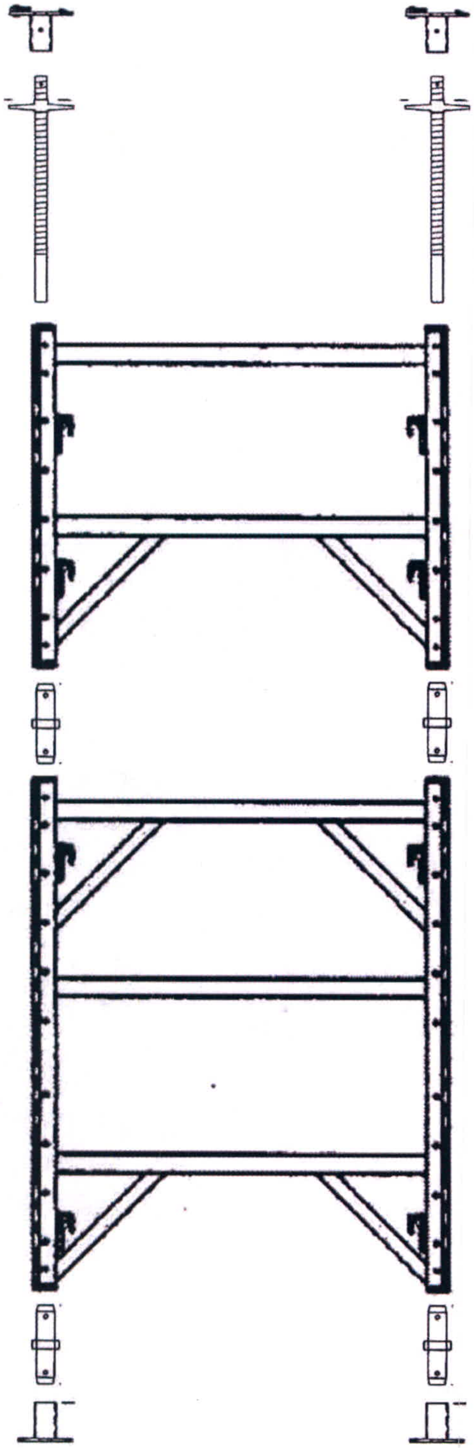
SH 41057
Top plate and Screw Jack
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41057



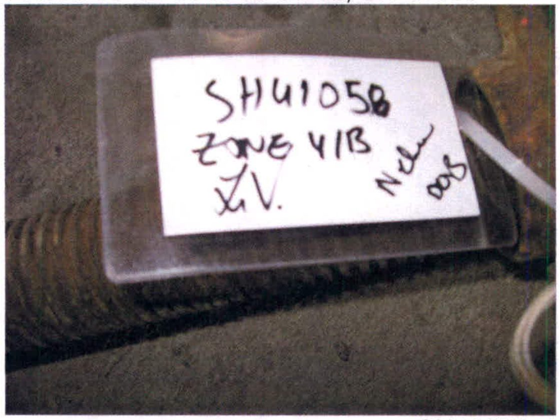
SH41058_1

SH41058_2



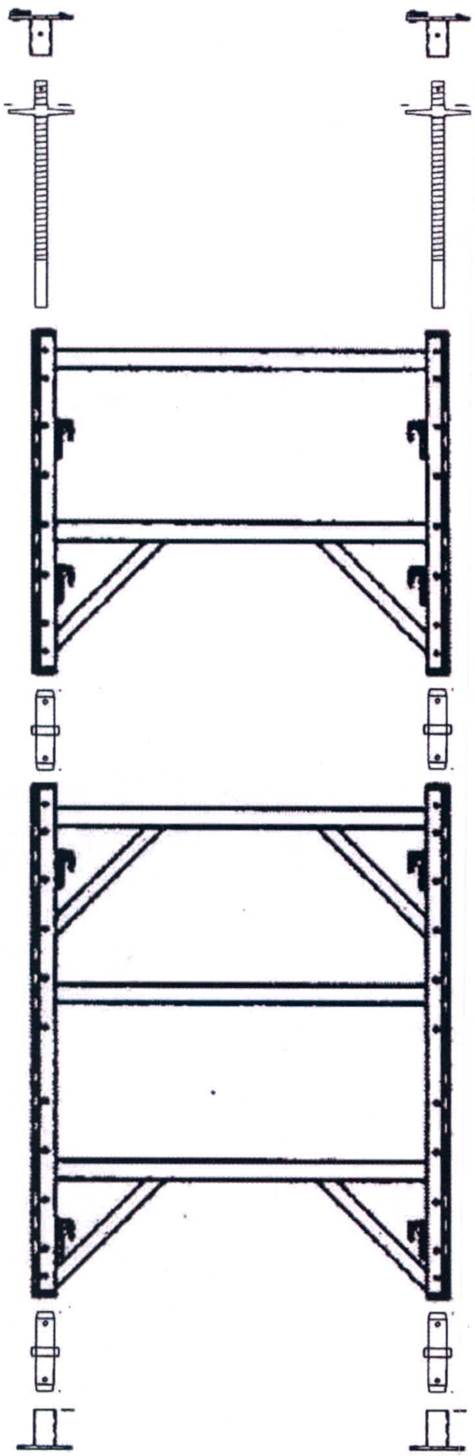
SH 41058
Top plate and Screw Jack
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41058



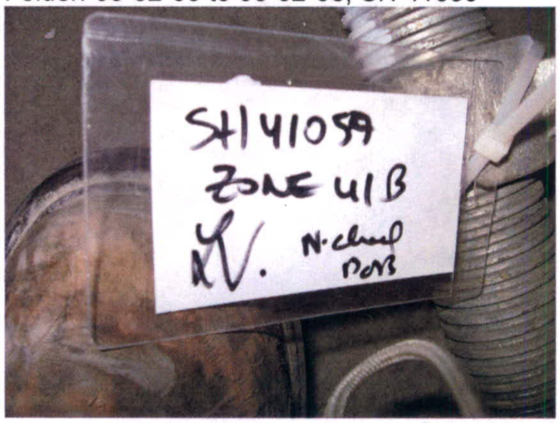
SH41058_1


SH41058_2

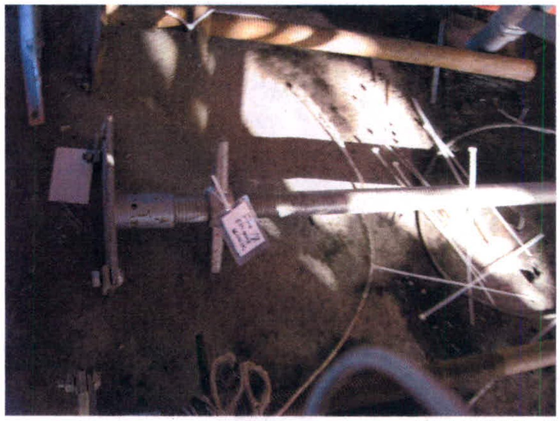


SH 41059
Top Plate and Screw Jack
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41059



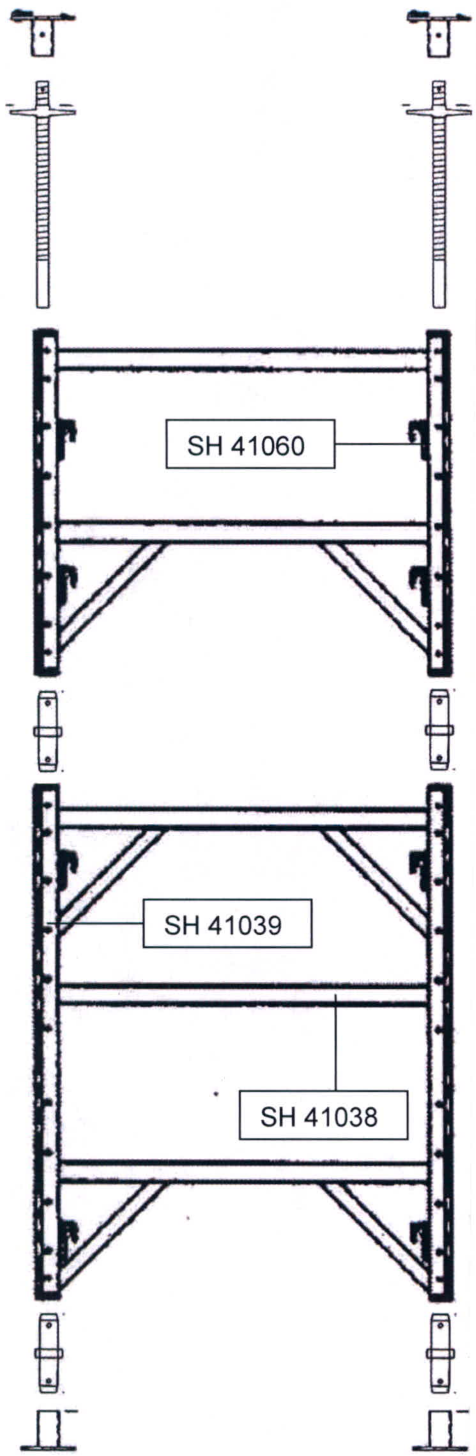
SH41059_1

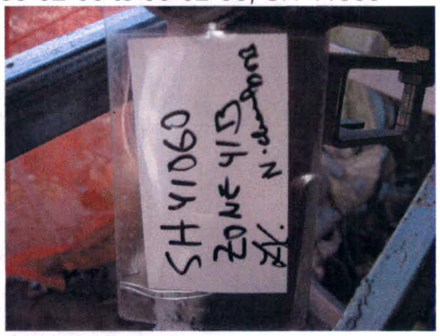


SH41059_2

SH 41060
Top frame of SH 41038
Zone 41B

Folder: 08-02-06 to 08-02-08; SH 41060

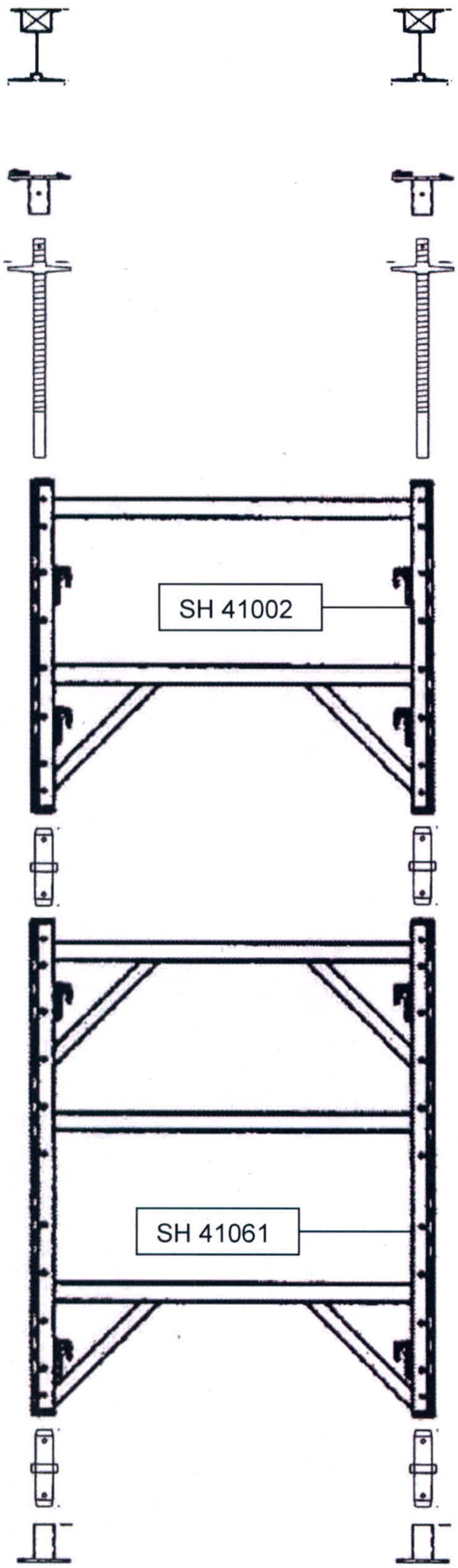




SH41060_1

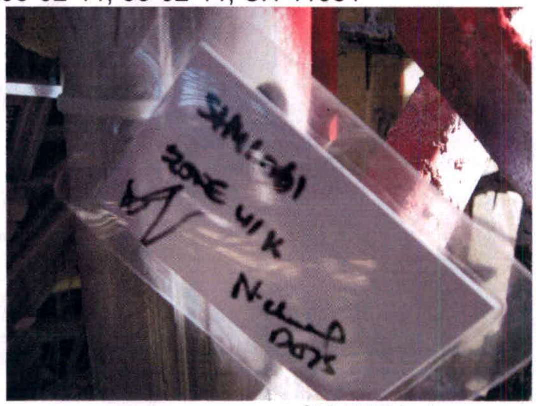

SH41060_2


SH41060_3

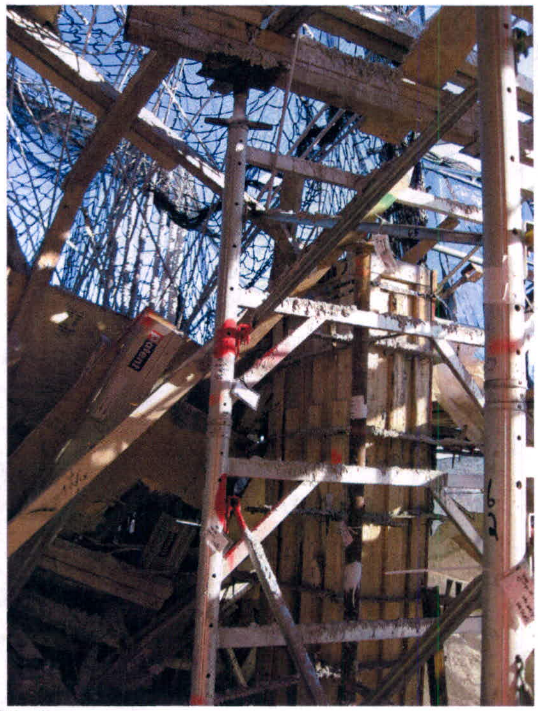


SH 41061
Bottom of SH 41002
Zone 41K

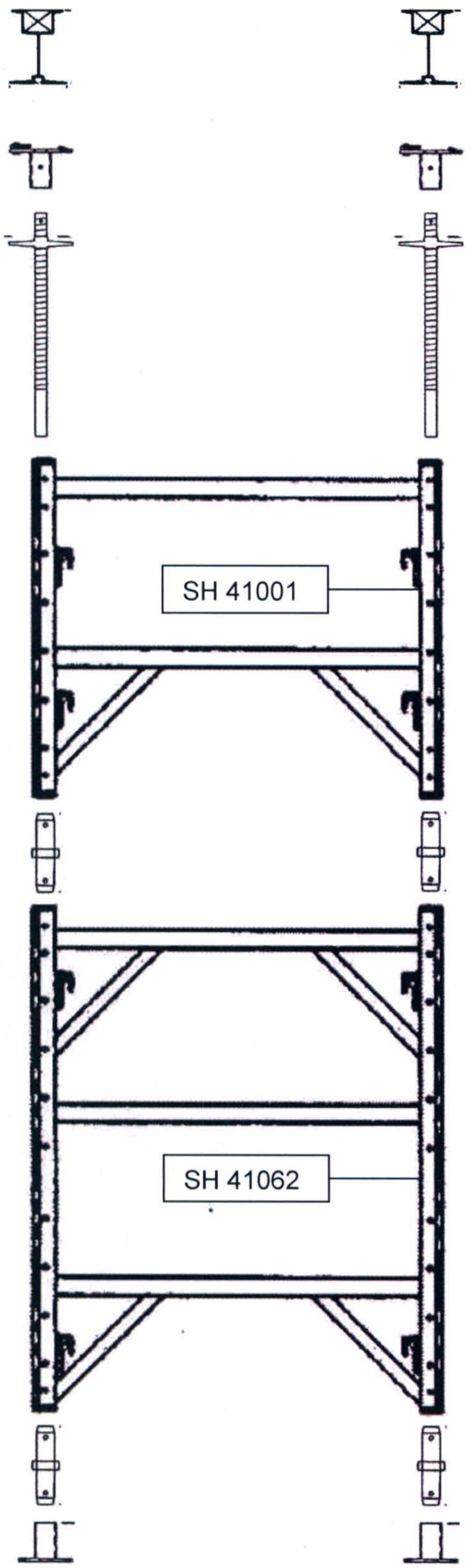
Folder: 08-02-11; 08-02-11; SH 41061



SH41061_1

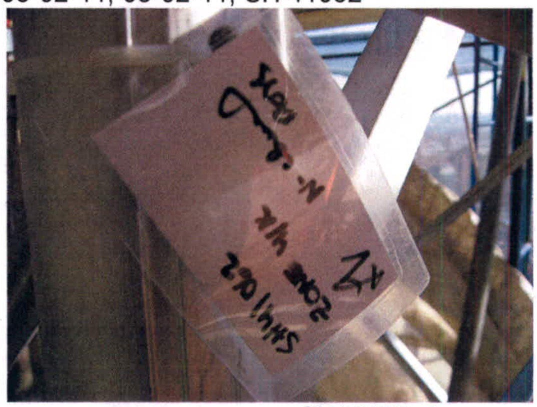


SH41061_2

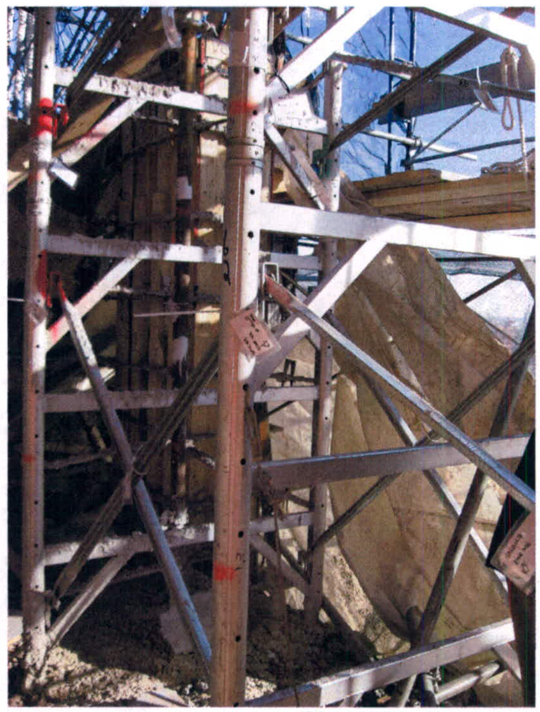


SH 41062
Bottom of SH 41001
Zone 41K

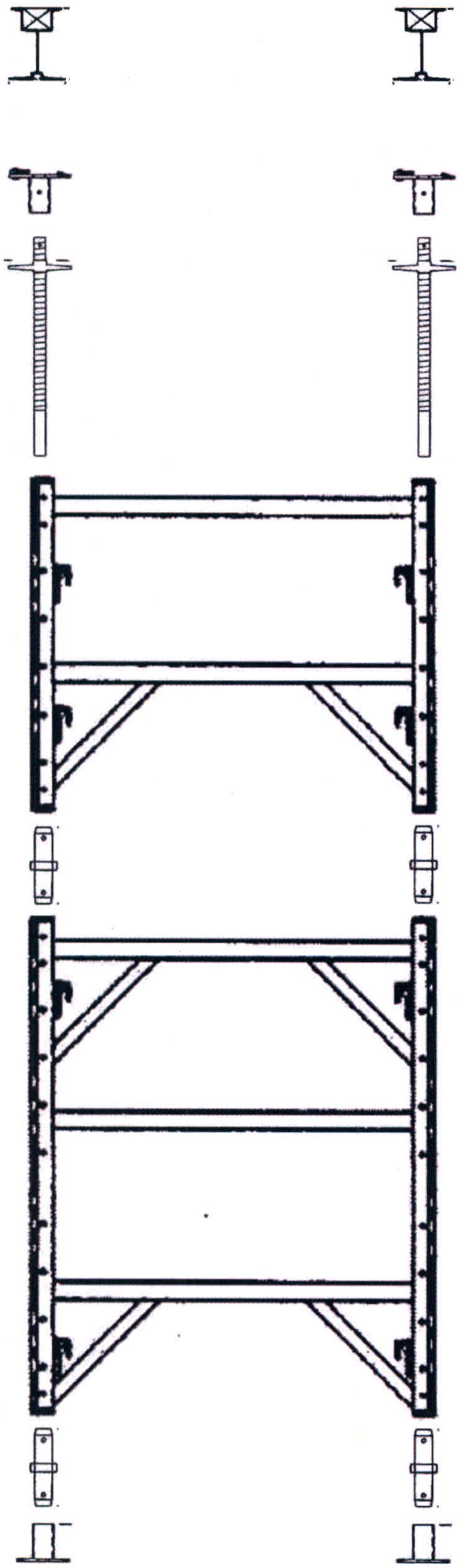
Folder: 08-02-11; 08-02-11; SH 41062



SH41062_2

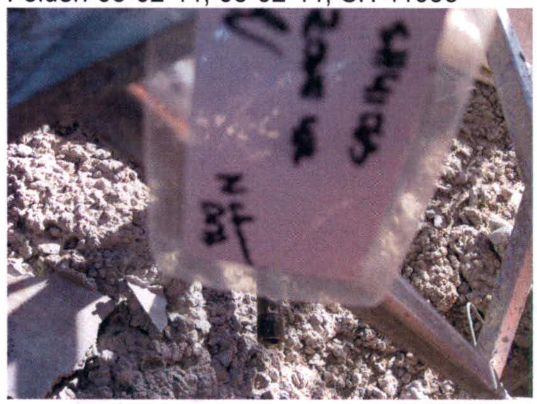


SH41062_3



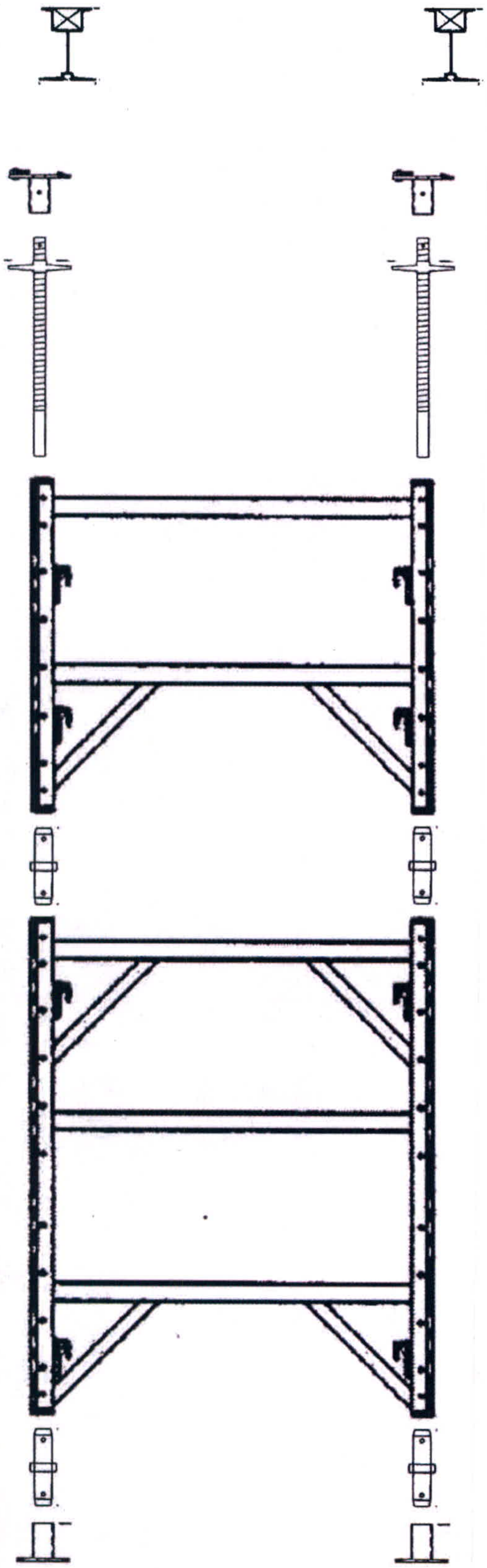
SH 41063
Brace
Connects SH 41062 to SH 41061
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41063



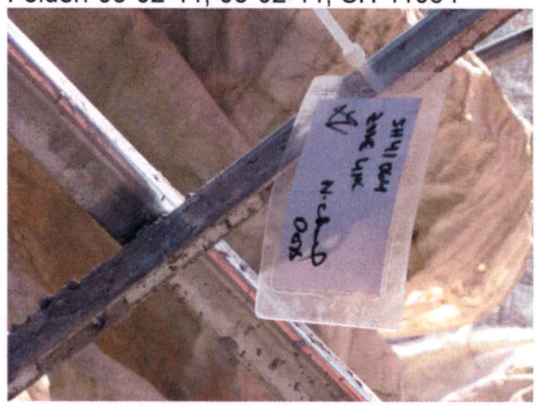
SH41063_1

SH41063_2



SH 41064
Brace
Connects SH 41062 to SH 41061
Zone 41K

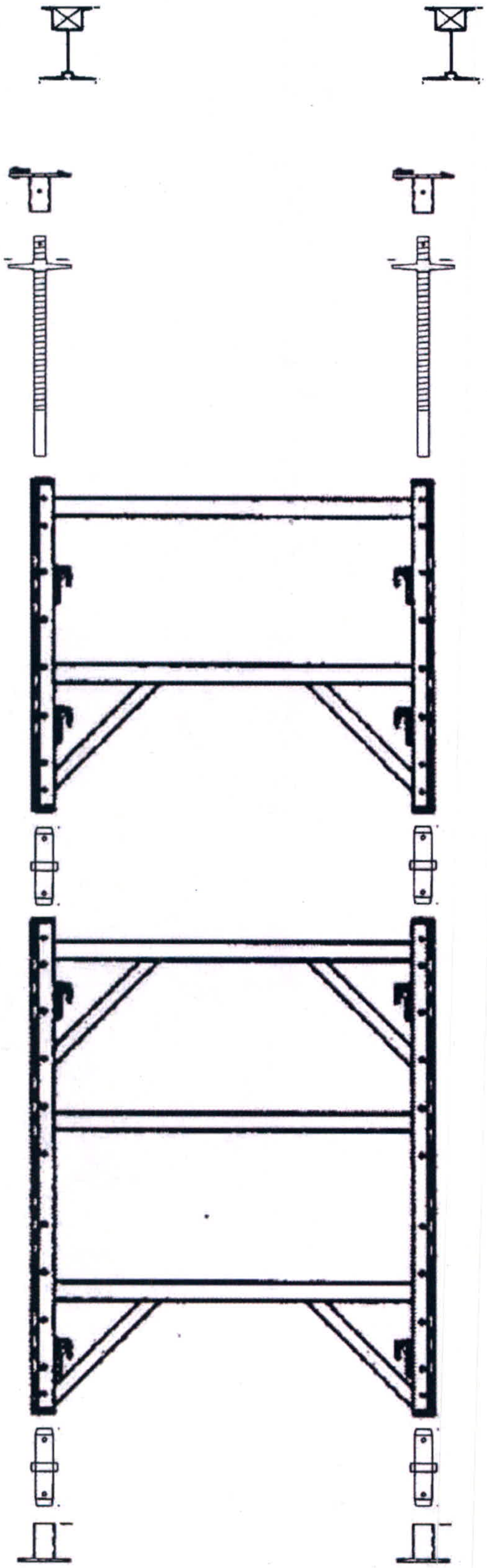
Folder: 08-02-11; 08-02-11; SH 41064



SH41064_1

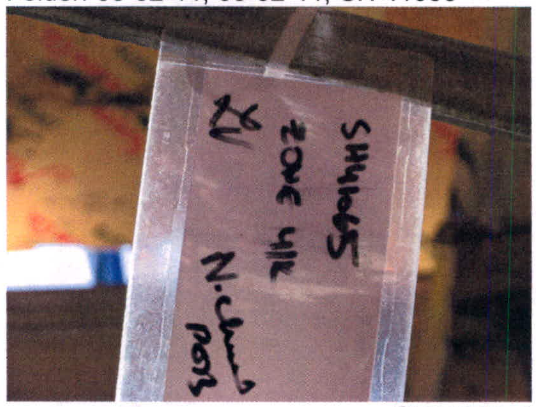
SH41064_2

SH41064_3



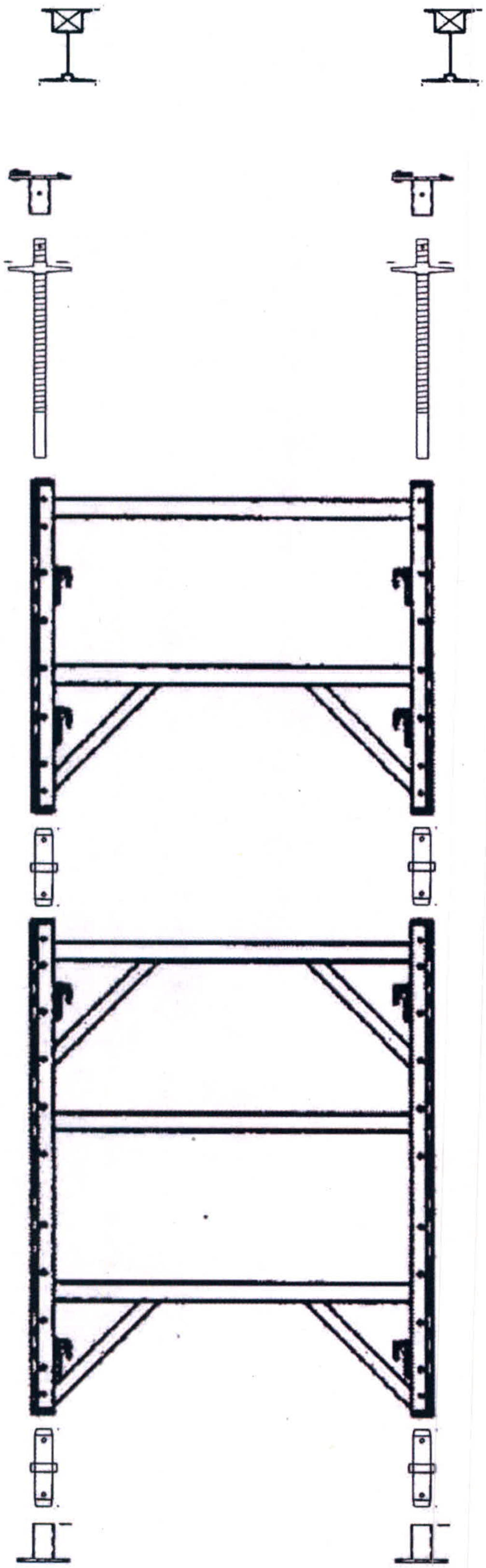
SH 41065
Brace
Connects SH 41001 to SH 41002
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41065



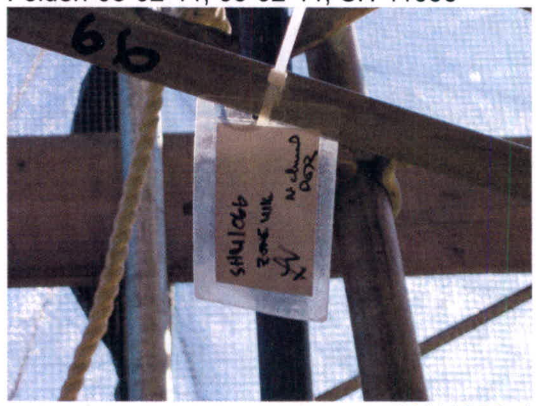
SH41065_1

SH41065_2



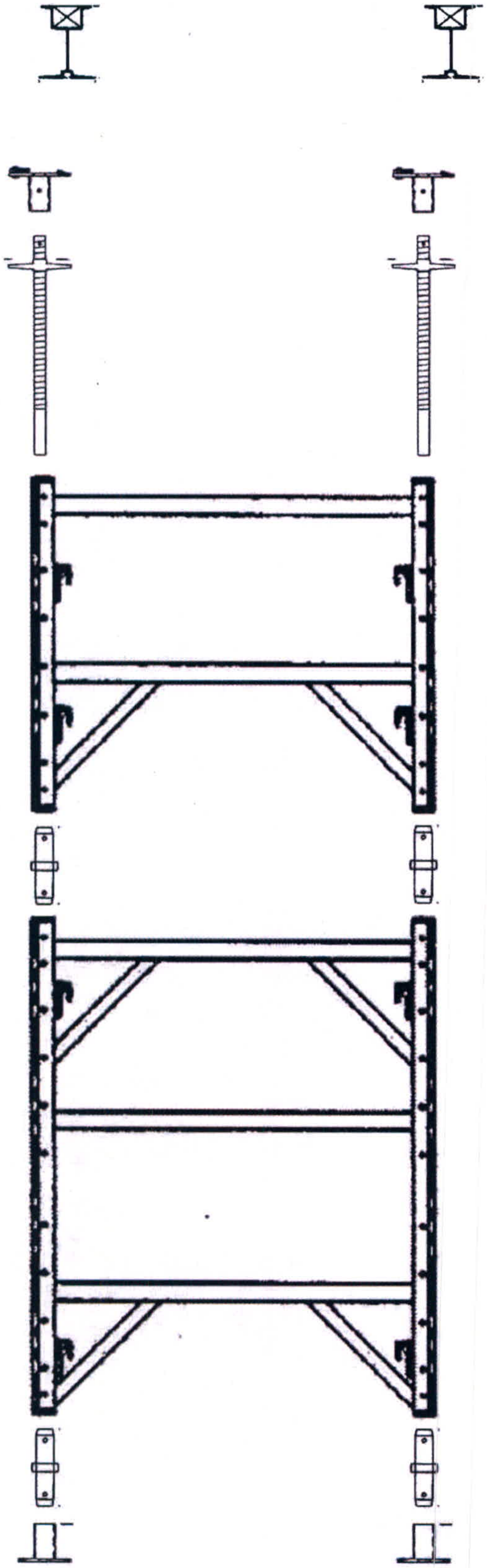
SH 41066
Brace
Connects SH 41001 to SH 41002
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41066



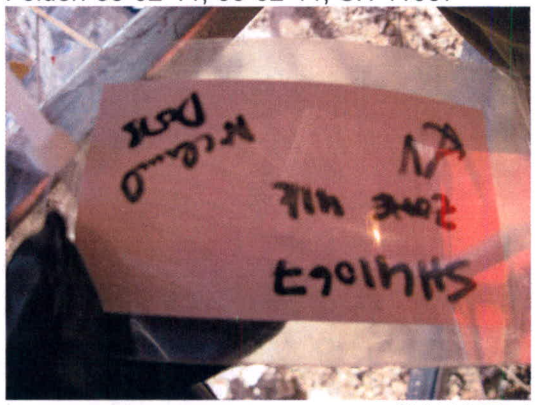
SH41066_1

SH41066_2



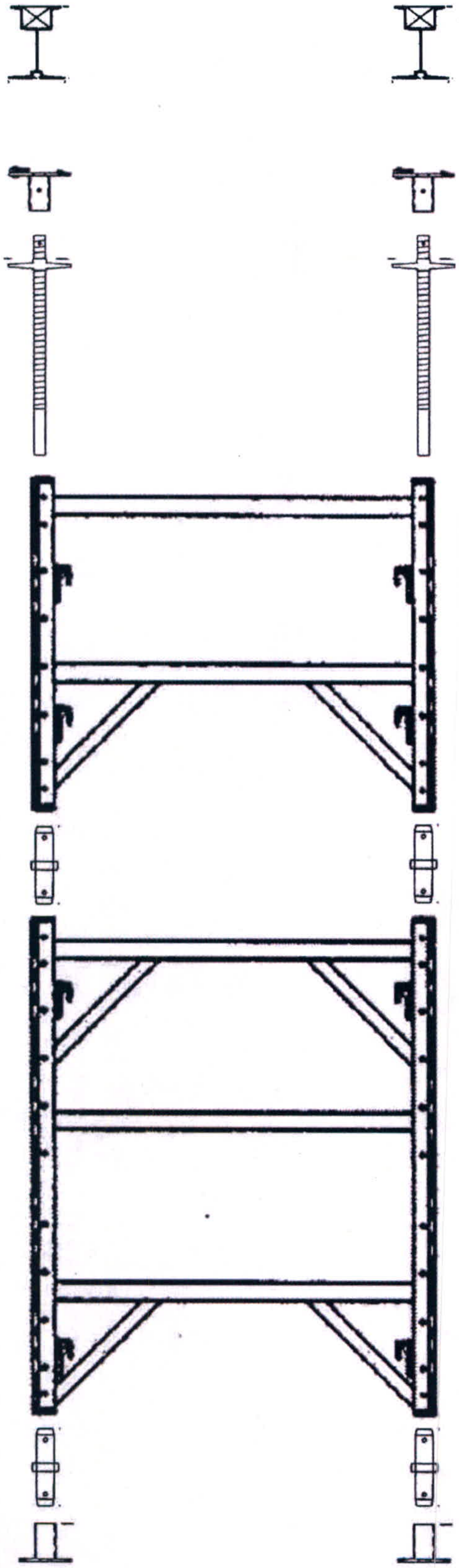
SH 41067
Brace connect to bottom SH 41062
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41067



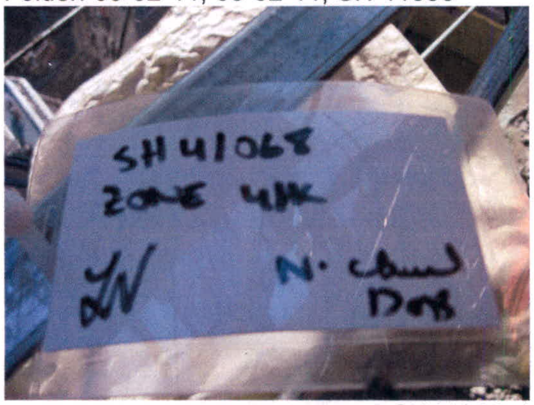
SH41067_1

SH41067_2



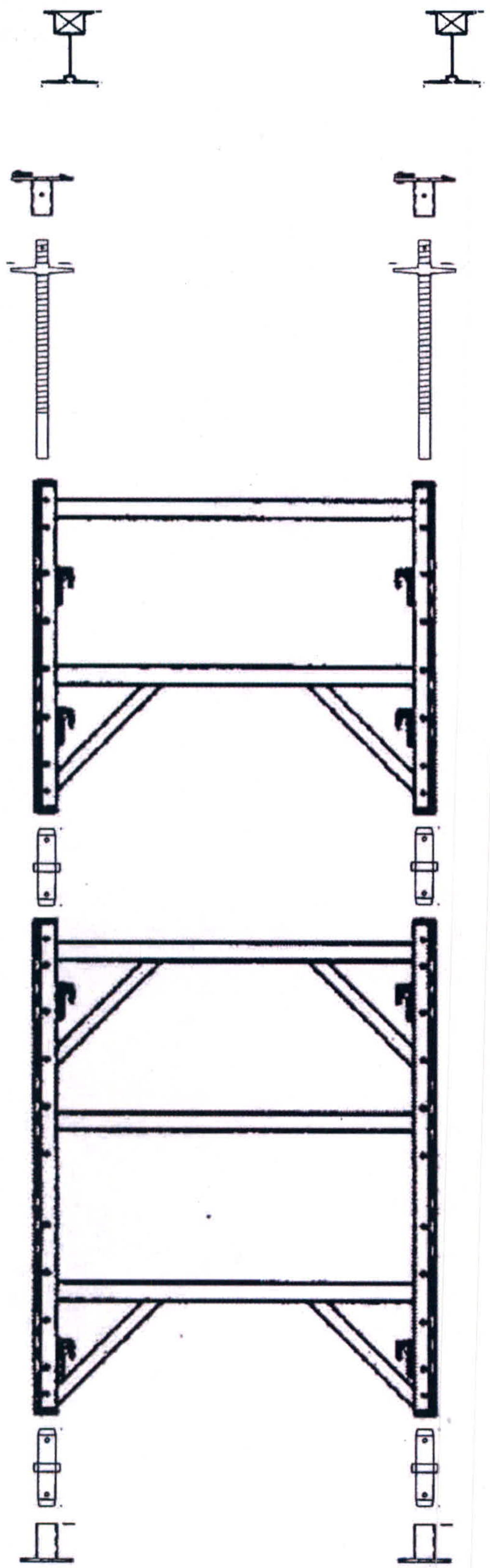
SH 41068
Brace connecting SH 41062 to SH 41083

Folder: 08-02-11; 08-02-11; SH 41068



SH41068_1

SH41067_2

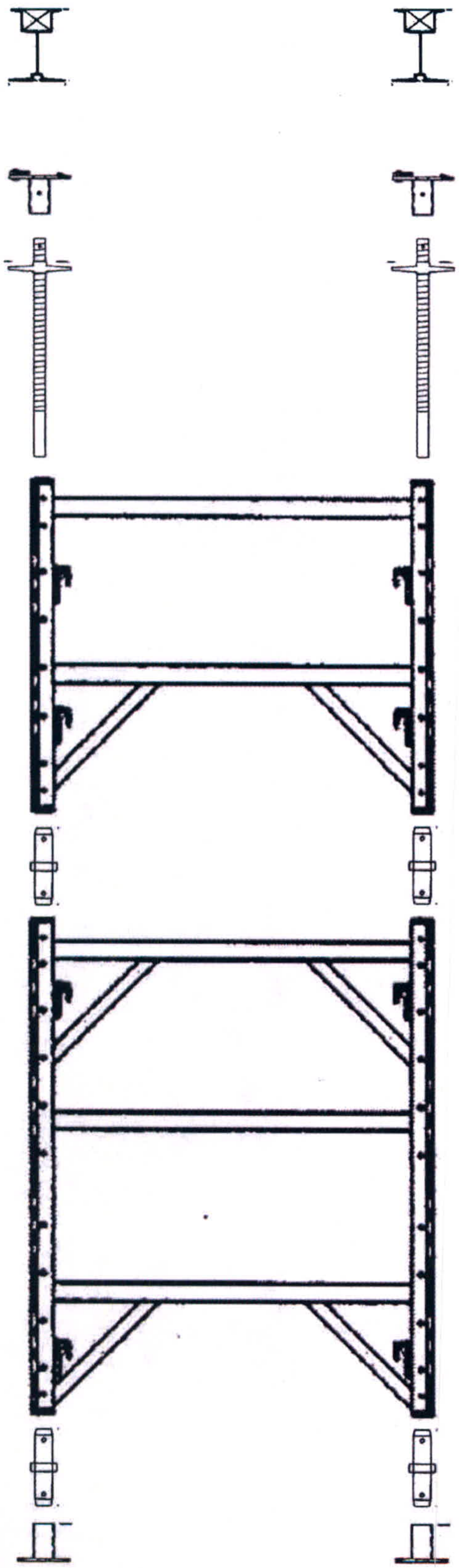


SH 41069
Top Brace
Connect SH 41001 to SH 41084
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41069

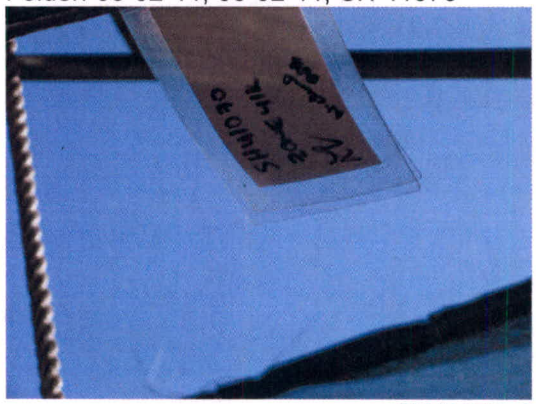
SH41069_1

SH41069_2



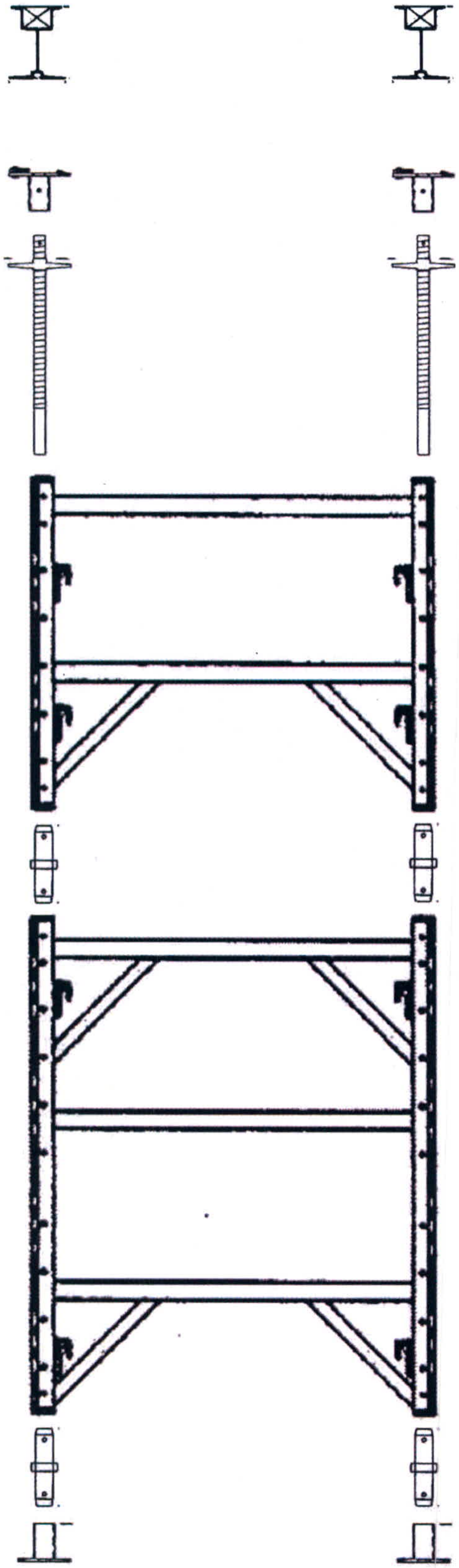
SH 41070
Brace
Connect SH 41001 to SH 41084
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41070



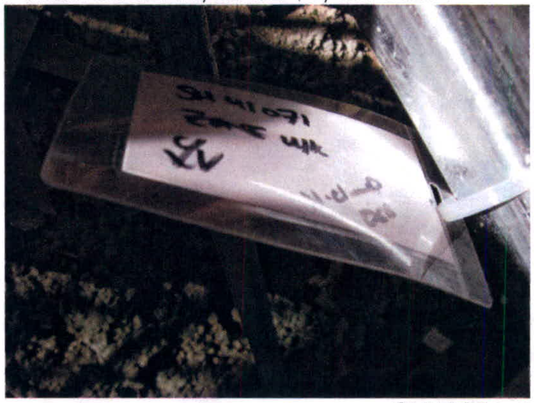
SH41070_1

SH41070_2



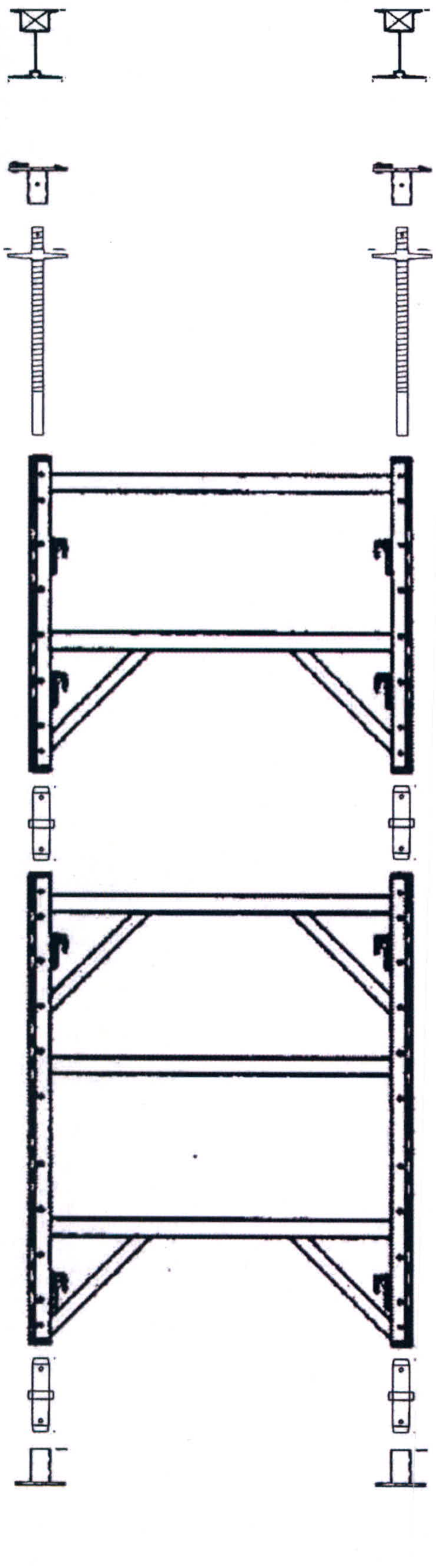
SH 41071
Brace
Connect SH 41074 to SH 41075
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41071



SH41071_1

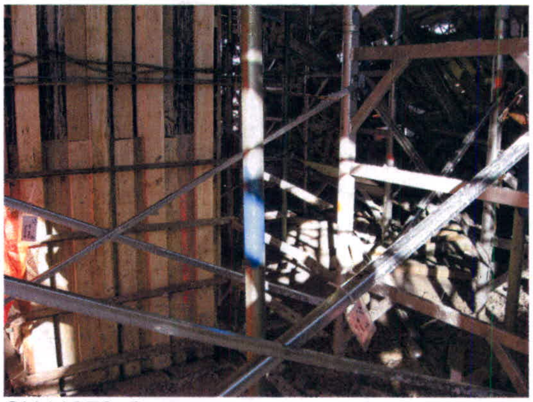
SH41071_2



SH 41072
Brace connects SH 41075
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41072

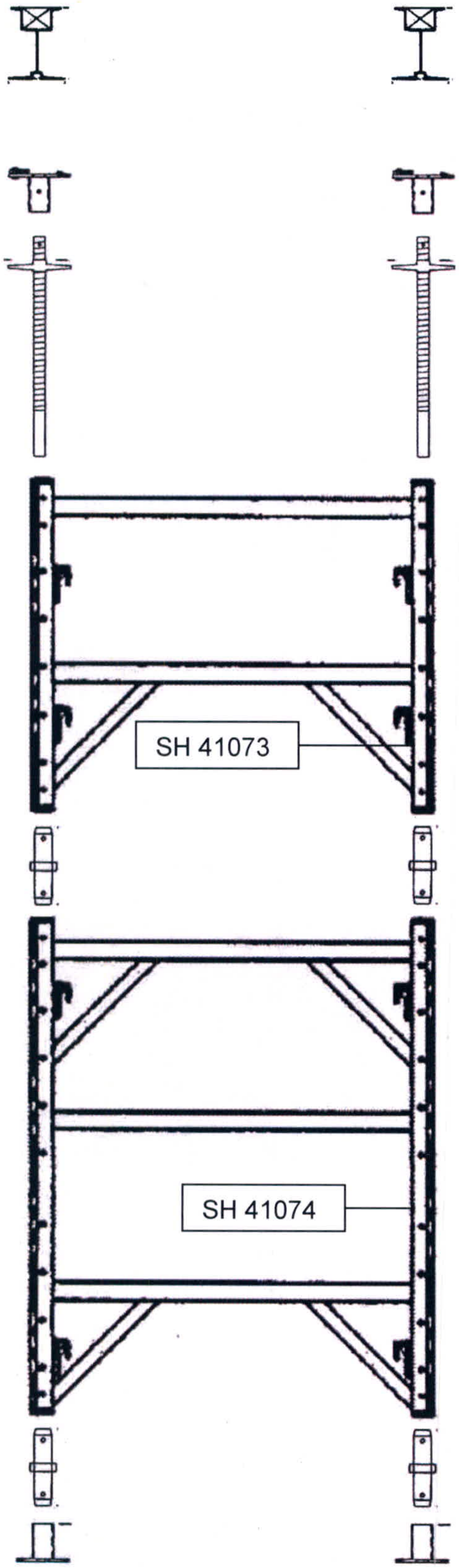
SH41072_1



SH41072_2

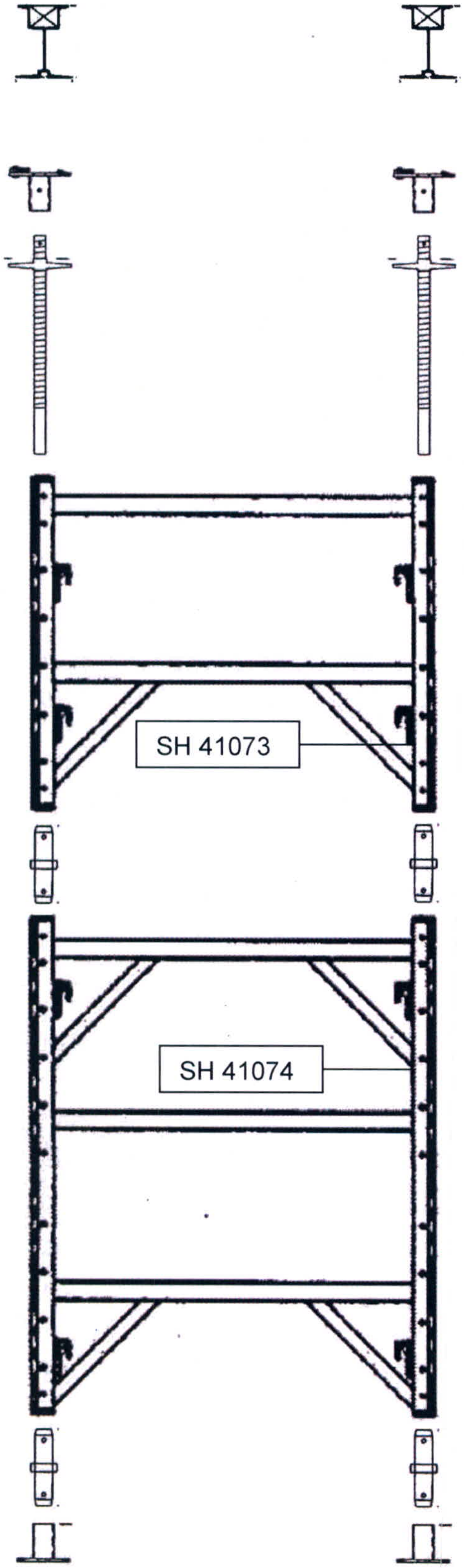
SH 41073
Top of SH 41074
Connects to top brace SH 41081
Connects to bottom brace SH 41071
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41073



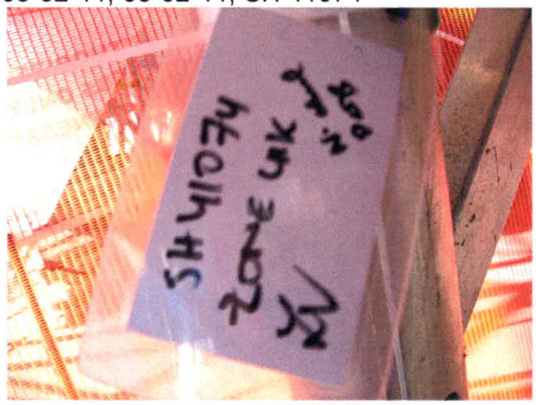

SH41073_1


SH41073_2



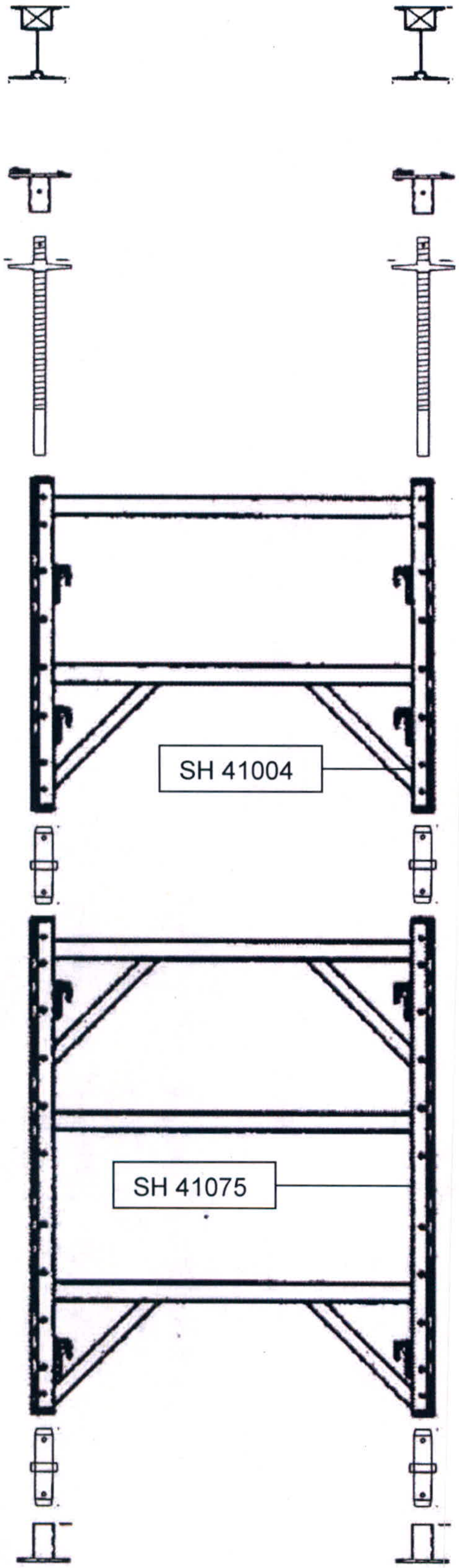
SH 41074
Bottom Frame of SH 41073
Connects to SH 41071 and SH 41072
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41074



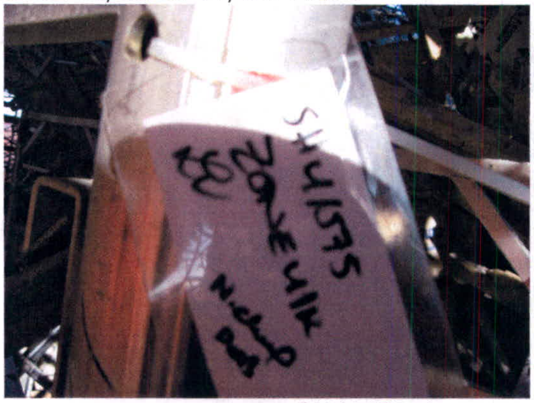
SH41074_1

SH41074_2

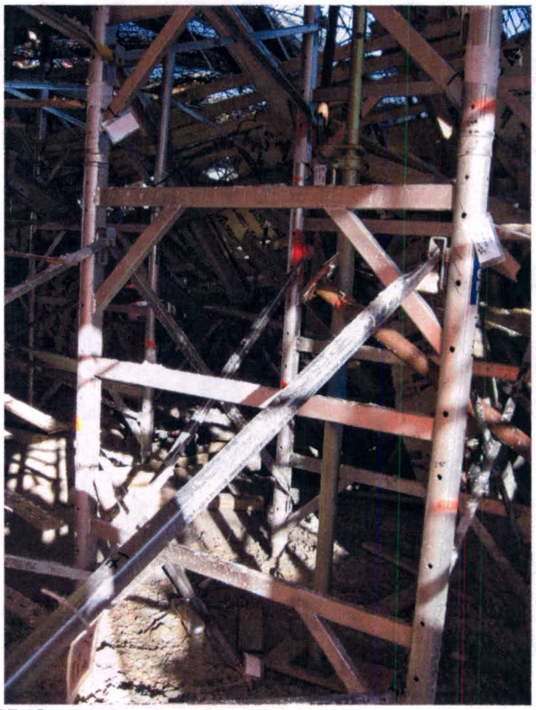


SH 41075
Bottom Frame of SH 41004
Connects to braces SH 41071 and SH 41072
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41075



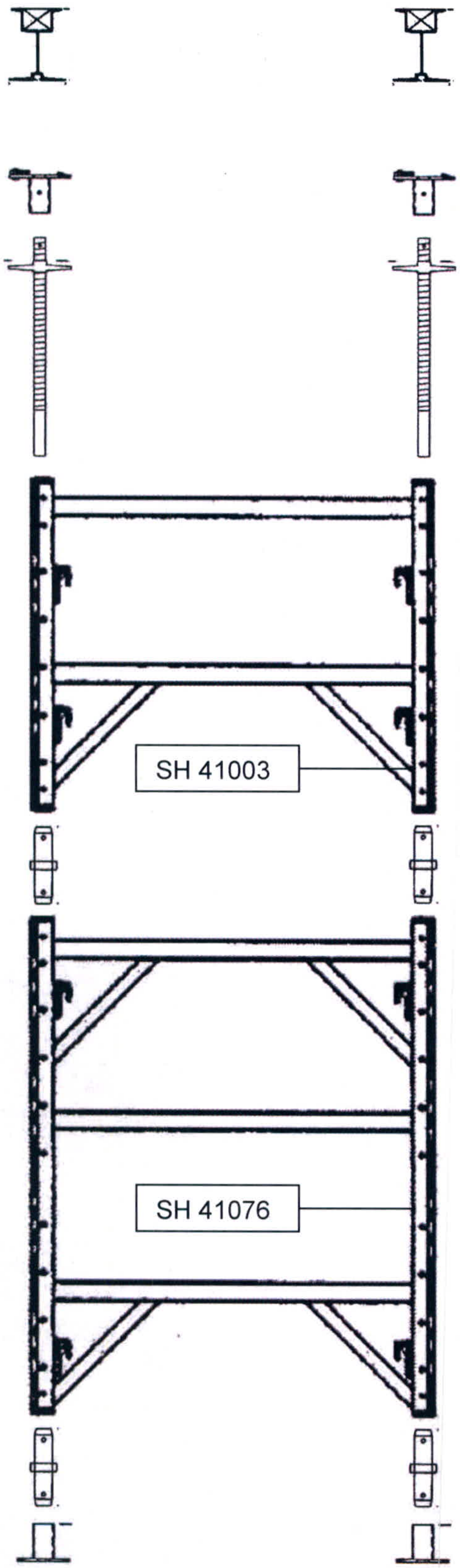
SH41075_1

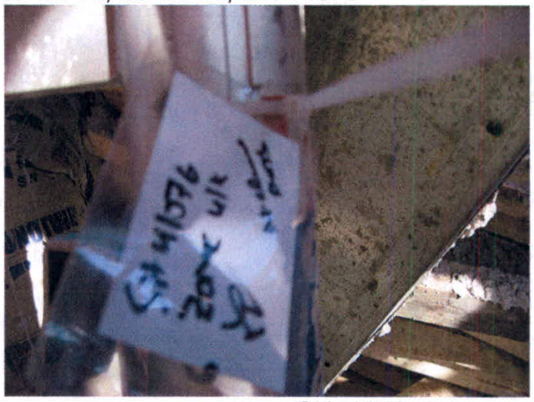


SH41075_2

SH 41076
Bottom Frame of SH 41003
Zone 41K

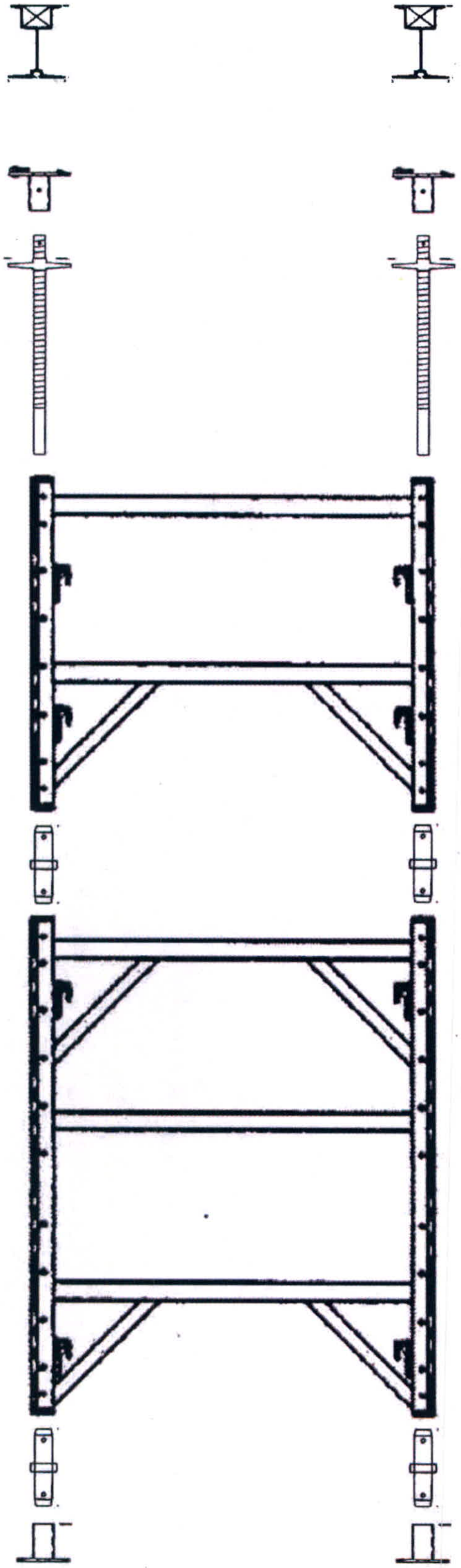
Folder: 08-02-11; 08-02-11; SH 41076





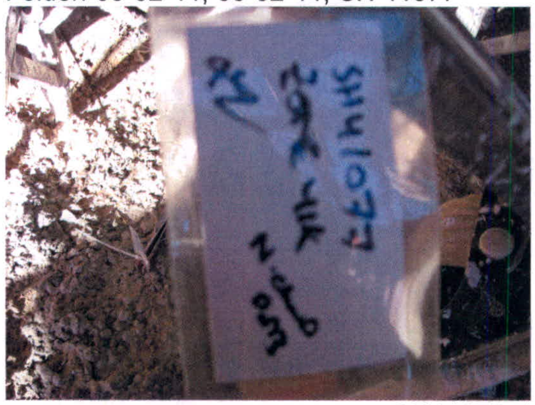
SH41076_1

SH41076_2



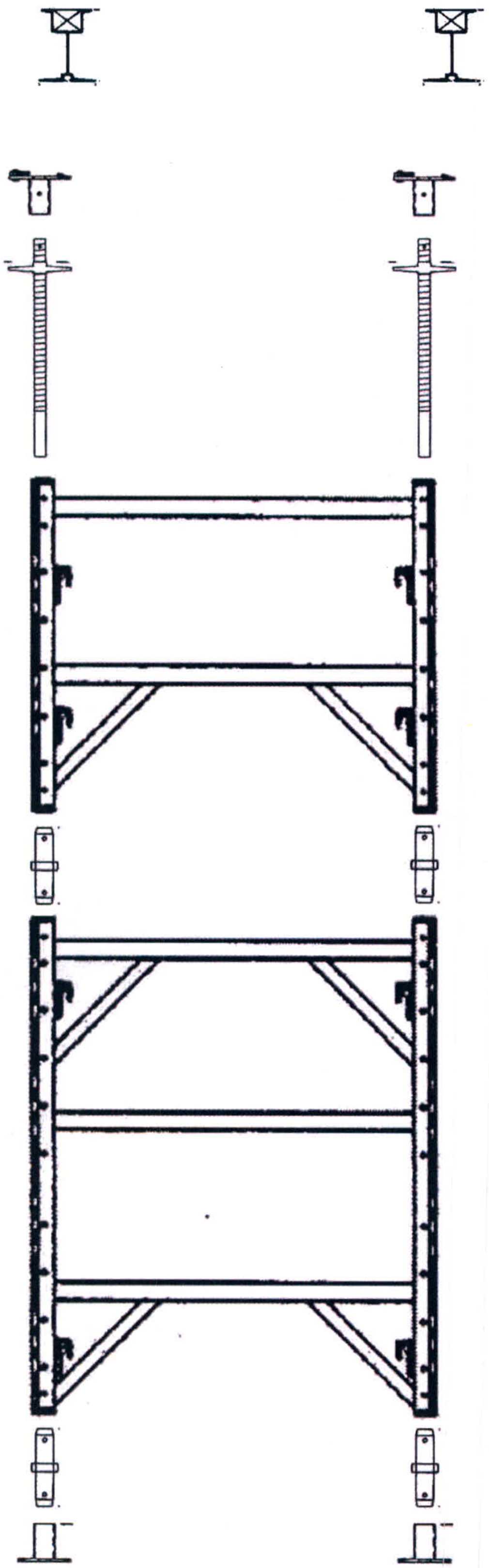
SH 41077
Brace connects SH 41075 to SH 41076
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41077



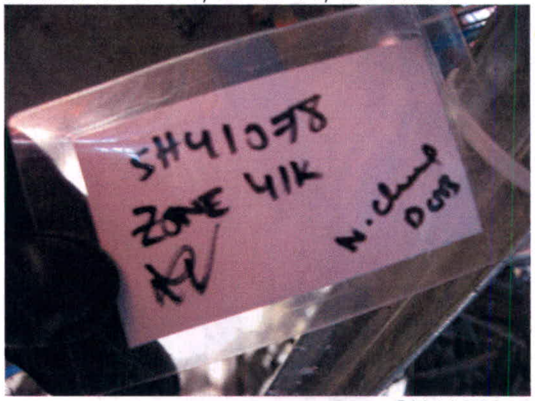
SH41077_1

SH41077_2



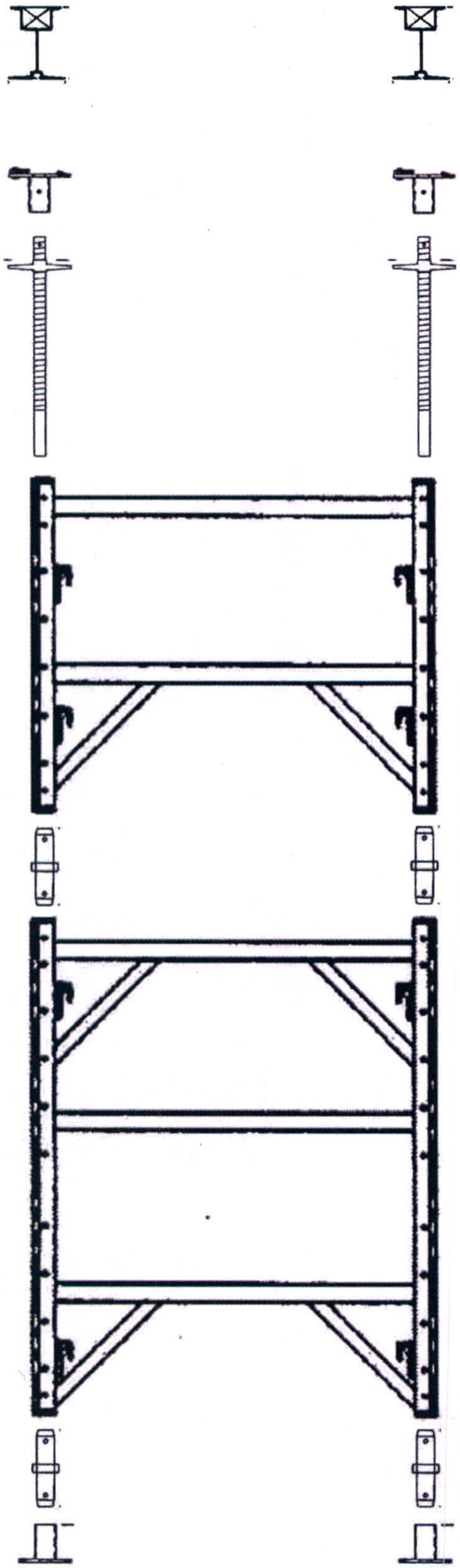
SH 41078
Brace connects SH 410745 to SH 41076
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41078



SH41078_1

SH41078_2

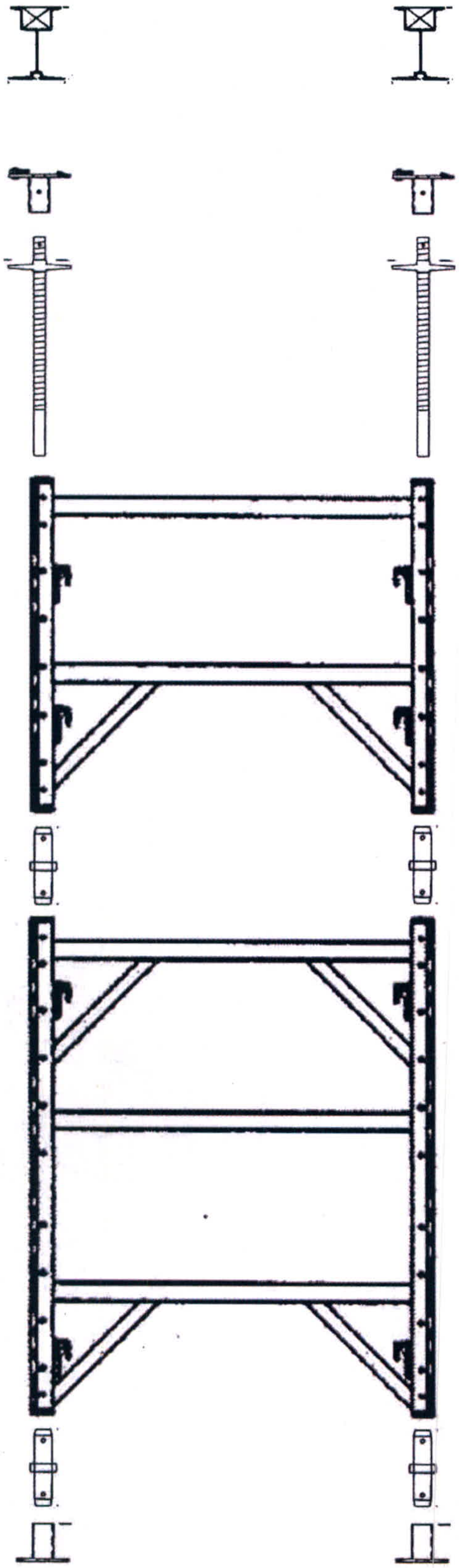


SH 41079
Top brace
Connects SH 41004 to SH 41003
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41079

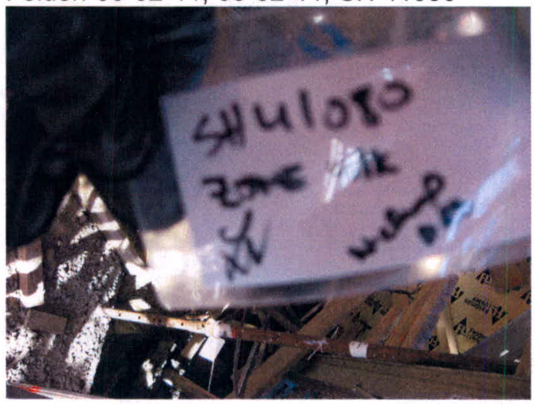
SH41079_1

SH41079_2



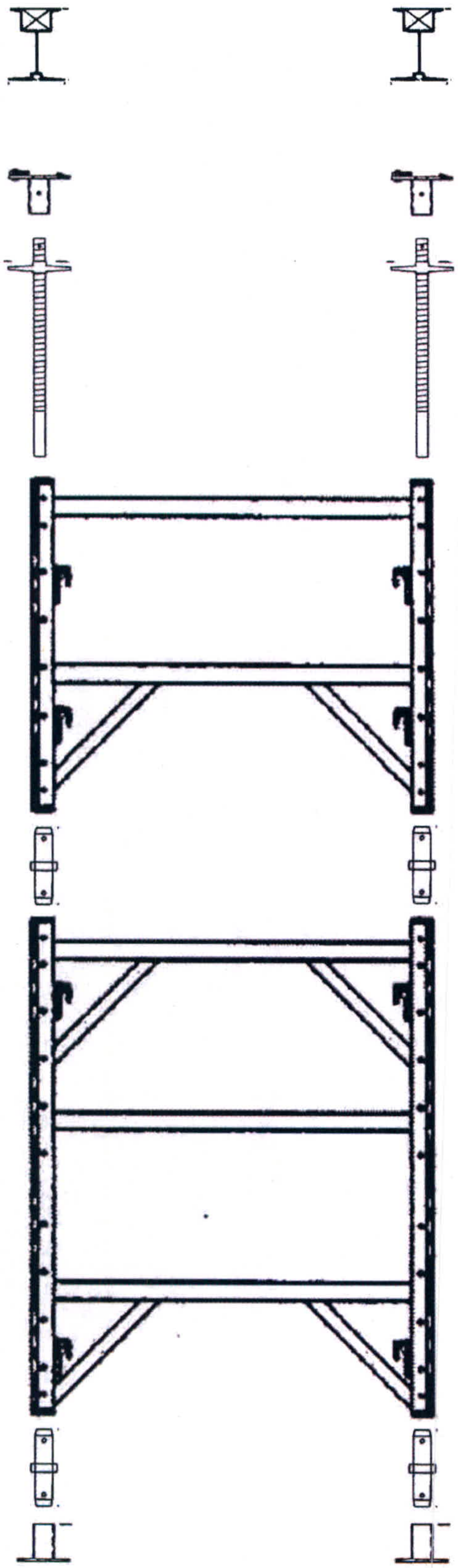
SH 41080
Brace connects SH 41004 to SH 41003
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41080



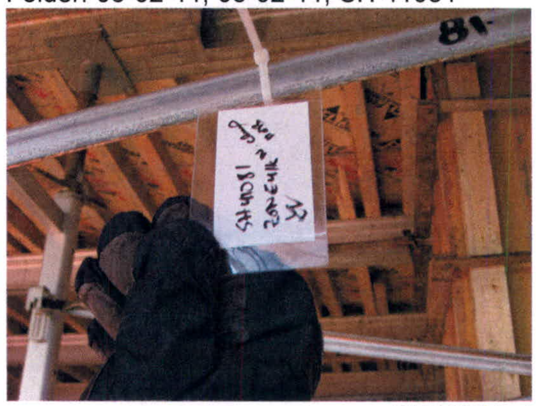
SH41080_1

SH41080_2



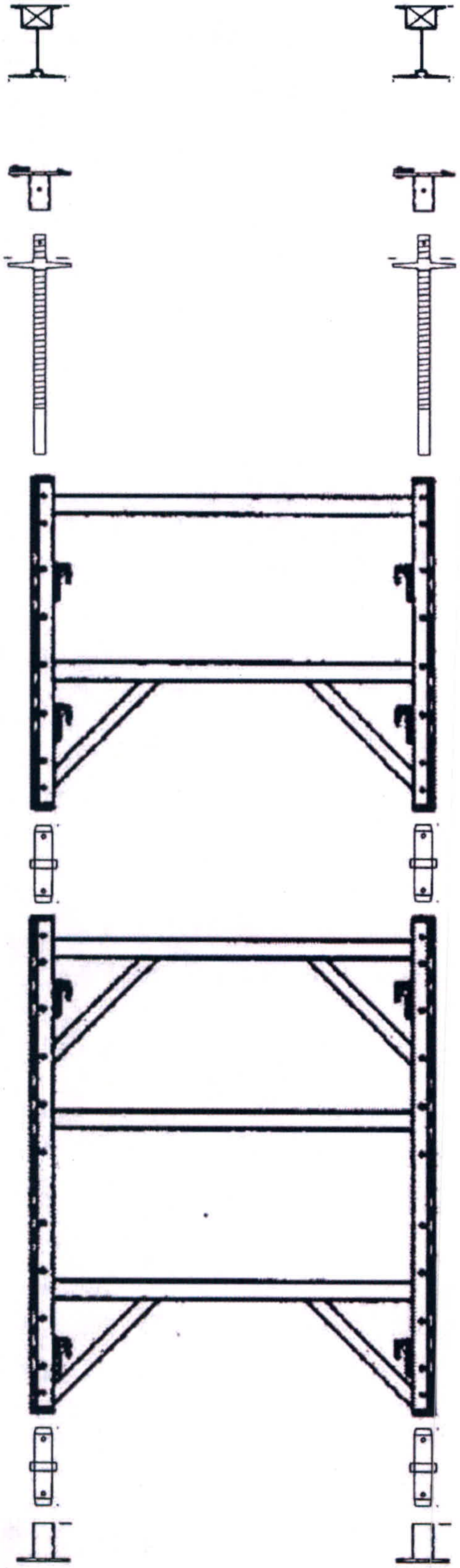
SH 41081
Top brace
Connects SH 41073 and SH 41004
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41081



SH41081_1

SH41081_2



SH 41082
Brace connects SH 41073 and SH 41004
Zone 41K

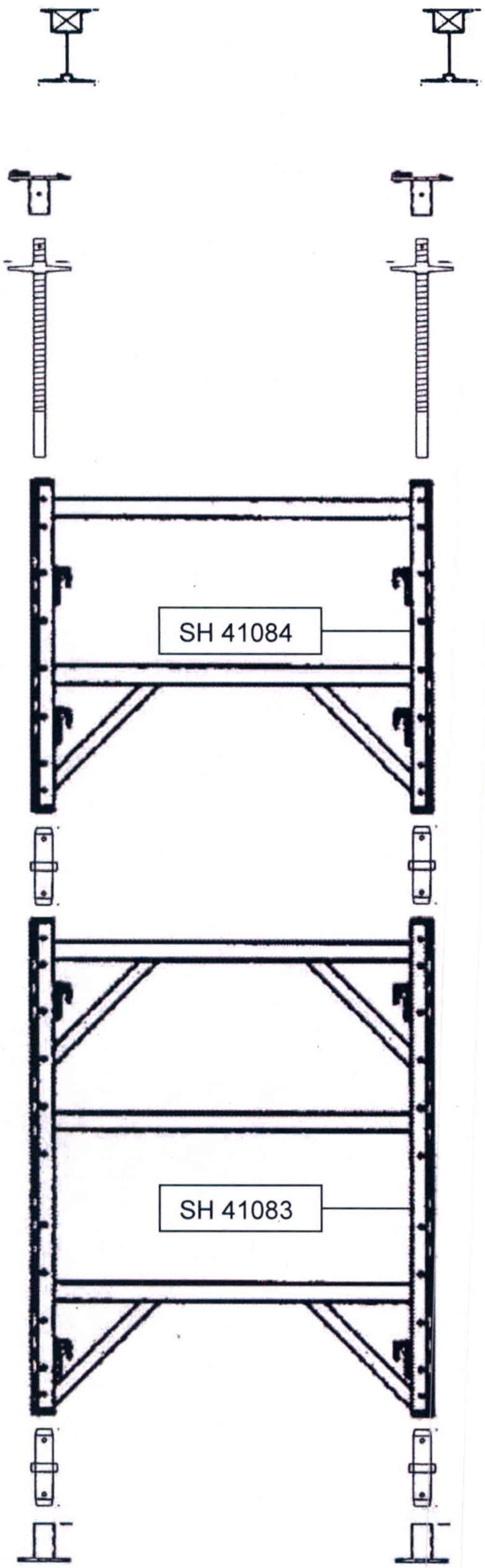
Folder: 08-02-11; 08-02-11; SH

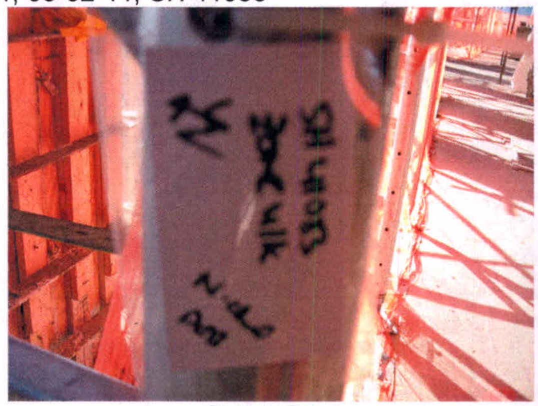
SH41082_1

SH41082_2

SH 41083
Bottom Frame of SH 41084
Zone 41K

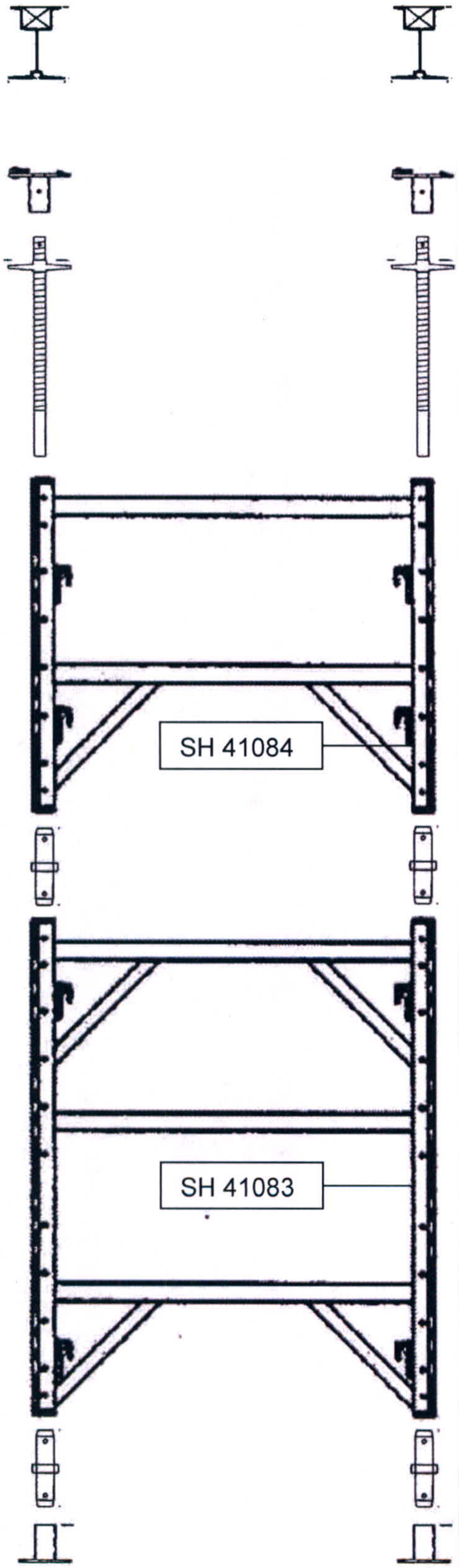
Folder: 08-02-11; 08-02-11; SH 41083





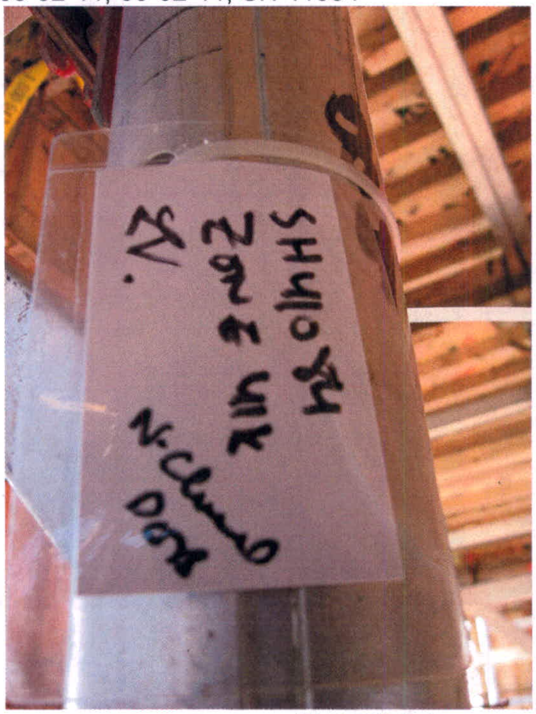
SH41083_1

SH41083_2



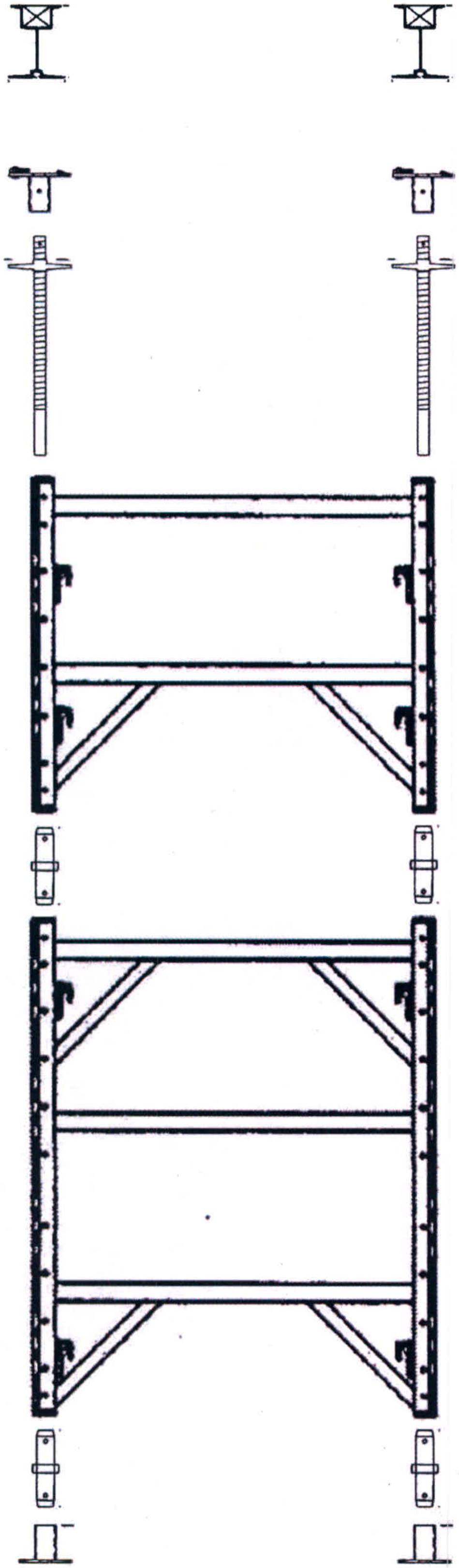
SH 41084
Top frame of SH 41083
Zone 41K

Folder: 08-02-11; 08-02-11; SH 41084



SH41084_1

SH41084_2

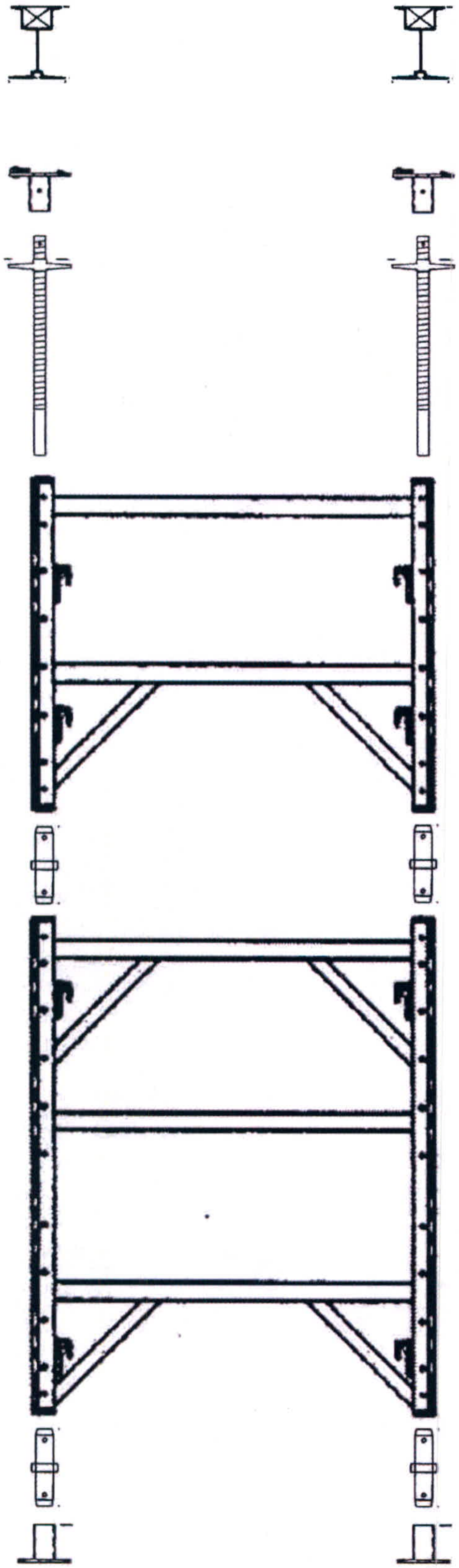


SH 41085
Screw Jack
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41085

SH41085_1

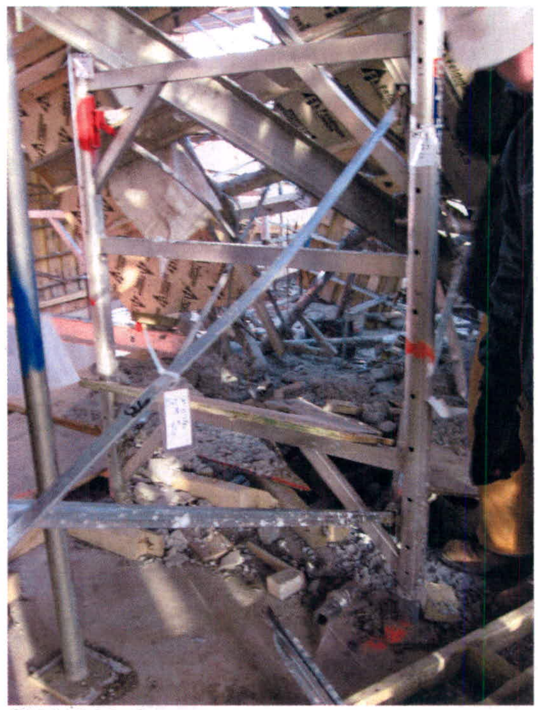
SH41085_2



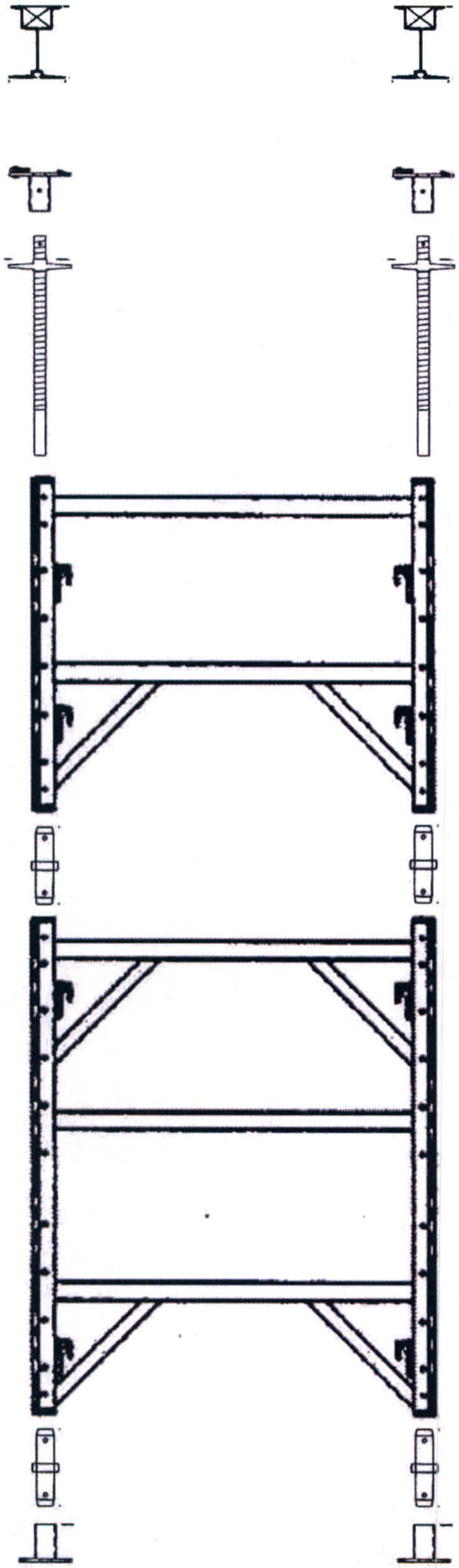
SH 41086
Brace connects SH 41093 and SH 41095
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41086

SH41086_1



SH41086_2

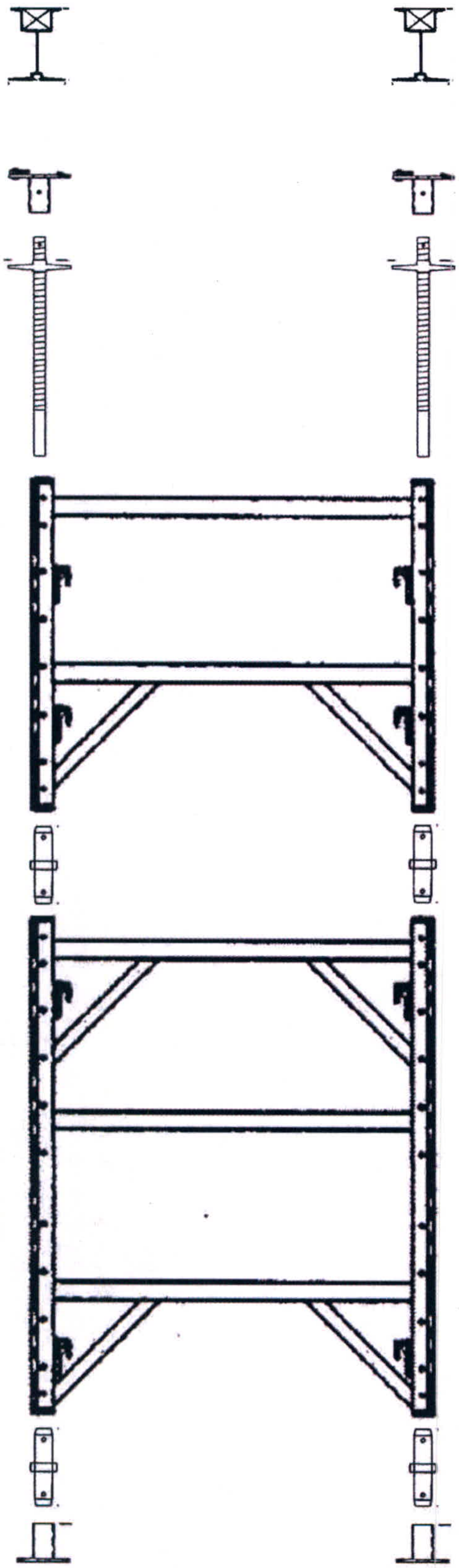


SH 41087
Brace Connects SH 41006 and SH 40194
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41087

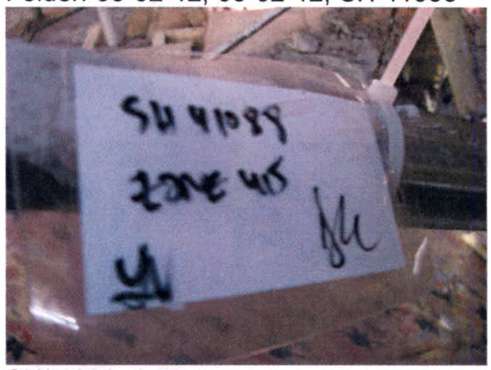
SH41087_1

SH41087_2



SH 41088
Brace connects SH 41093 to SH 41044
Zone 41F

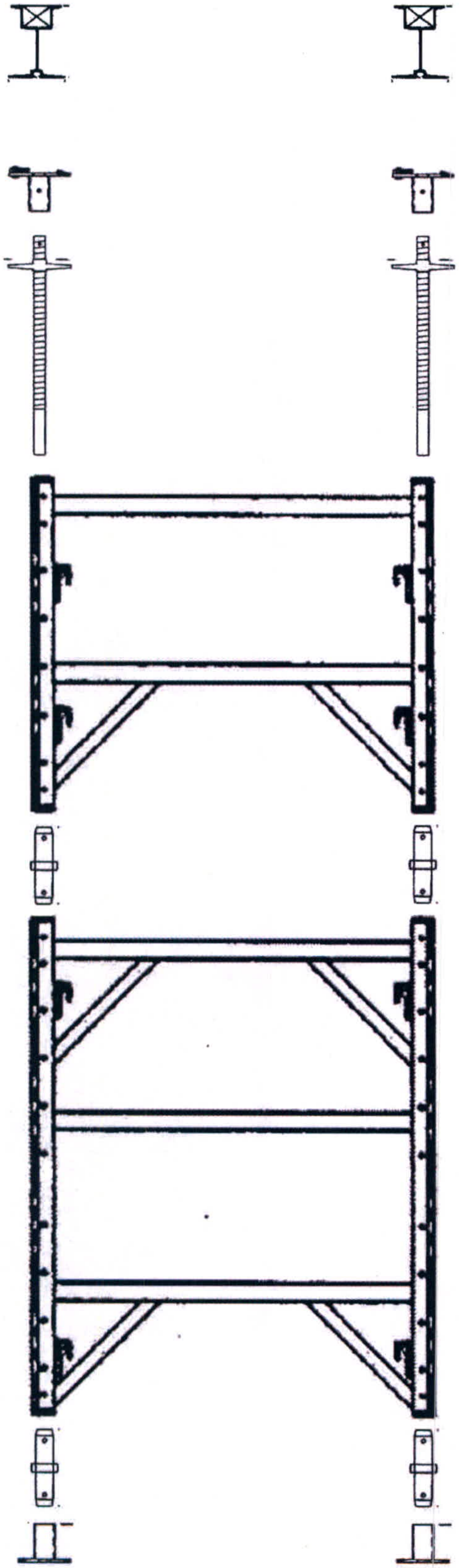
Folder: 08-02-12; 08-02-12; SH 41088



SH41088_1

SH41088_2

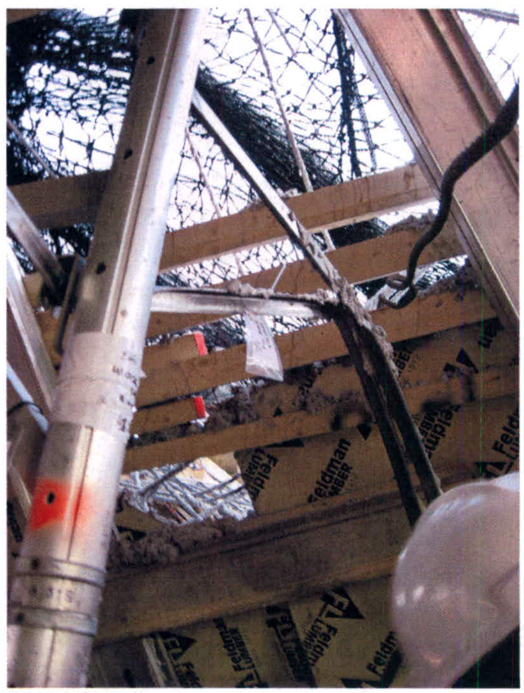
SH41088_3



SH 41089
Brace connects SH 41090 and SH 41006
Zone 41F

Folder: 08-02-12; 08-02-12; SH 41089

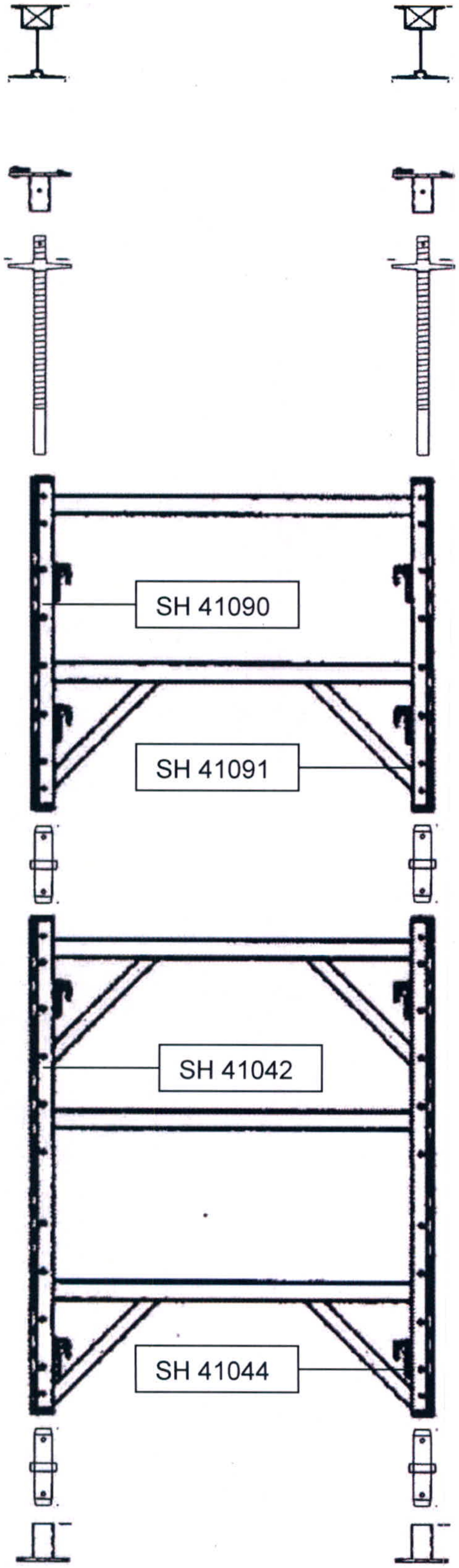
SH41089_1

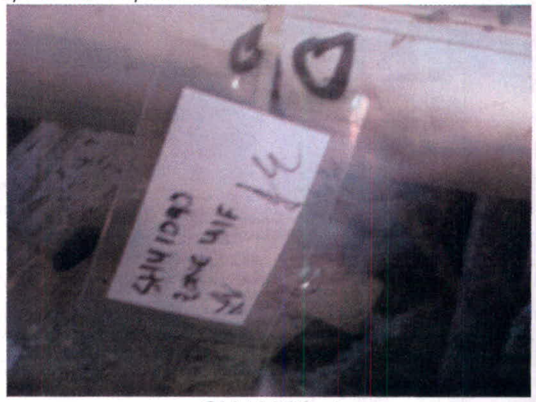


SH41089_2

SH 41090
Top Frame with SH 41091
Top Frame of SH 41042 & SH 41044

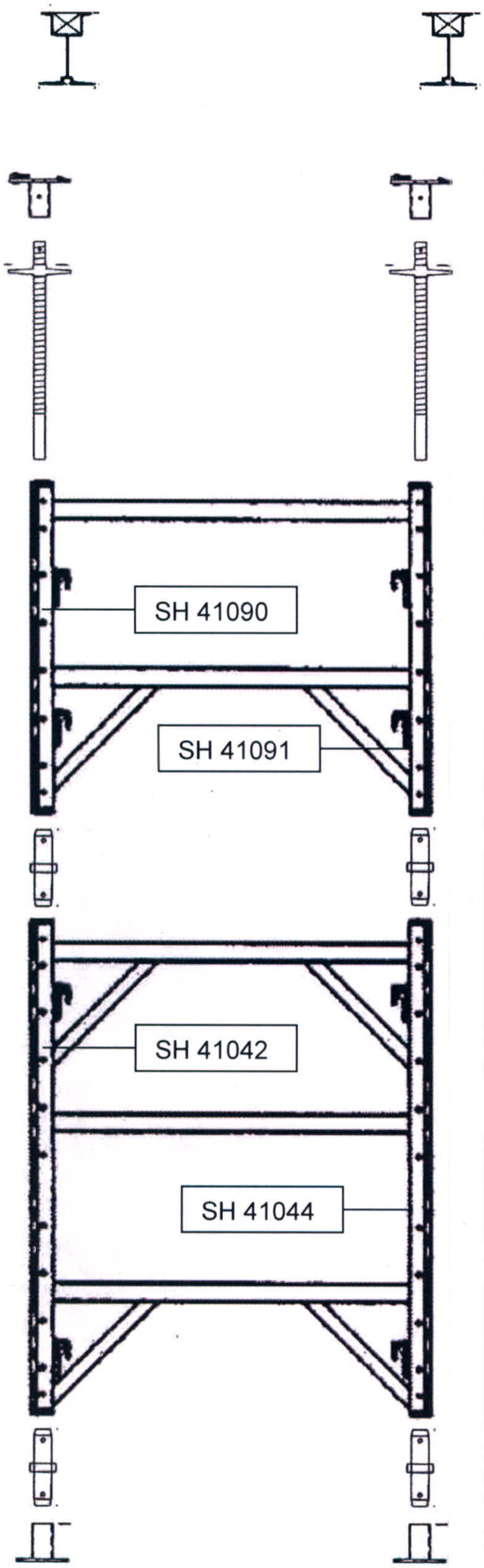
Folder: 08-02-12; 08-02-12; SH 41090




SH41090_1

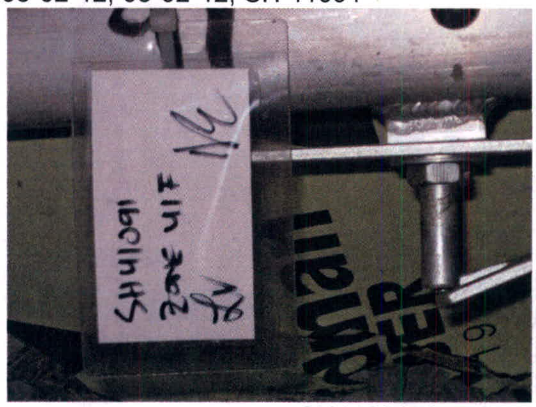

SH41090_2


SH41090_7

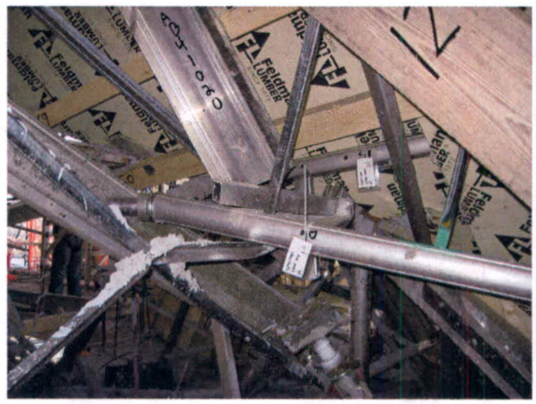


SH 41091
Broken with SH 41090
Top Frame of SH 41042 & SH 41044
Half piece of SH 41092
Zone 41F

Folder: 08-02-12; 08-02-12; SH 41091



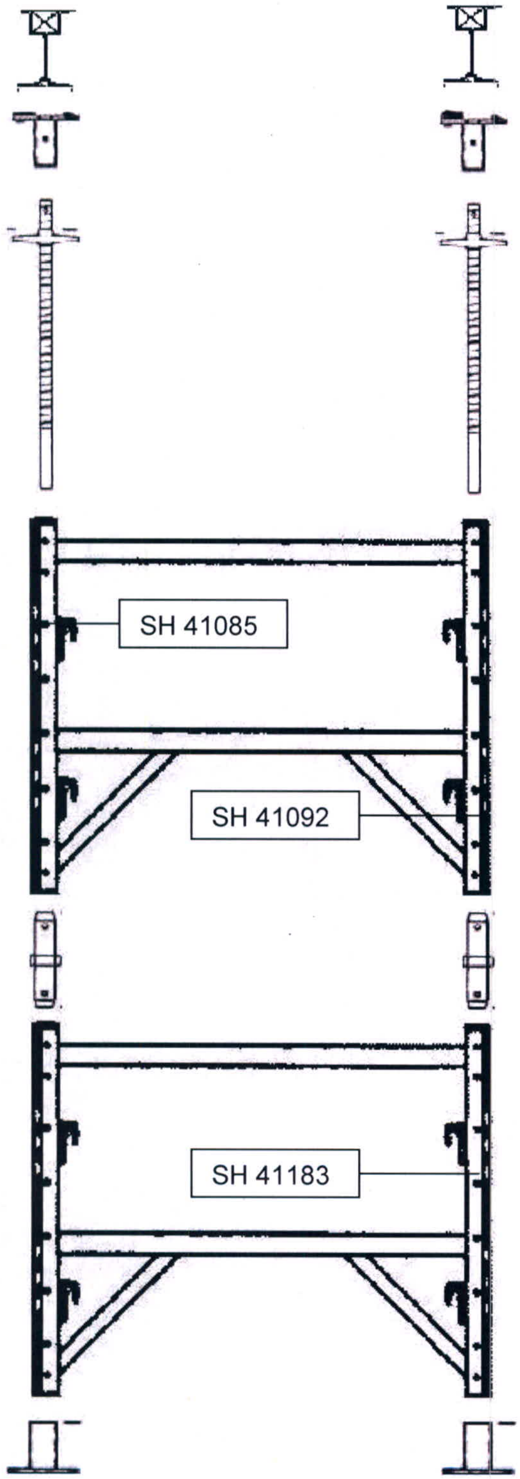
SH41091_3



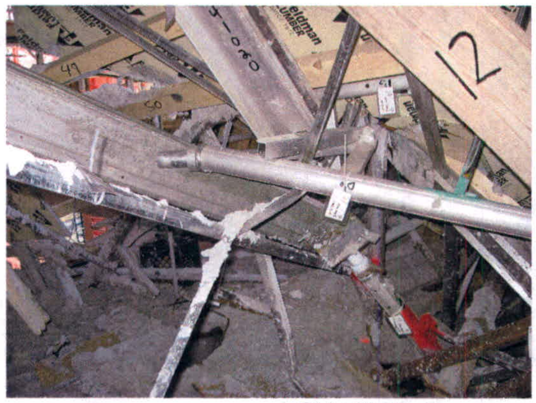
SH41091_4

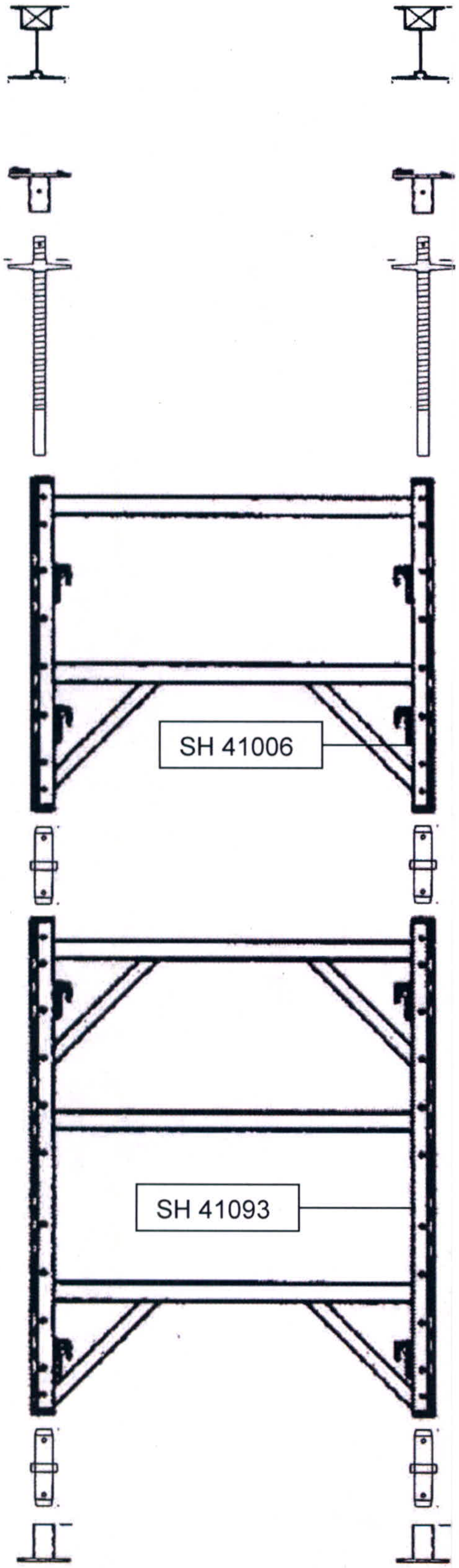
SH 41092
Broken with SH 41185
Top Frame of SH 41183
Connects to SH 41029 and SH 41005
Zone 41F-G

Folder: 08-02-12; 08-02-12; SH 41092



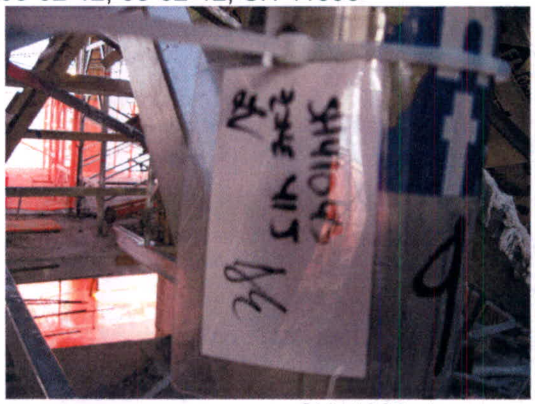

SH41092_1


SH41092_2



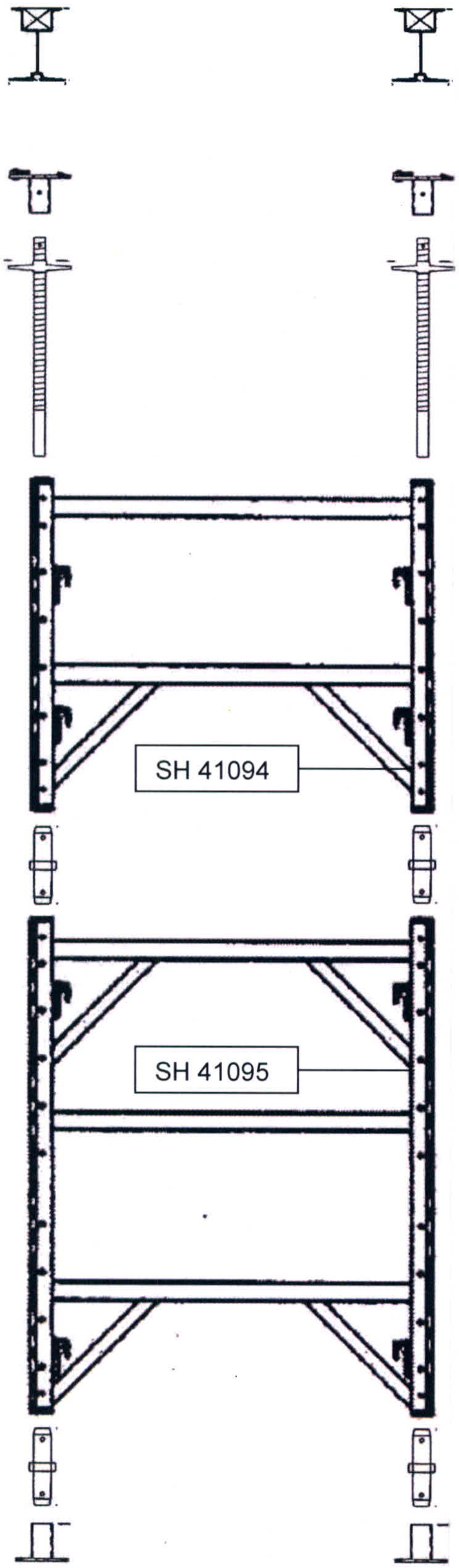
SH 41093
Bottom Frame of SH 41006
Connect to SH 41029
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41093



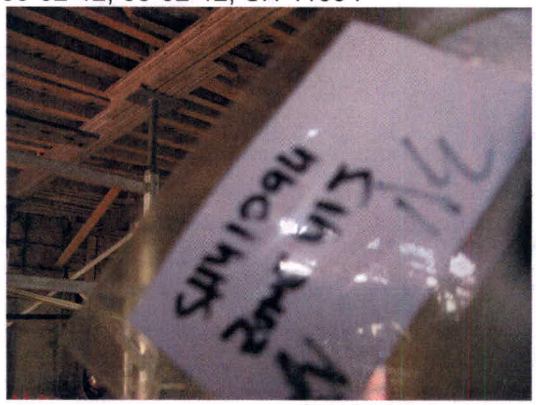
SH41093_1

SH41093_2



SH 41094
Top frame of SH 41095
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41094

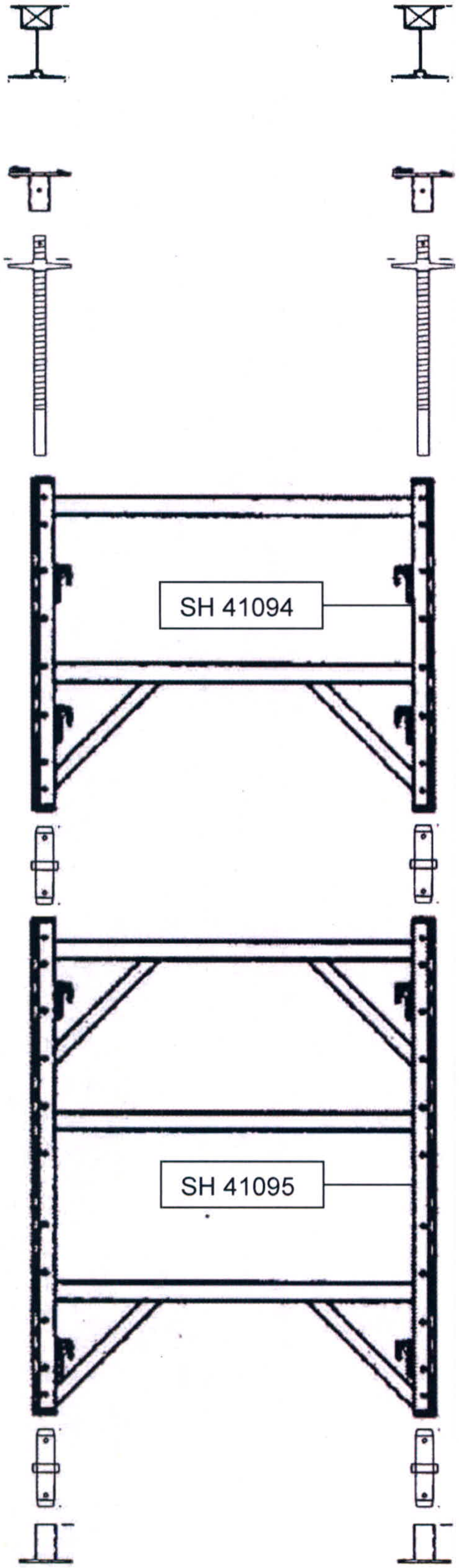


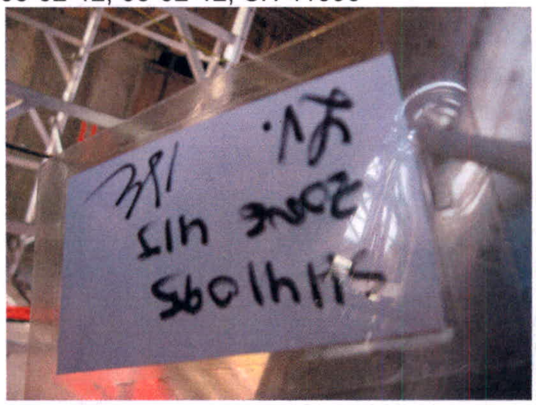
SH41094_1

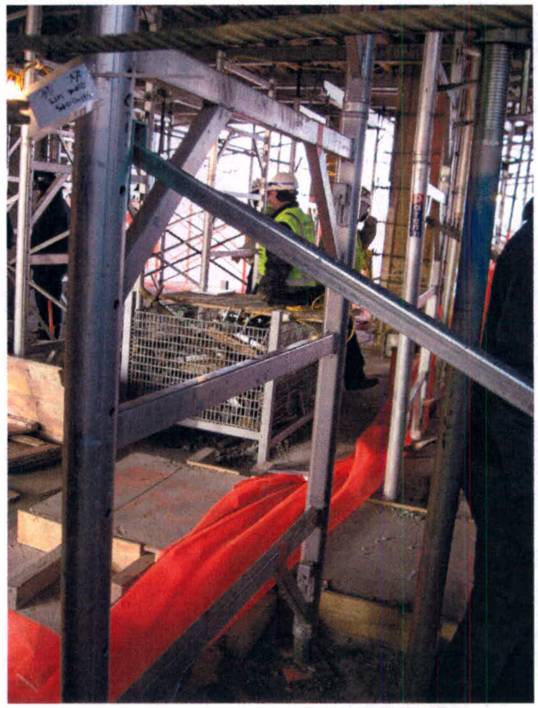
SH41094_2

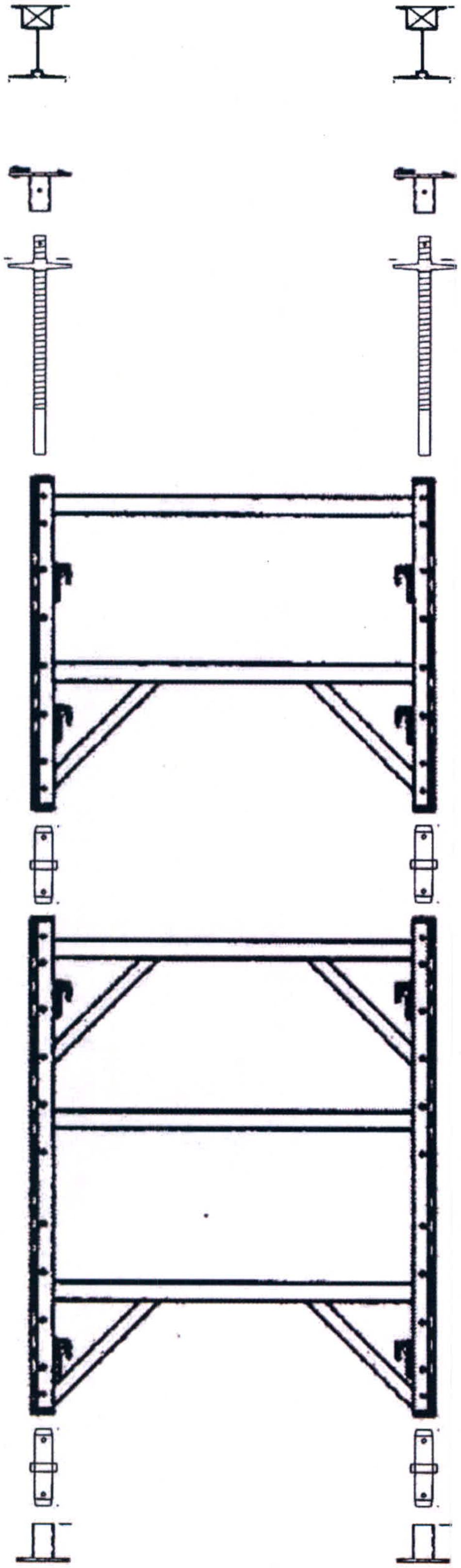
SH 41095
Bottom Frame of SH 41094
Connects to SH 41087 and SH 41086
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41095




SH41095_1


SH41095_2

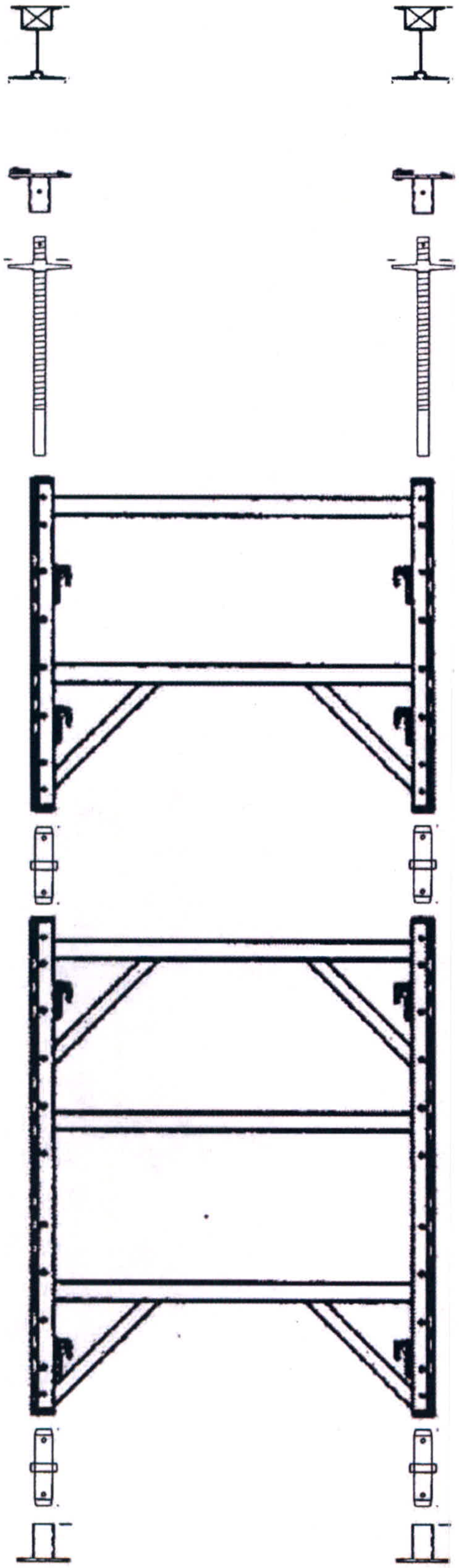


SH 41096
Brace connects SH 41094 and SH 41006
Zone 41F

Folder: 08-02-12; 08-02-12; SH 41096

SH41096_1

SH41096_2

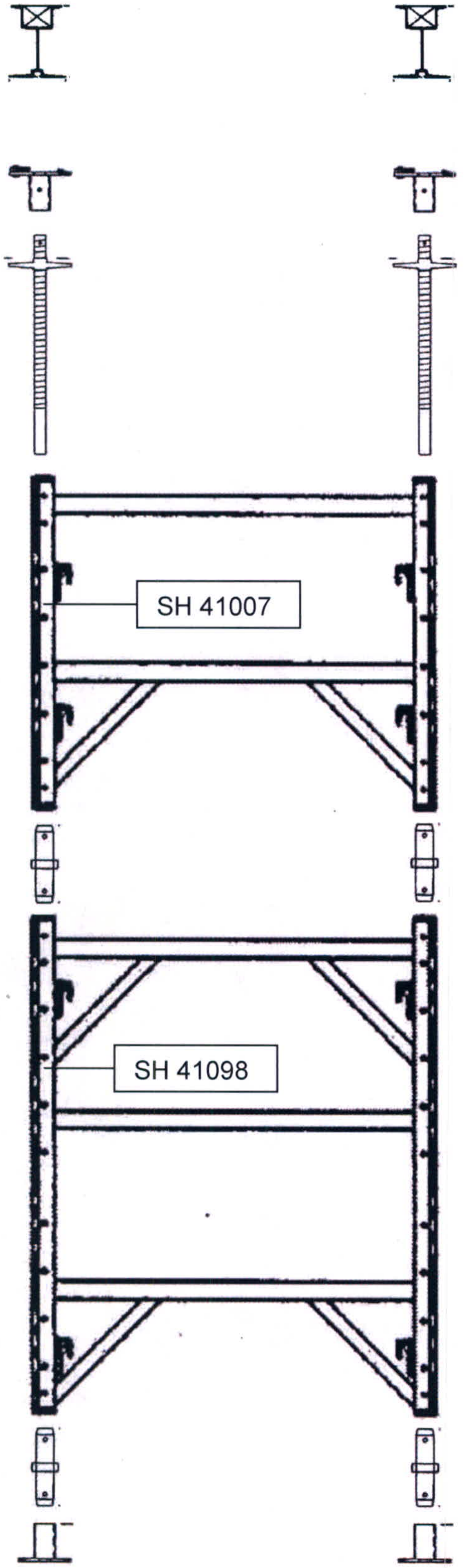


SH 41097
Top Brace connects SH 41091
Zone 41F

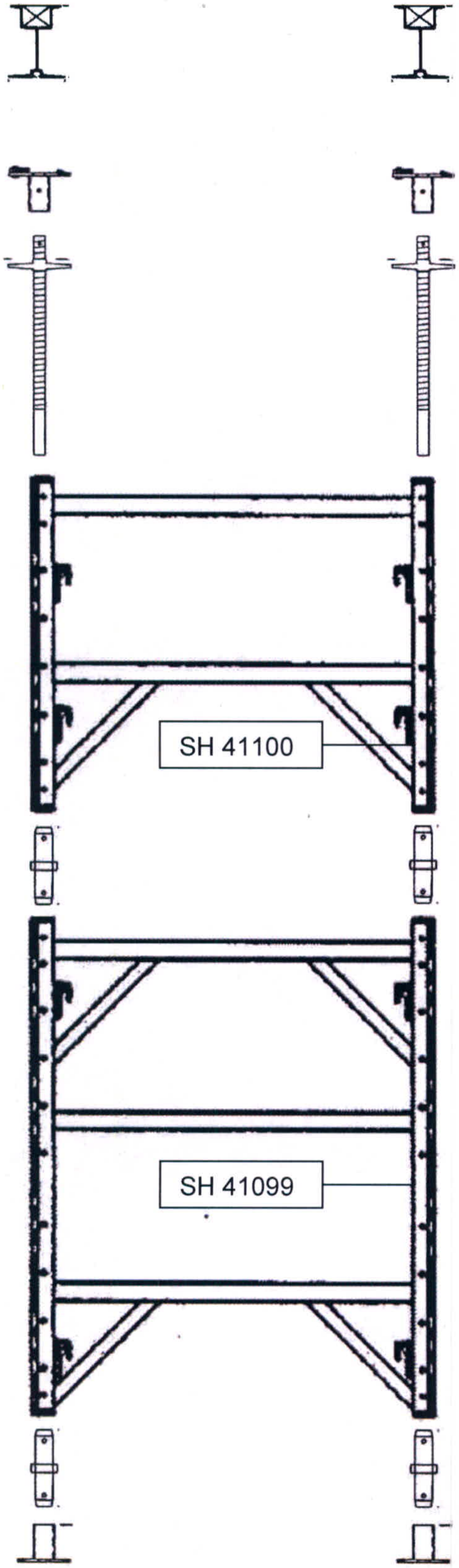
Folder: 08-02-12; 08-02-12; SH 41097

SH41097_1

SH41097_3

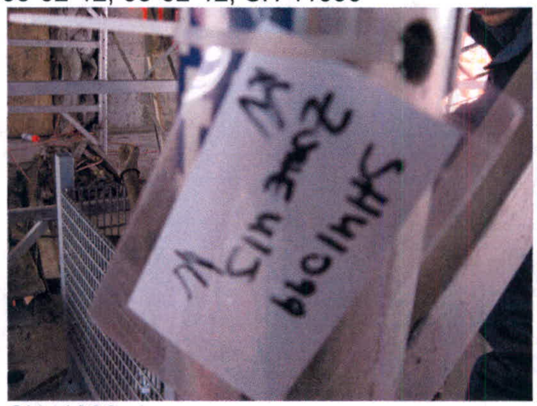


SH 41098
Bottom Frame of SH 41007
See SH 41007 for Details
Tag was skipped



SH 41099
Bottom Frame of SH 41100
Connect to brace SH 41101
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41099

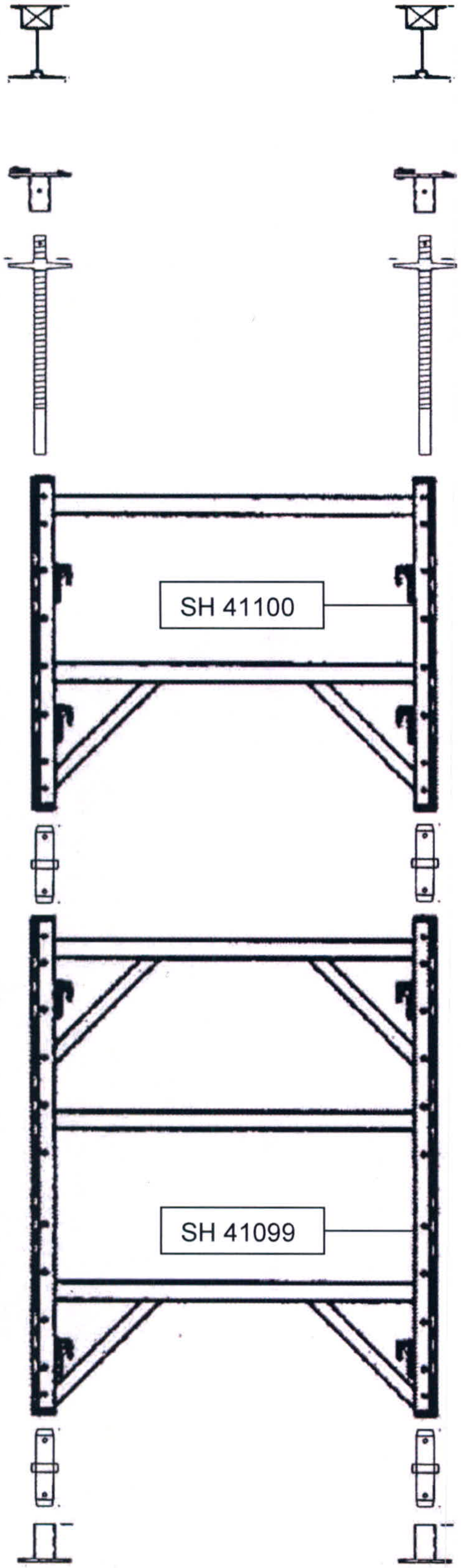


SH41099_1

SH41099_2

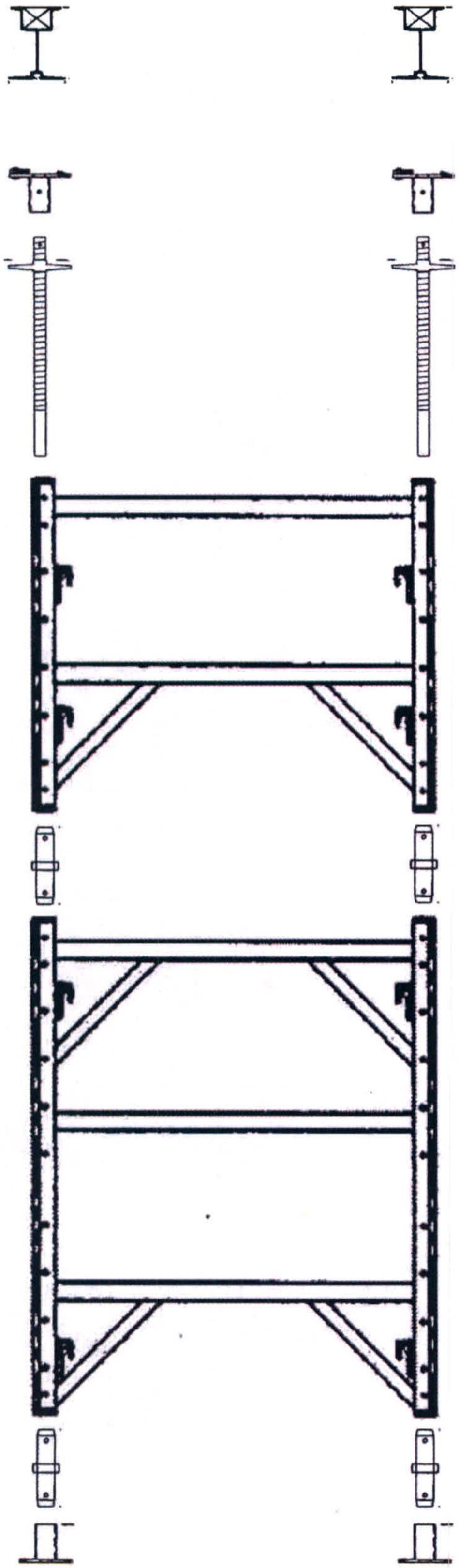
SH 41100
Top frame of SH 41099
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41100



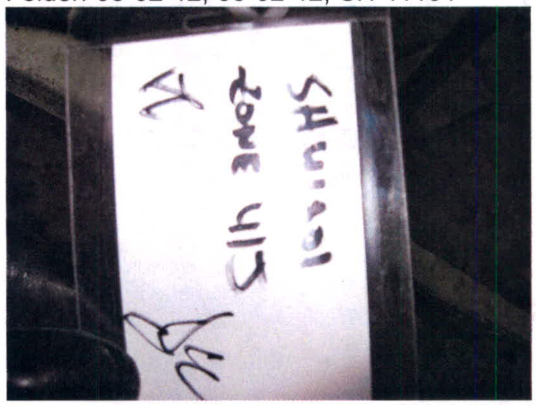
SH41100_1

SH41100_2



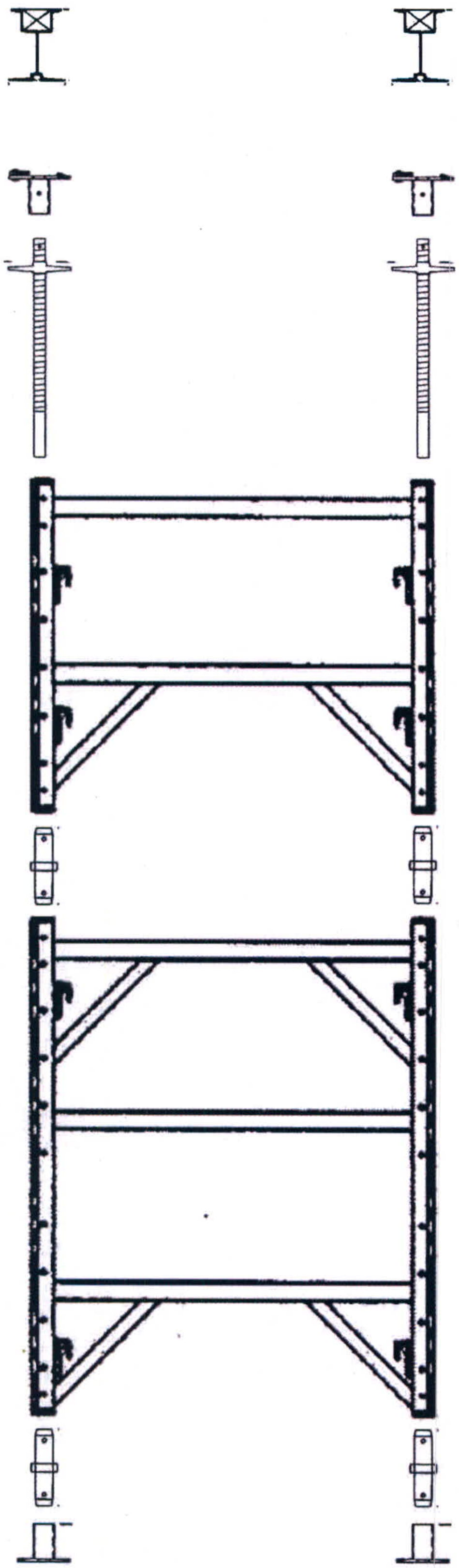
SH 41101
Brace of SH 41099
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41101



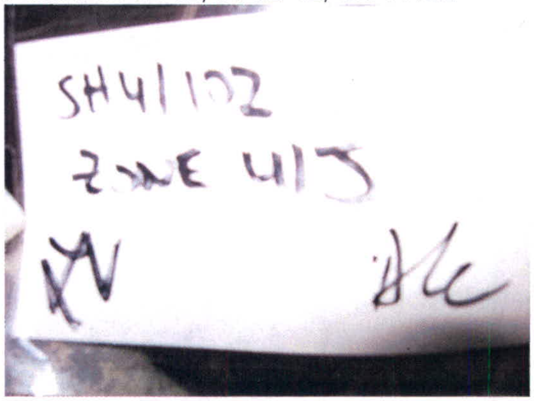
SH41101_1

SH41101_2



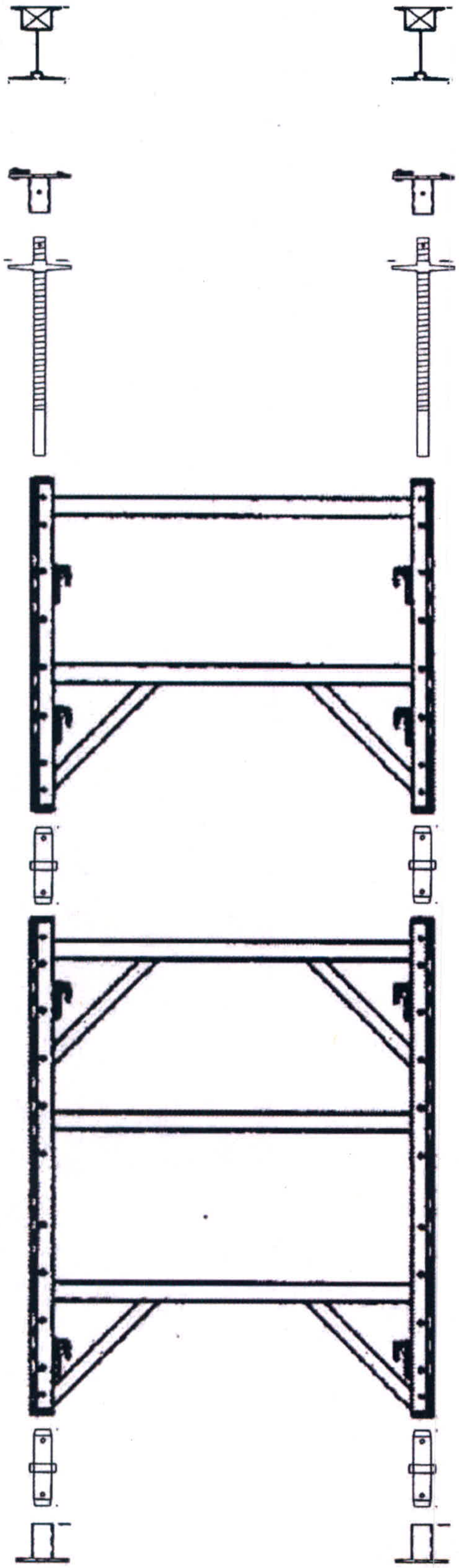
SH 41102
Bottom Brace for SH 41099
Top Shore is SH 41007
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41102



SH41102_1

SH41102_2

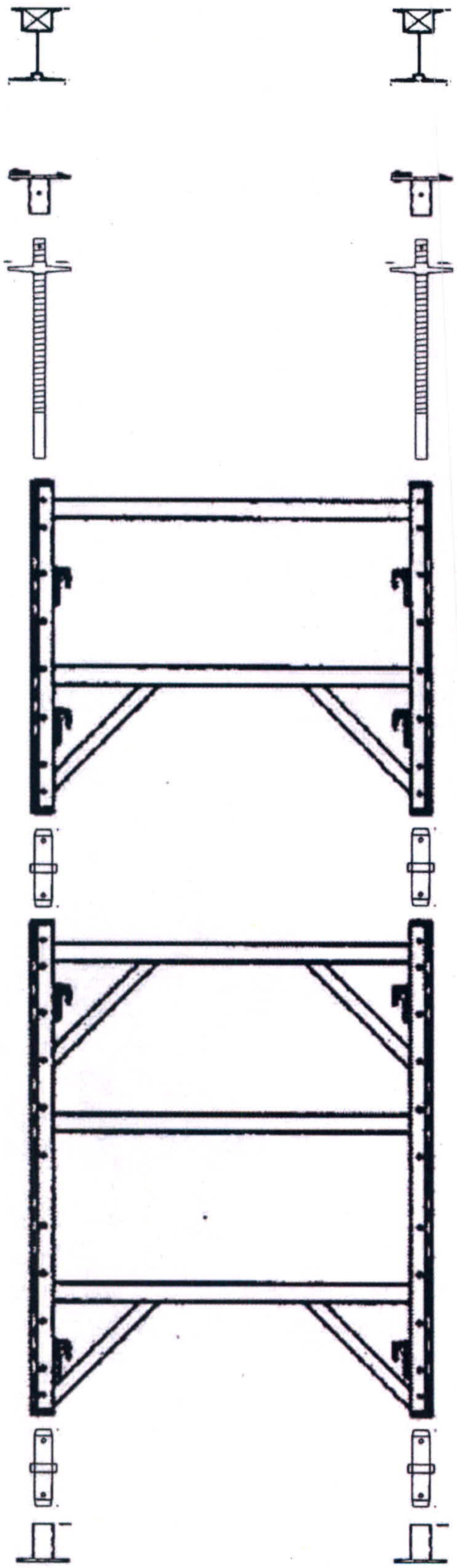


SH 41103
Top brace
Brace connects SH 41007 and SH 41100
Zone 41J

Folder: 08-02-12; 08-02-12; SH 41103

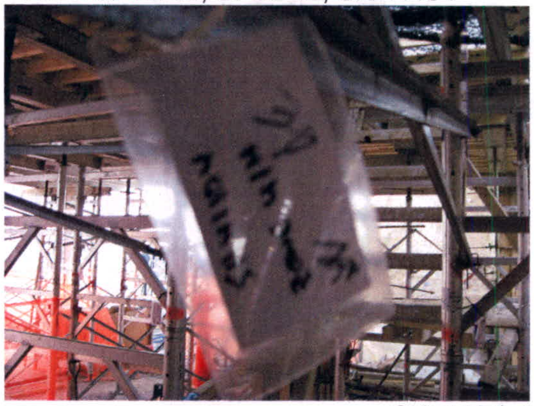
SH41103_1

SH41103_2



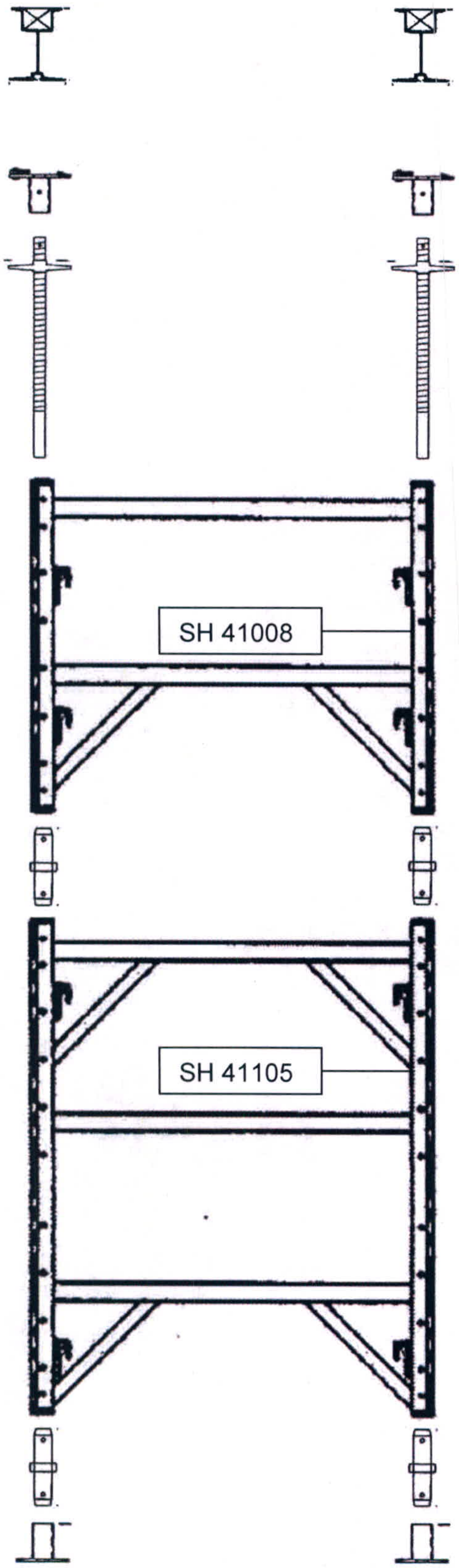
SH 41104
Top brace
Brace Connects to SH 41007 and SH 41100
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41104



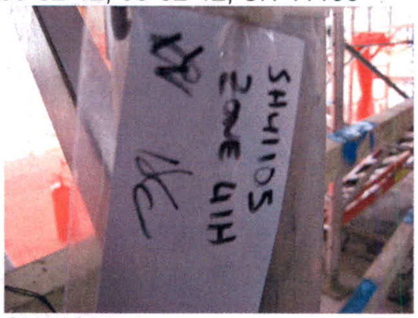
SH41104_1

SH41104_2



SH 41105
Bottom Frame of SH 41008
Connects to brace SH 41108 and SH 41109
Zone 41H

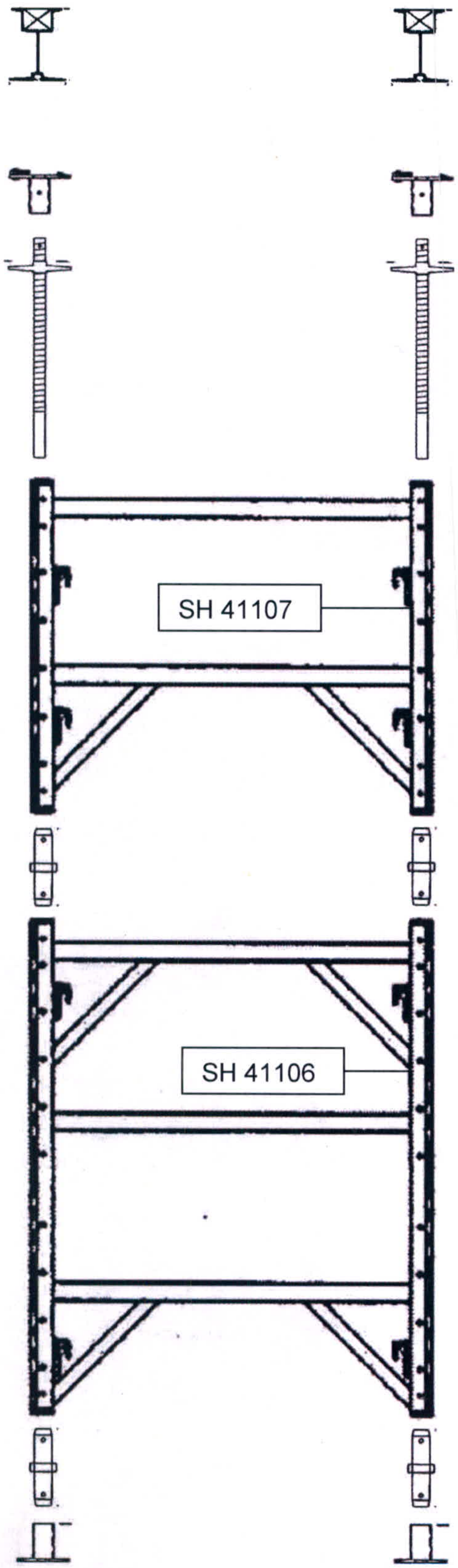
Folder: 08-02-12; 08-02-12; SH 41105



SH41105_1

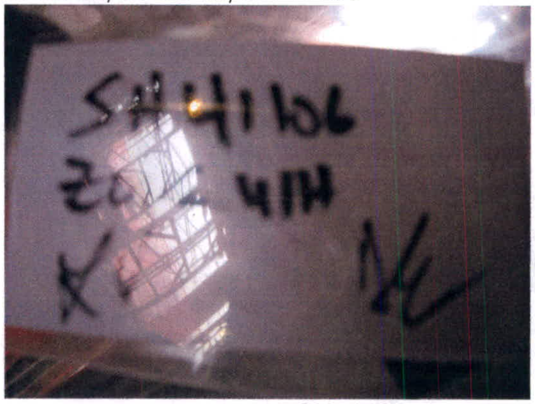
SH41105_2

SH41105_3



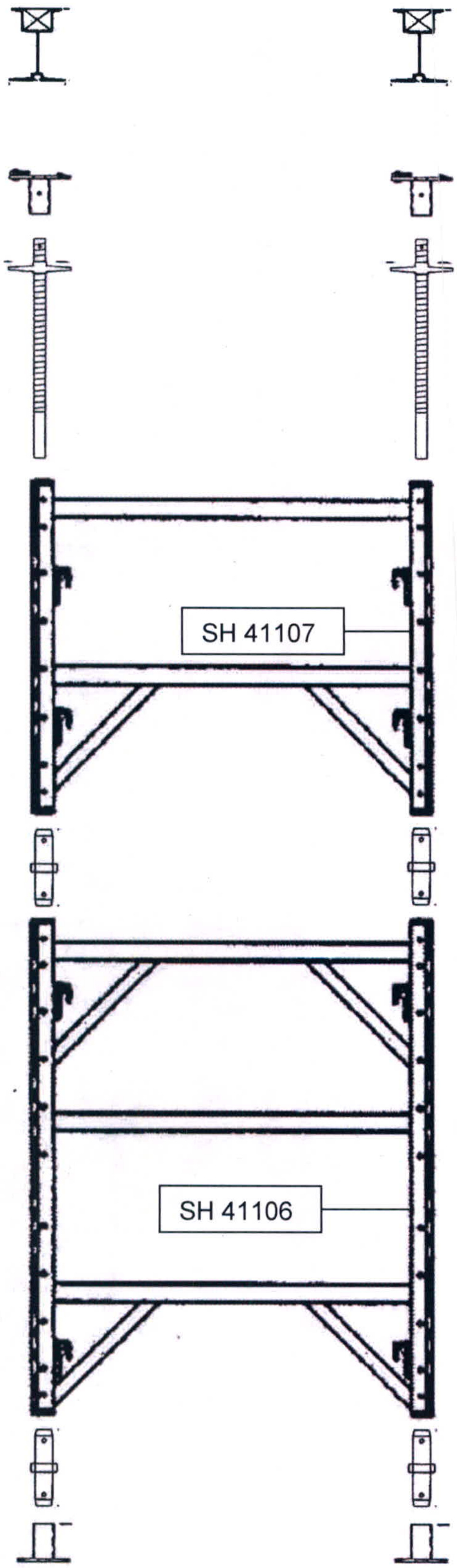
SH 41106
Bottom Frame of SH 41107
Connects to brace SH 41108 and SH 41109
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41106



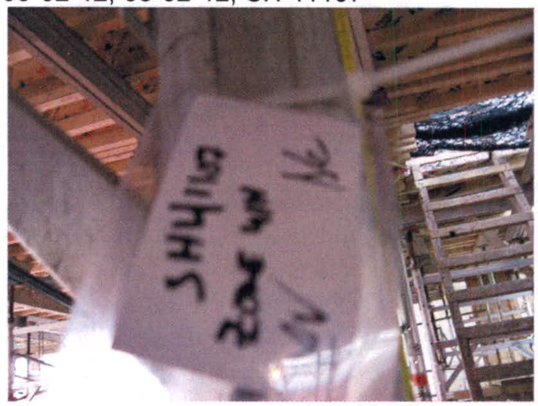
SH41106_1


SH41106_2



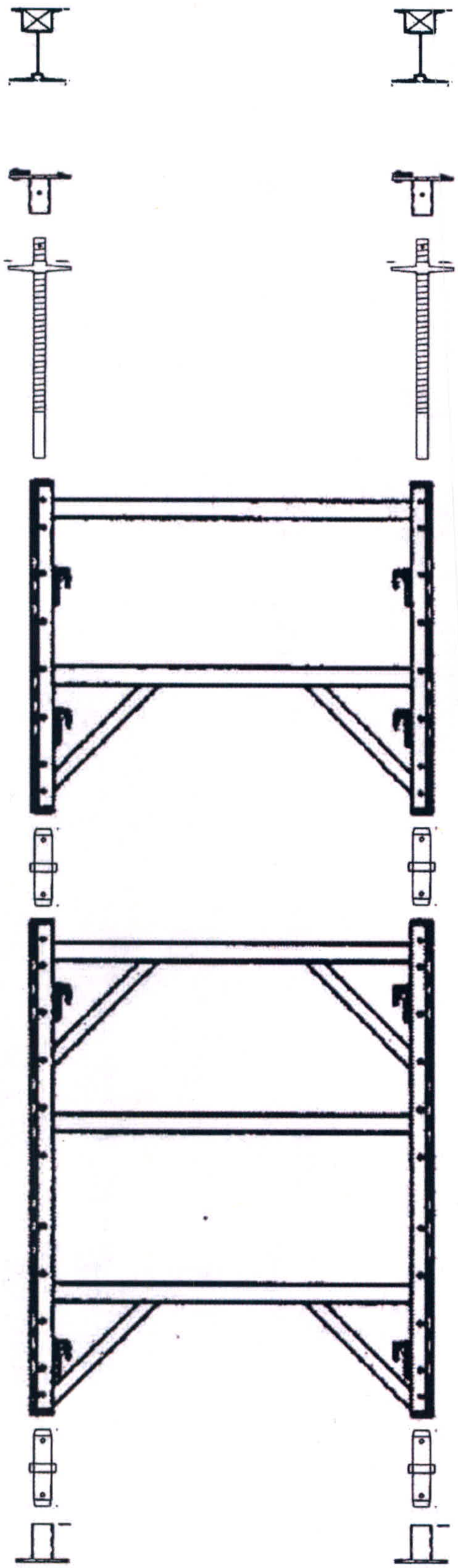
SH 41107
Top frame of SH 41106
Connects to SH 4110 and SH 41111
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41107



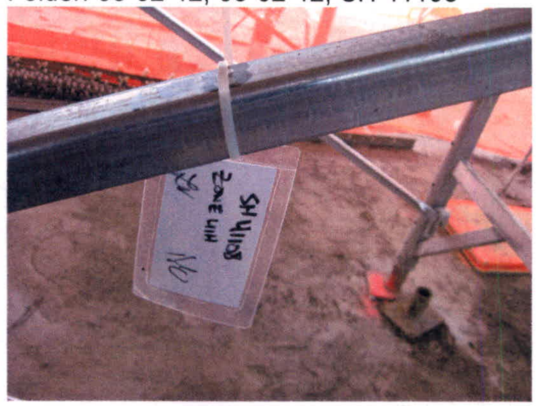
SH41107_1

SH41107_2



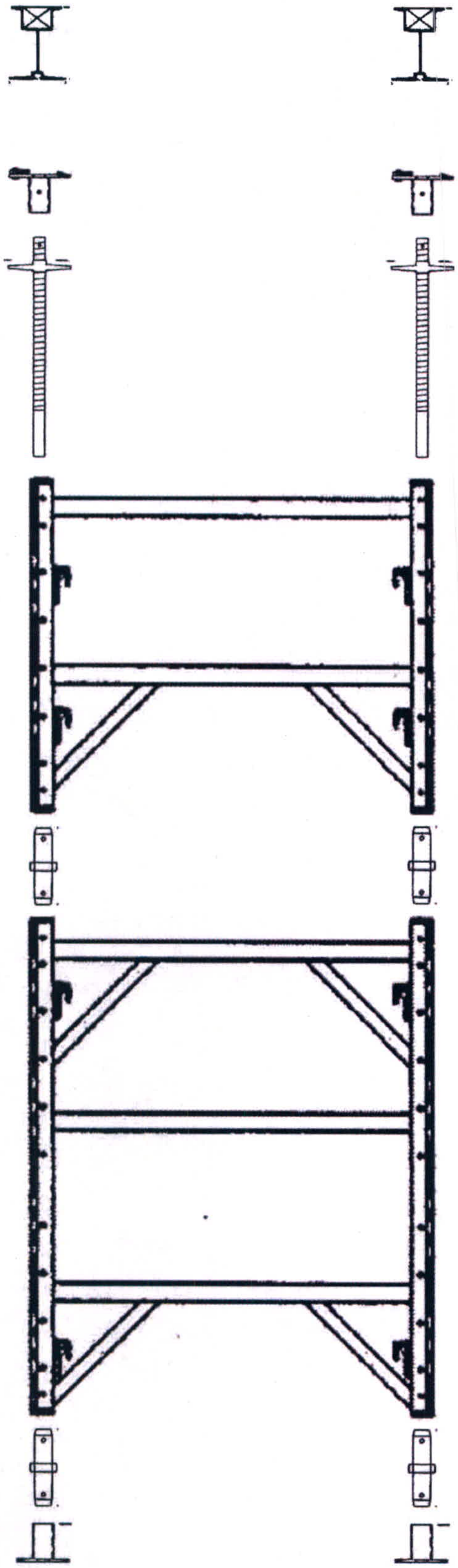
SH 41108
Bottom Brace
Brace of SH 41105 and SH 41106
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41108



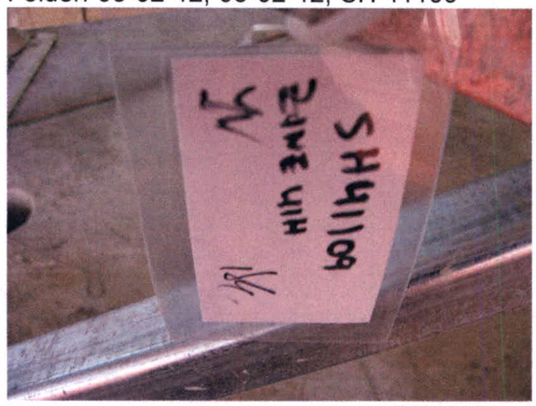
SH41108_1

SH41108_2



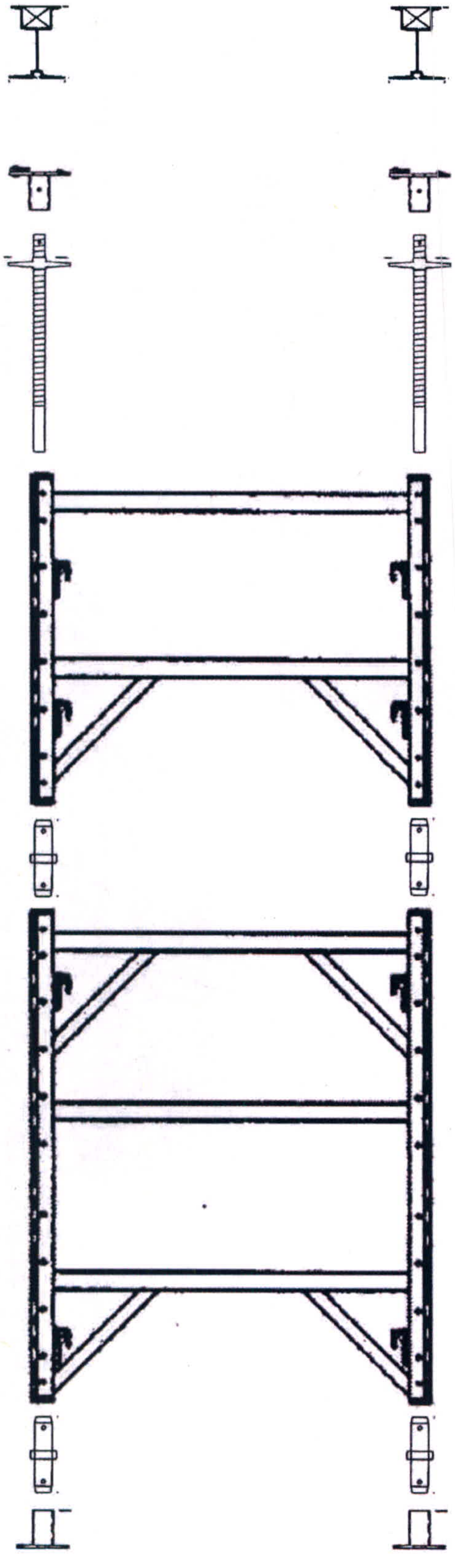
SH 41109
Brace
Connects SH 41105 and SH 41106
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41109



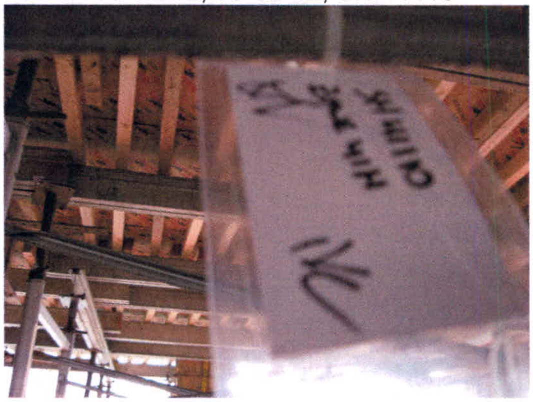
SH41109_1

SH41109_2



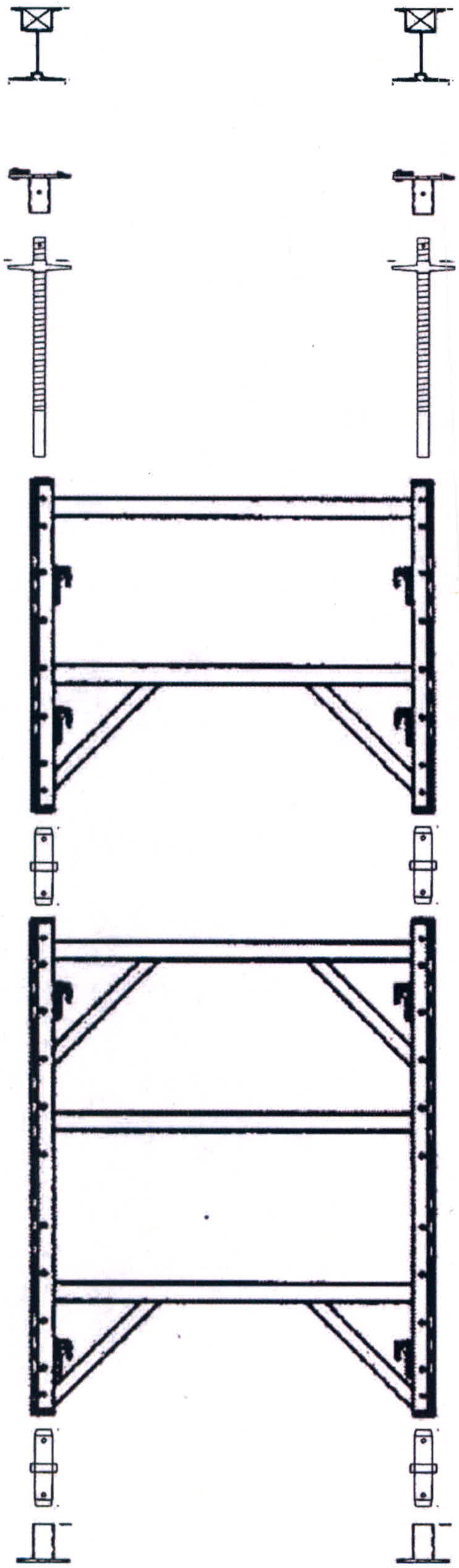
SH 41110
Brace
Connects SH 41108 and SH 41107
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41110



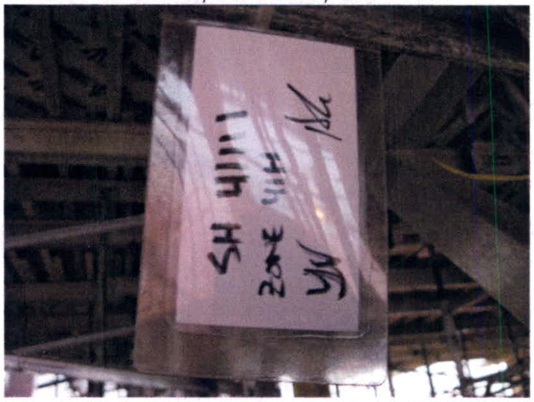
SH41110_1

SH41110_2



SH 41111
Brace
Connects to SH 41008 and SH 41107
Zone 41H

Folder: 08-02-12; 08-02-12; SH 41111

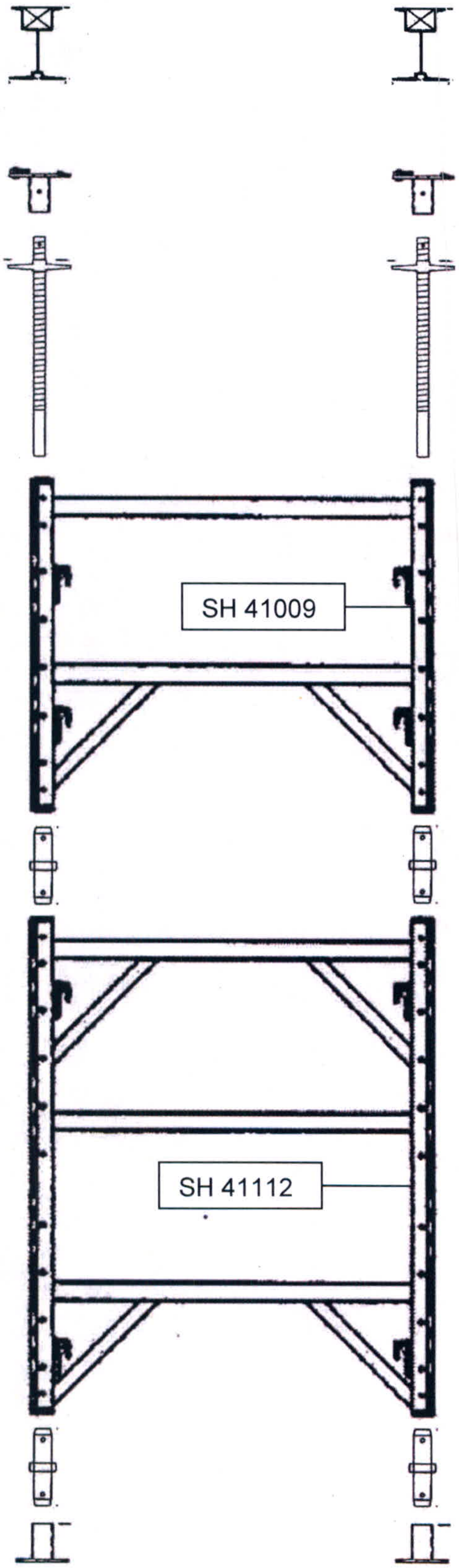


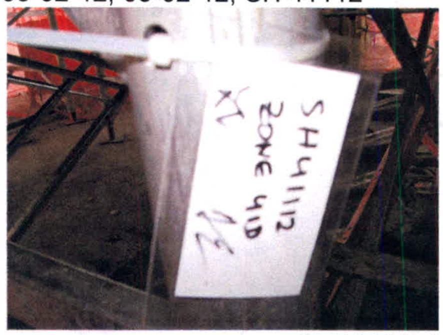
SH41111_1

SH41111_2

SH 41112
Bottom Frame of SH 41009
Zone 41D

Folder: 08-02-12; 08-02-12; SH 41112

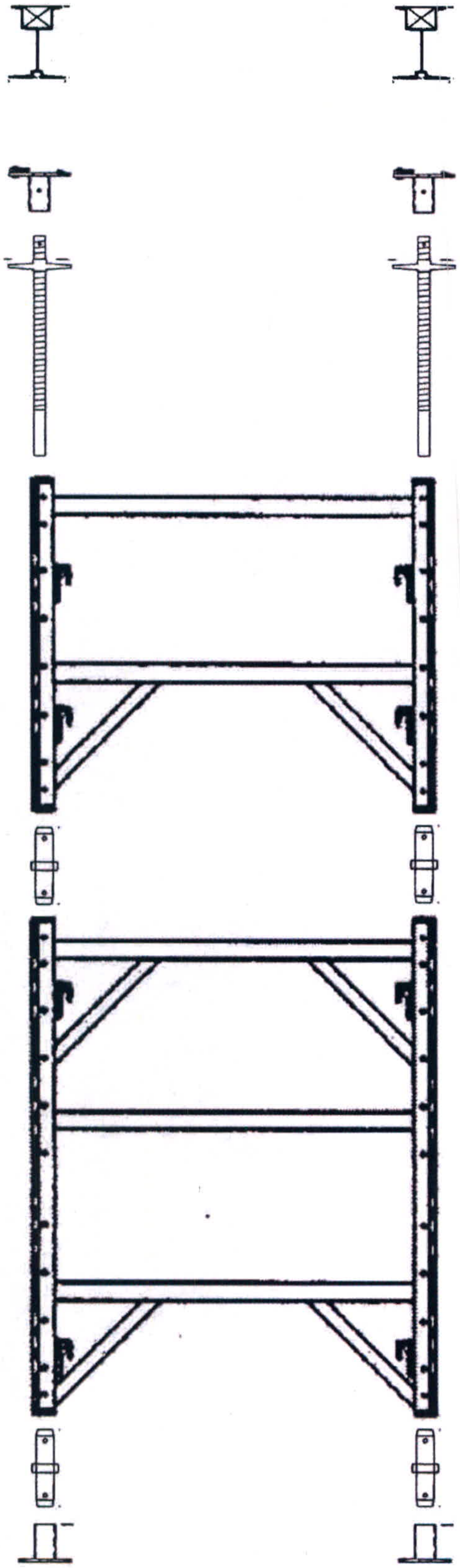




SH41112_1

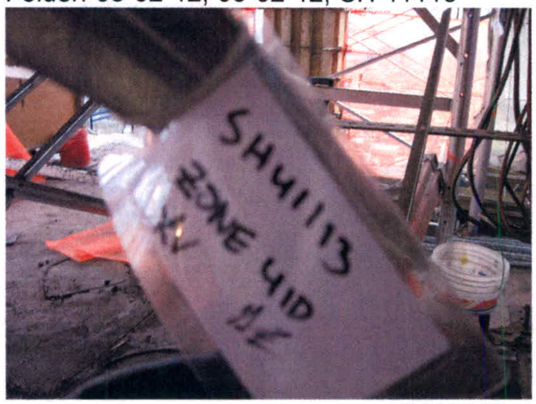
SH41112_2

SH41112_3



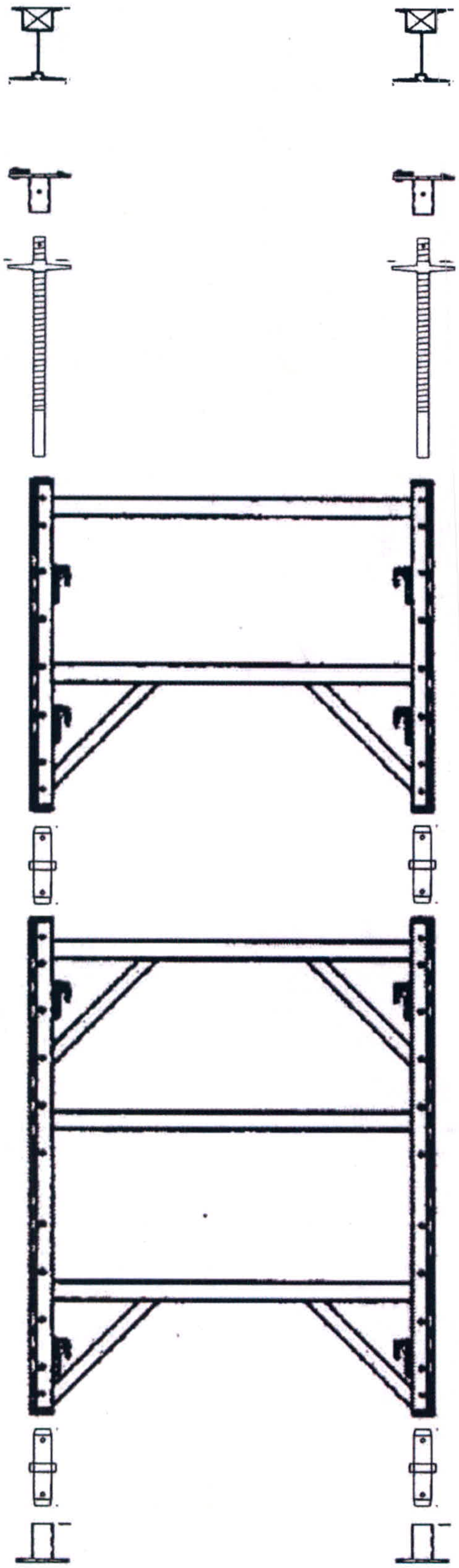
SH 41113
Brace
Connect SH 41115 and SH 41112
Zone 41D

Folder: 08-02-12; 08-02-12; SH 41113



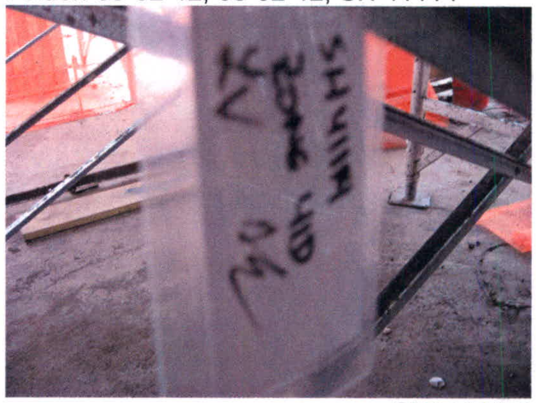
SH41113_1

SH41113_2



SH 41114
Brace
Connect to SH 41115 and SH 41112
Zone 41D

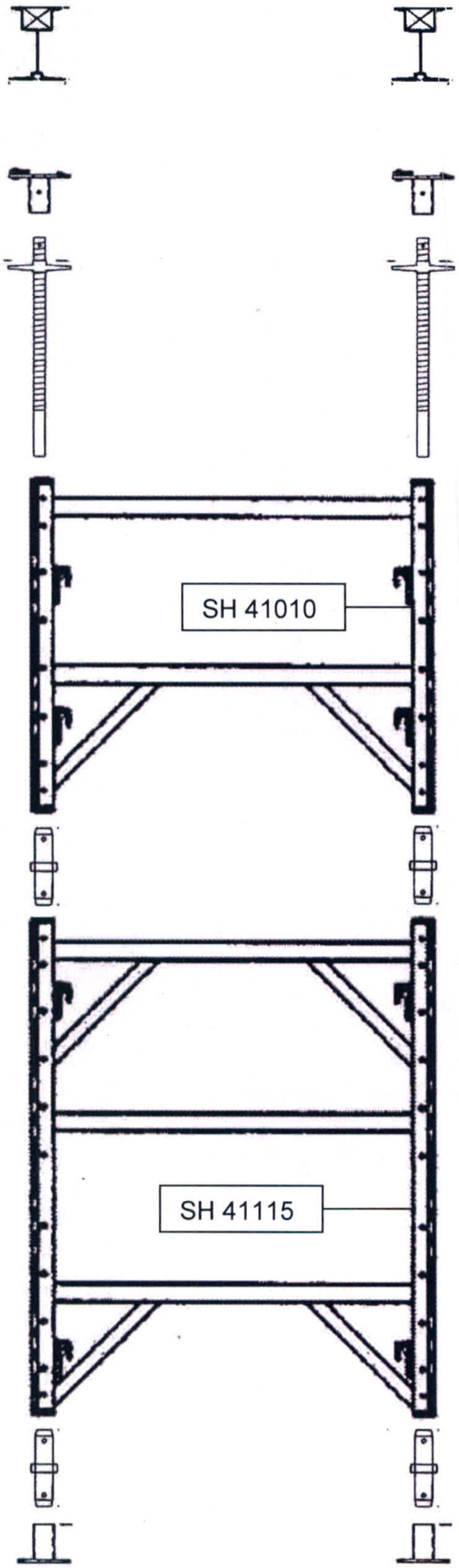
Folder: 08-02-12; 08-02-12; SH 41114

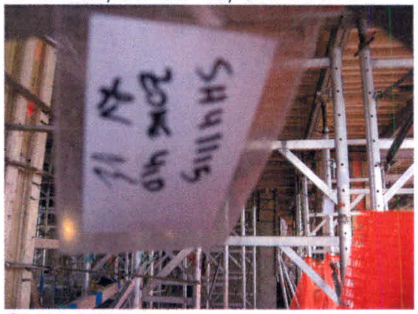


SH41114_1

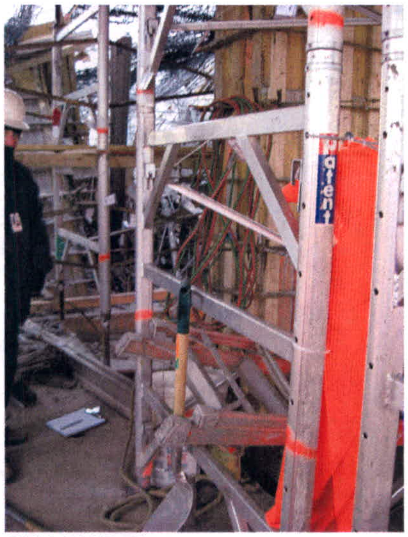
SH41114_2

SH 41115
Bottom Frame of SH 41010
Zone 41D
Folder: 08-02-12; 08-02-12; SH 41115





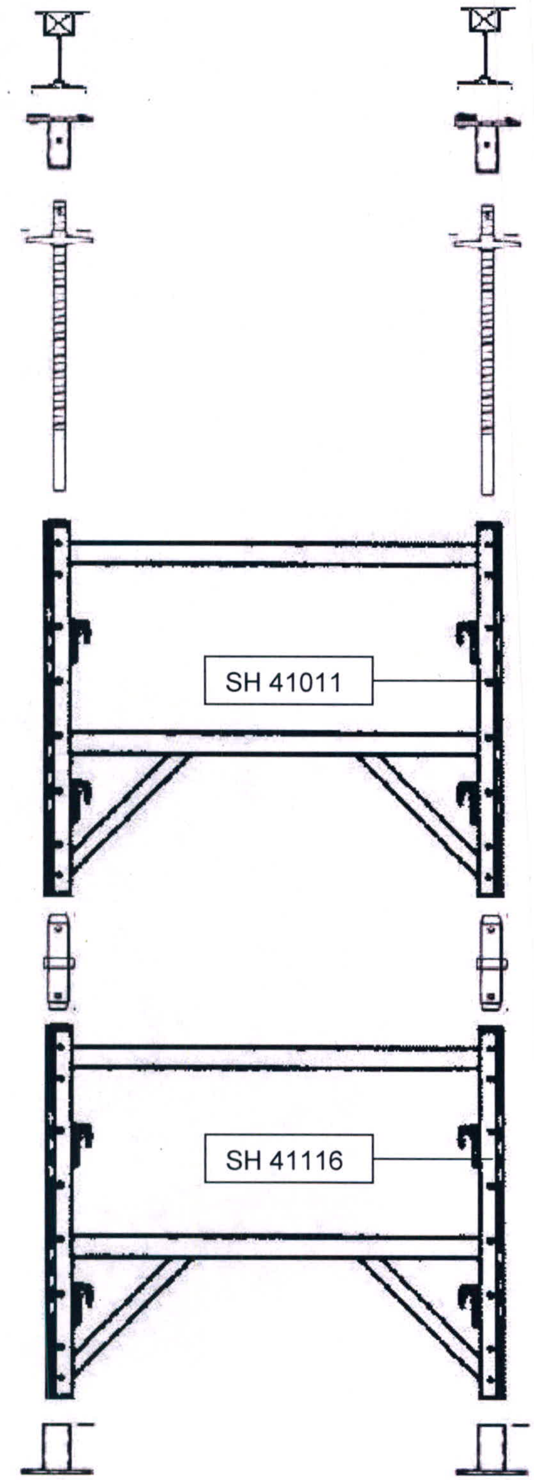
SH41115_1

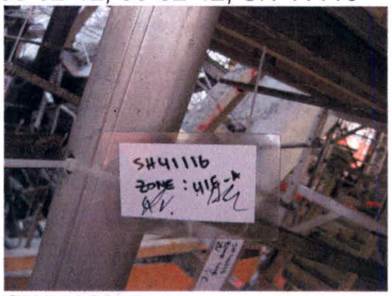

SH41115_2


SH41115_3

SH 41116
Bottom Frame of SH 41011
Connected to Brace SH 41118
Zone 41E-A

Folder: 08-02-12; 08-02-12; SH 41116





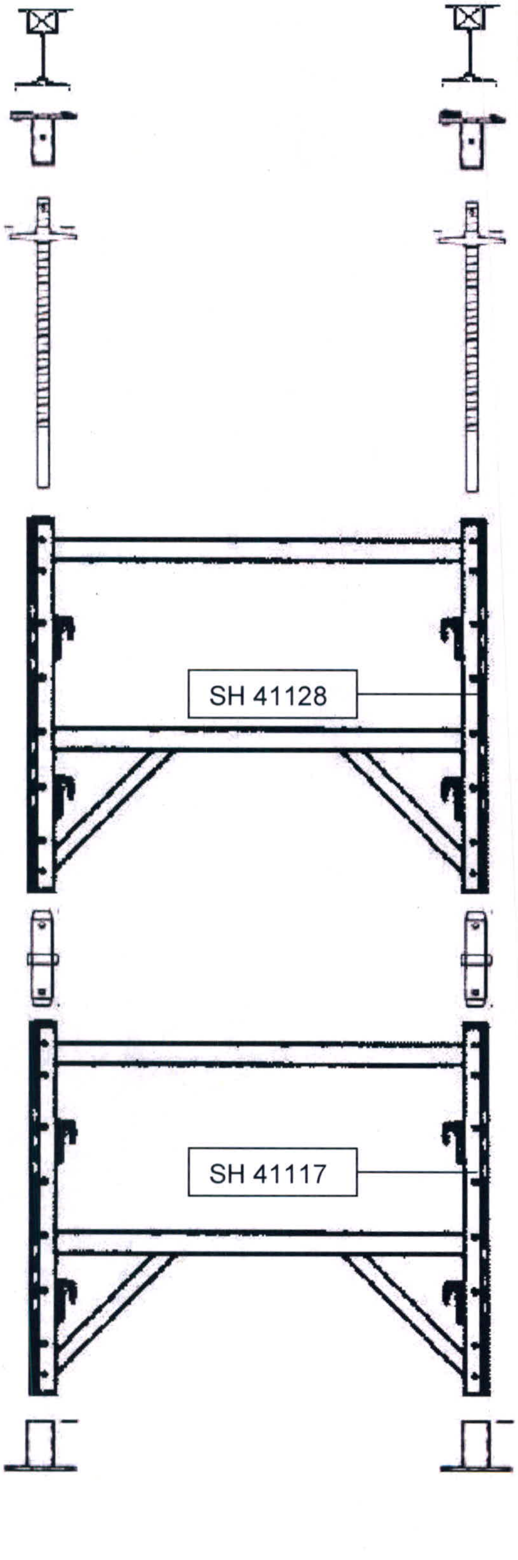
SH41116_1


SH41116_2


SH41116_3

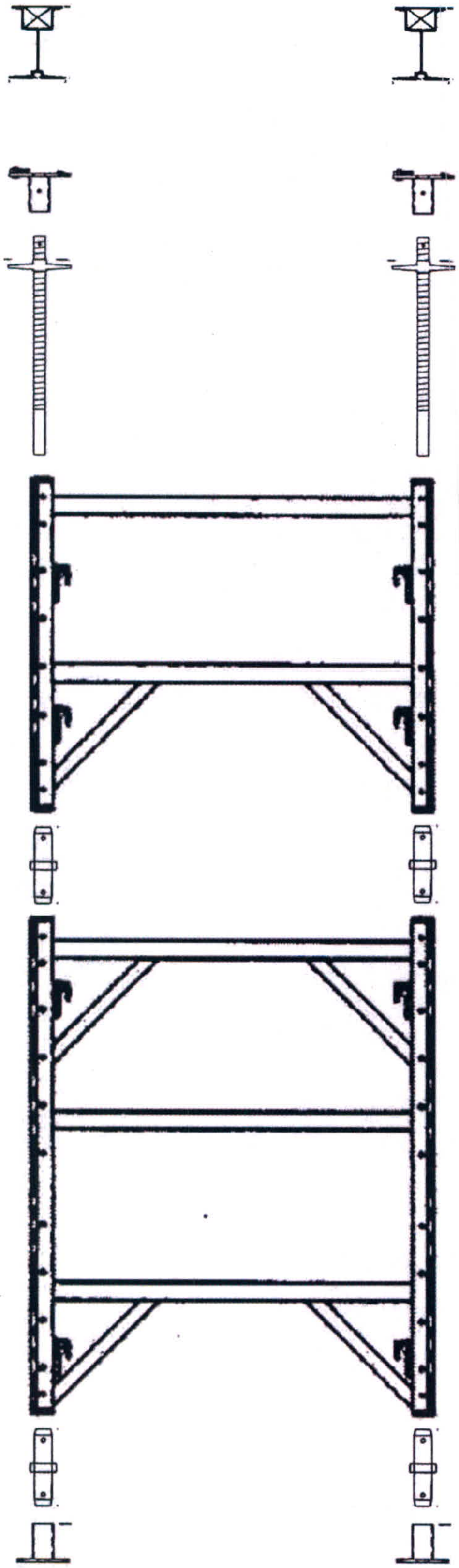
SH 41117
Bottom Frame of SH 41128
Zone 41E

Folder: 08-02-12; 08-02-12; SH 41117



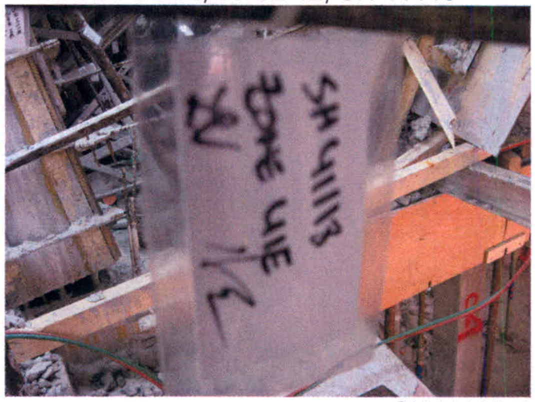

SH41117_1


SH41117_2



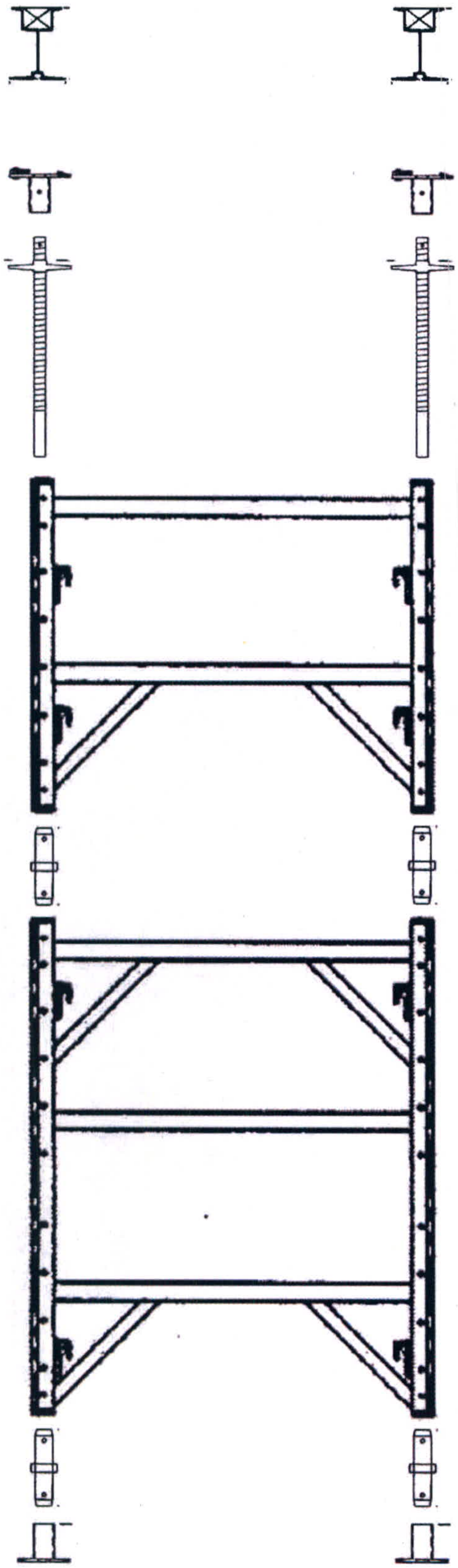
SH 41118
Bottom Brace
Connecting SH 41119 and SH 41116
Zone 41E-A

Folder: 08-02-12; 08-02-12; SH 41118



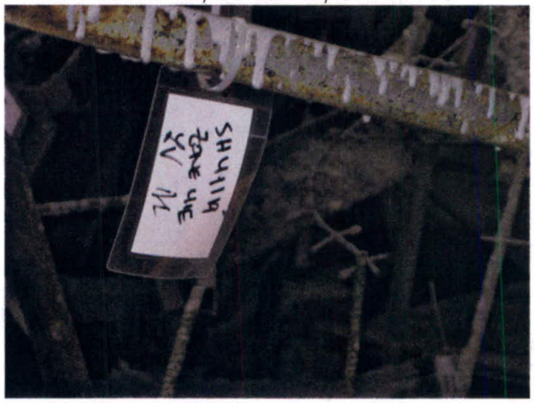
SH41118_1

SH41118_2



SH 41119
Bottom Brace
Connecting SH 41117 and SH 41116
Zone 41E-A

Folder: 08-02-12; 08-02-12; SH 41119

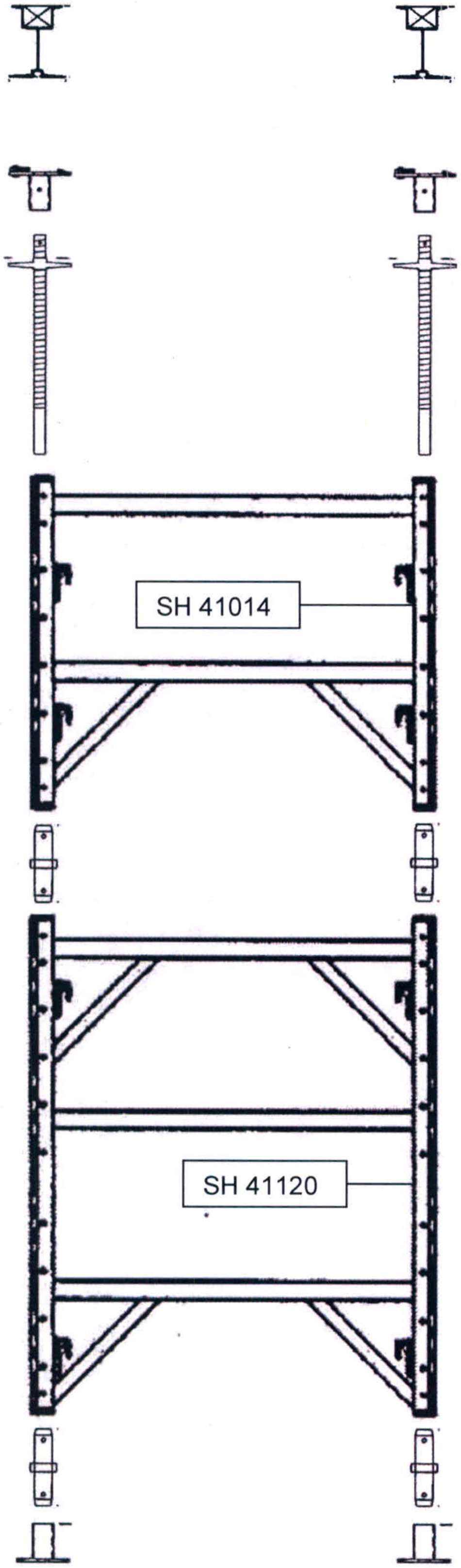


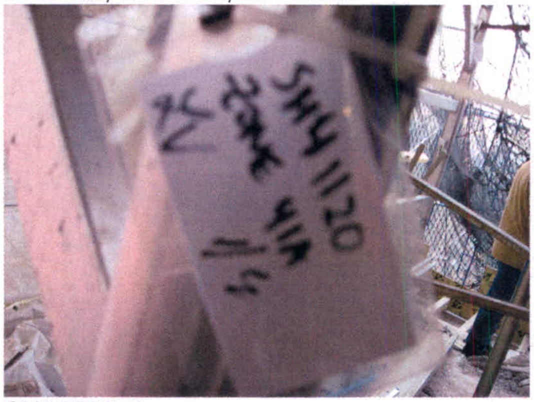
SH41119_1

SH41119_2

SH 41120
Bottom Frame of SH 41014
Connects to brace SH 41122
Zone 41A

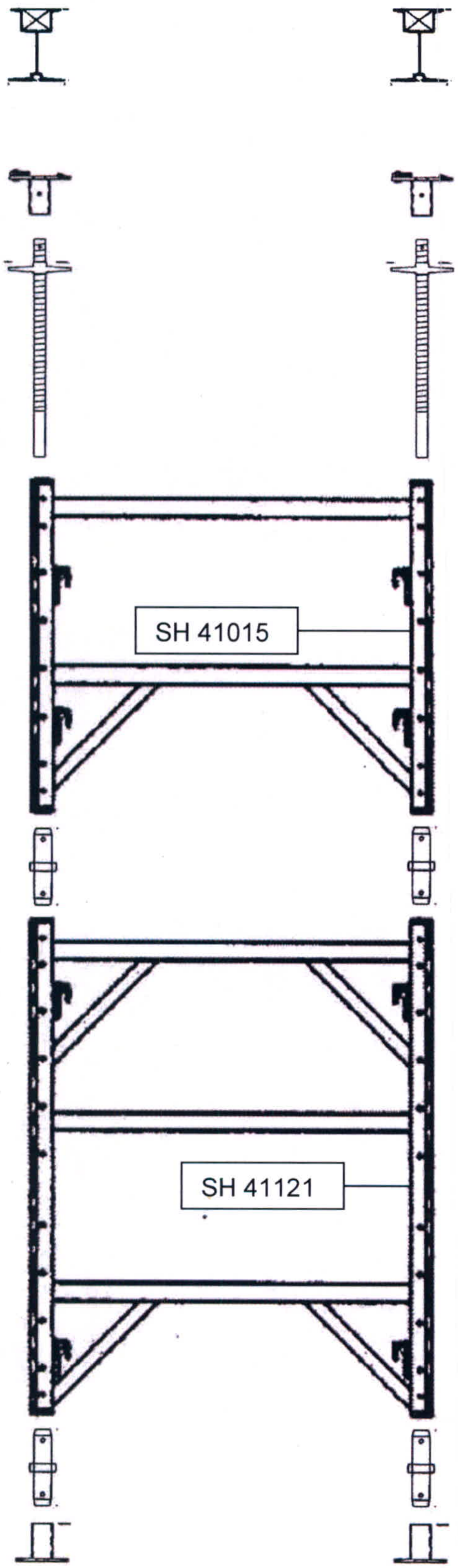
Folder: 08-02-12; 08-02-12; SH 41020





SH41120_1

SH41120_2



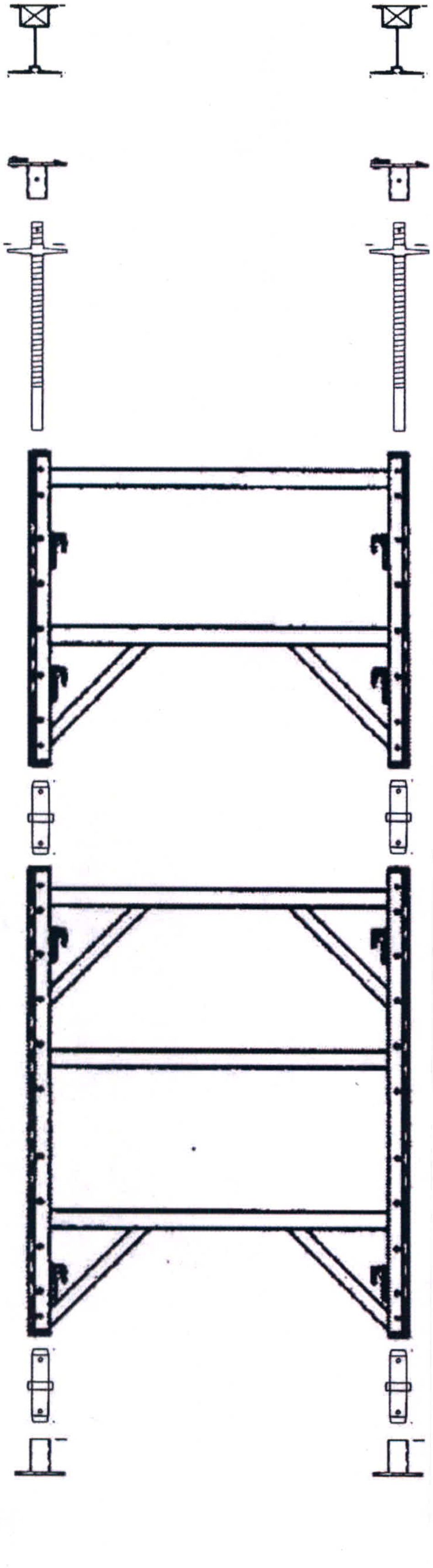
SH 41121
Bottom Shore of SH 41015
Connects by SH
Zone 41A

Folder: 08-02-12; 08-02-12; SH 41121

SH41121_1

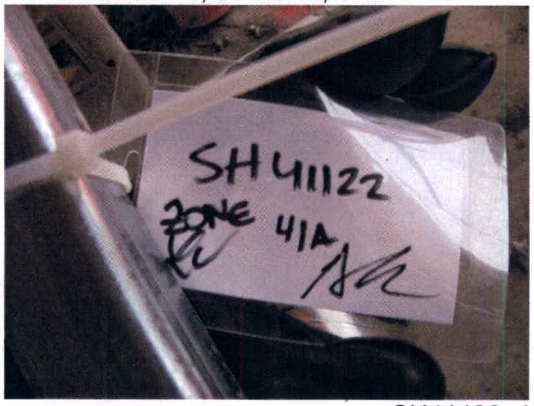
SH41121_2

SH41121_3



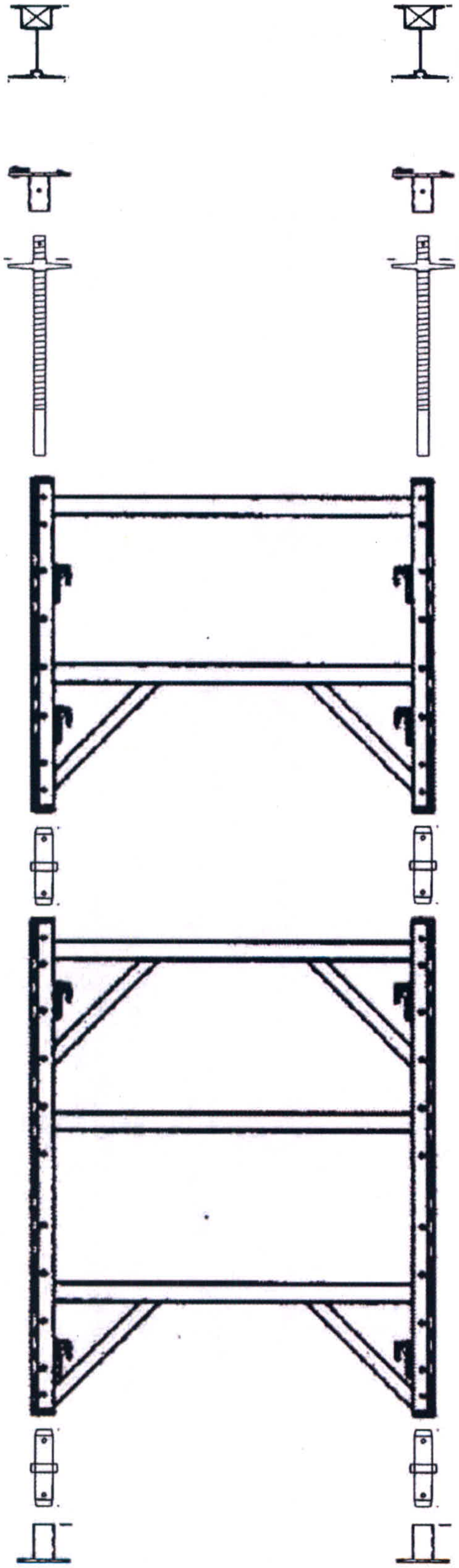
SH 41122
Bottom Brace
Connects SH 41121 & SH 41120
Zone 41A

Folder: 08-02-12; 08-02-12; SH 41122



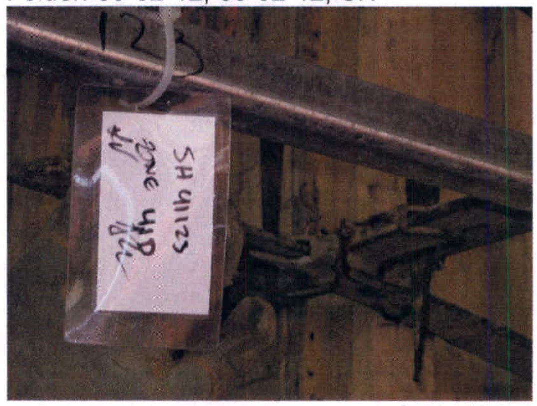
SH41122_1

SH41122_2



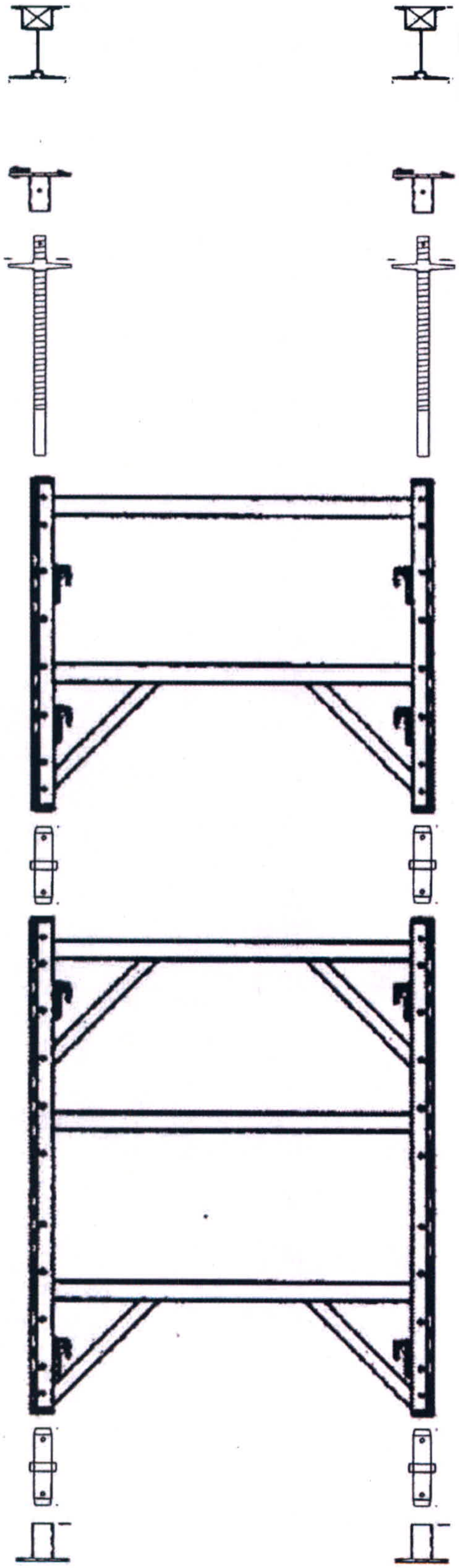
SH 41123
Top Brace
Connects SH 41110 to SH 41112
Zone 41D

Folder: 08-02-12; 08-02-12; SH



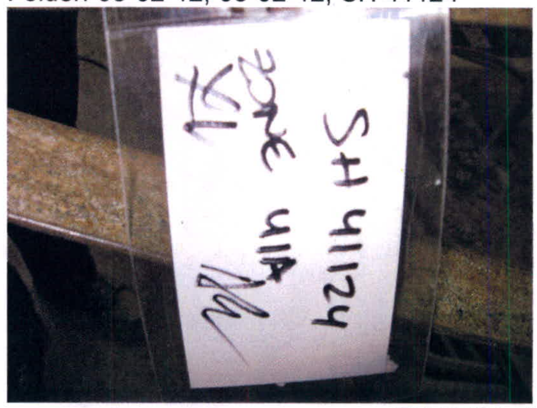
SH41123_1

SH41123_2



SH 41124
Bottom Brace
Connects SH 41125 & SH 41116
Zone 41A

Folder: 08-02-12; 08-02-12; SH 41124

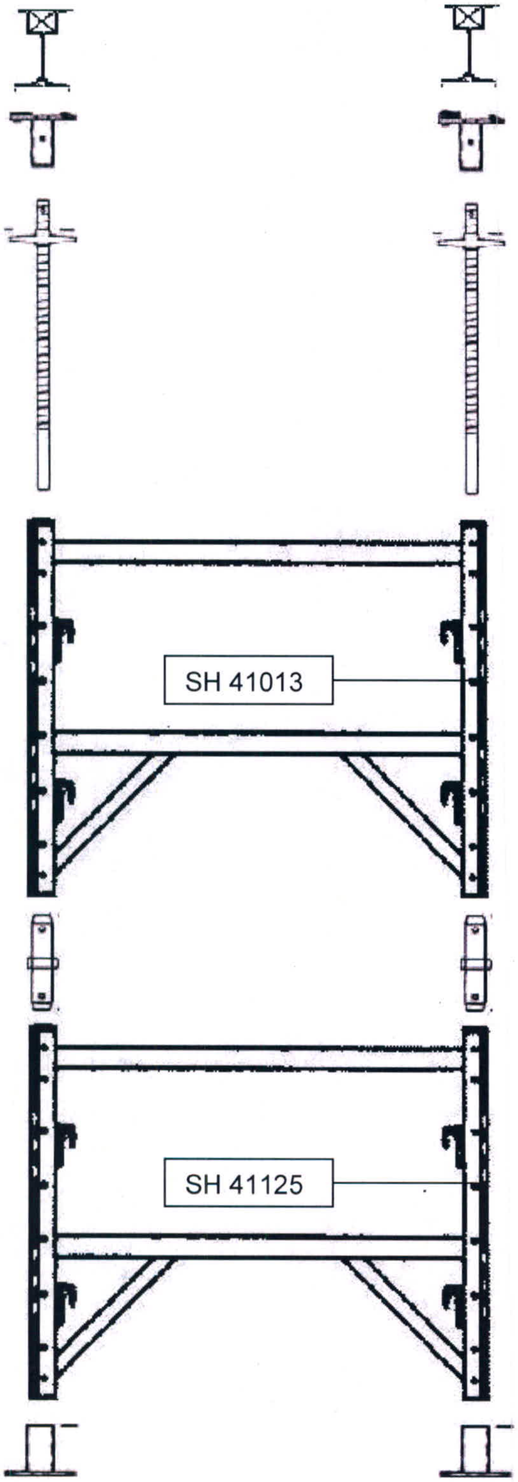


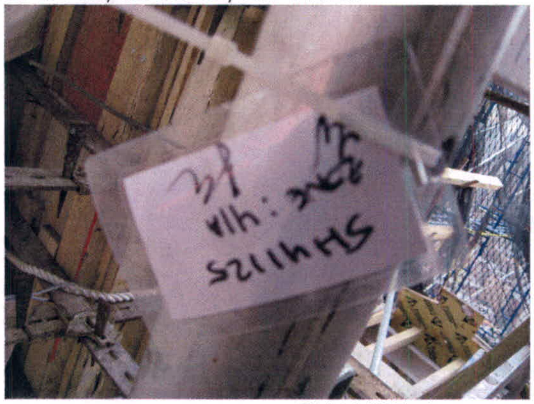
SH41124_1

SH41124_2

SH 41125
Bottom Shore of SH 41013
Connect to Brace SH 41124
Zone 41A

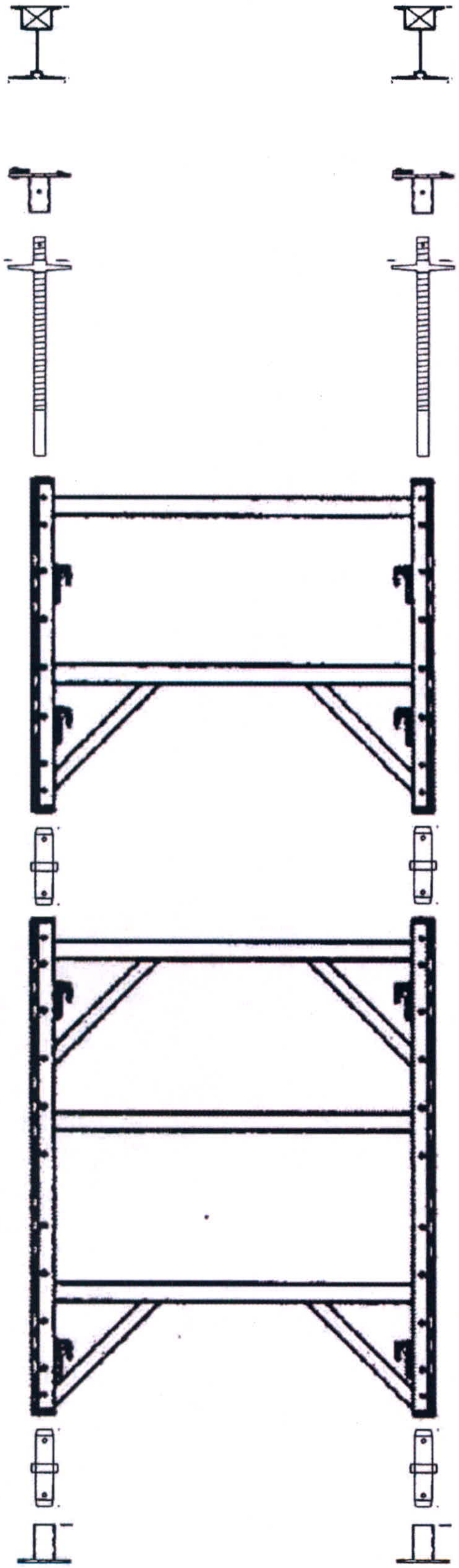
Folder: 08-02-12; 08-02-12; SH 41125





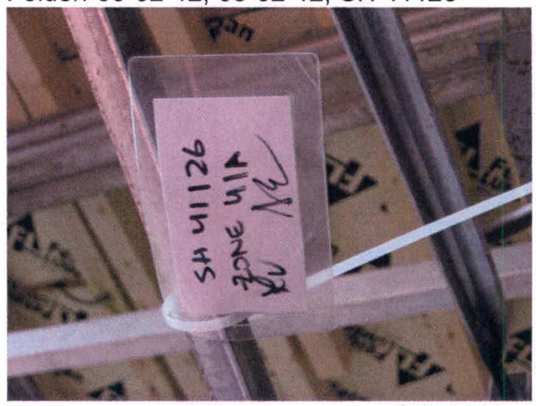
SH41125_1

SH41125_2



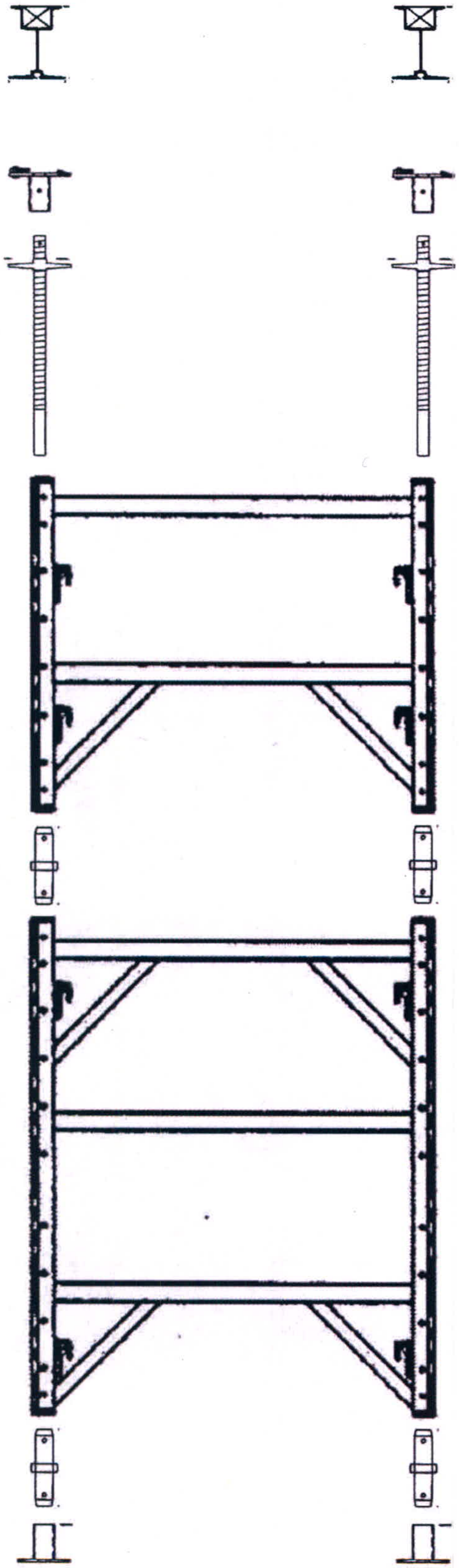
SH 41126
Brace
Connects SH 41014 and SH 41012
Zone 41A

Folder: 08-02-12; 08-02-12; SH 41126



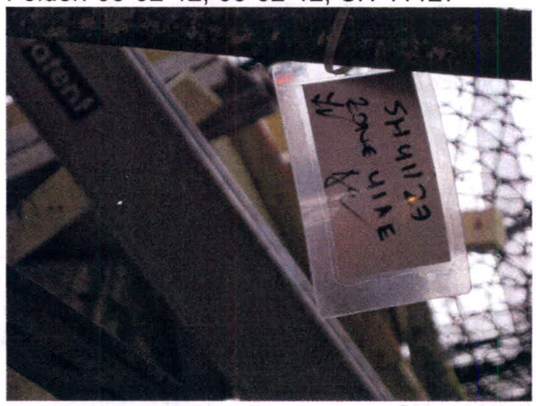
SH41126_1


SH41126_2



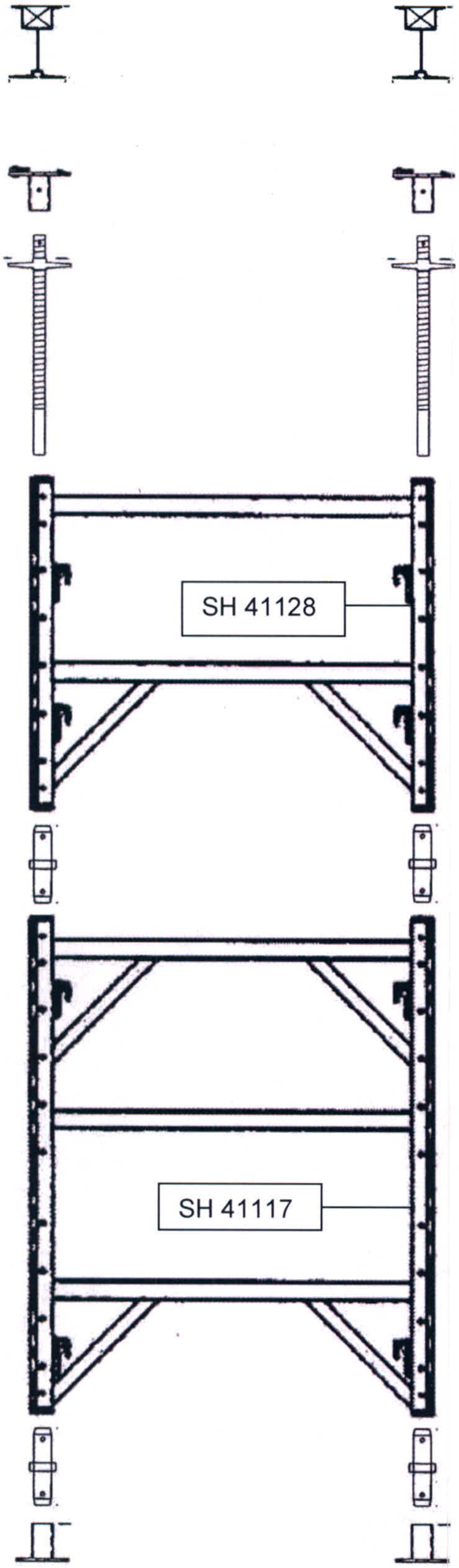
SH 41127
Top Brace
Top Shore SH 41128 Bottom Shore SH
41117 Connecting SH 41011
Zone 41E-A

Folder: 08-02-12; 08-02-12; SH 41127



SH41127_1

SH41127_2



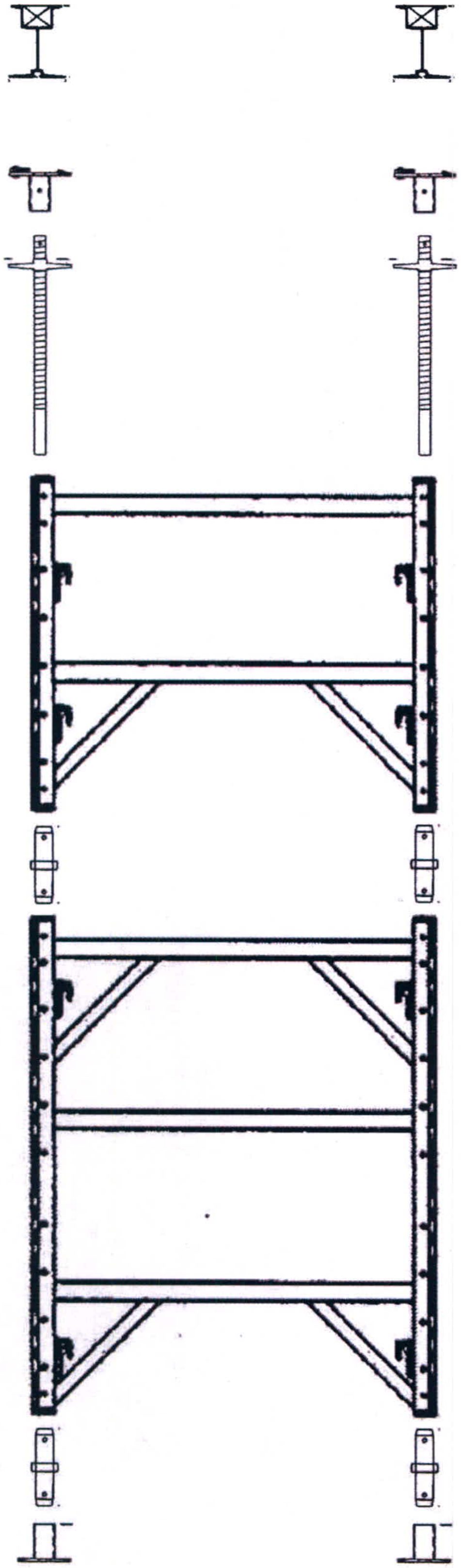
SH 41128
Top Shore of SH 41117
Zone 41B

Folder: 08-02-12; 08-02-12; SH 41128

SH41128_1

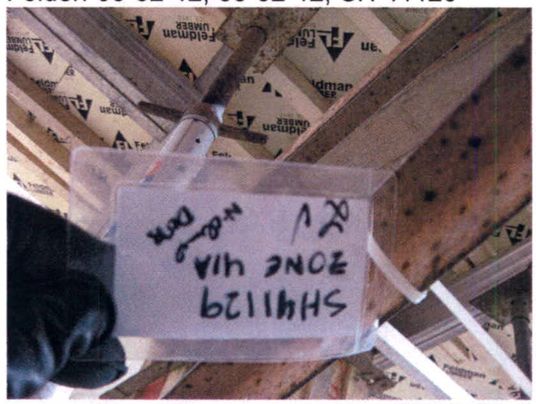
SH41128_2

SH41128_3



SH 41129
Top Brace
Connect to SH 41128
Zone 41

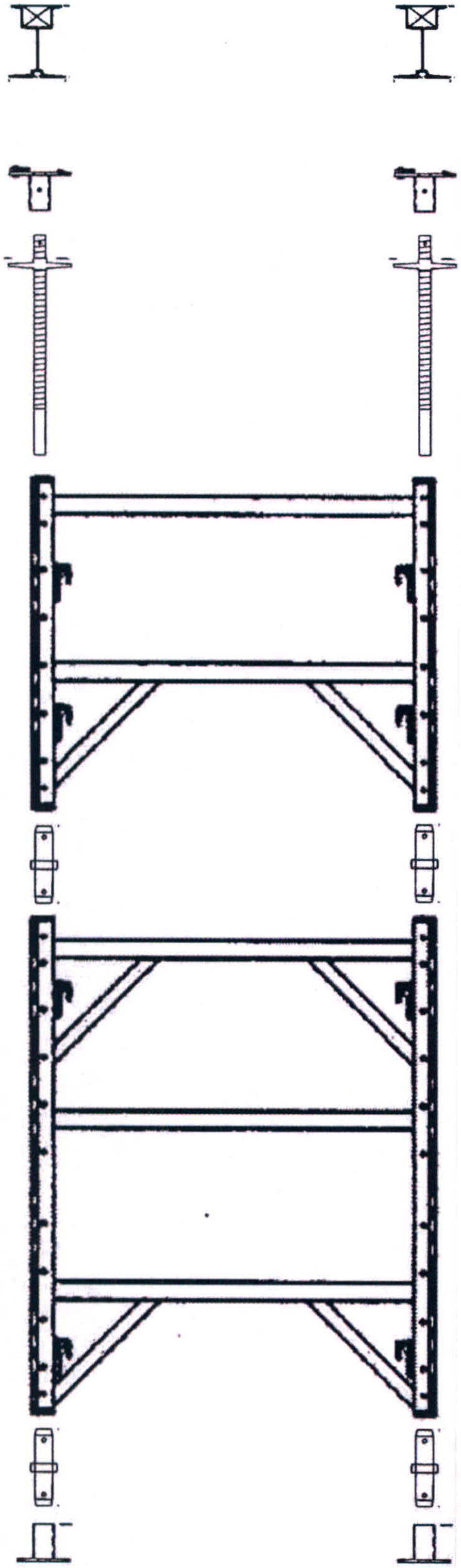
Folder: 08-02-12; 08-02-12; SH 41129



SH41129_1

SH41129_2

SH41129_3



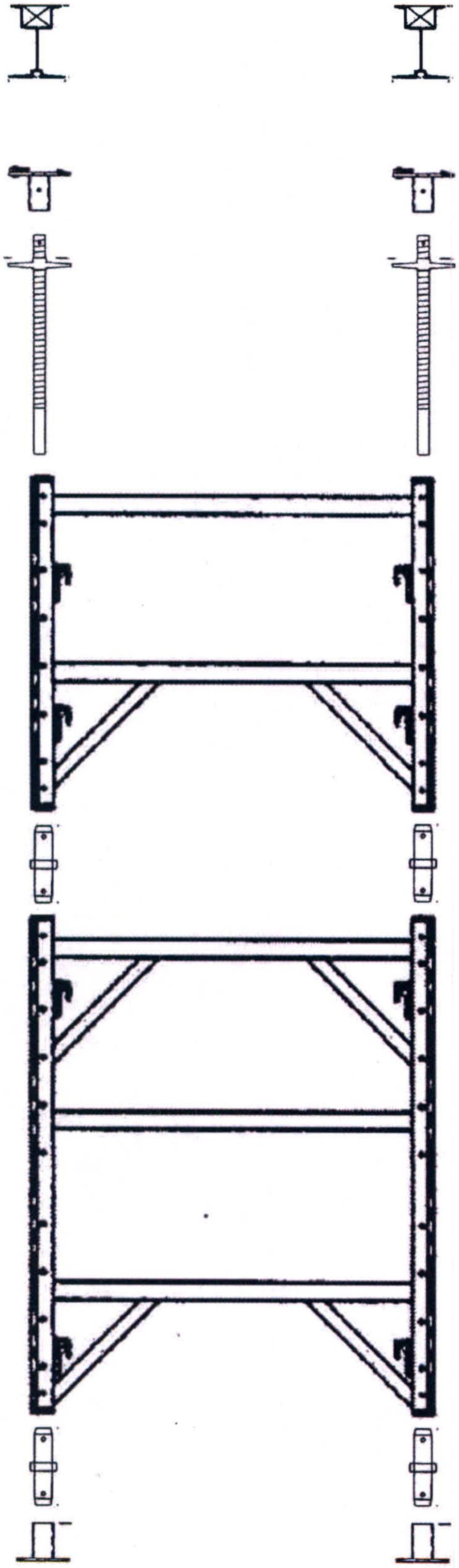
SH 41130
Brace
Connect to SH 41128
Zone 41B

Folder: 08-02-12; 08-02-12; SH 41130

SH41130_3

SH41130_4

SH41130_6



SH 41131
Top plate
Zone 41G

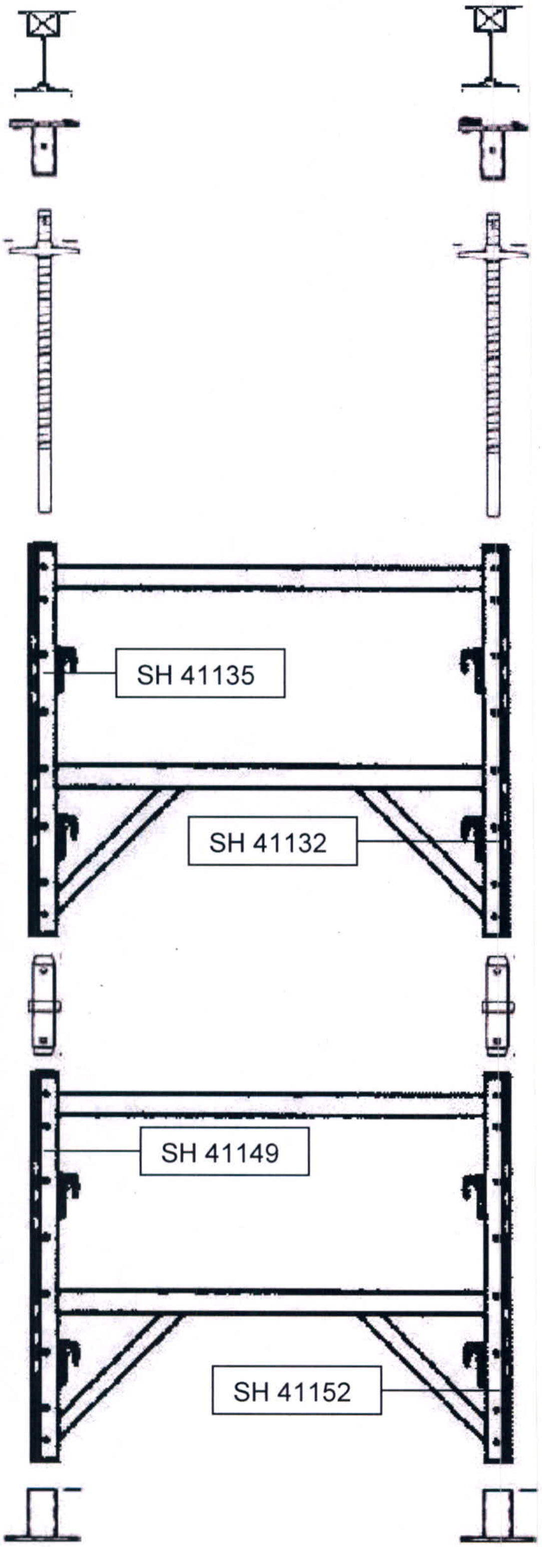
Folder: 08-02-13; 08-02-13; SH 41131

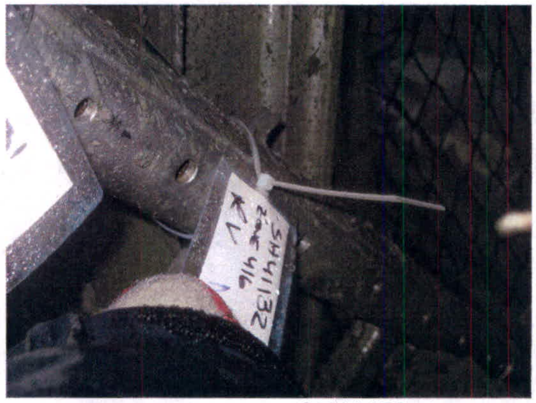
SH41131_1

SH41131_2

SH 41132
Top Frame of SH 41152 and SH 41149
Frame Connects to SH 41135
Zone 41G

Folder: 08-02-13; 08-02-13; SH 41132



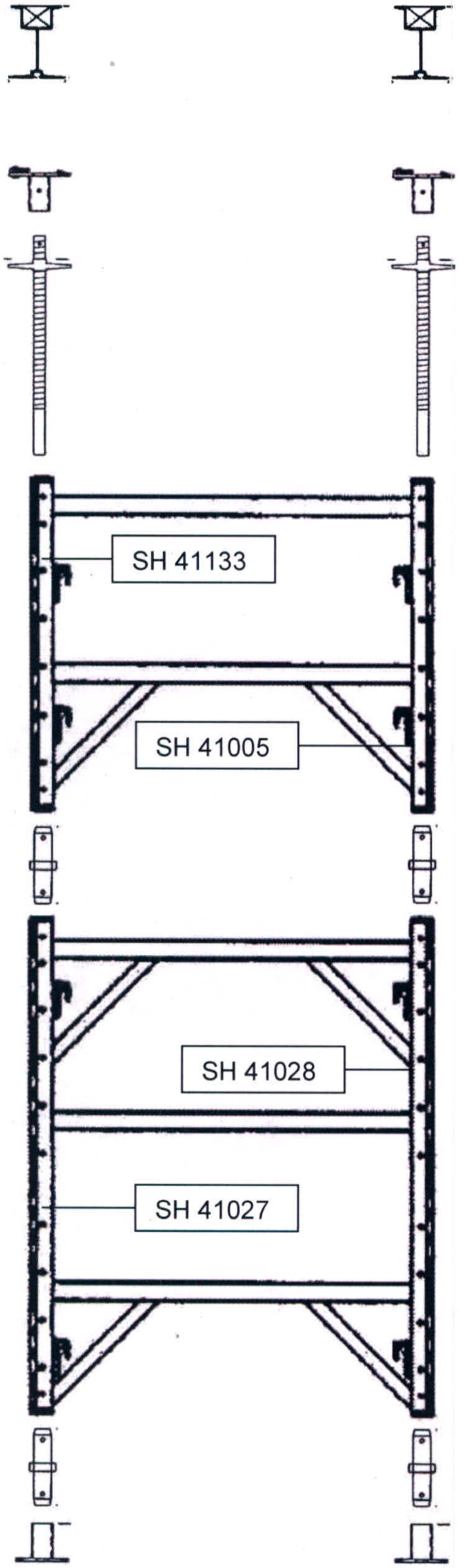


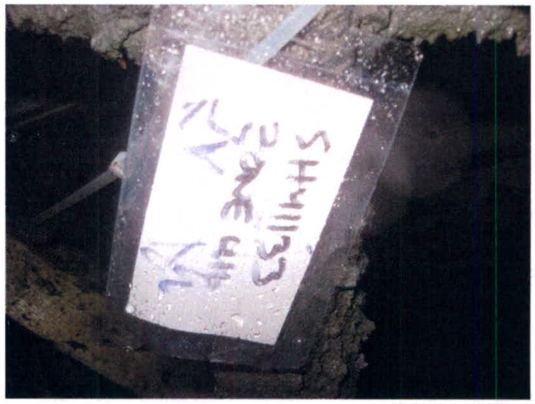
SH41132_1

SH41132_2

SH 41133
Top Frame
Frame connecting SH 41026 and AB 41001
Zone 41G

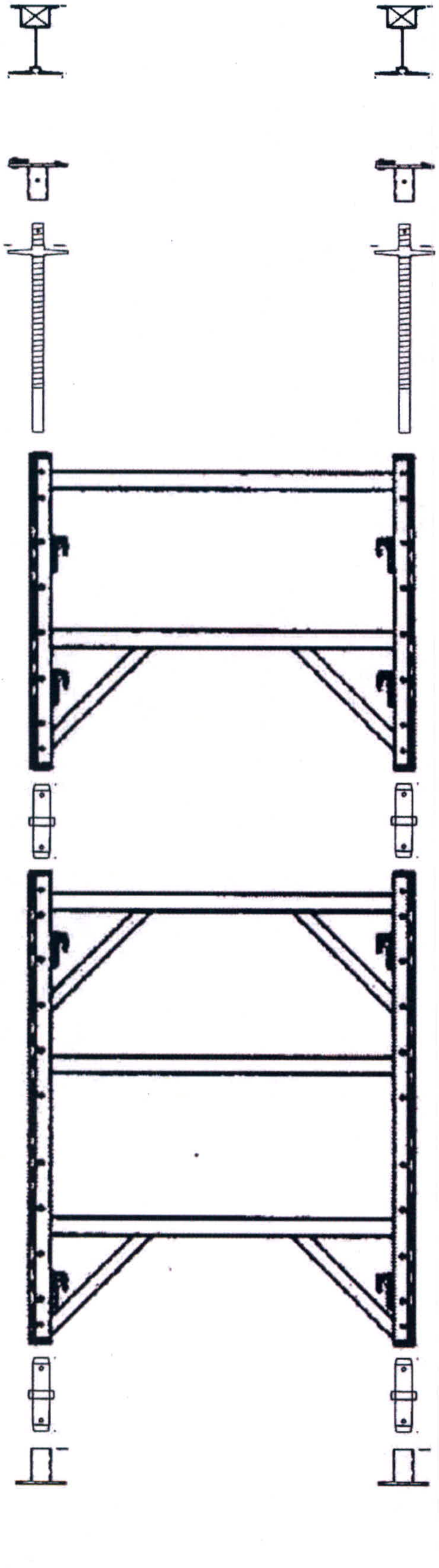
Folder: 08-02-13 SH 41133





SH41133_1

SH41133_2



SH 41134
Brace
Connects SH 41133
Zone 41G

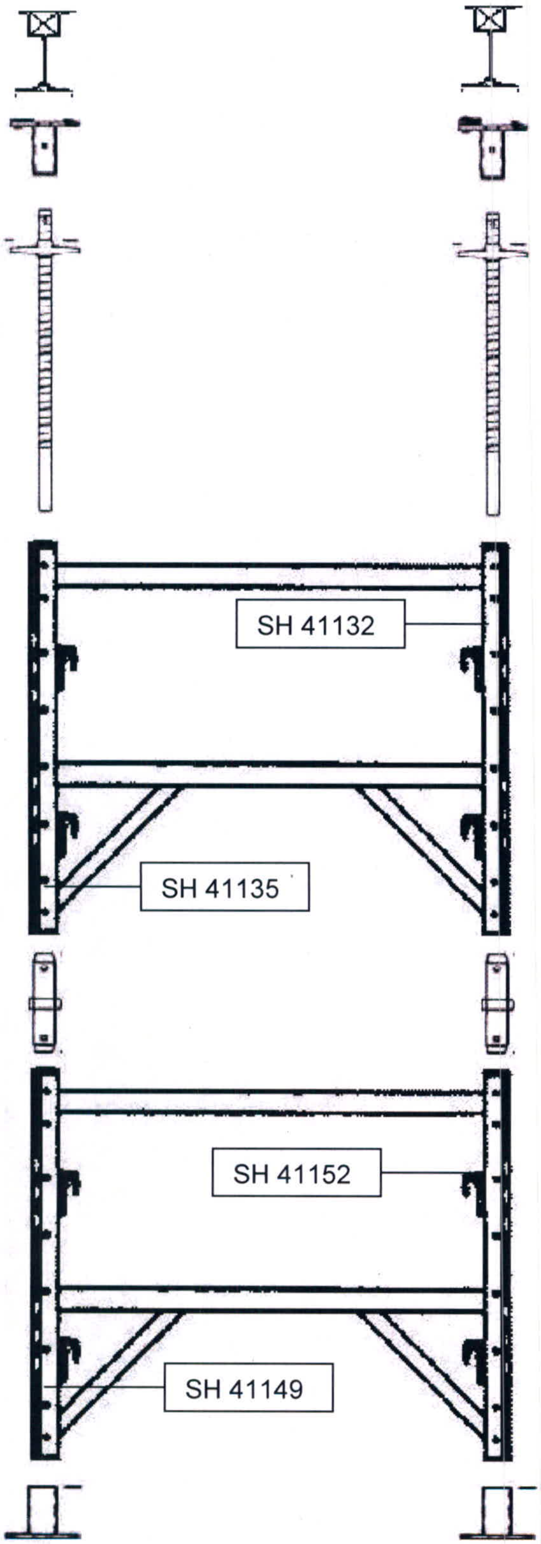
Folder: 08-02-13; 08-02-13; SH 41134

SH41134_1

SH41134_2

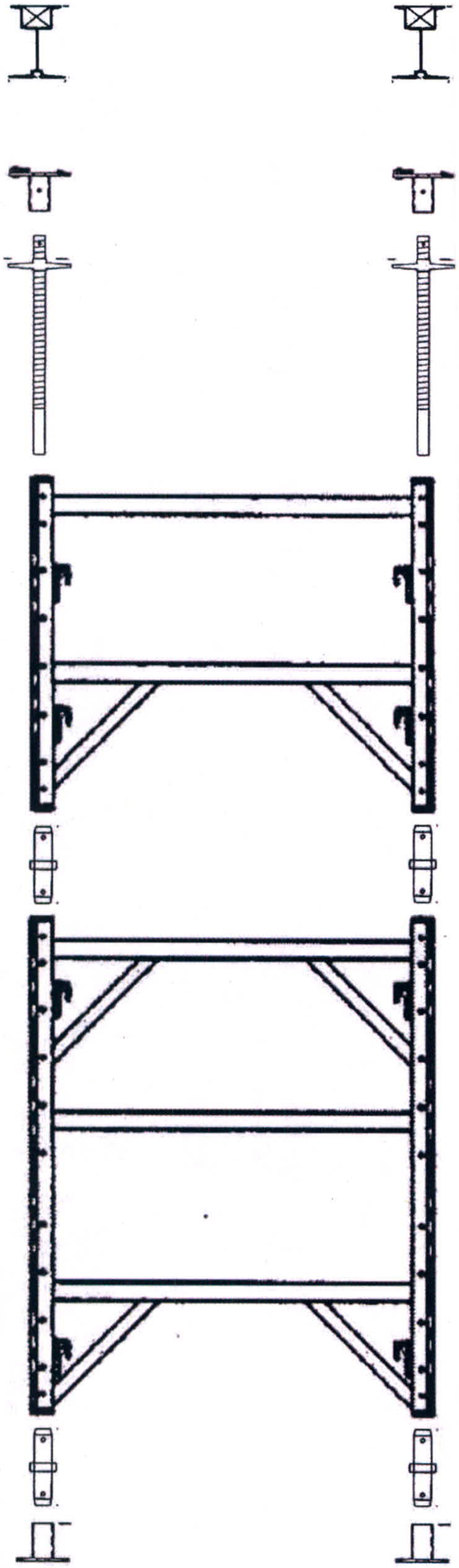
SH 41135
Frame Connects to AB 41002
Zone 41G

Folder: 08-02-13; 08-02-13; SH 41135



SH41135_1

SH41135_2



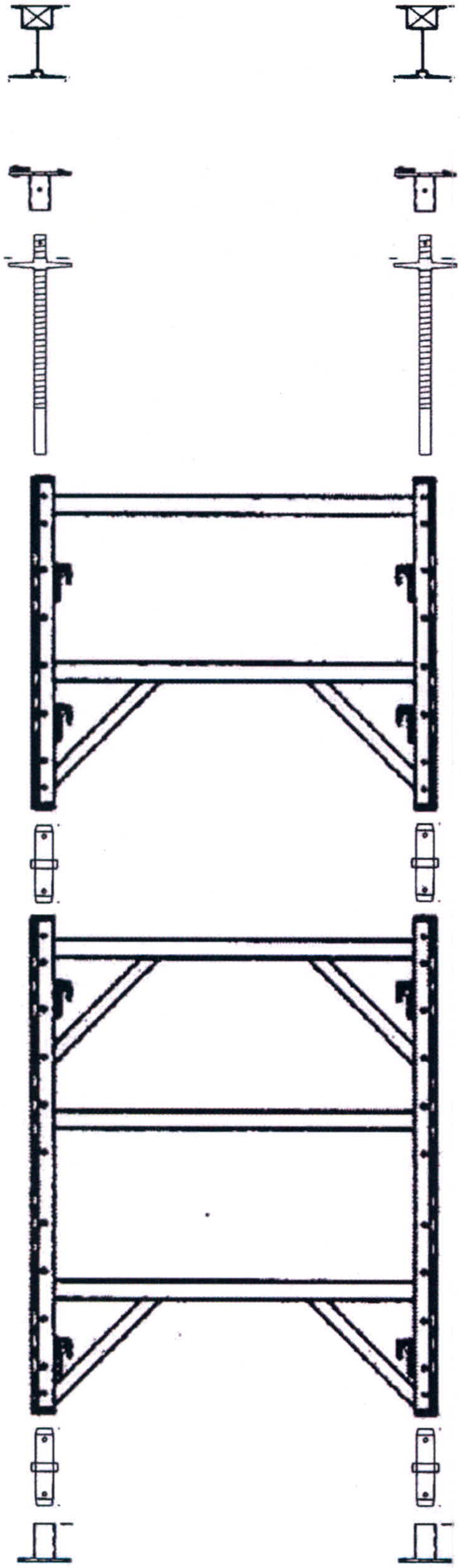
SH 41136
Screw Jack Leg SH 41042
Zone 41G

Folder: 08-02-13; 08-02-13; SH 41136

SH41136_1

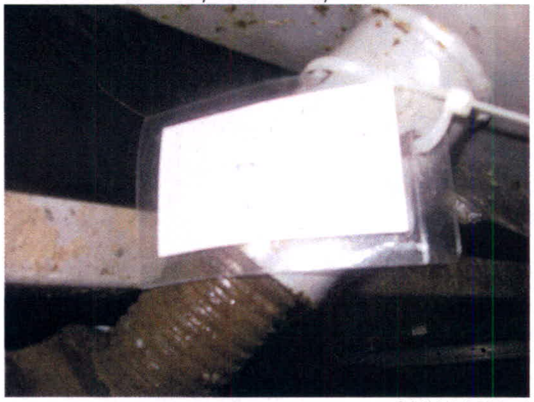
SH41136_2

SH41136_3

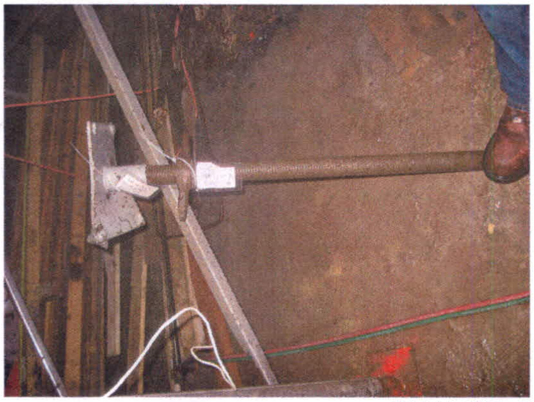


SH 41137
Top plate attached to PW 41006
Zone 41F

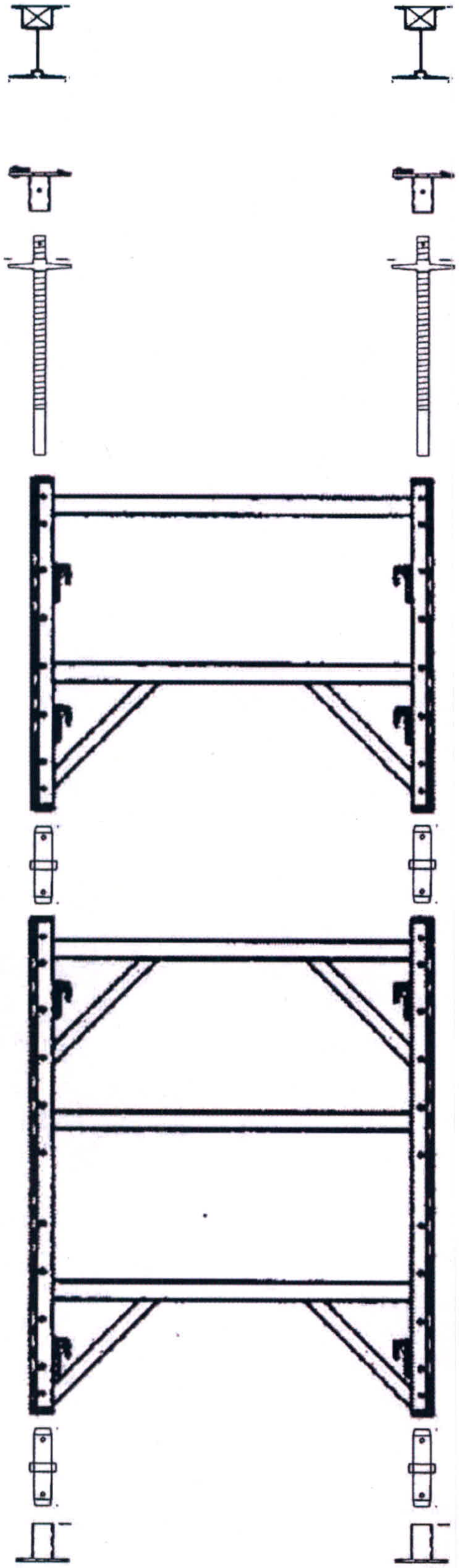
Folder: 08-02-13; 08-02-13; SH 41137



SH41137_1

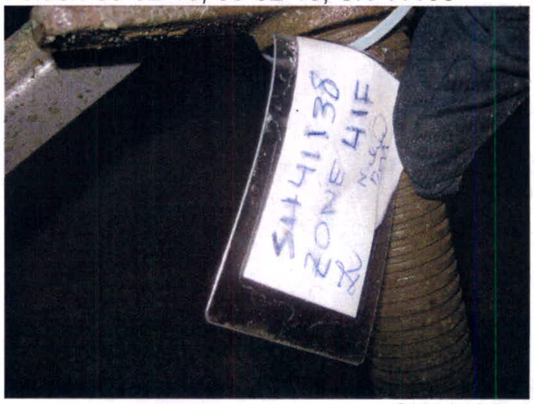


SH41137_2



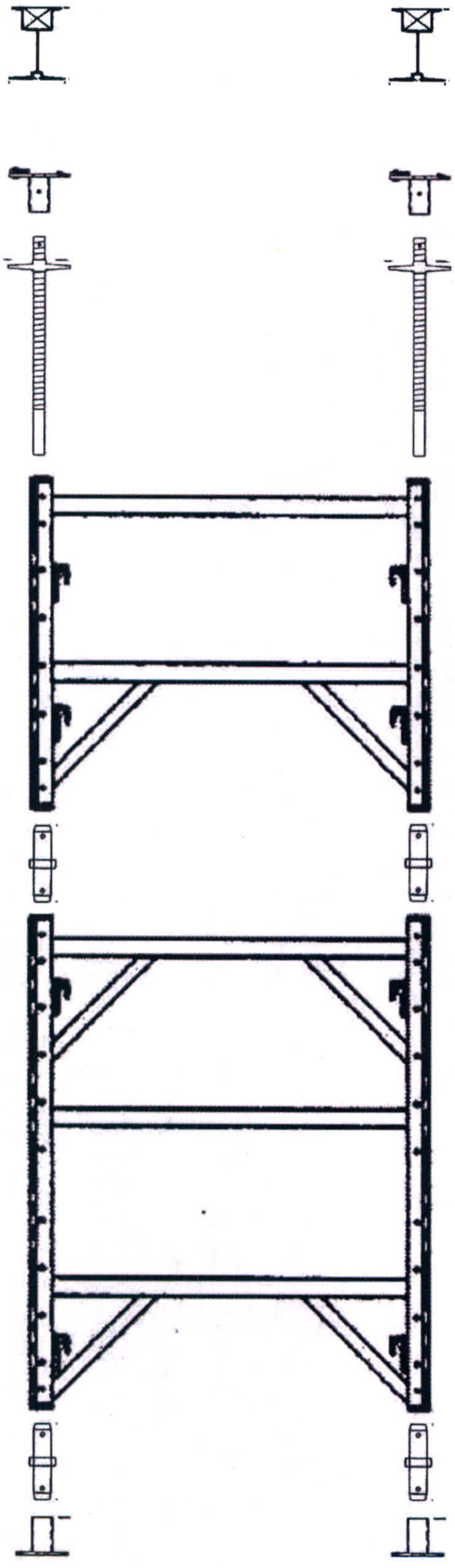
SH 41138
Screw Jack Leg
Zone 41F

Folder: 08-02-13; 08-02-13; SH 41138



SH41138_1

SH41138_2



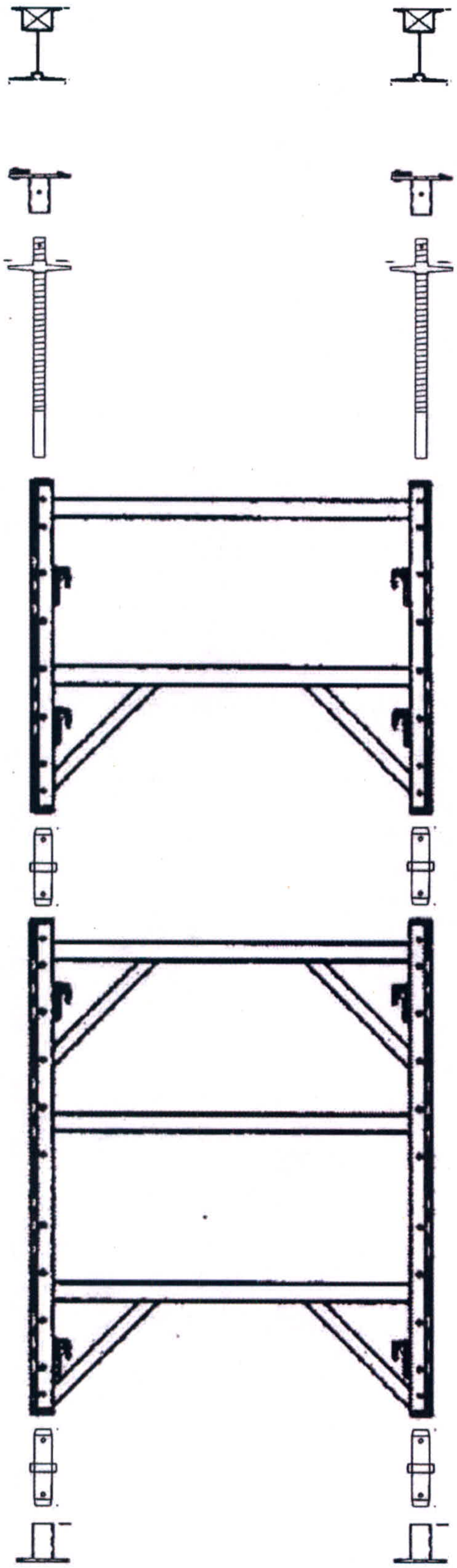
SH 41139
Brace
Zone 41F

Folder: 08-02-14; 08-02-14; SH 41139

SH41139_1

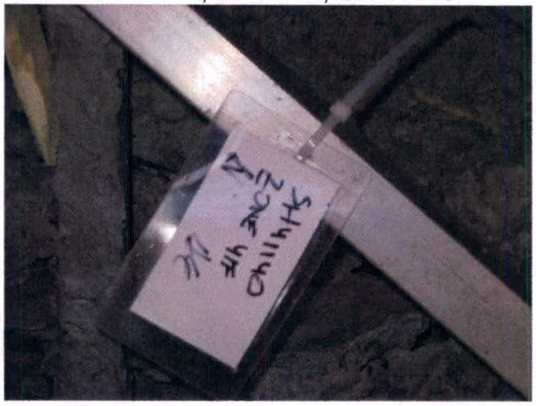
SH41139_2

SH41139_3



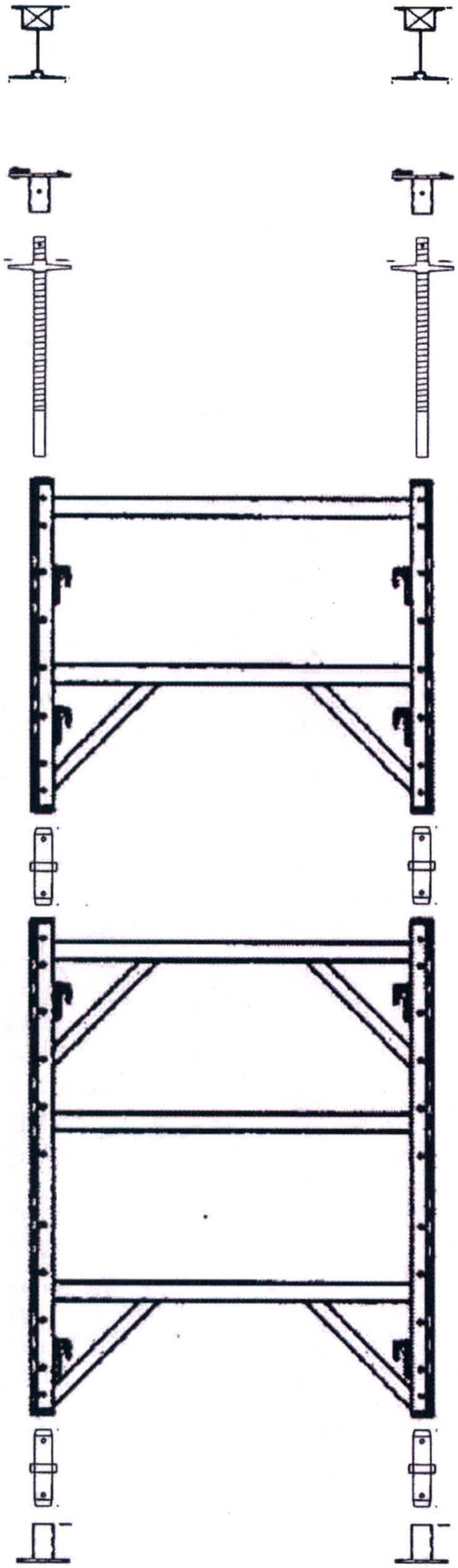
SH 41140
Piece of Frame
Zone 41F

Folder: 08-02-14; 08-02-14; SH 41140



SH41140_1

SH41140_2



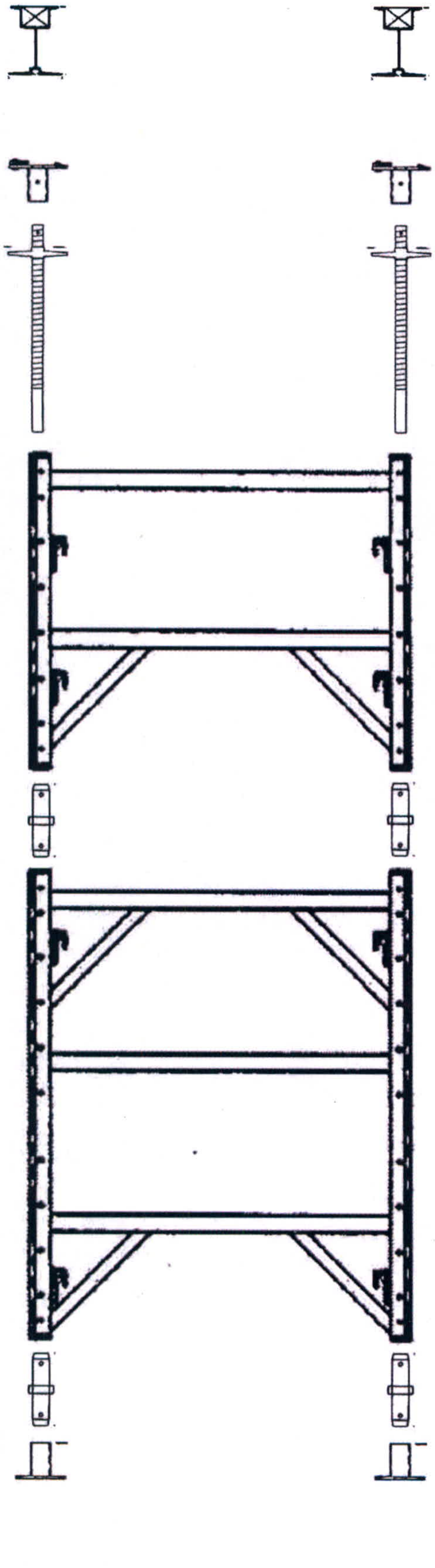
SH 41141
Brace
Zone 41F

Folder: 08-02-14; 08-02-14; SH 41141

SH41141_1

SH41141_2

SH41141_3



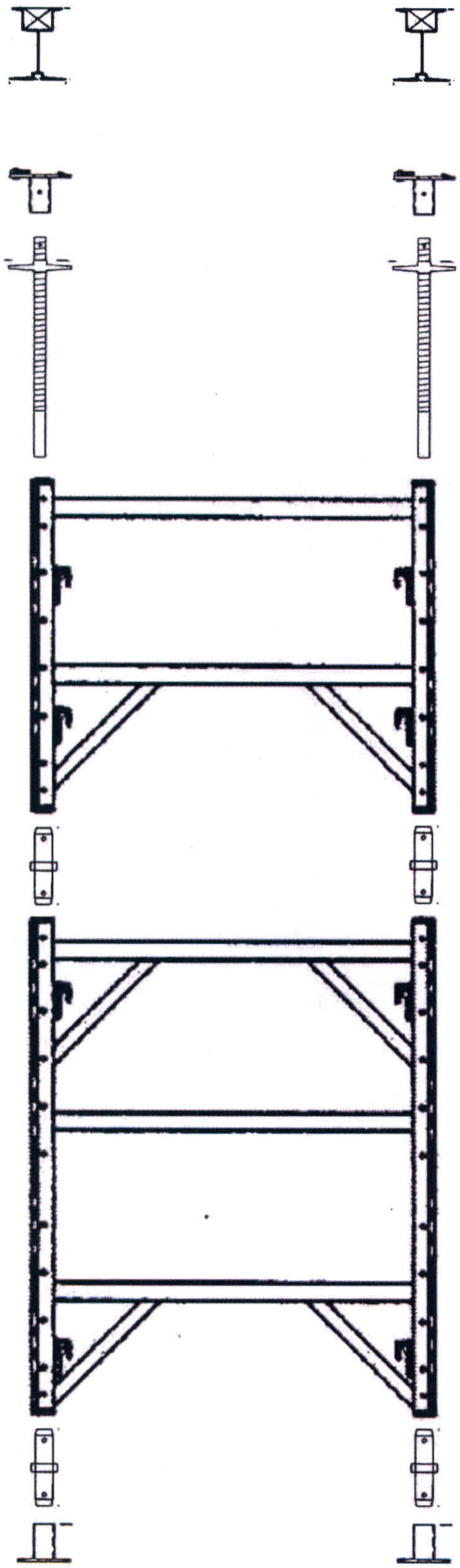
SH 41142
Frame connects to SH 41029 and SH 41039
Zone 41F

Folder: 08-02-14; 08-02-14; SH 41142

SH41142_1

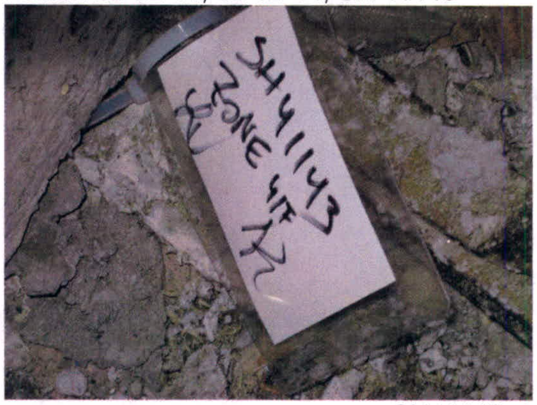
SH41142_2

SH41142_3



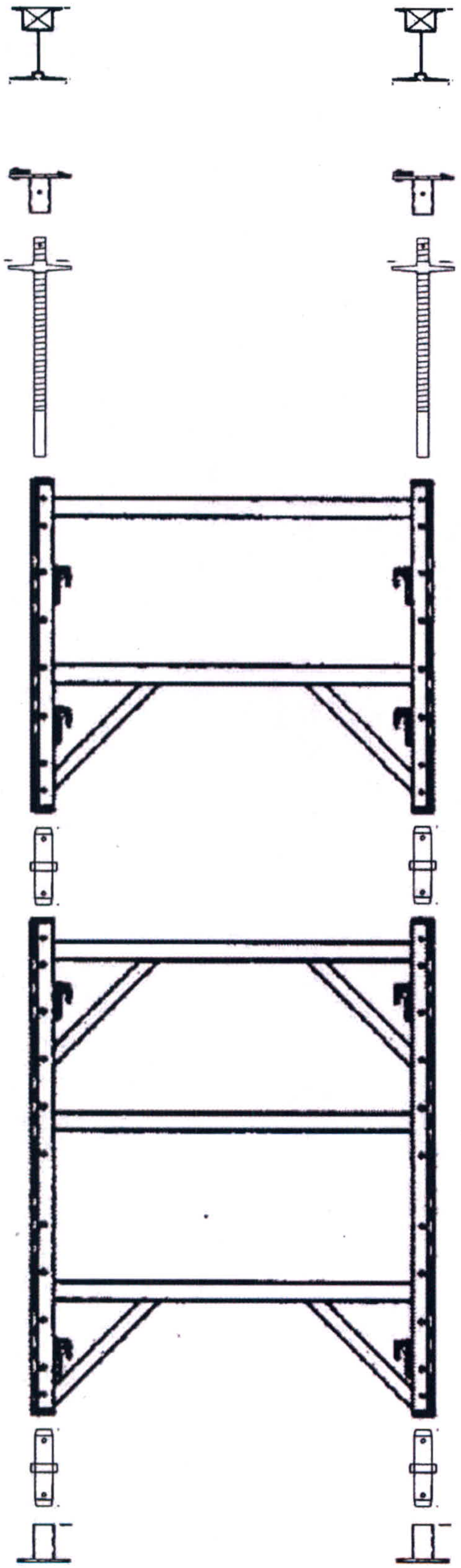
SH 41143
Brace
Zone 41F

Folder: 08-02-14; 08-02-14; SH 41143

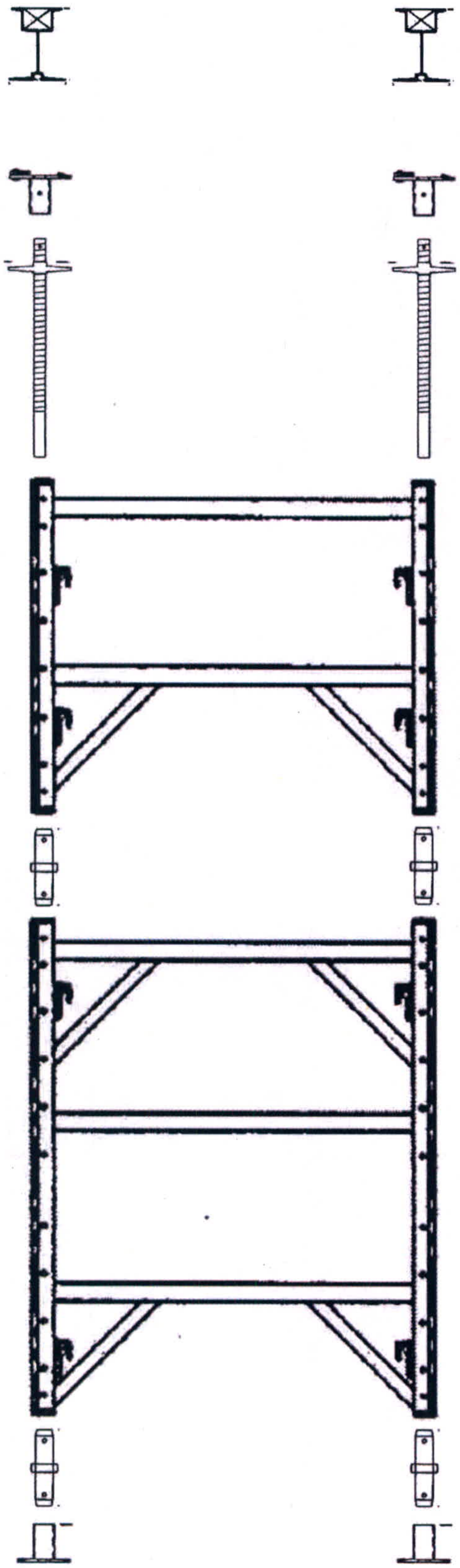


SH41143_1

SH41143_2

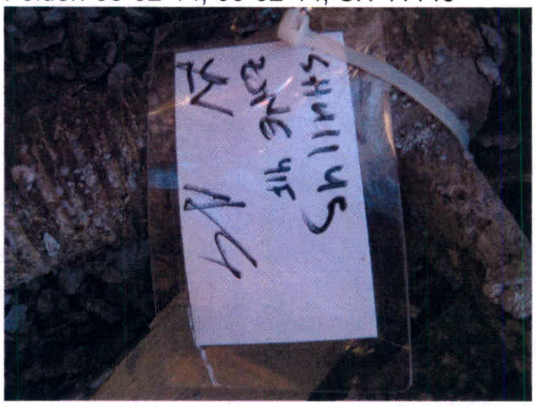


SH 41144
Brace
No photo
Zone 41F



SH 41145
Top Screw Jack
Zone 41G

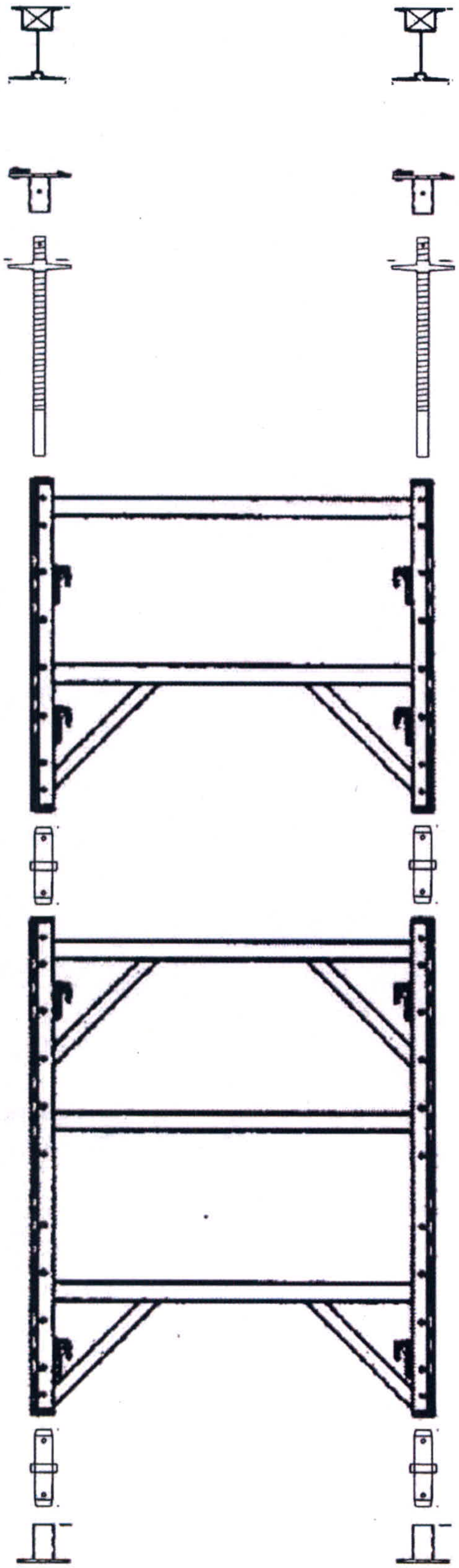
Folder: 08-02-14; 08-02-14; SH 41145



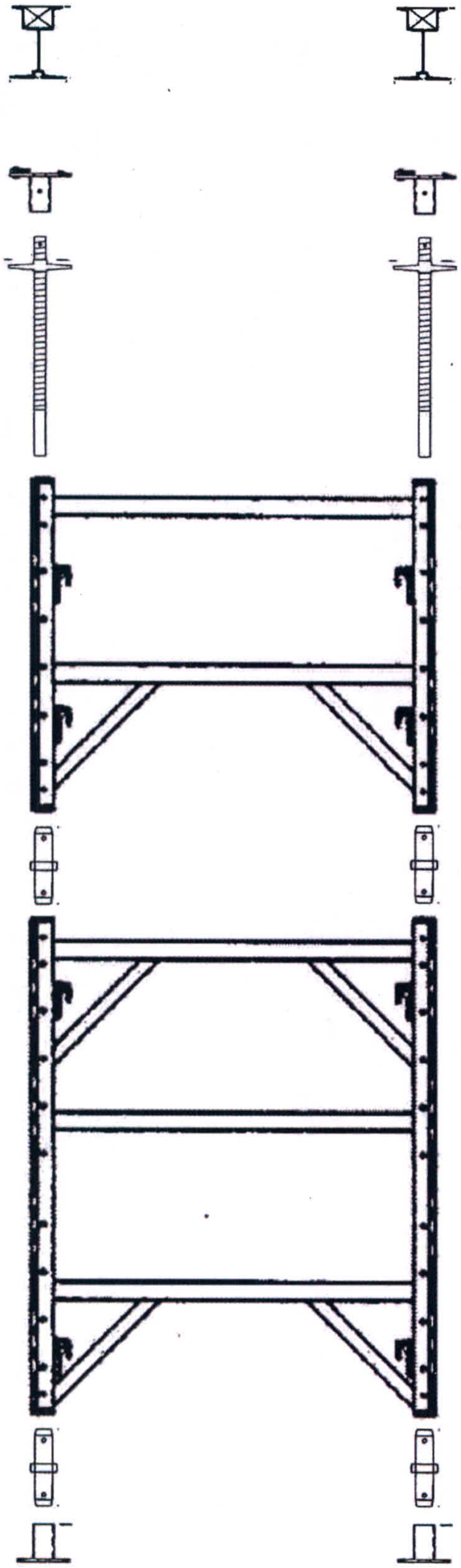
SH41145_1

SH41145_2

SH41145_4

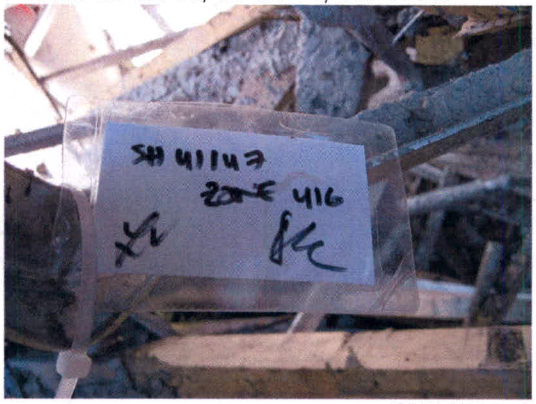


SH 41146
Tag skipped

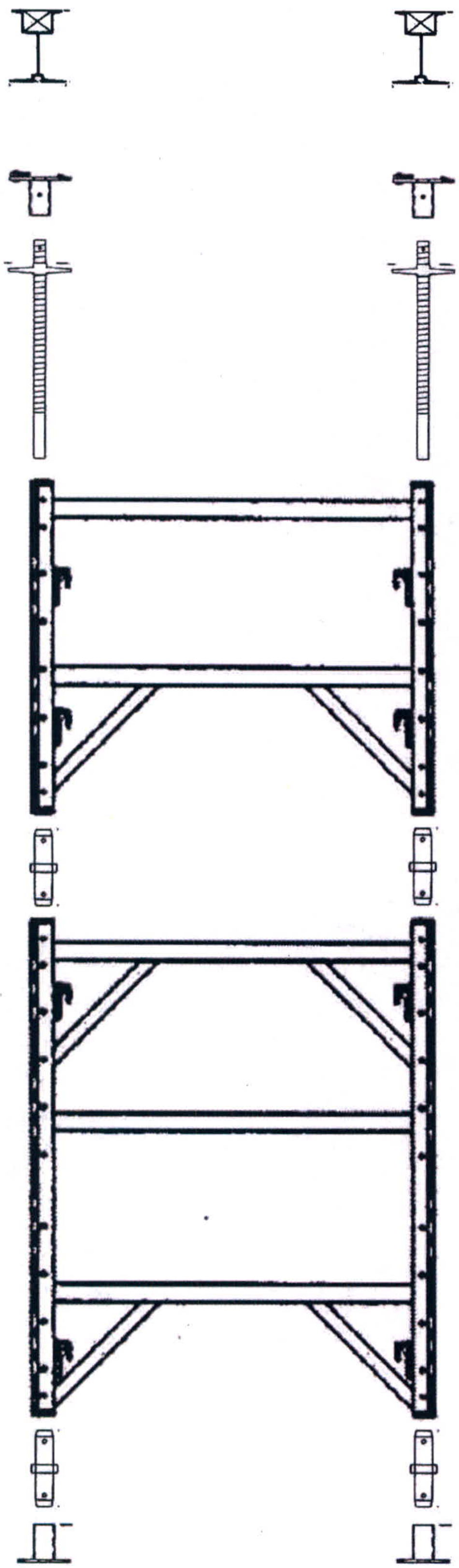


SH 41147
Brace
Attached to SH 41135
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41147

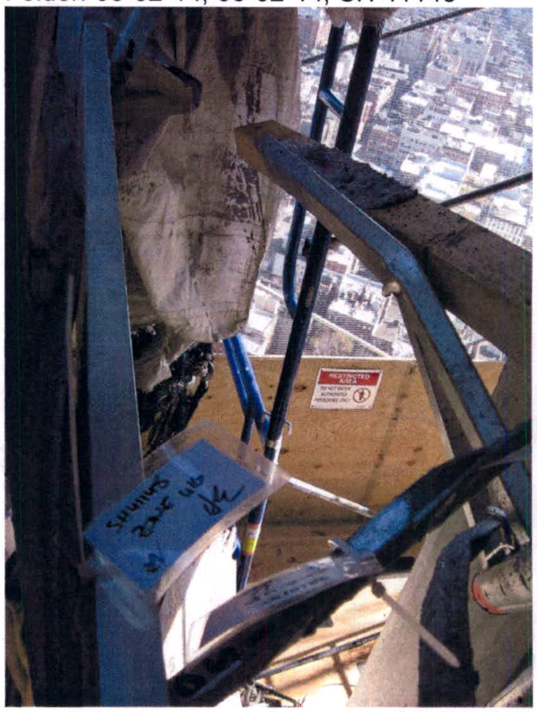


SH41147_1

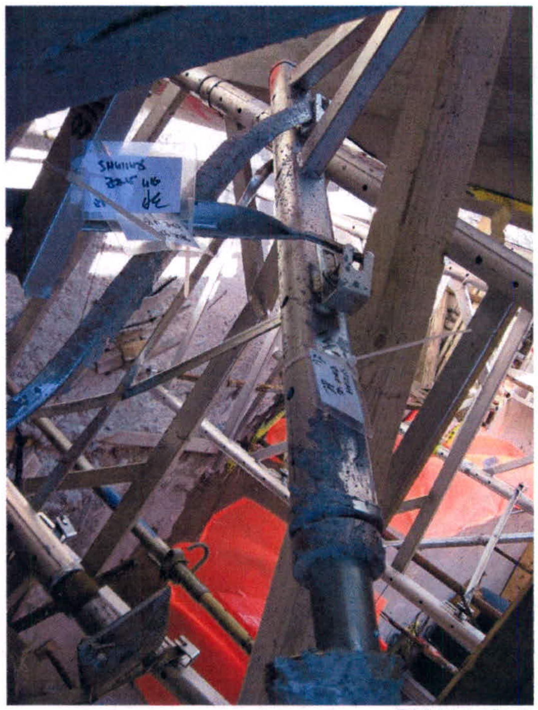


SH 41148
Piece of frame
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41148



SH41148_1



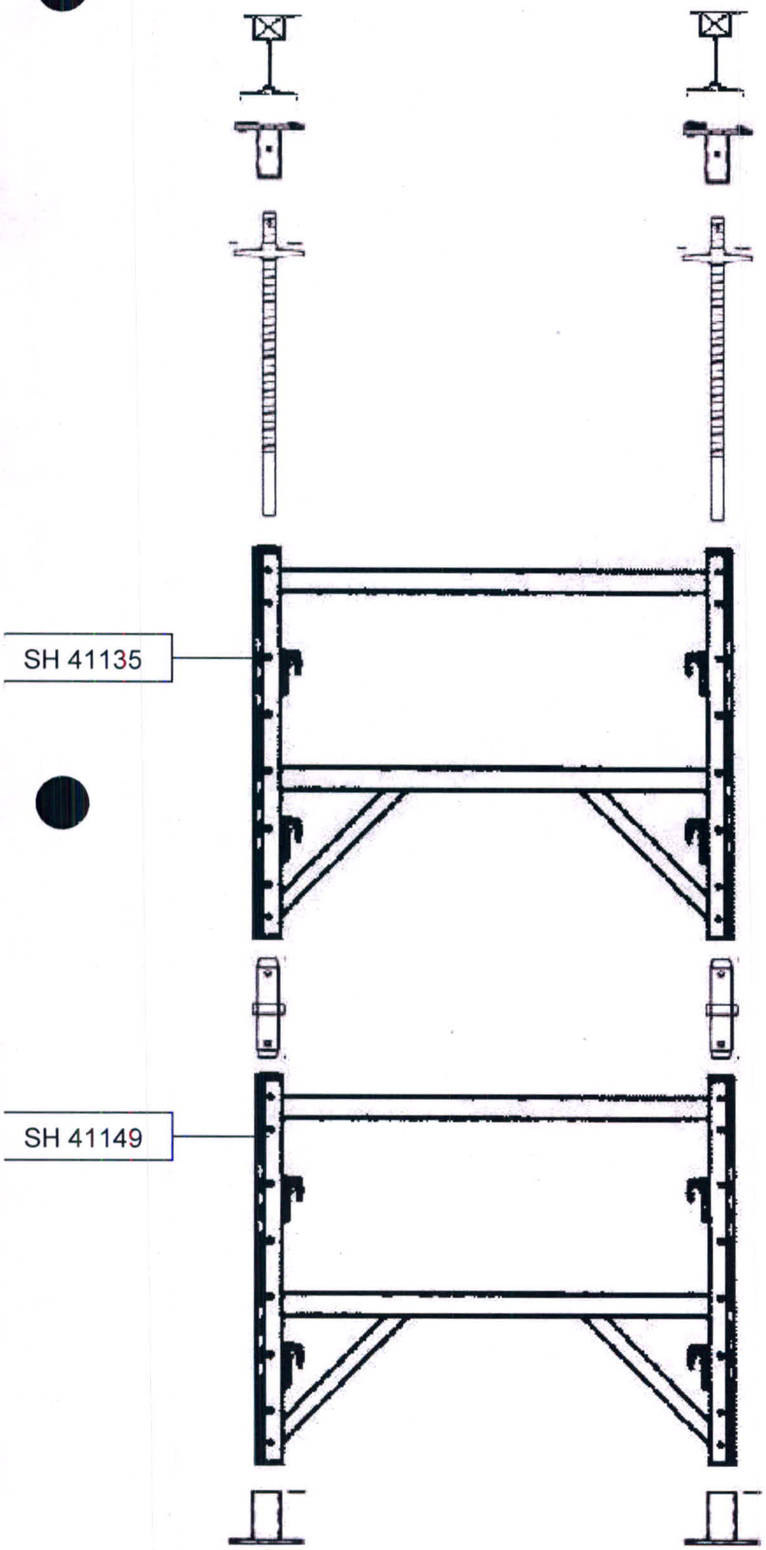
SH41148_2

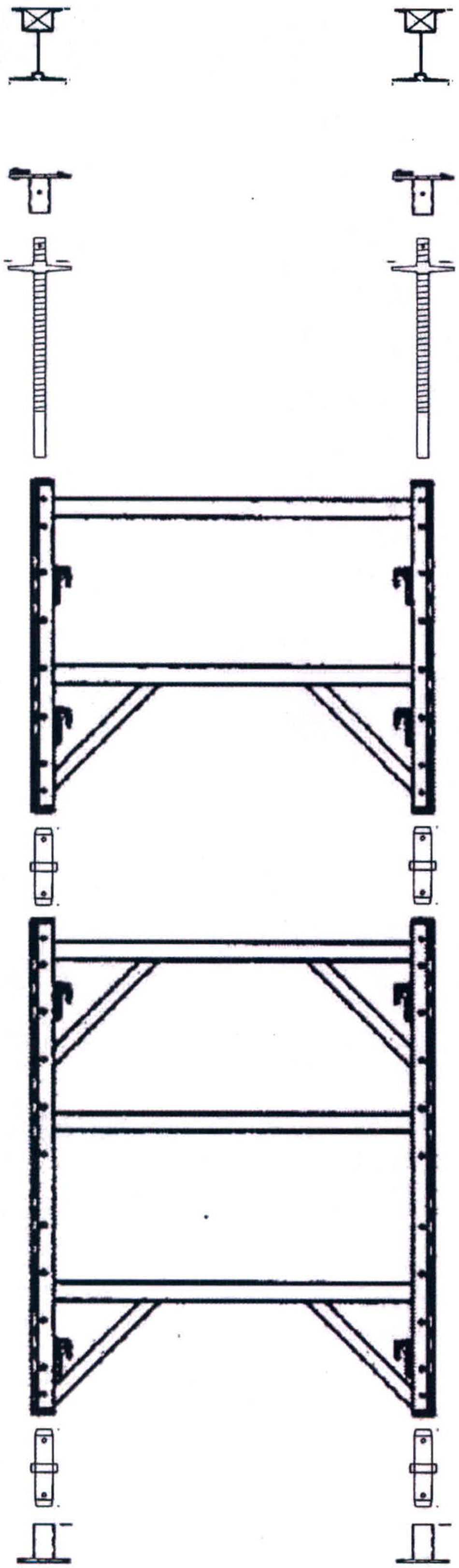
SH 41149
Bottom part of SH 41135
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41149

SH41149_1

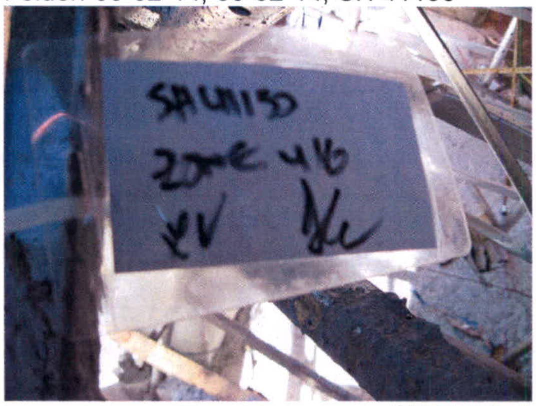
SH41149_2



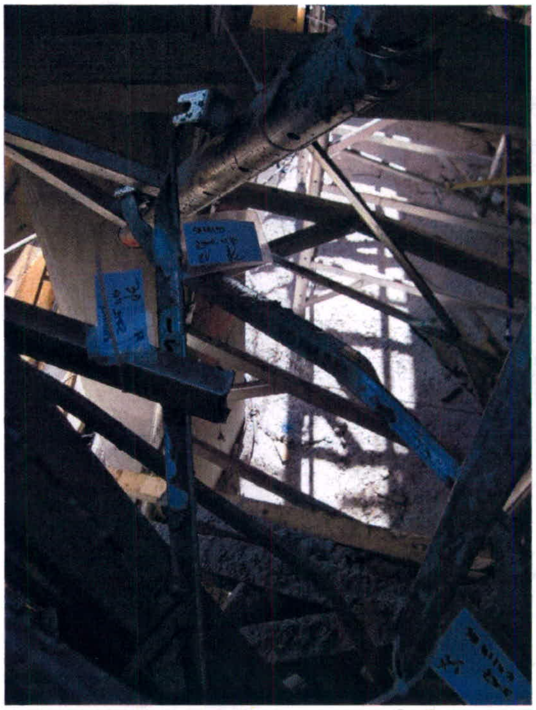


SH 41150
Brace that Connects o SH 41149
Zone 41G

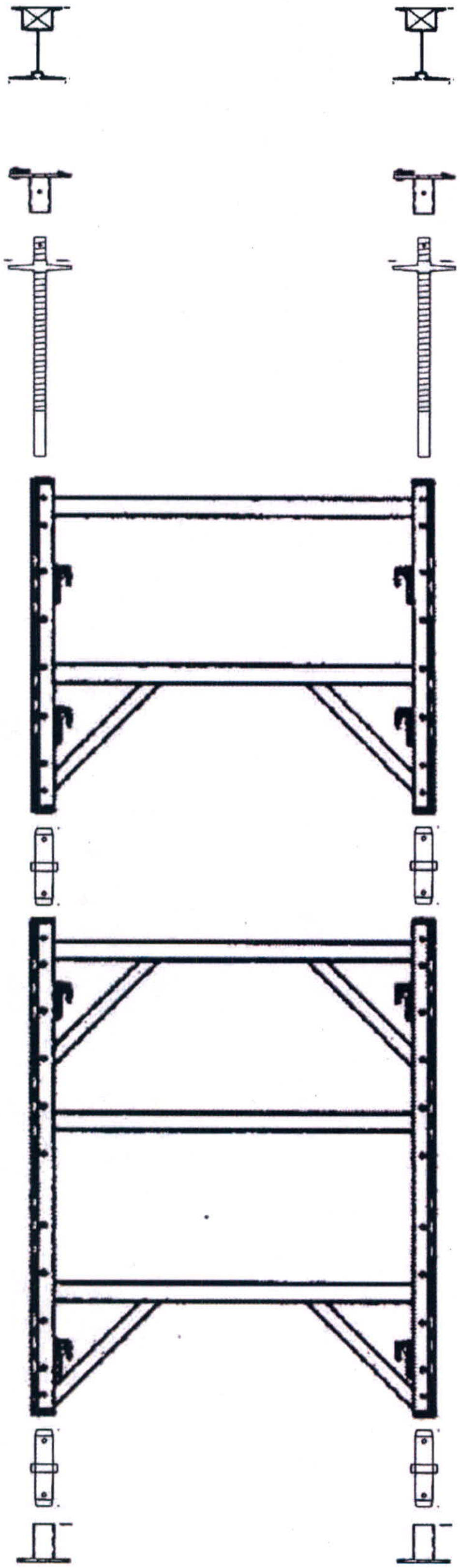
Folder: 08-02-14; 08-02-14; SH 41150



SH41150_1

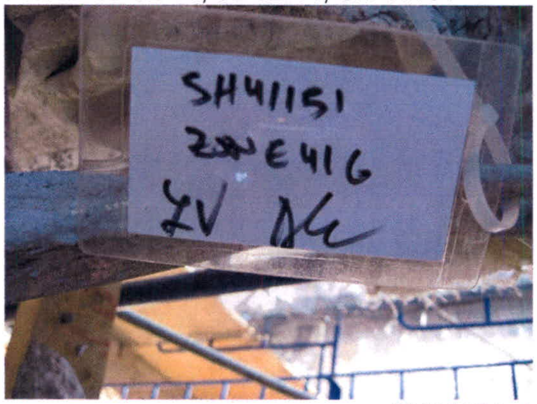


SH41150_2



SH 41151
Brace that Connects to SH 41152
Zone 41G

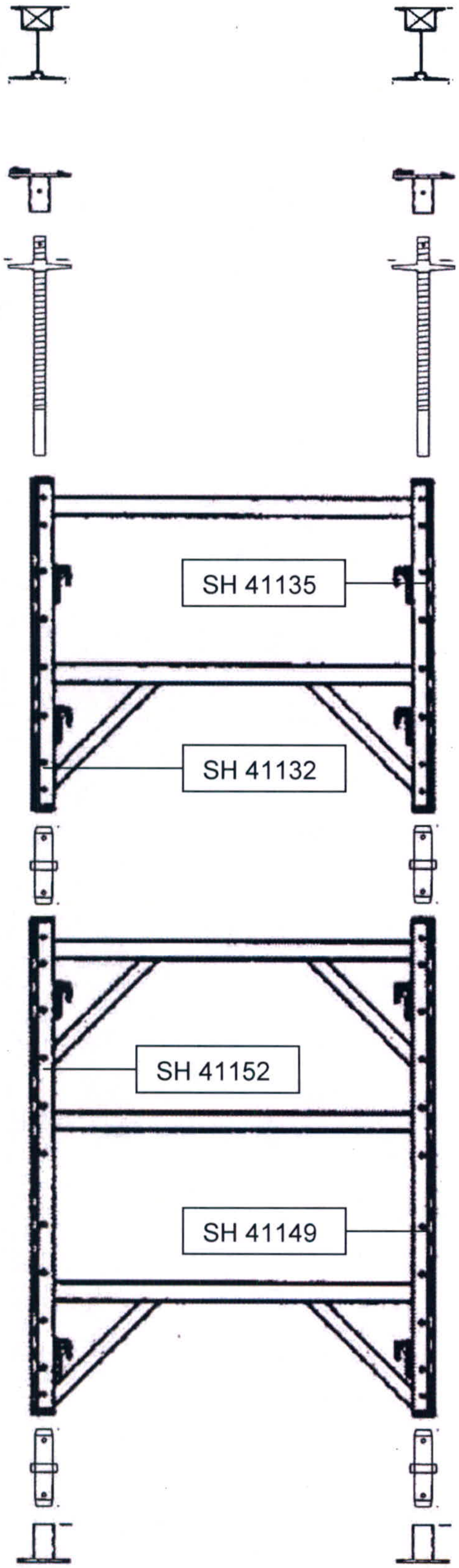
Folder: 08-02-14; 08-02-14; SH 41151



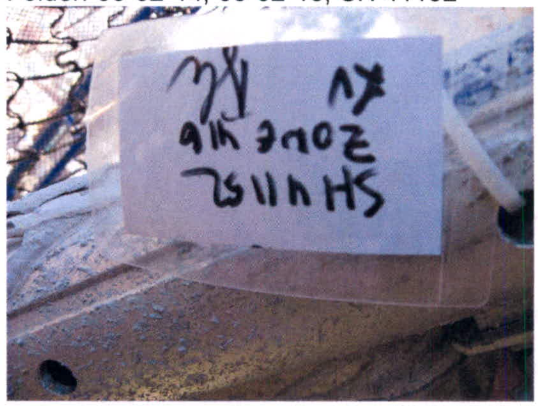
SH41151_1

SH41151_2

SH 41152
Bottom Frame of SH 41132
Zone 41G

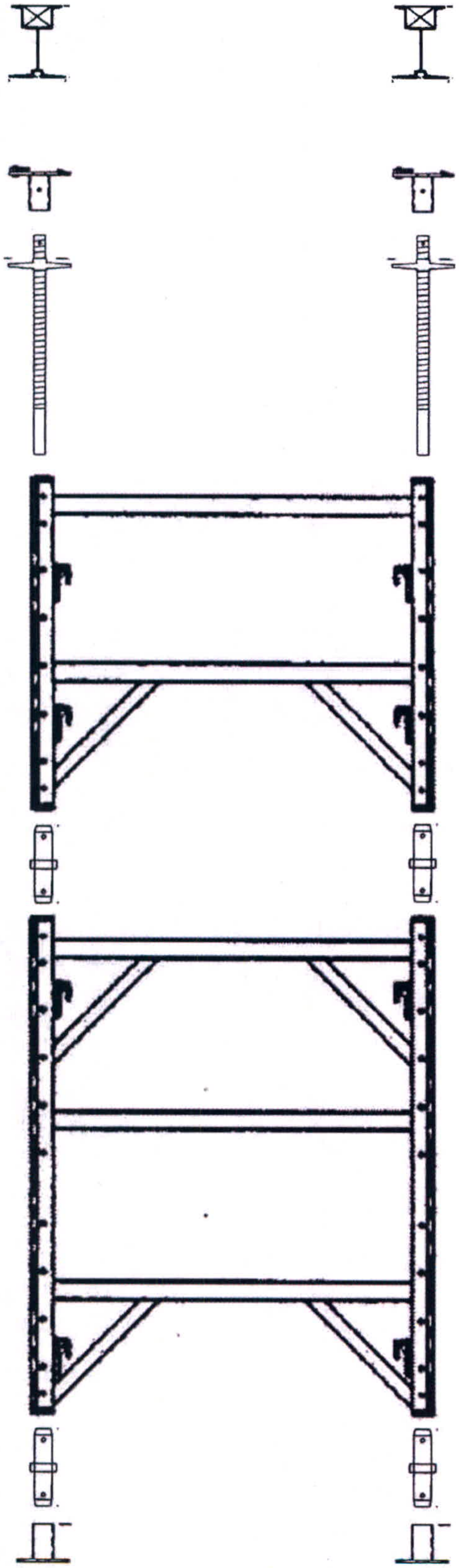


Folder: 08-02-14; 08-02-13; SH 41152



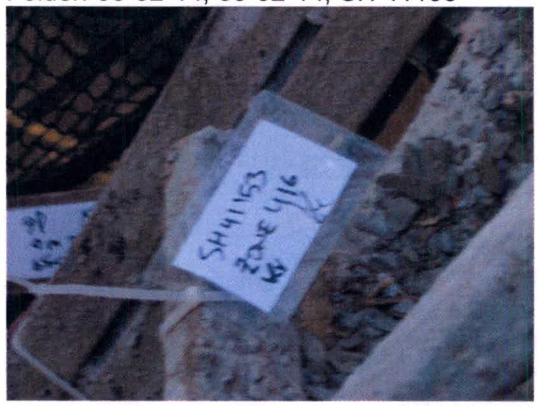
SH41152_1

SH41152_2



SH 41153
Piece of frame
41G

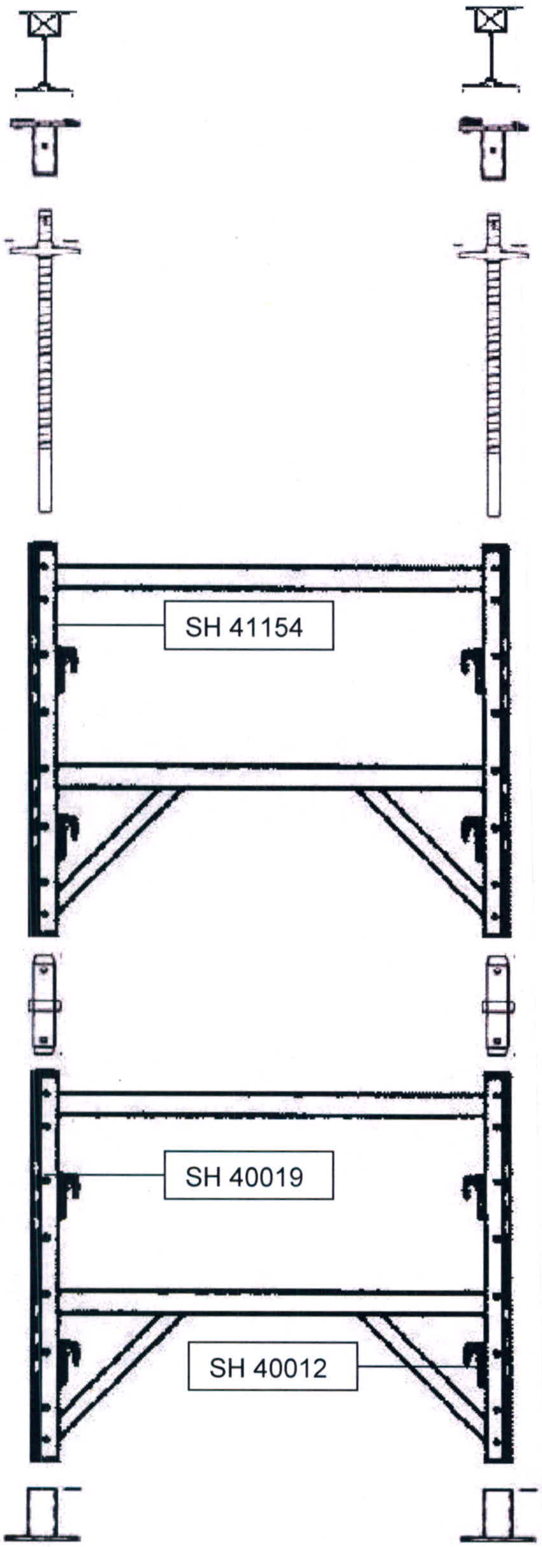
Folder: 08-02-14; 08-02-14; SH 41153



SH41153_1

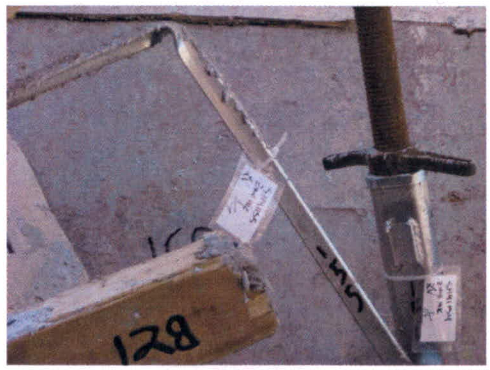
SH41153_2

SH 41154
Top Shore
Connect to brace SH 41155
Zone 41C



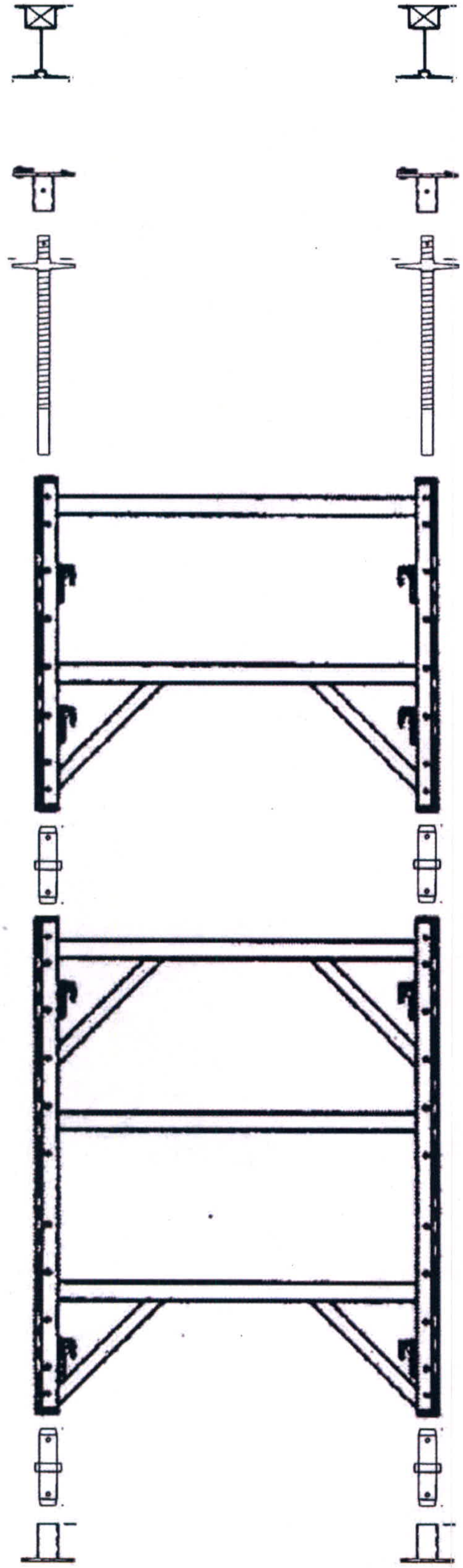
Folder: 08-02-14; 08-02-14; SH 41154


SH41154_1



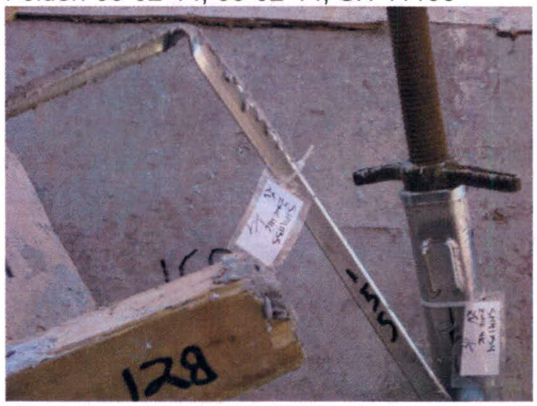
SH41154_3


SH41154_2



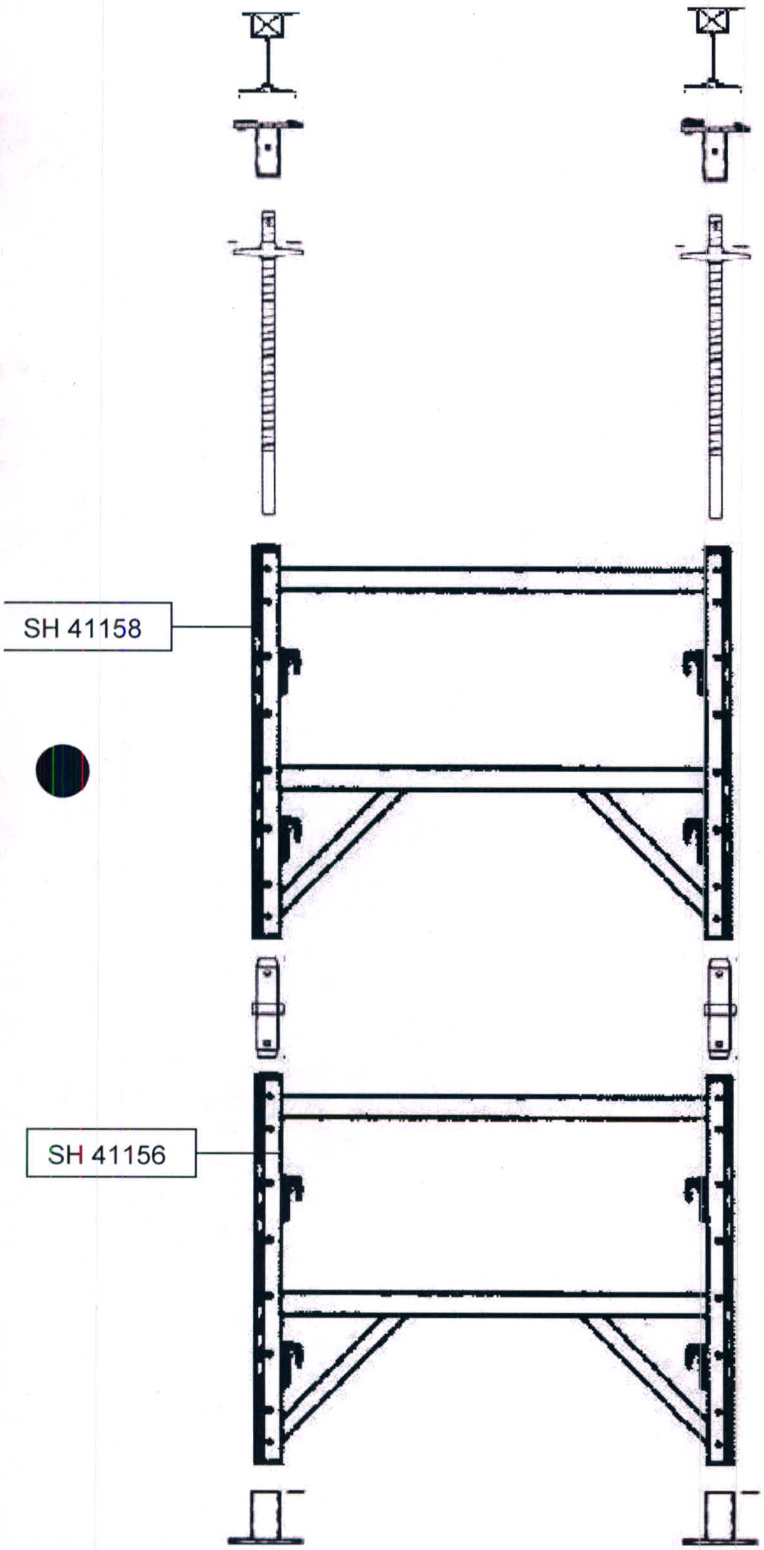
SH 41155
Brace
Connected to leg SH 41154
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41155



SH41154_3

SH 41156
Bottom Frame of SH 41158
Zone 41C



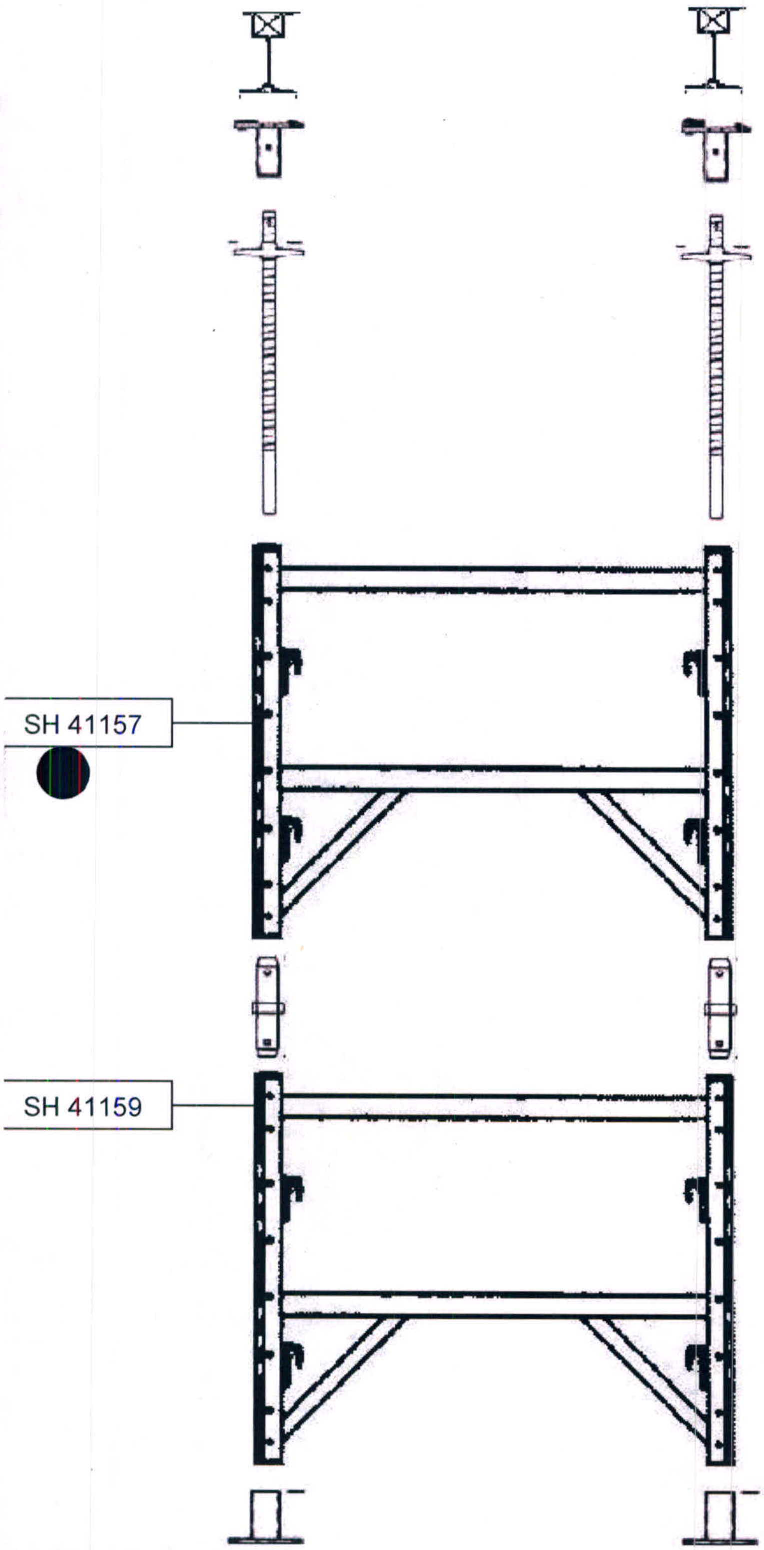
Folder: 08-02-14; 08-02-14; SH 41156

SH41156_1

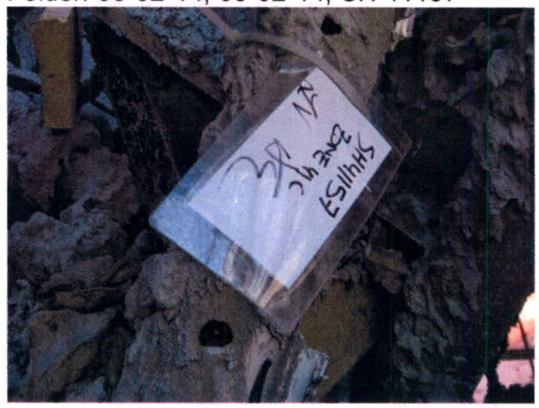
SH41156_2

SH41156_3

SH 41157
Top frame of SH 41159
Zone 41C



Folder: 08-02-14; 08-02-14; SH 41157



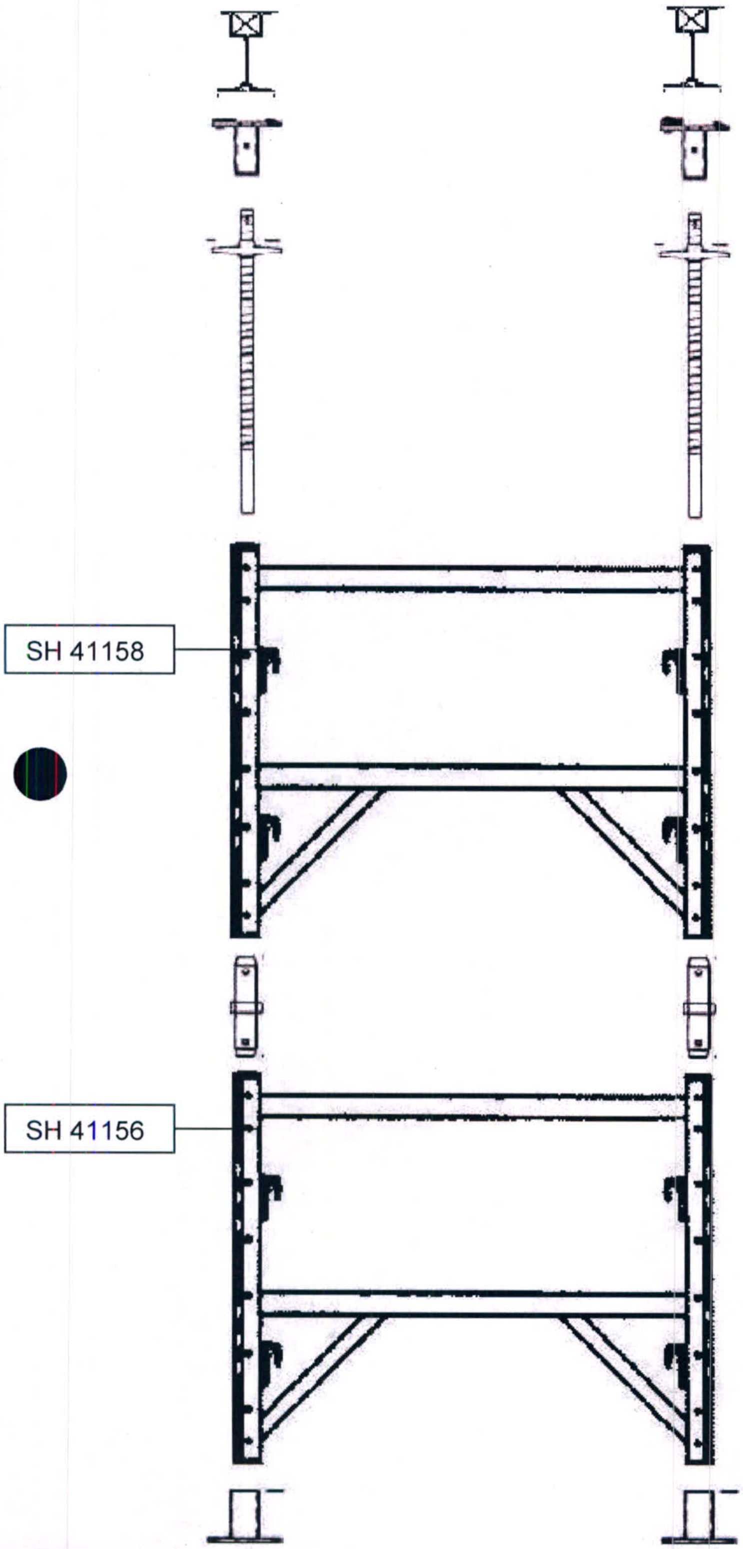
SH41157_1

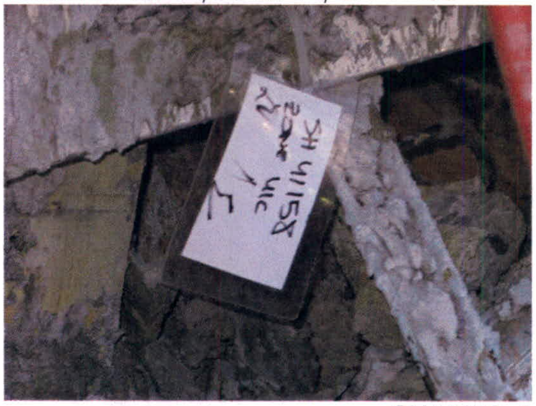
SH41157_2

SH41157_3

SH 41158
Top frame of SH 41156
Connected to brace SH 41155
Zone 41C

Folder: 08-02-14; 08-02-14; SH 41158





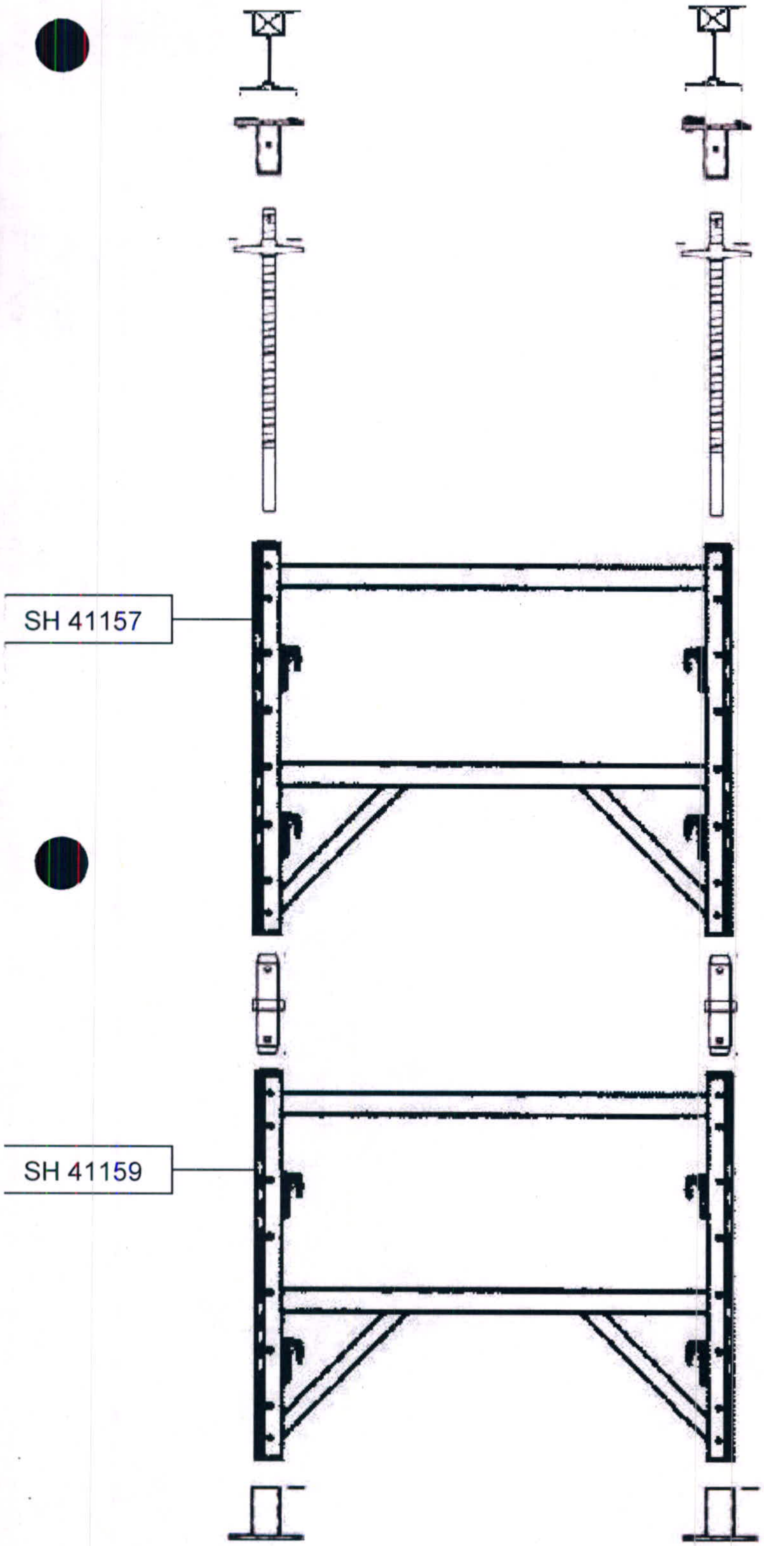
SH41158_1

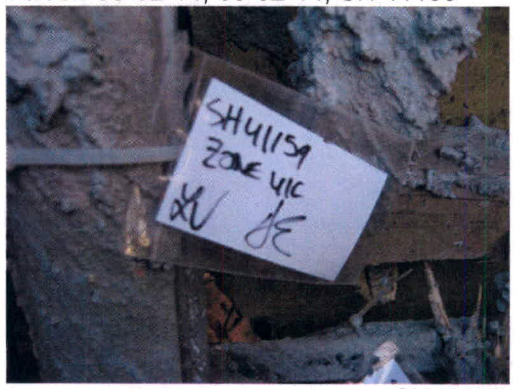
SH41158_2

SH41158_3

SH 41159
Bottom Frame of SH 41157
Zone 41C

Folder: 08-02-14; 08-02-14; SH 41159

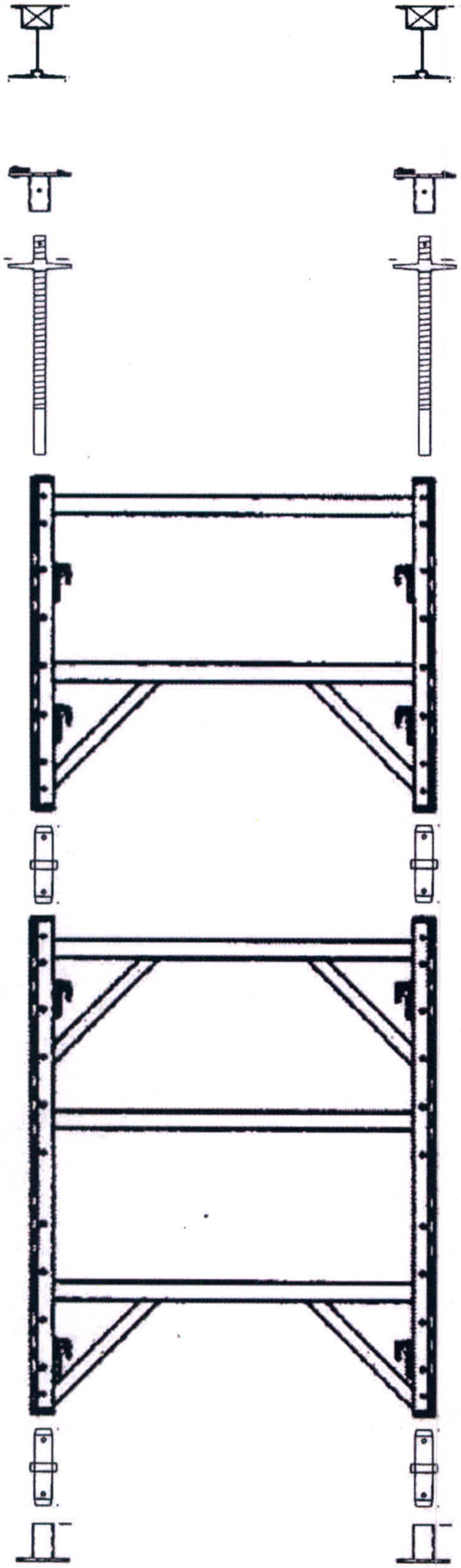




SH41159_2

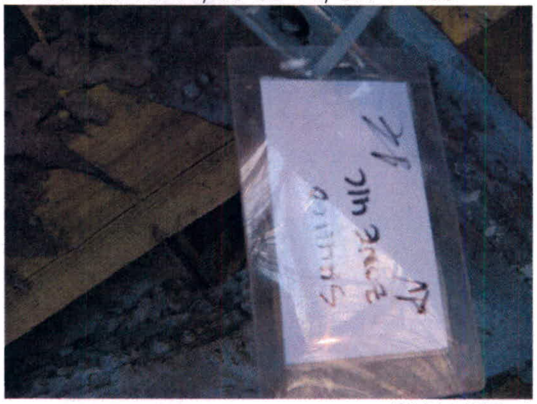

SH41159_1


SH41159_3



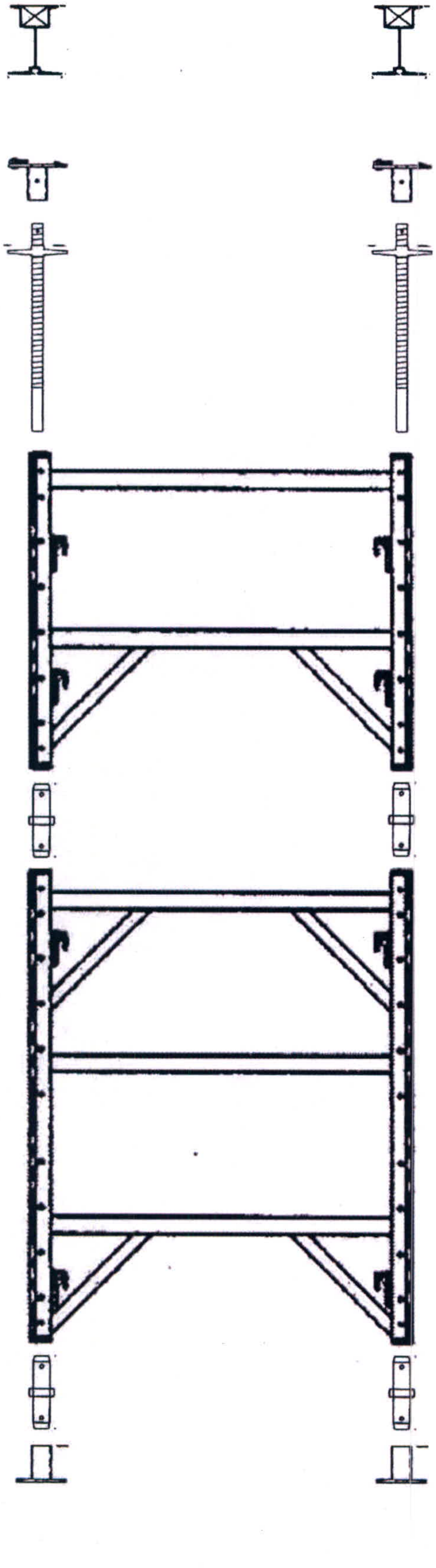
SH 41160
Brace Connected to SH 41157
Zone 41C

Folder: 08-02-14; 08-02-14; SH 41160



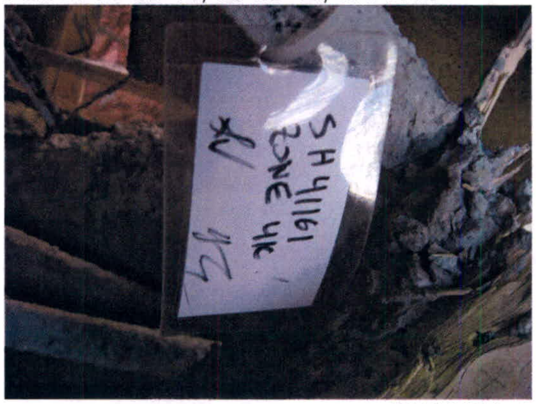
SH41160_1

SH41160_2



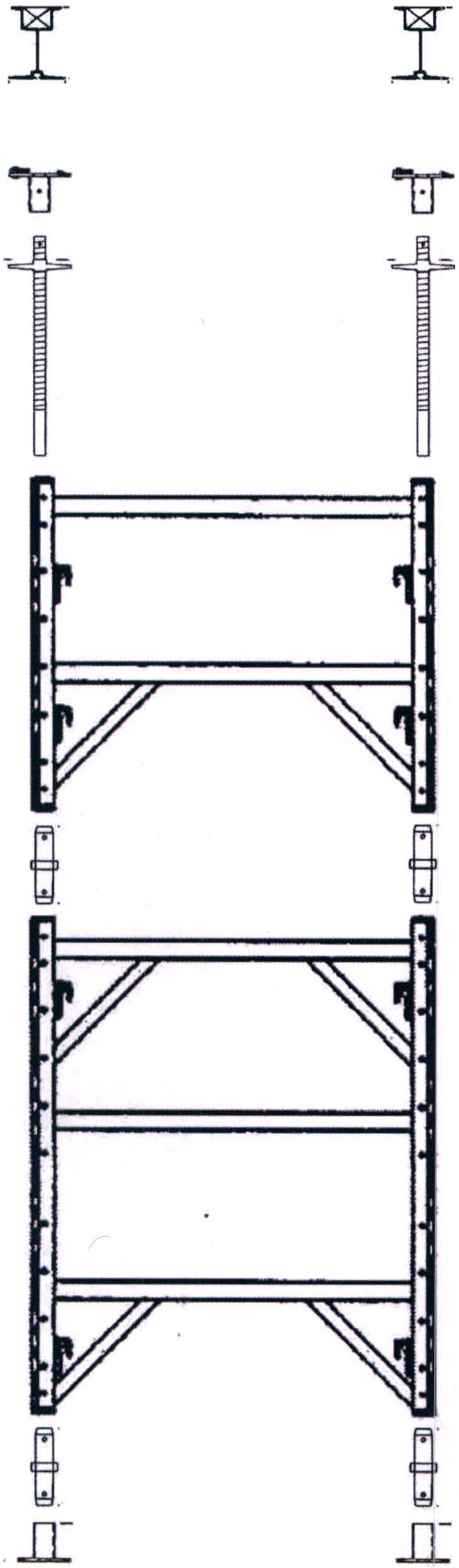
SH 41161
Brace connected to SH 41159
Zone 41C

Folder: 08-02-14; 08-02-14; SH 41161



SH41161_1

SH41161_2



SH 41162
Brace
Connected to SH 41156
Zone 41C

Folder: 08-02-14; 08-02-14; SH 41162

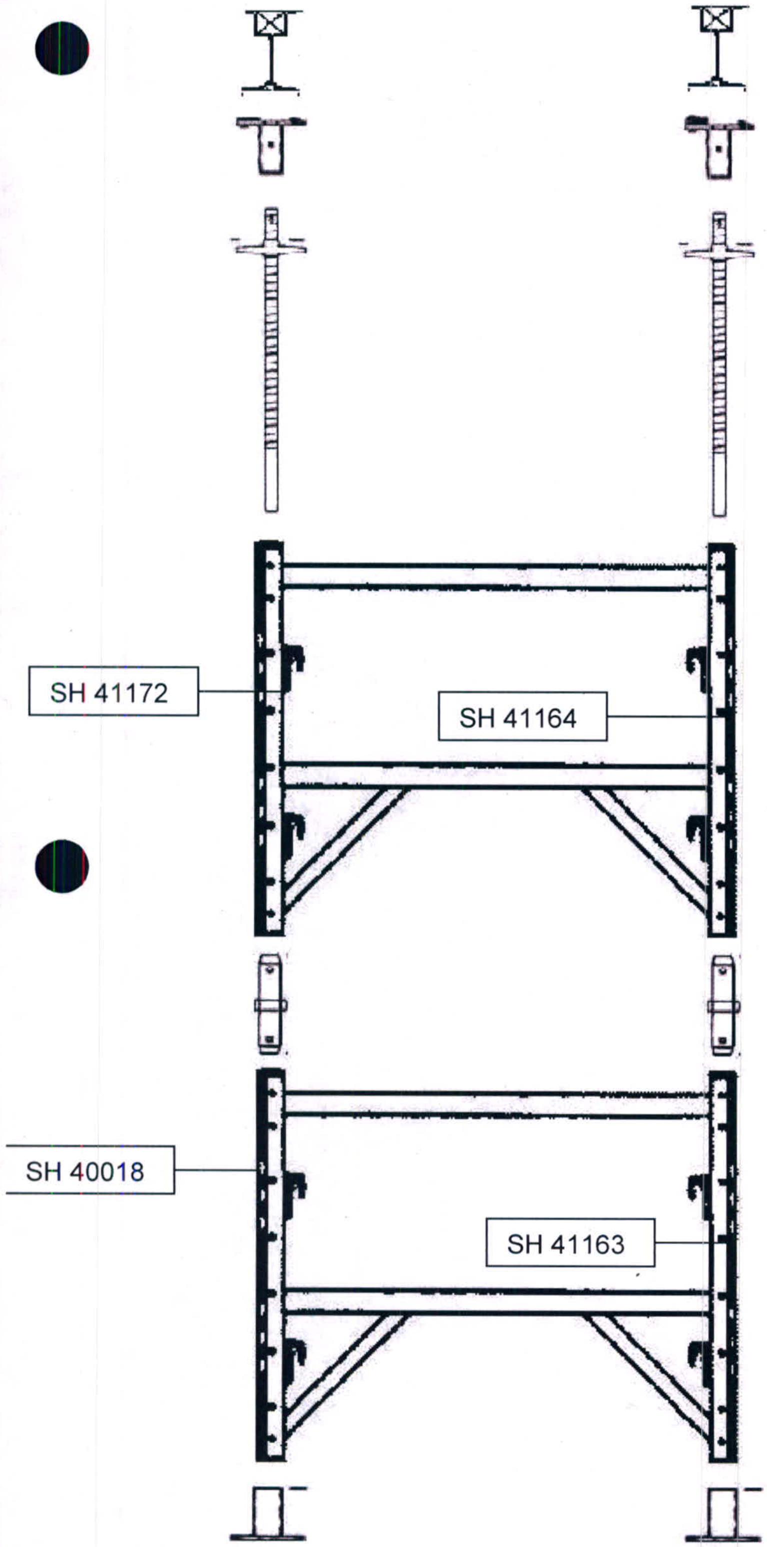
SH41162_1

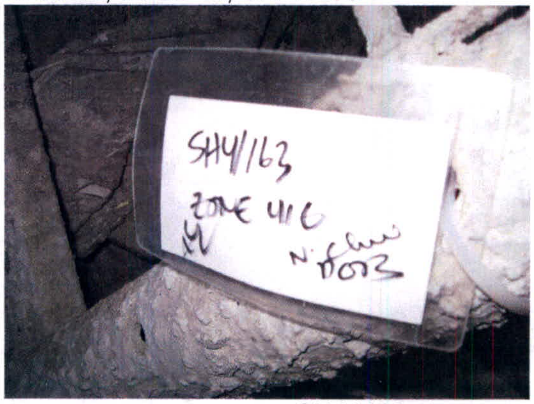
SH41162_2

SH41162_3

SH 41163
Bottom Shore
Frame attached to brace SH 41164 and SH 41151 Brace
Zone 41G

Folder: 08-02-13; 08-02-14; SH 41163



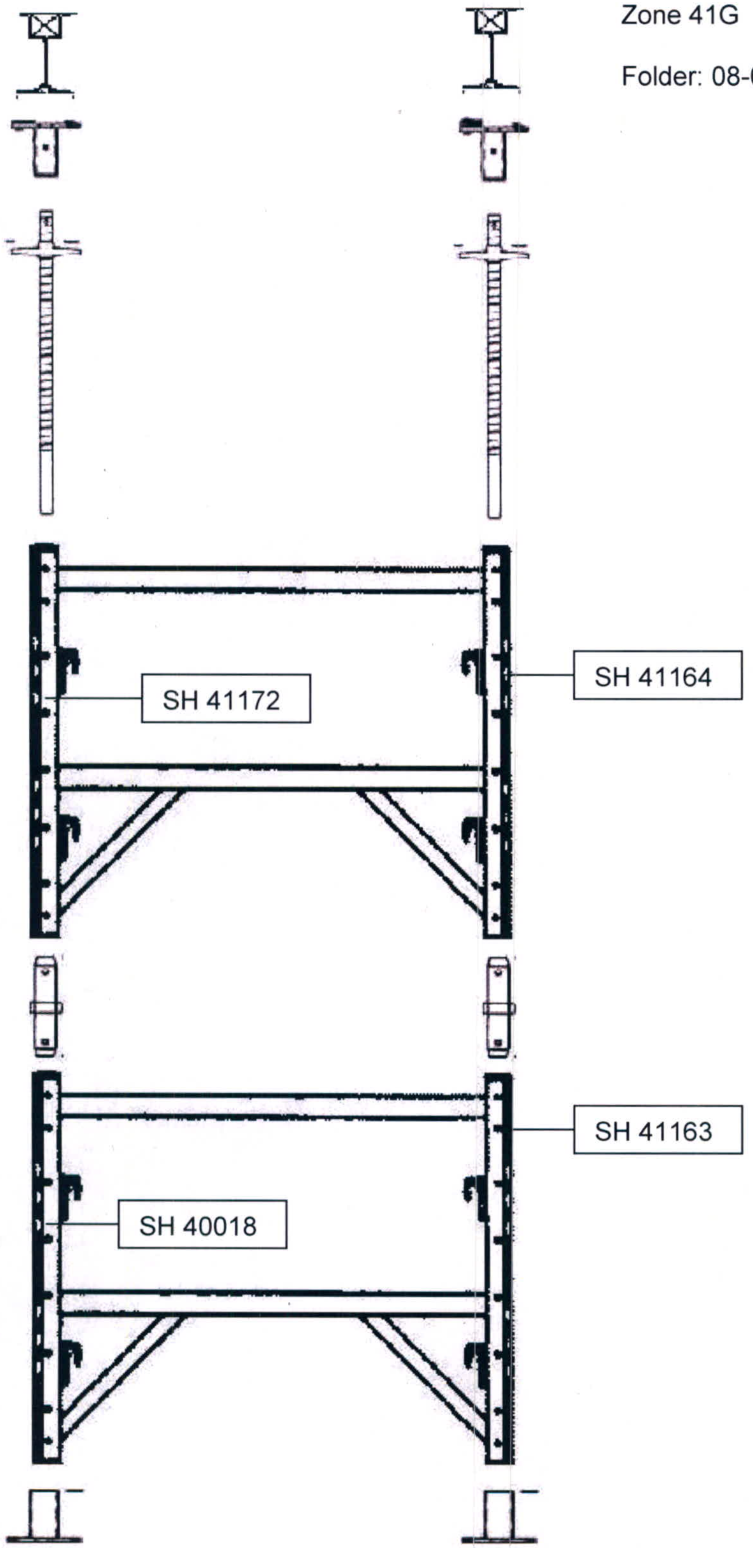


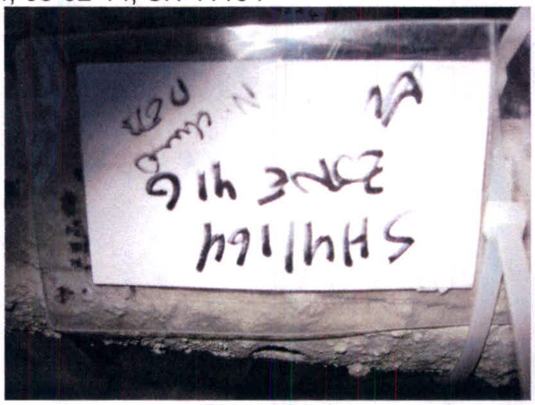
SH41163_1


SH41163_2

SH 41164
Frame attached to SH 41163
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41164





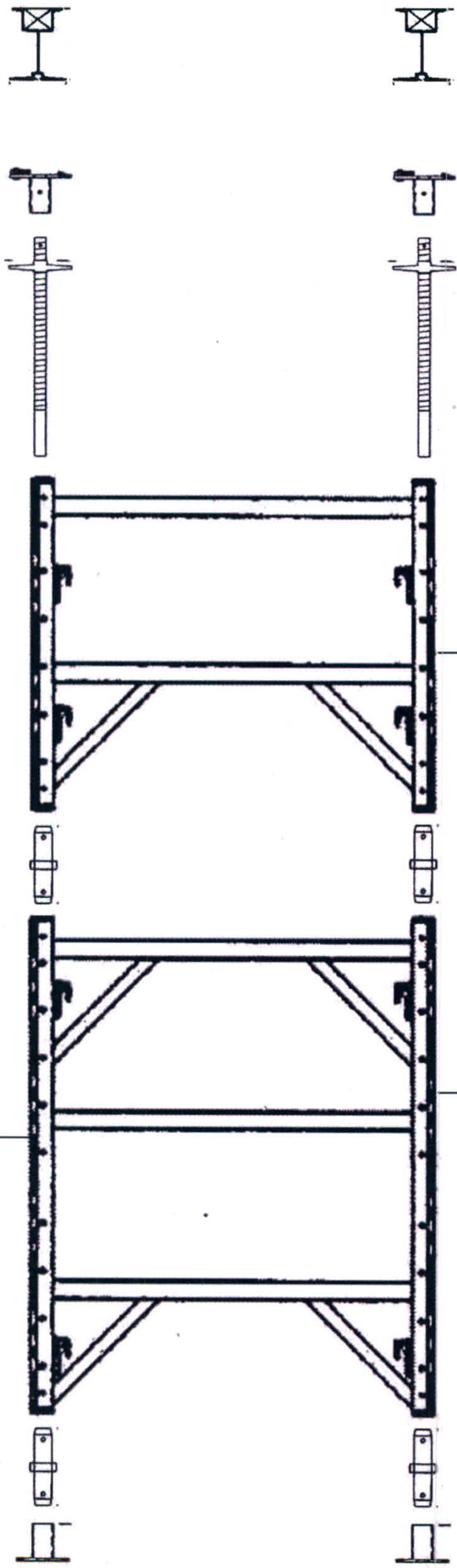
SH41164_1

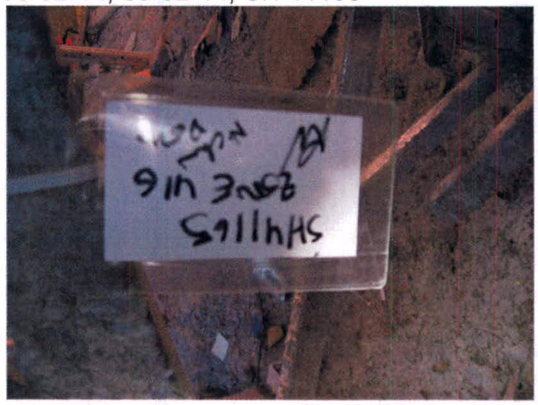
SH41164_2

SH41164_3

SH 41165 on 40th Floor
Frame attached to SH 40004
Zone 41G

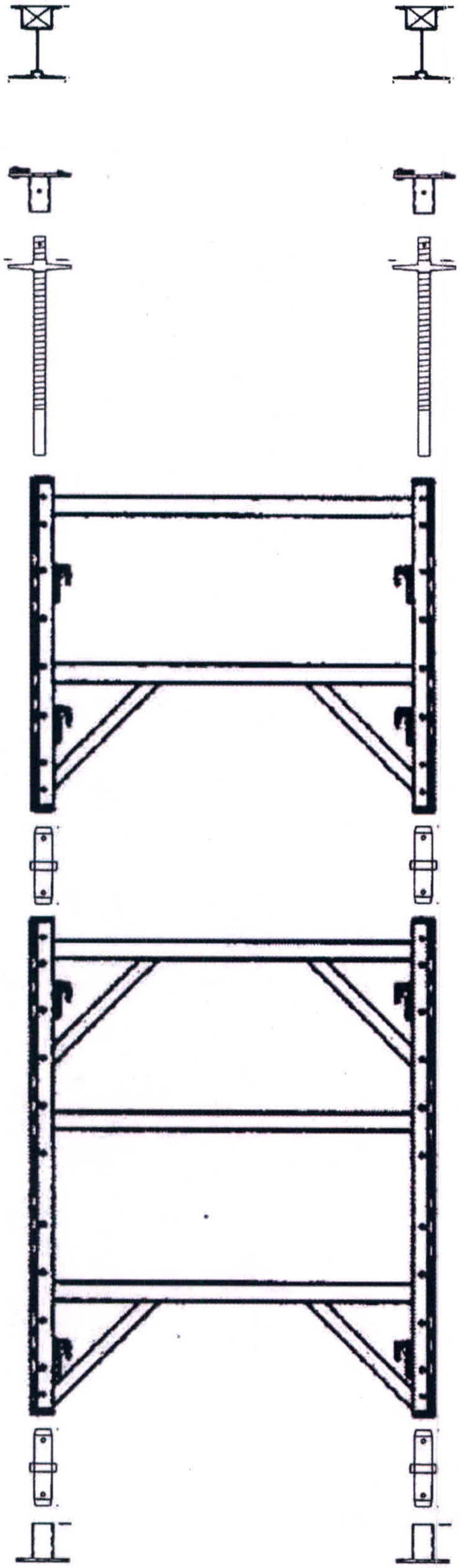
Folder: 08-02-14; 08-02-14; SH 41165





SH41165_1

SH41165_2

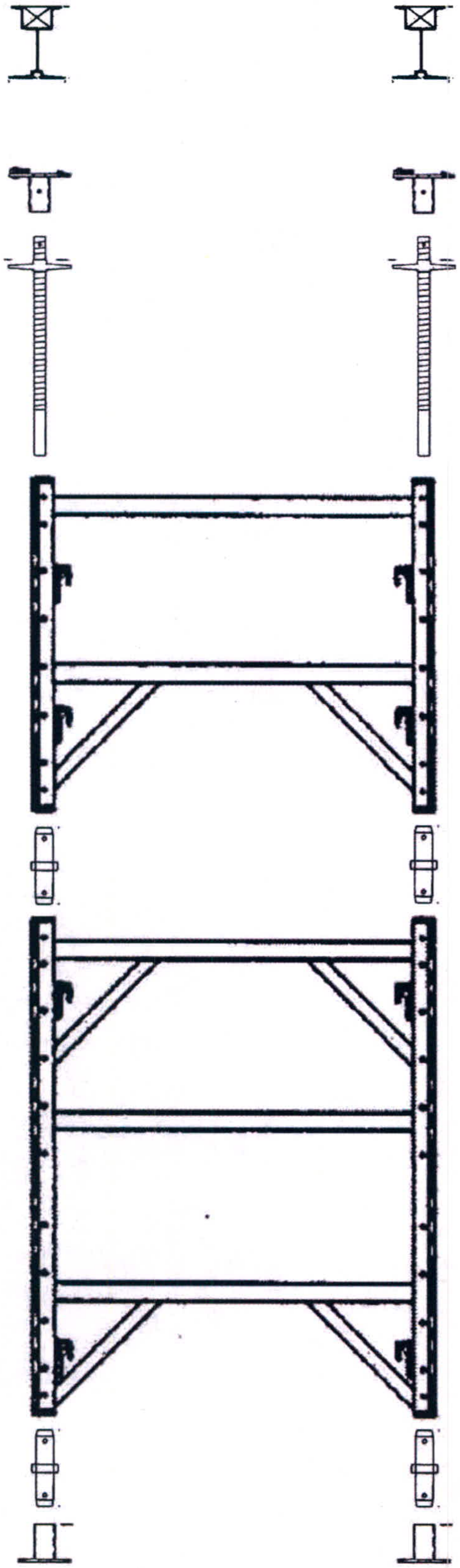


SH 41166
Brace attached to SH 40003 and SH 40004
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41166

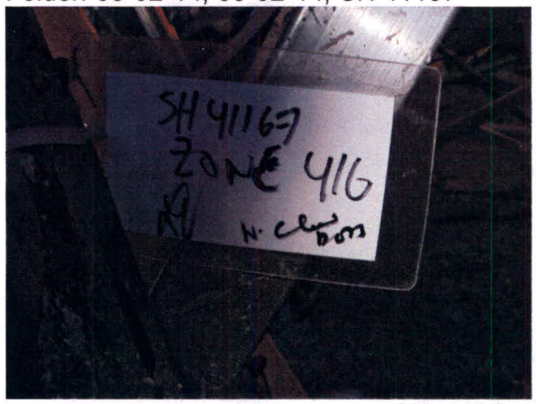
SH41166_1

SH41166_2



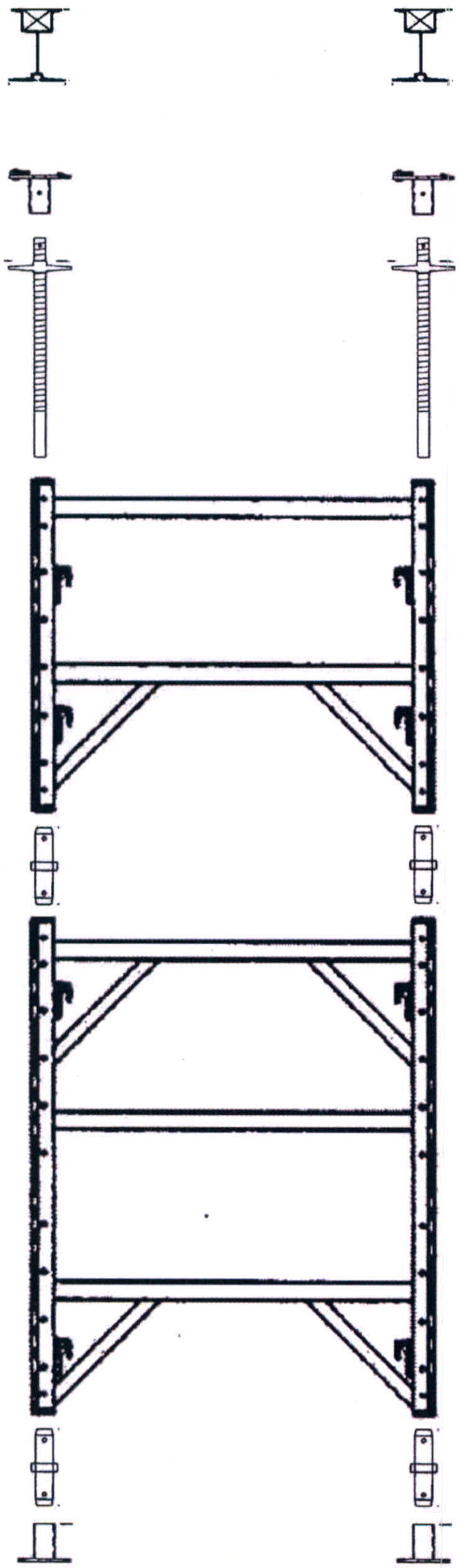
SH 41167 on 40th Floor
Connected to Frame SH 41175
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41167



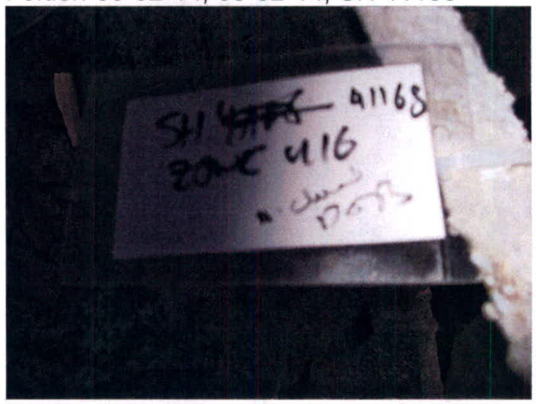
SH41167_1

SH41167_2



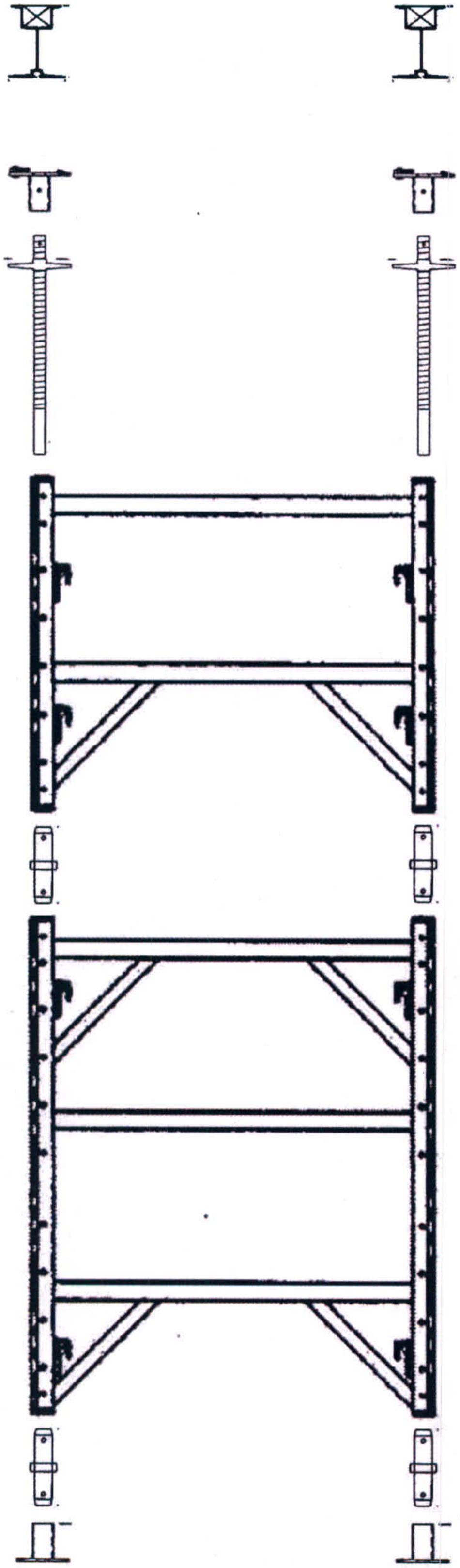
SH 41168
Bottom Brace connected to SH 41175
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41168



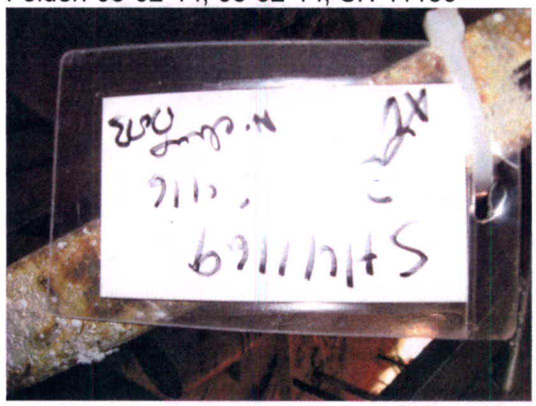
SH41168_1

SH41168_2



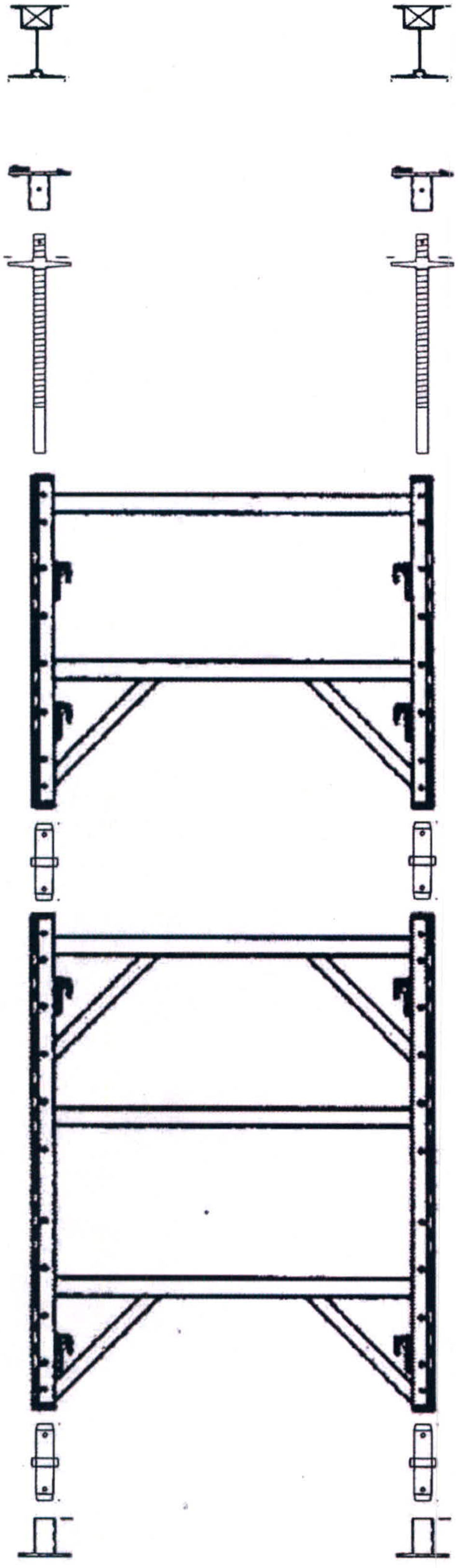
SH 41169
Brace connects to SH 41175
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41169



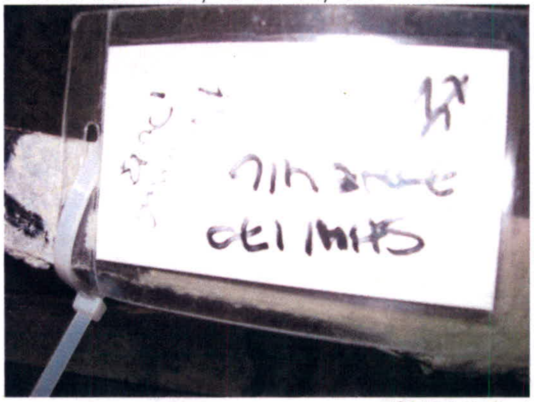
SH41169_1

SH41169_2



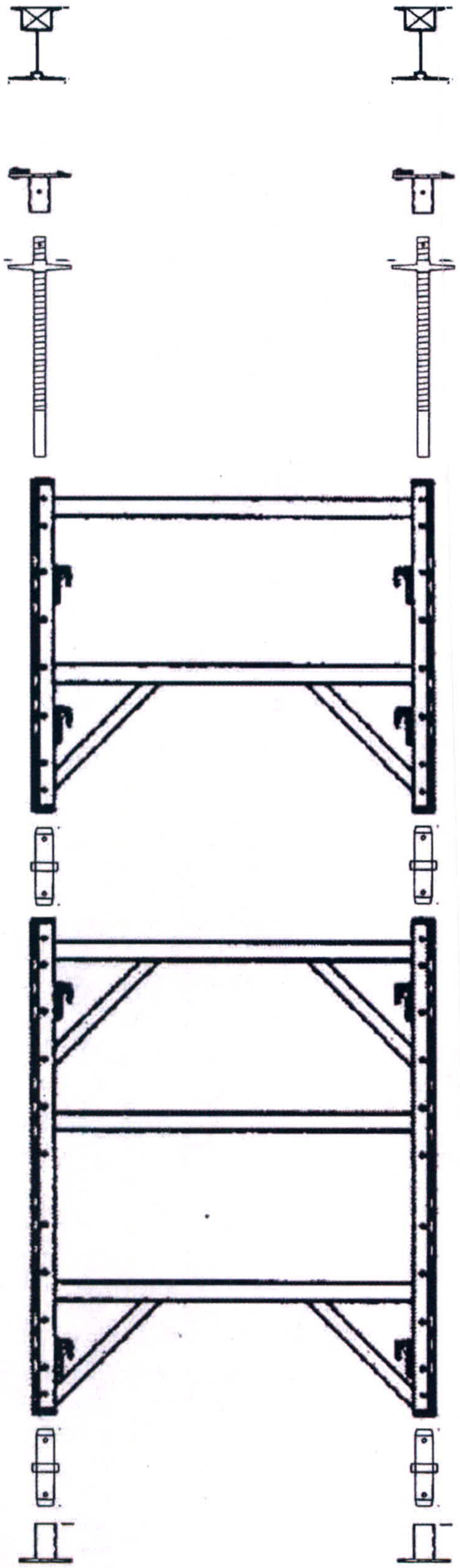
SH 41170 on 40th Floor
Piece of frame leaning SH 41169
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41170



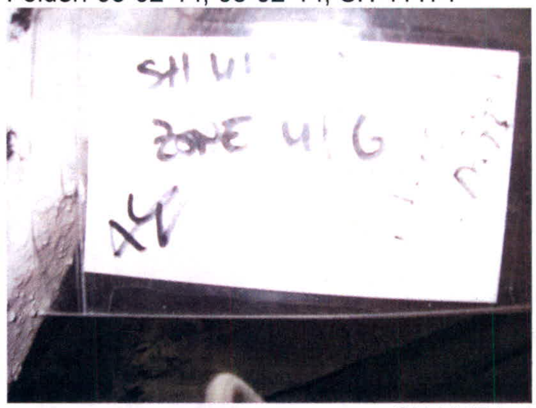
SH41170_1

SH41170_2



SH 41171
Piece of Frame
Frame Connected to Brace SH 41147
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41171

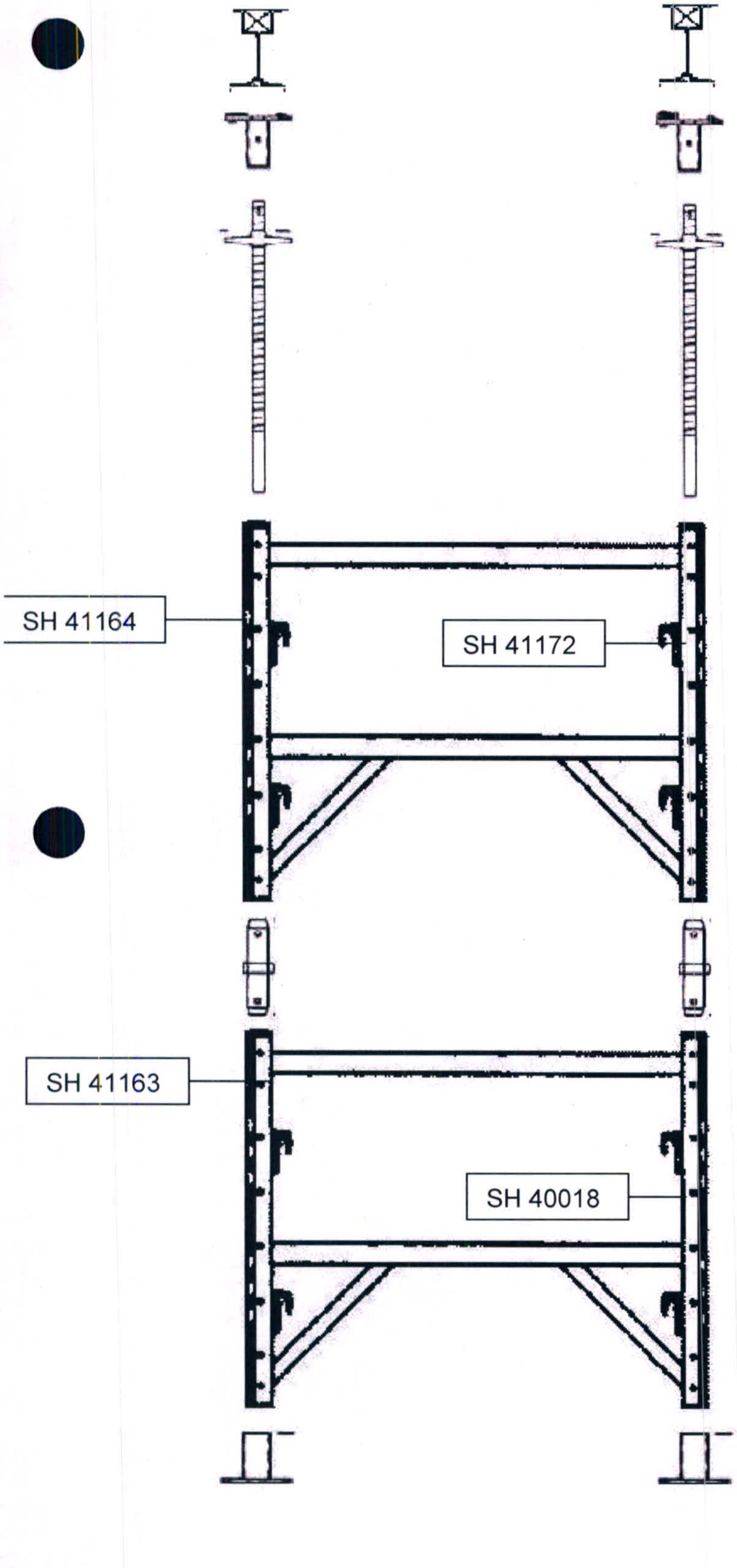


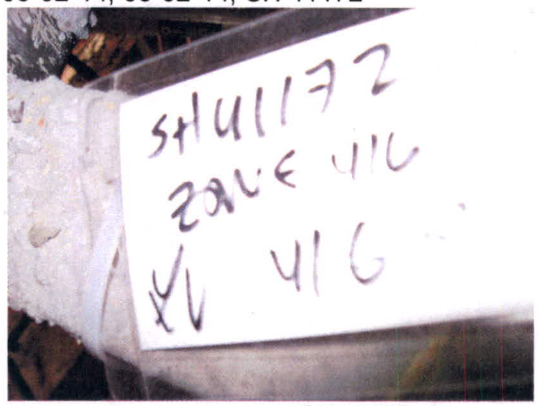
SH41171_1


SH41171_2

SH 41172
Top plate below AB 41001
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41172





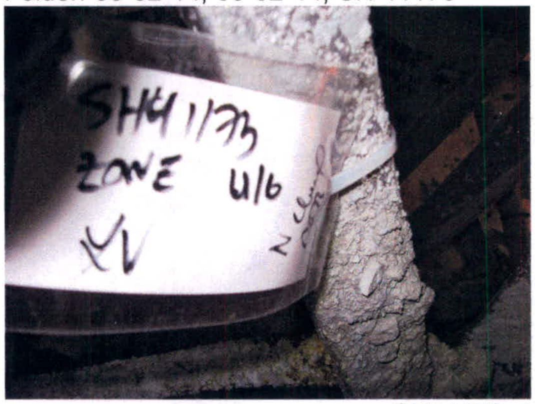
SH41172_1


SH41172_2

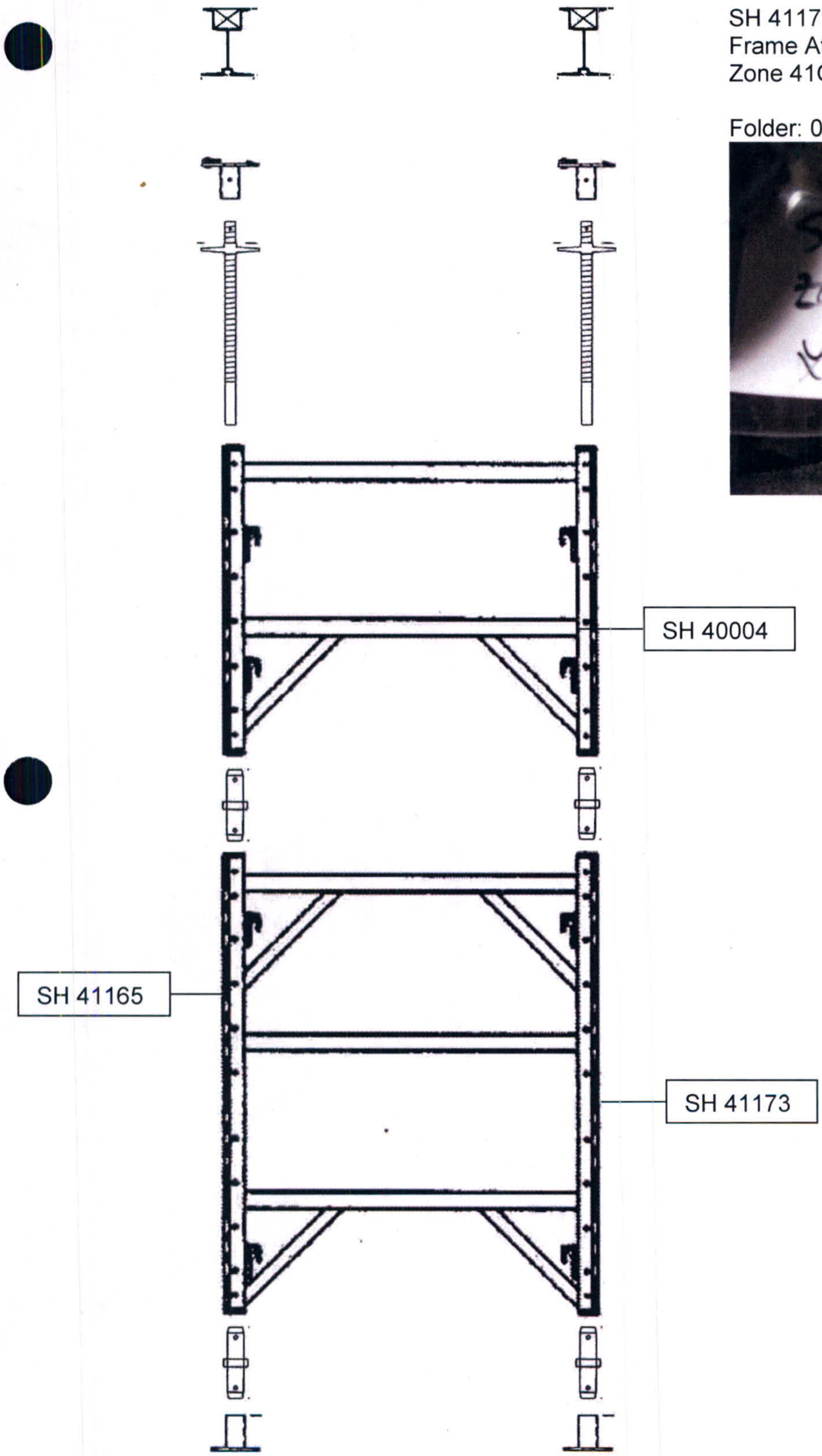

SH41172_3

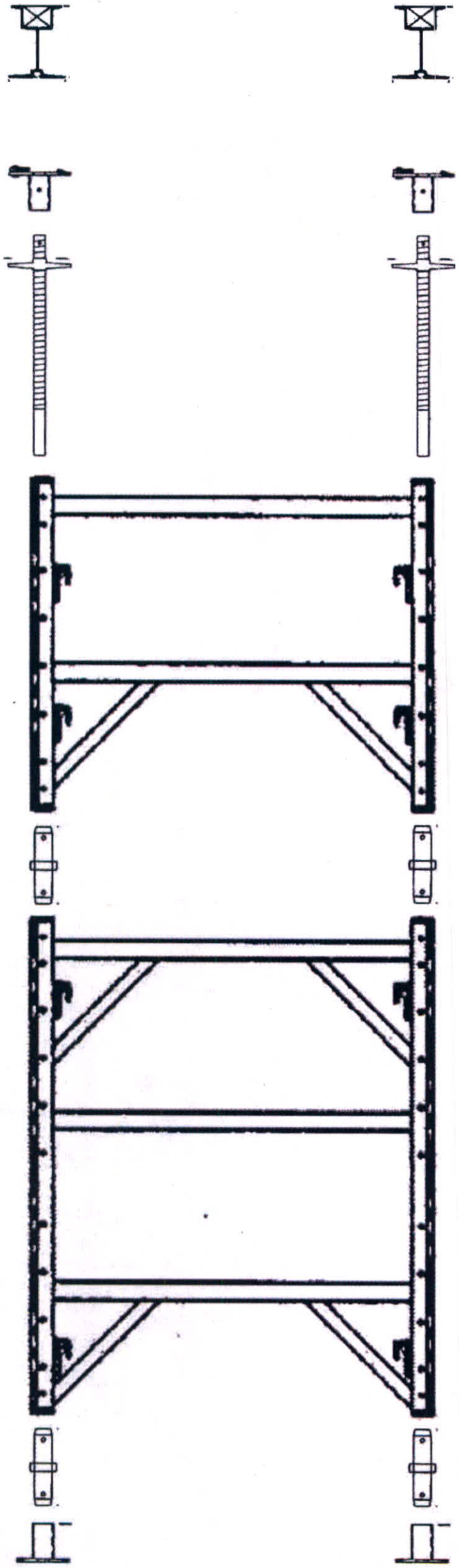
SH 41173
Frame Attached to SH 41169
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41173



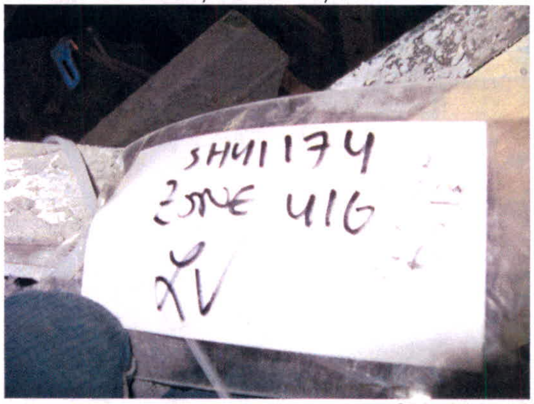
SH41173_1





SH 41174
Brace connects SH 41166 and SH 40004
Zone 41G

Folder: 08-02-14; 08-02-14; SH 41174



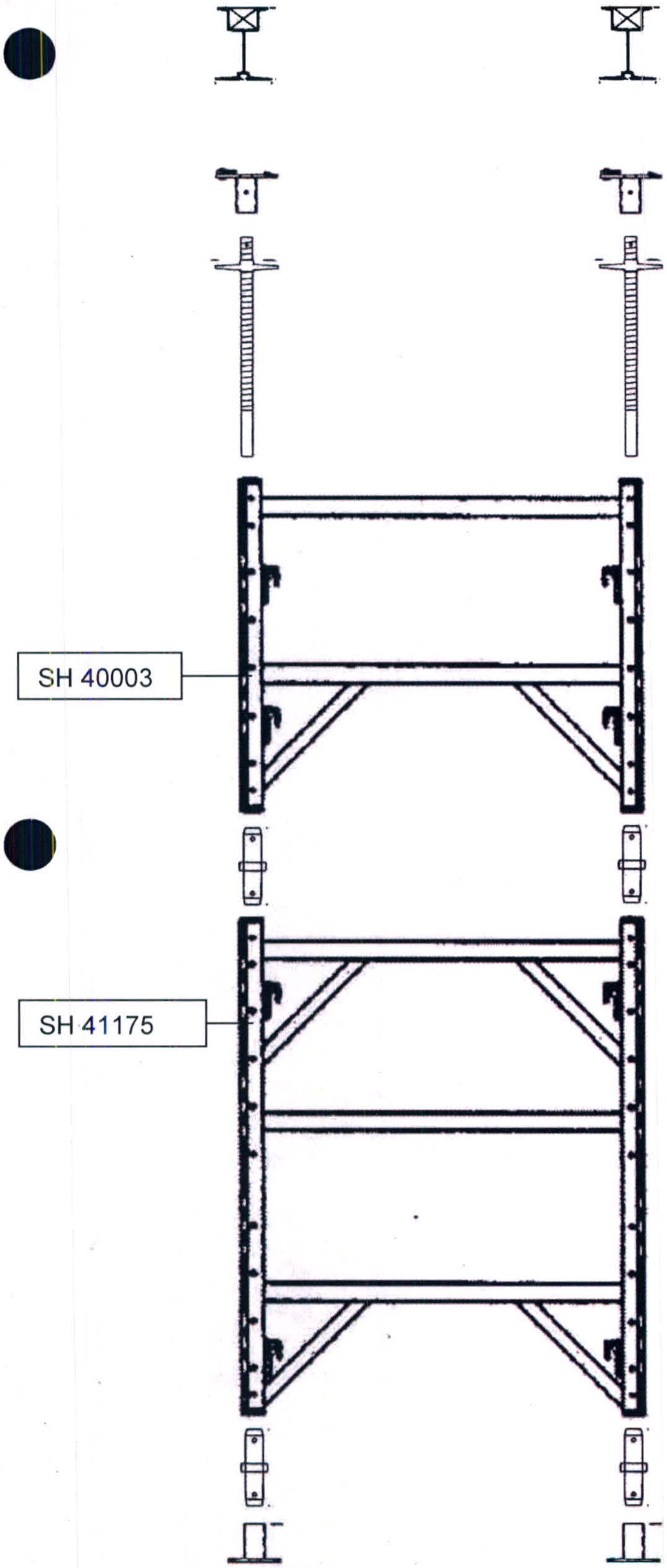
SH41174_1

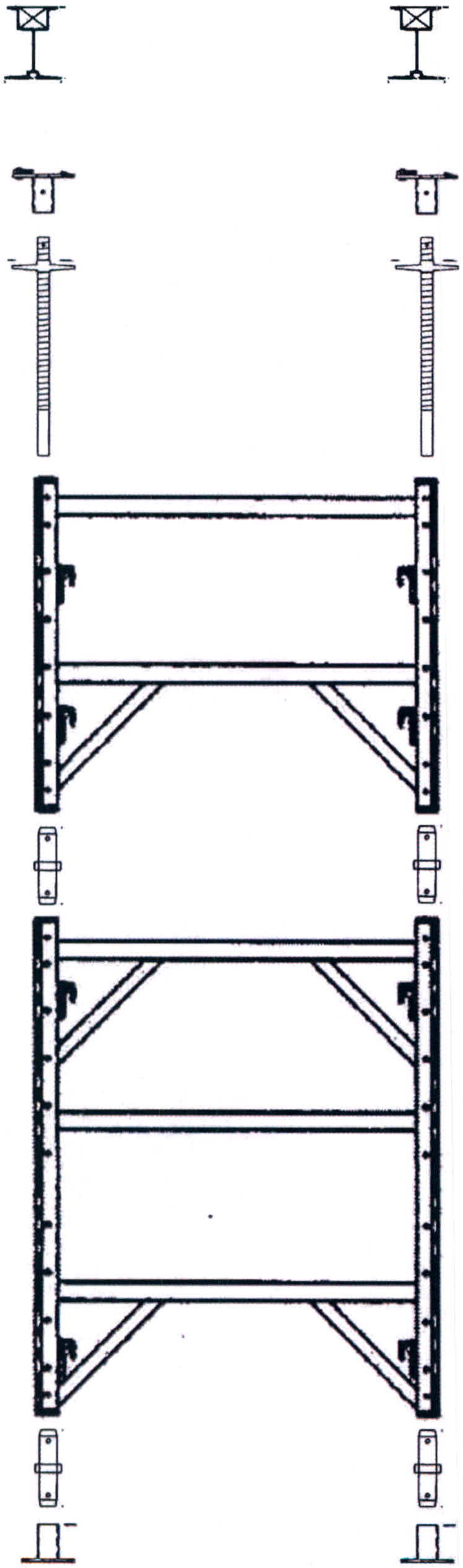
SH41174_2

SH 41175 on 40th floor
Bottom Frame of SH 40003
Zone 41G

Folder: 08-02-15; SH 41175

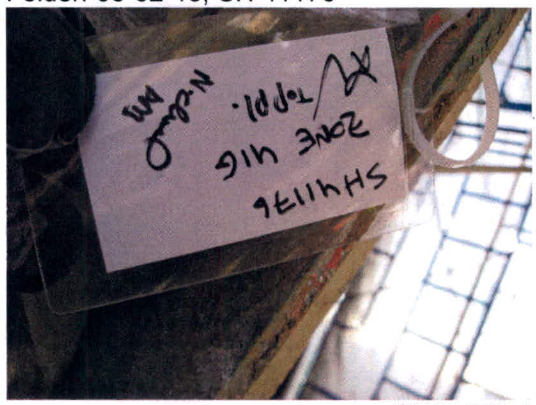
SH41175_1



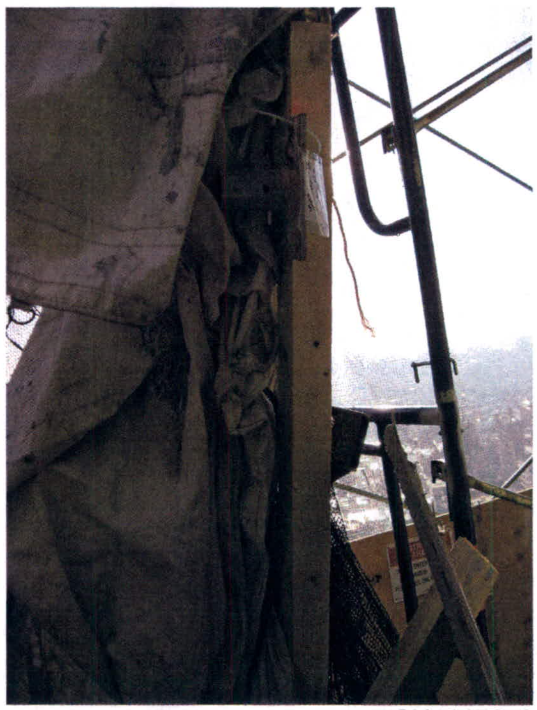


SH 41176
Top plate
Zone 41G

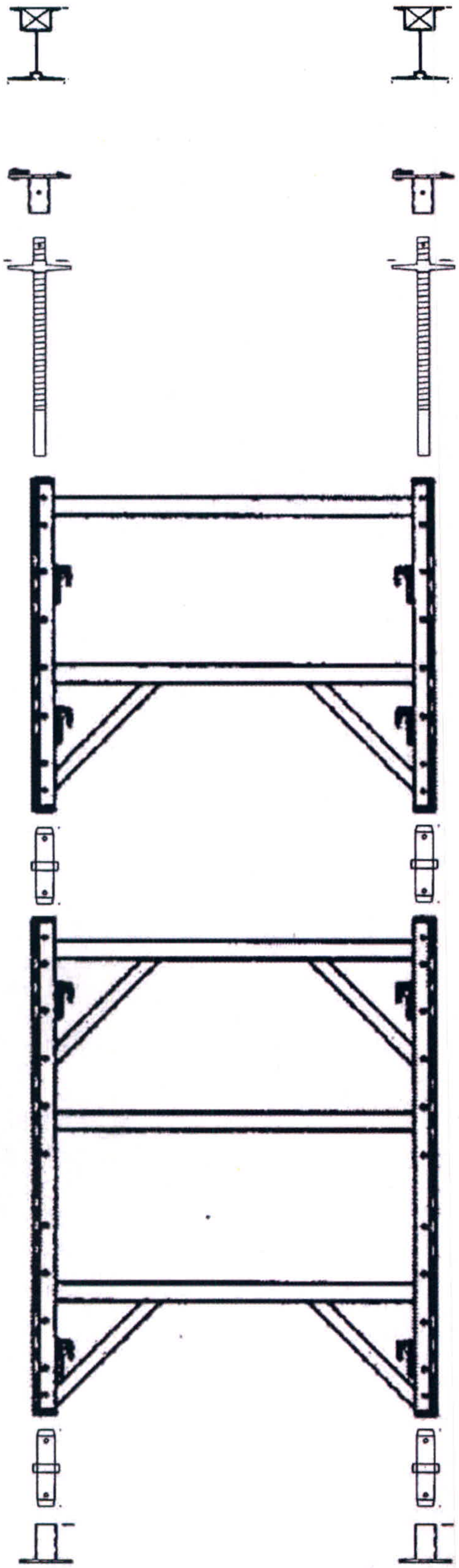
Folder: 08-02-15; SH 41176



SH41176_1

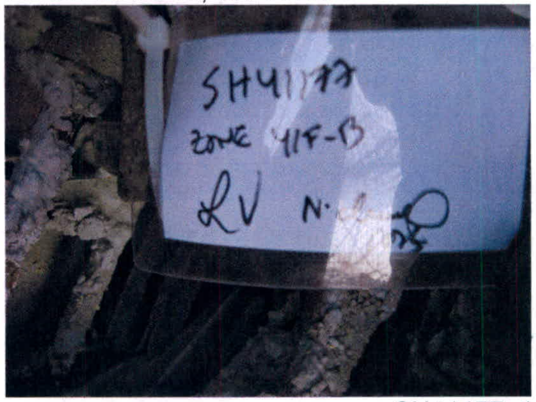


SH41176_2



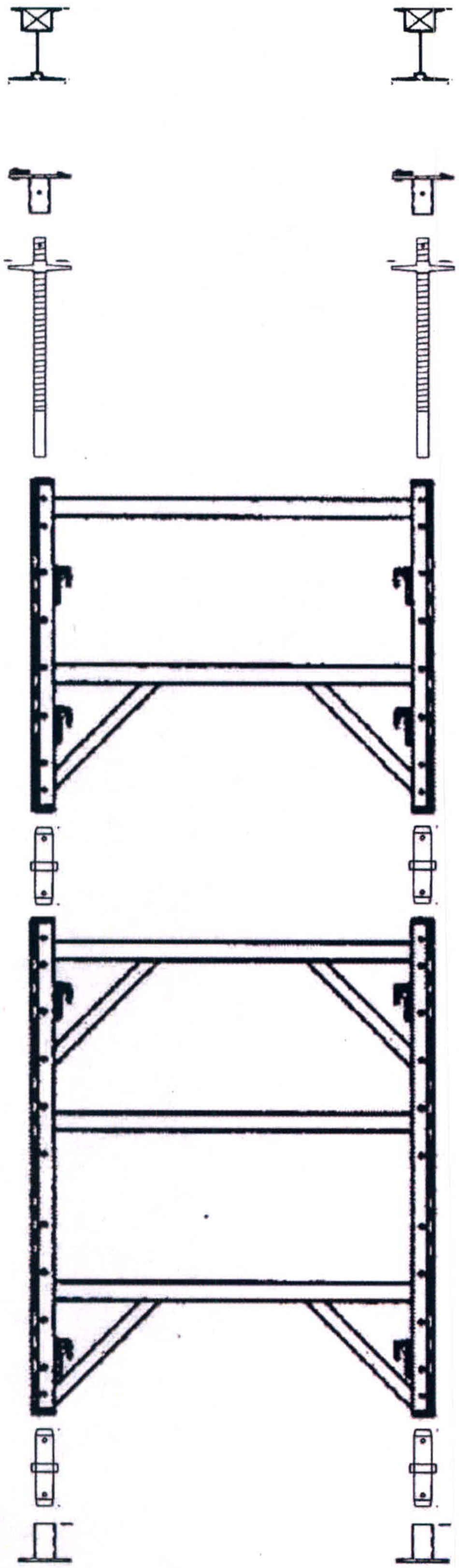
SH 41177
Top plate with Screw Jack
Zone 41F-B

Folder: 08-02-15; SH 41177



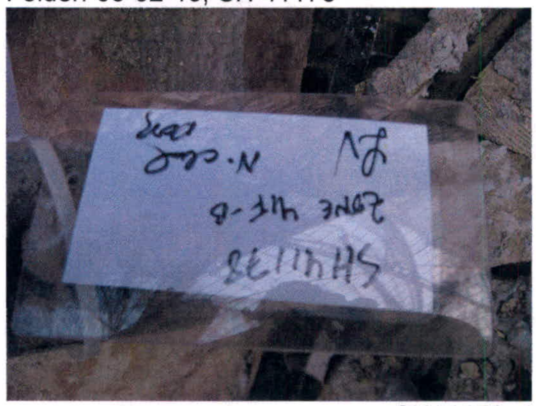
SH41177_1

SH41177_2

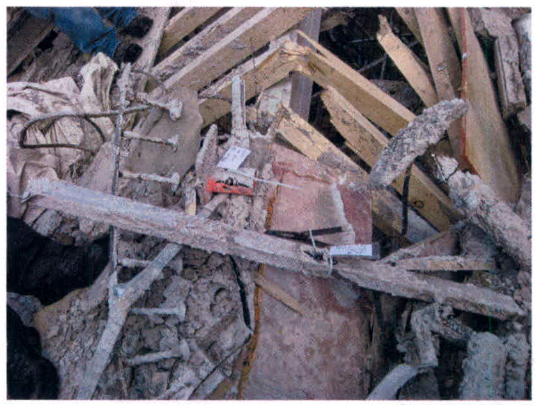


SH 41178
Piece of Frame
Zone 41F-B

Folder: 08-02-15; SH 41178



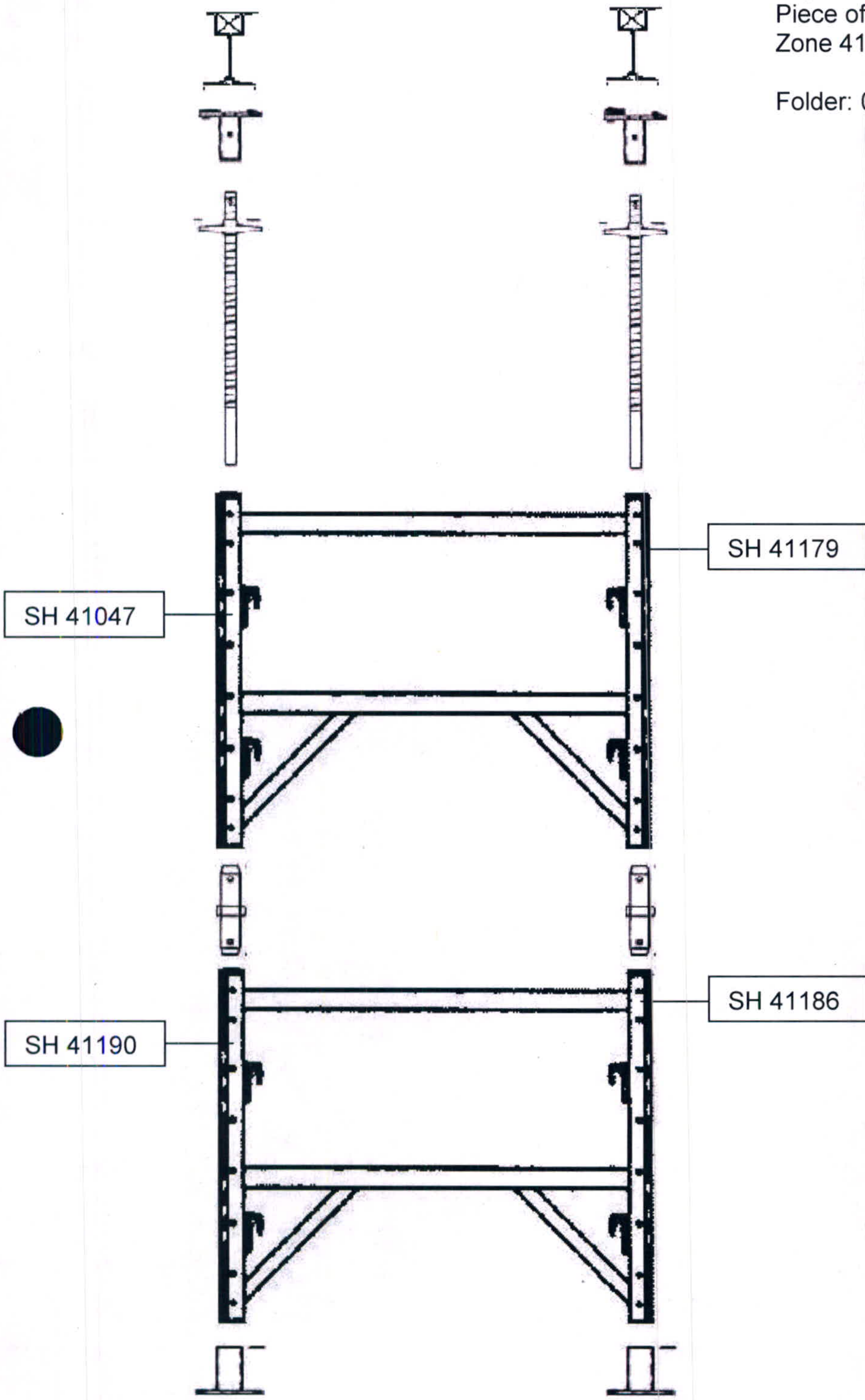
SH41178_1

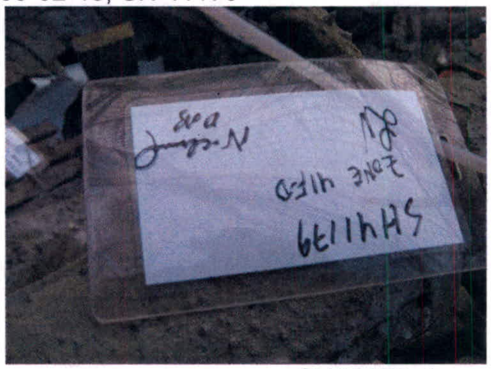


SH41178_2

SH 41179
Broken with SH 41047
Top Frame
Piece of SH 41177
Zone 41F-B

Folder: 08-02-15; SH 41179

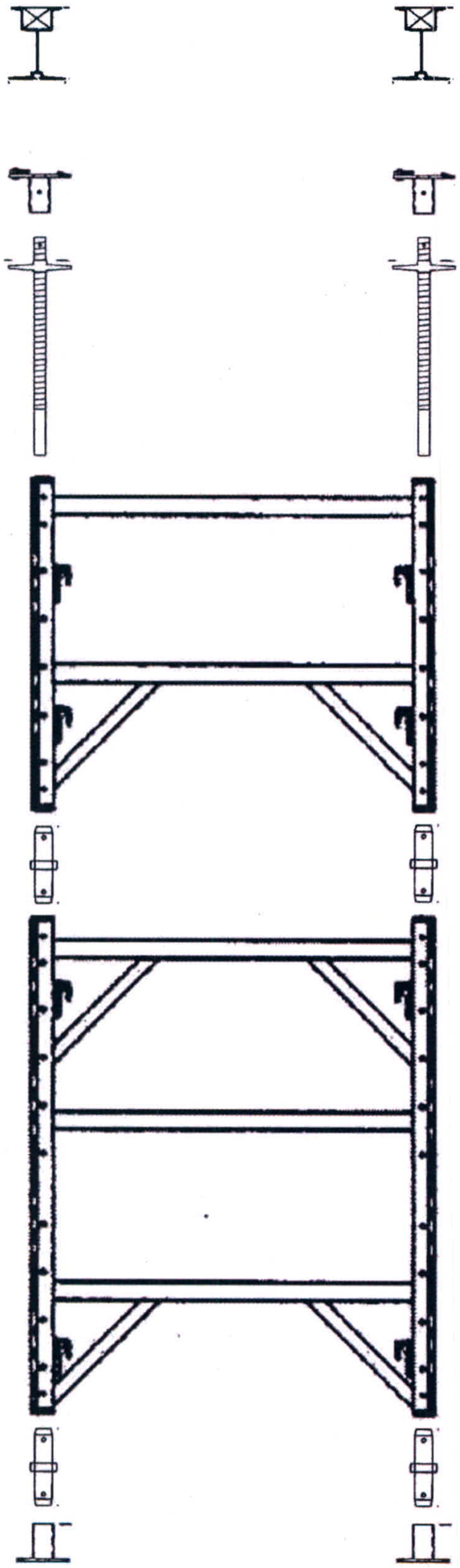




SH41179_1

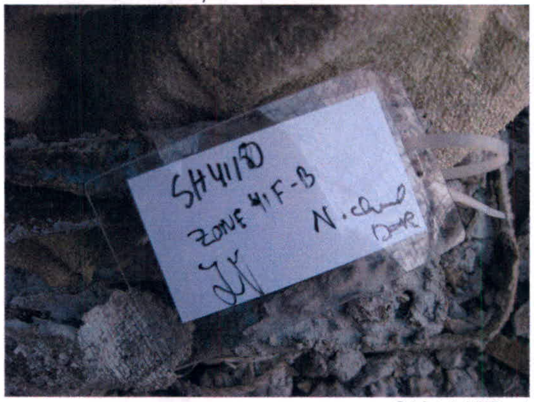
SH41179_2

SH41179_3



SH 41180
Brace
Zone 41F-B

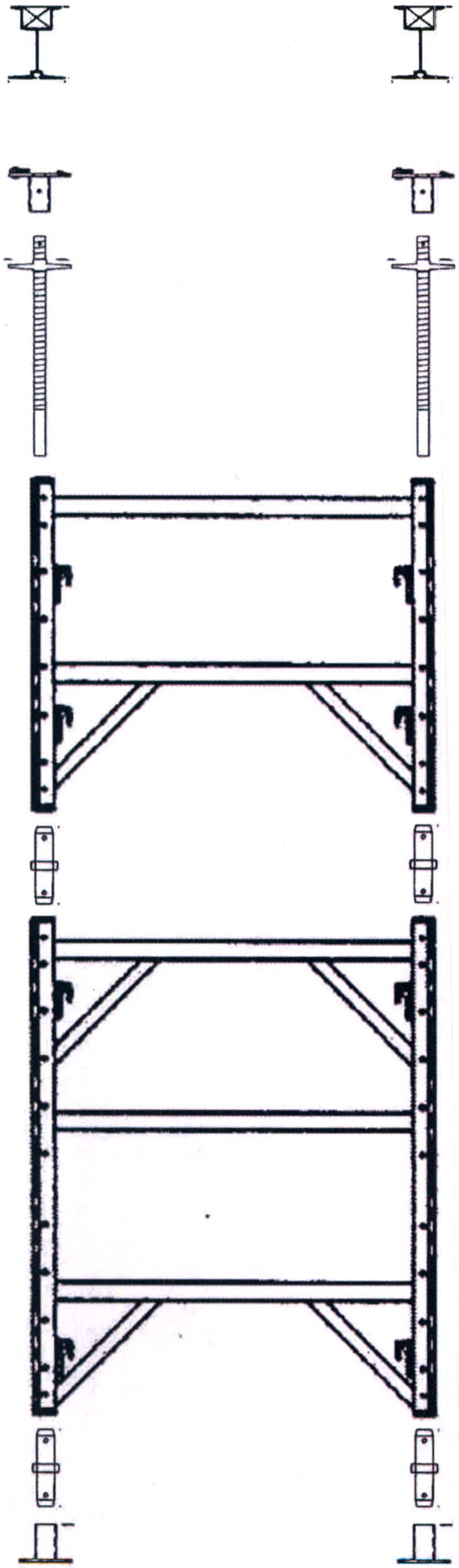
Folder: 08-02-15; SH 41180



SH41180_1

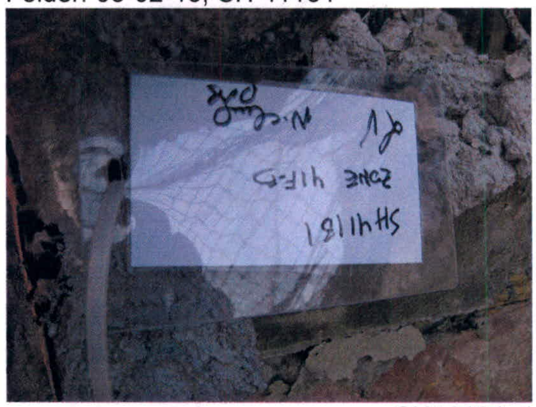
SH41180_2

SH41180_3



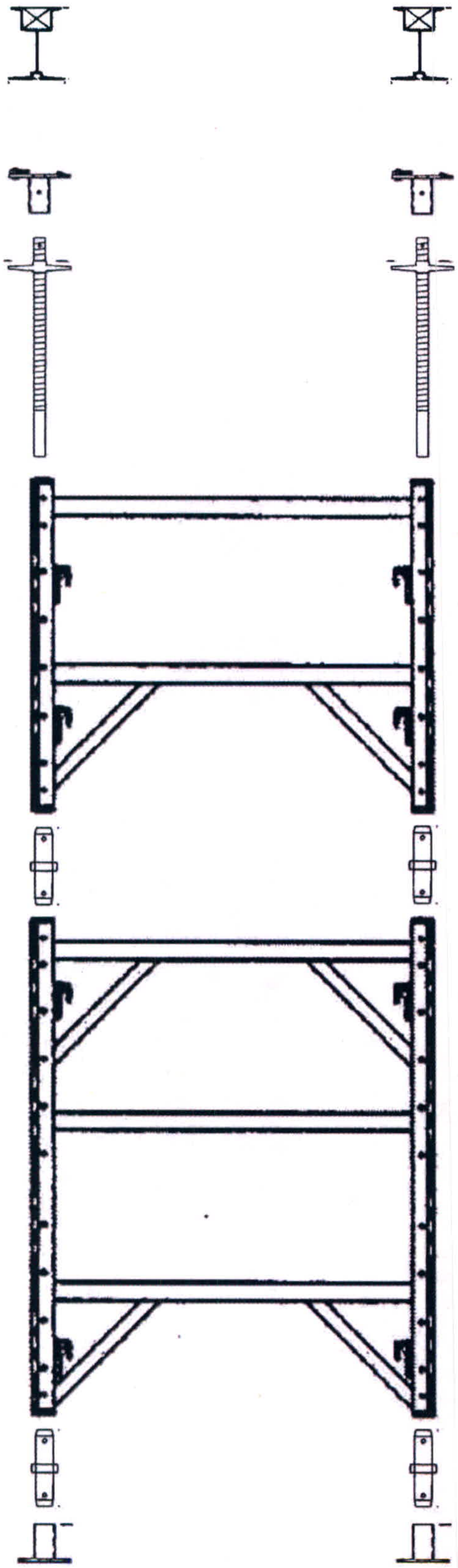
SH 41181
Top plate
Zone 41F-B

Folder: 08-02-15; SH 41181



SH41181_1

SH41181_2



SH 41182
Top plate
Zone 41F

Folder: 08-02-15; SH 41182

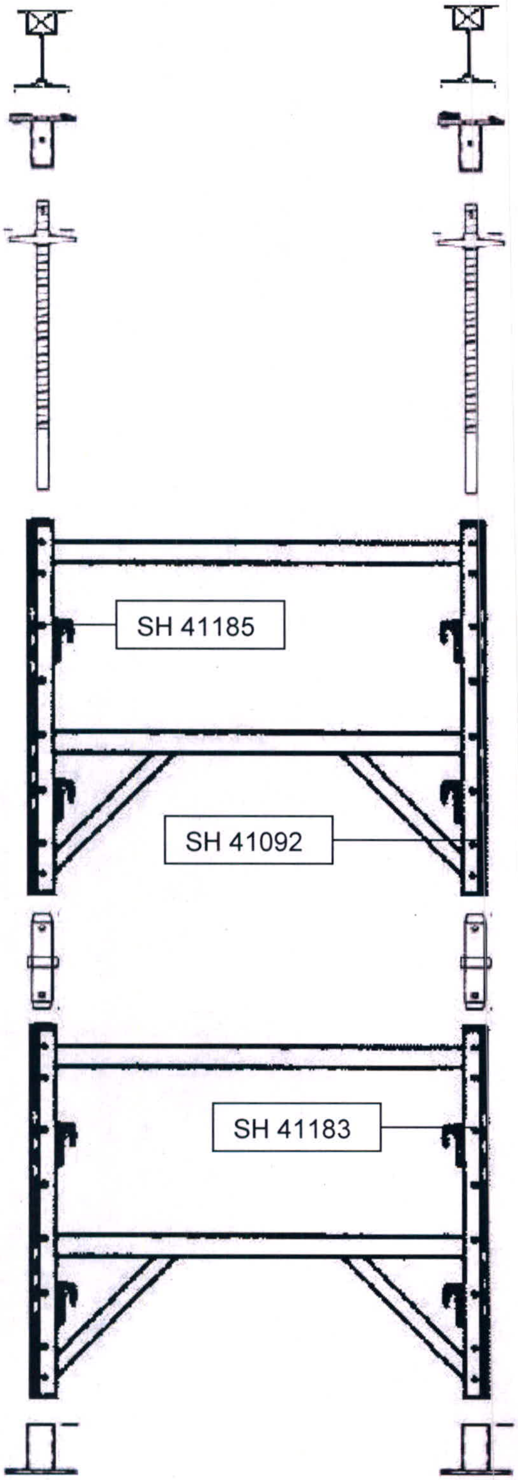
SH41182_1

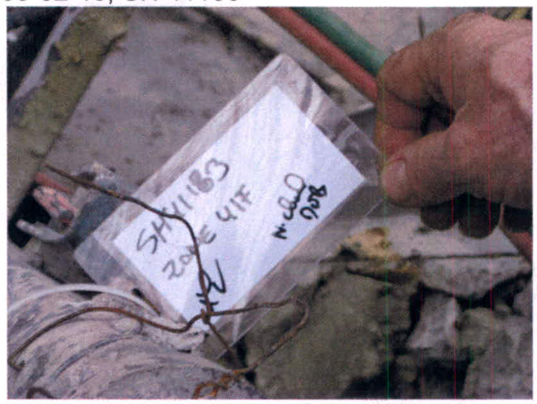
SH41182_2

SH41182_3

SH 41183
Bottom Frame of SH 41092 & SH 41185
Zone 41F

Folder: 08-02-15; SH 41183

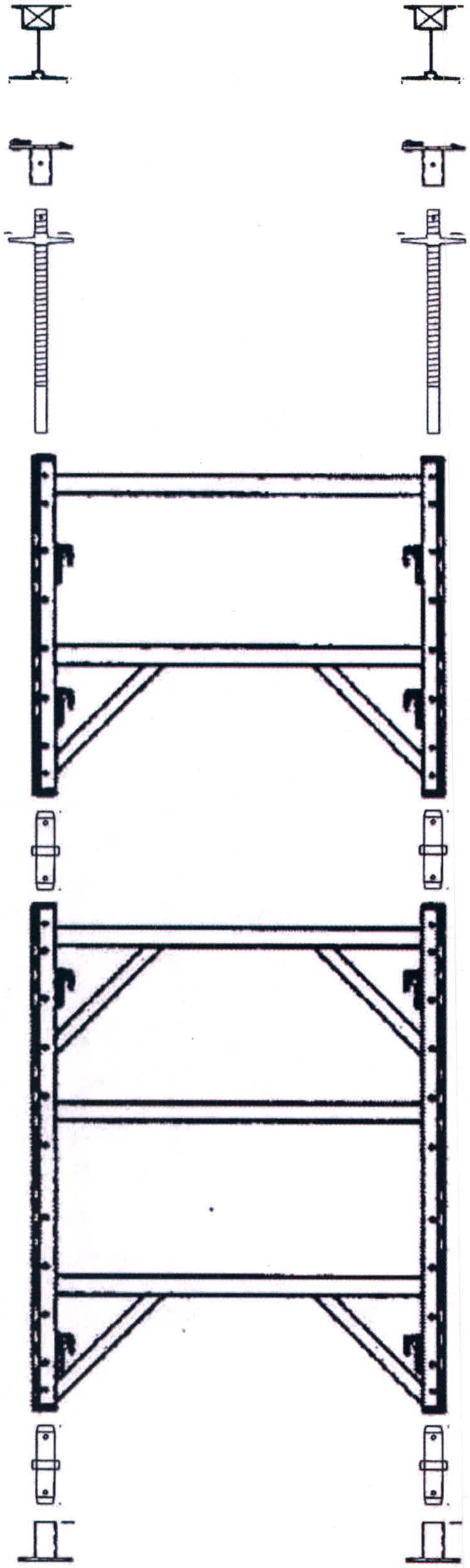




SH41183_1

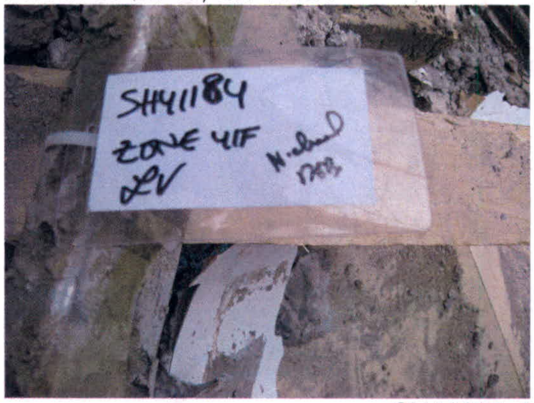
SH41183_2

SH41183_3



SH 41184
Piece of Frame attached to SH 41183
Zone 41F

Folder: 08-02-15; SH 41184

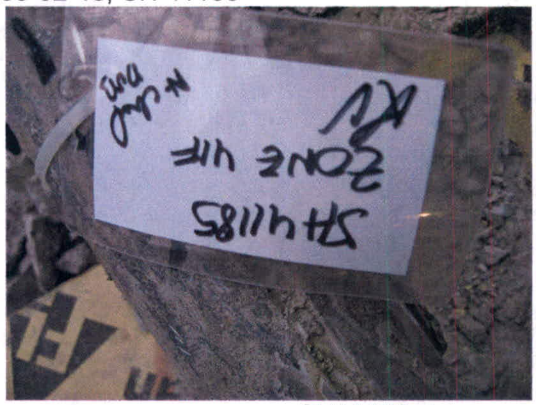


SH41184_1


SH41184_2

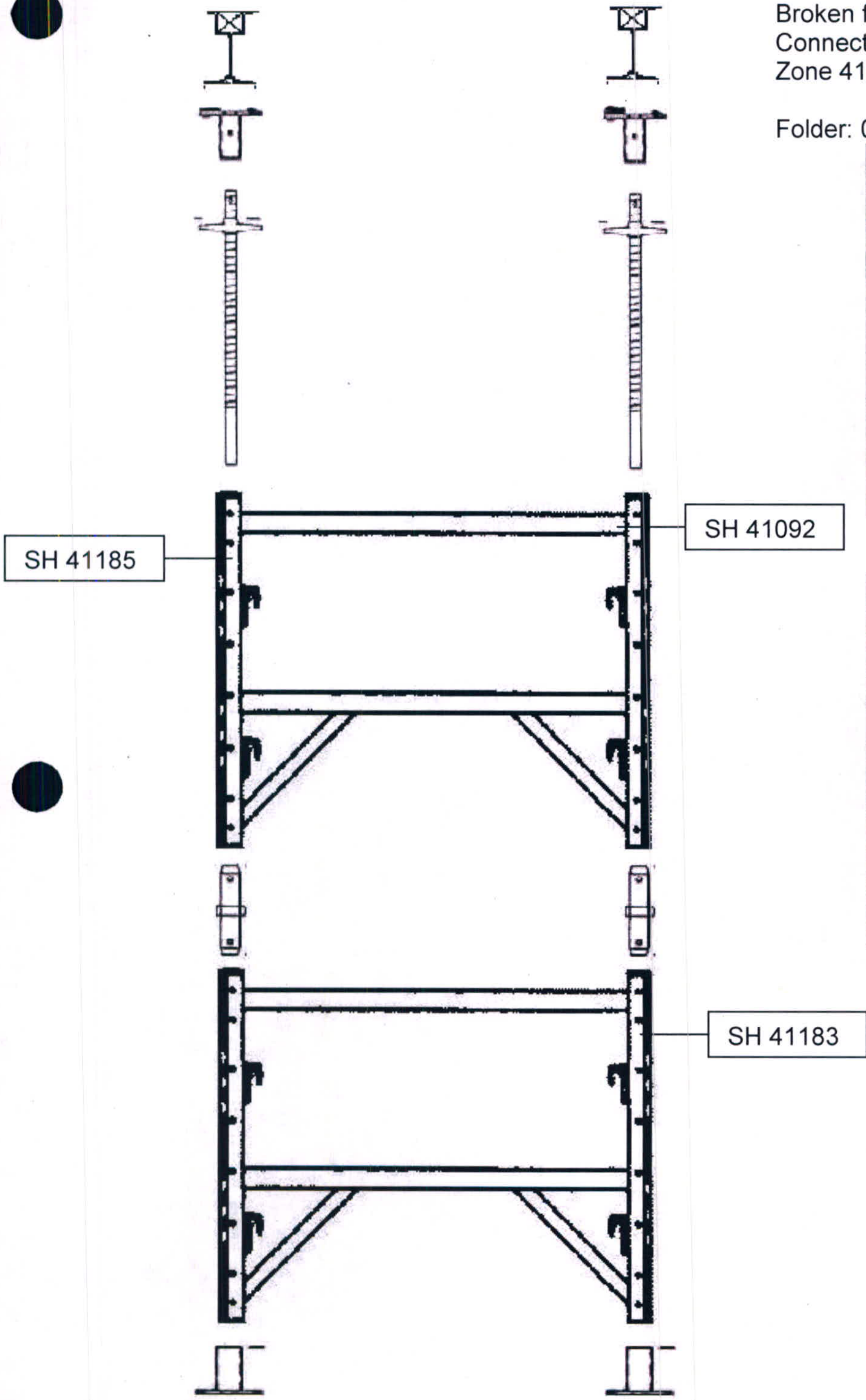
SH 41185
Broken frame attached to top SH 41134
Connected to SH 41133
Zone 41F

Folder: 08-02-15; SH 41185



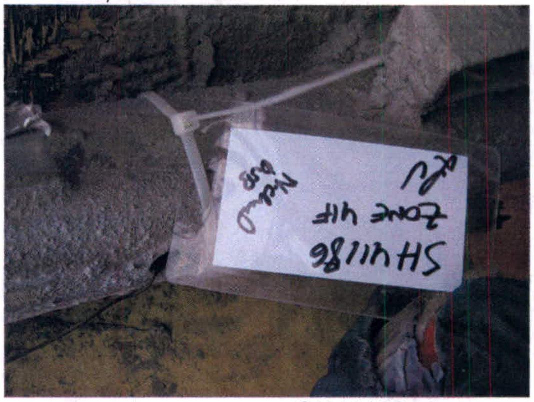
SH41185_1


SH41185_2



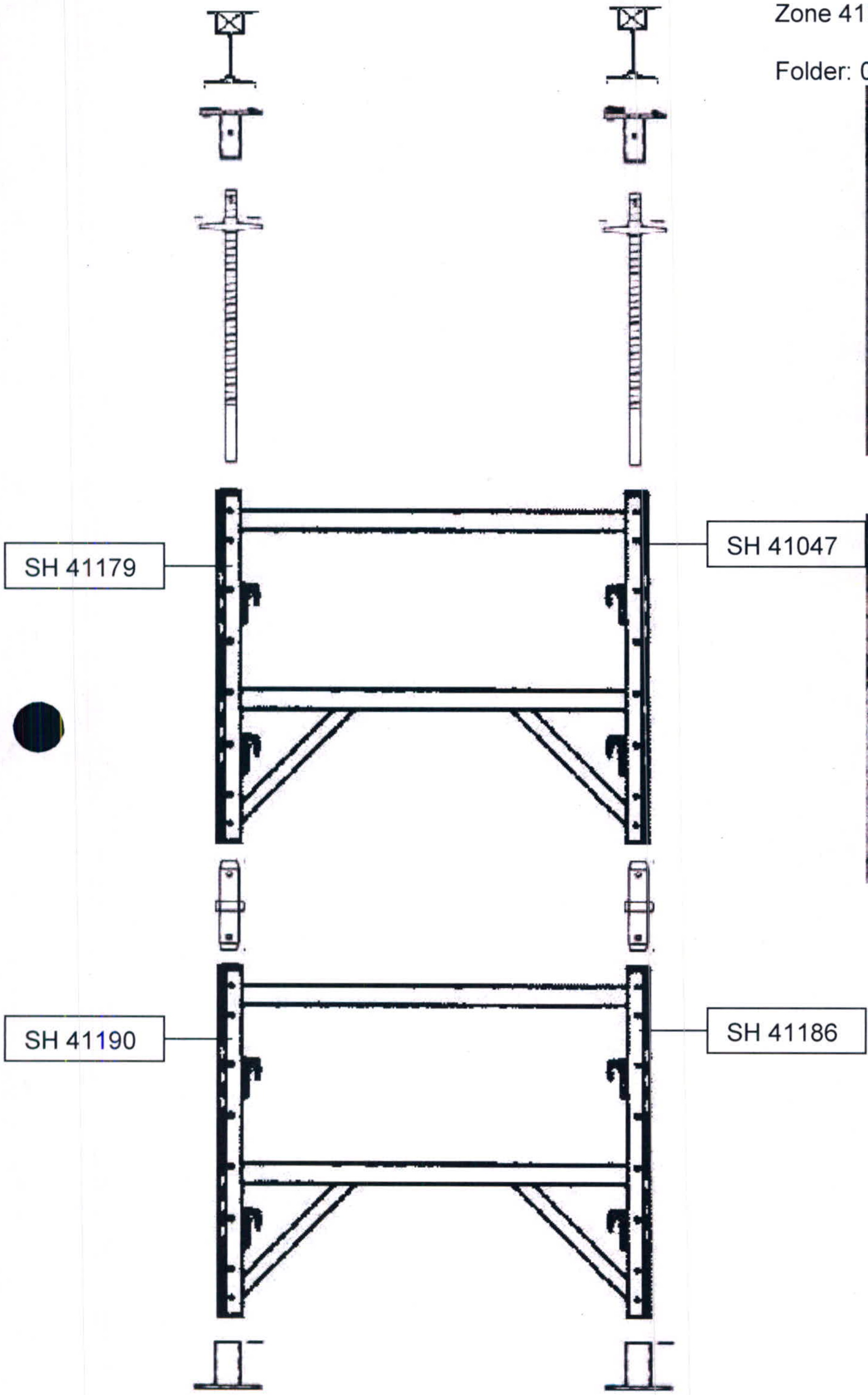
SH 41186
Broken with SH 41190
Bottom frame of SH 41179 and SH 41047
Zone 41F

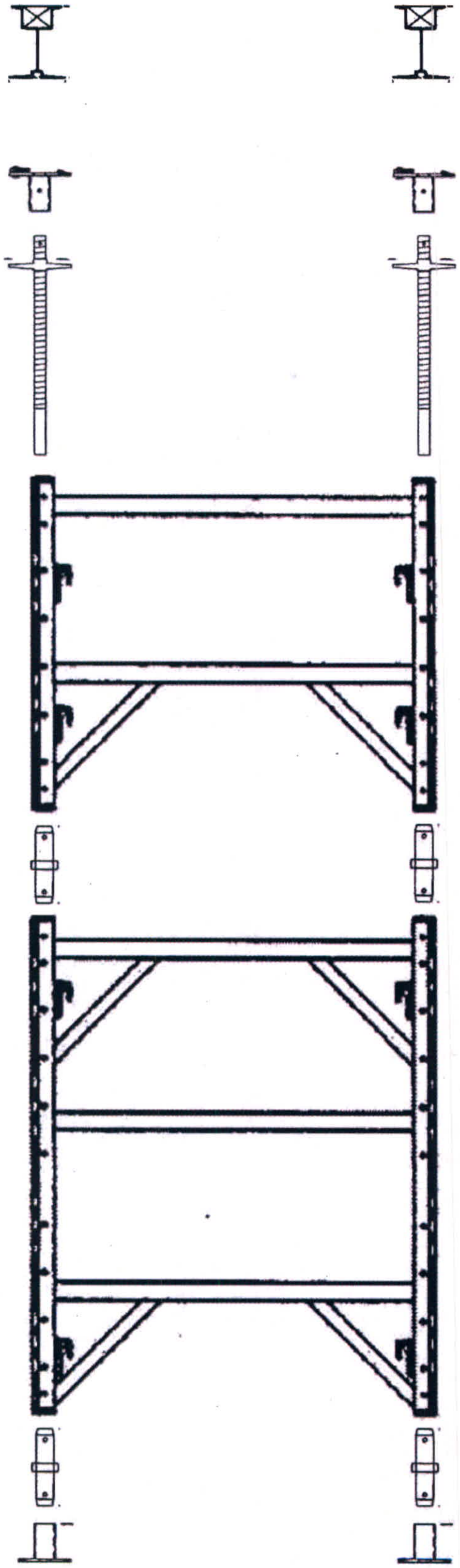
Folder: 08-02-15; SH 41186



SH41186_1

SH41186_2





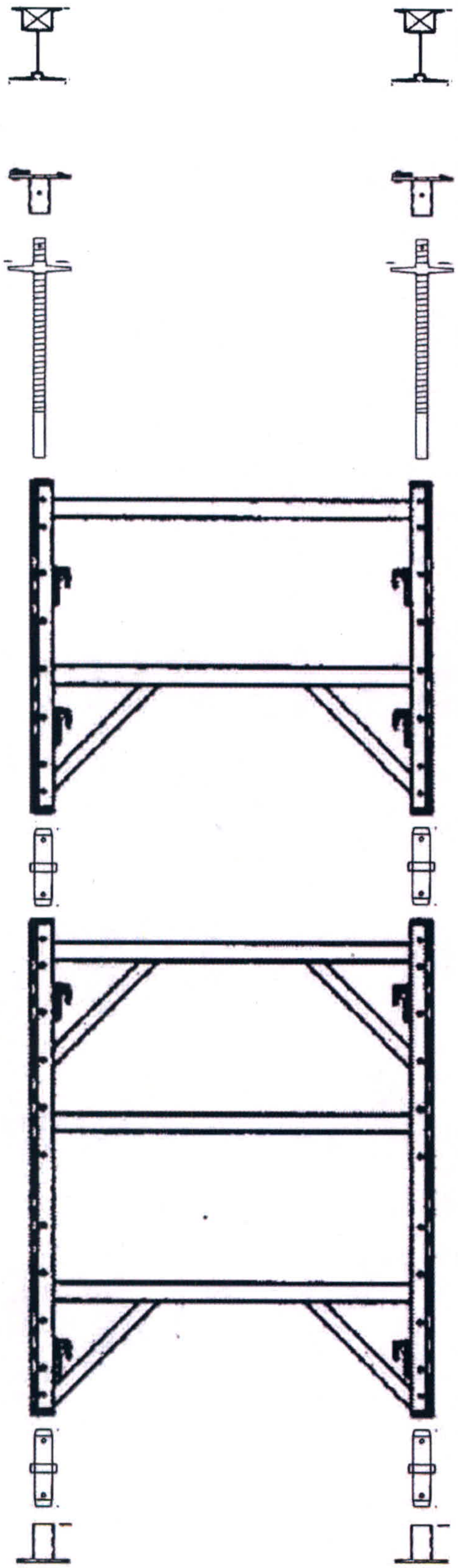
SH 41187
Piece of frame
Zone 41F

Folder: 08-02-15; SH 41187

SH41187_1

SH41187_2

SH41187_3



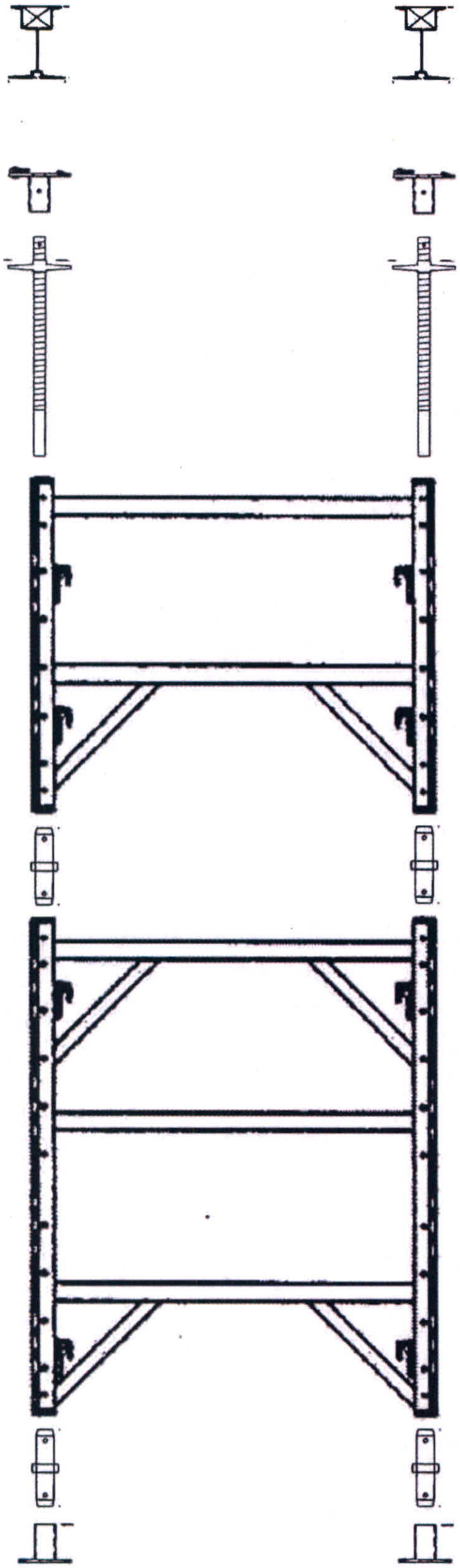
SH 41188
Screw Jack
Zone 41F

Folder: 08-02-15; SH 41188

SH41188_1

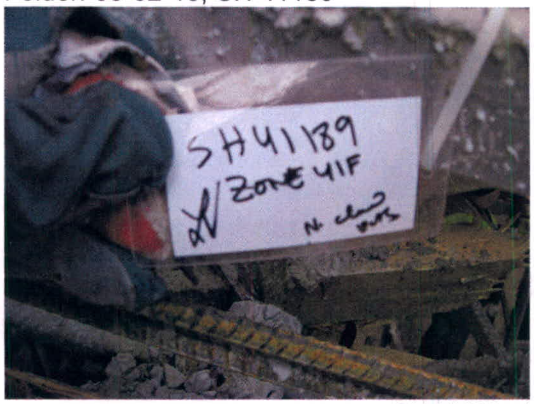
SH41188_2

SH41188_3



SH 41189
Piece of frame SH 41190
Zone 41F

Folder: 08-02-15; SH 41189

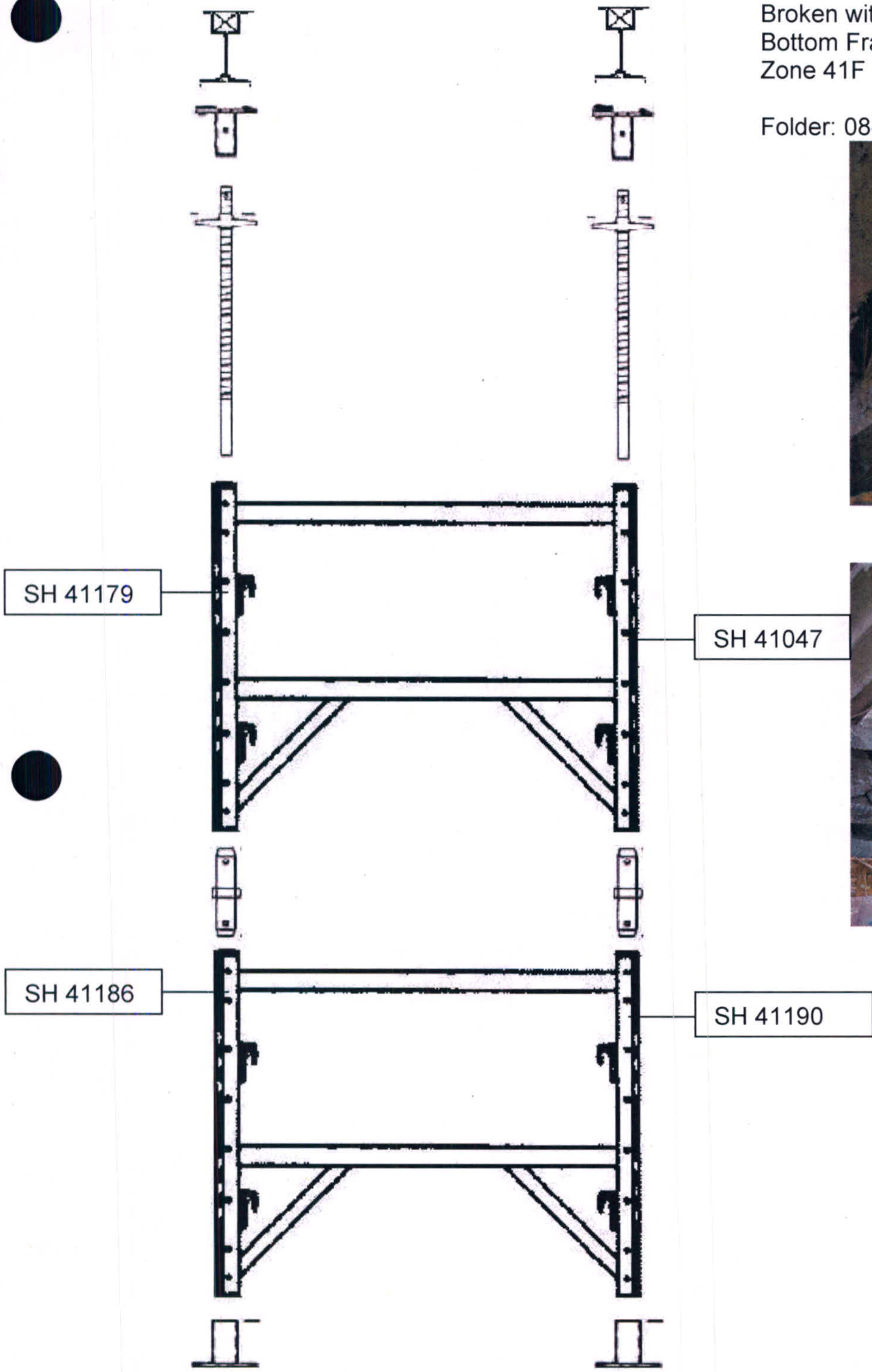


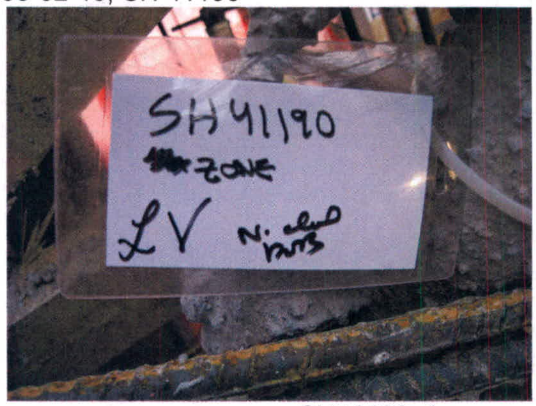
SH41189_1

SH41189_2

SH 41190
Broken with SH 41186
Bottom Frame of SH 41047 & SH 41179
Zone 41F

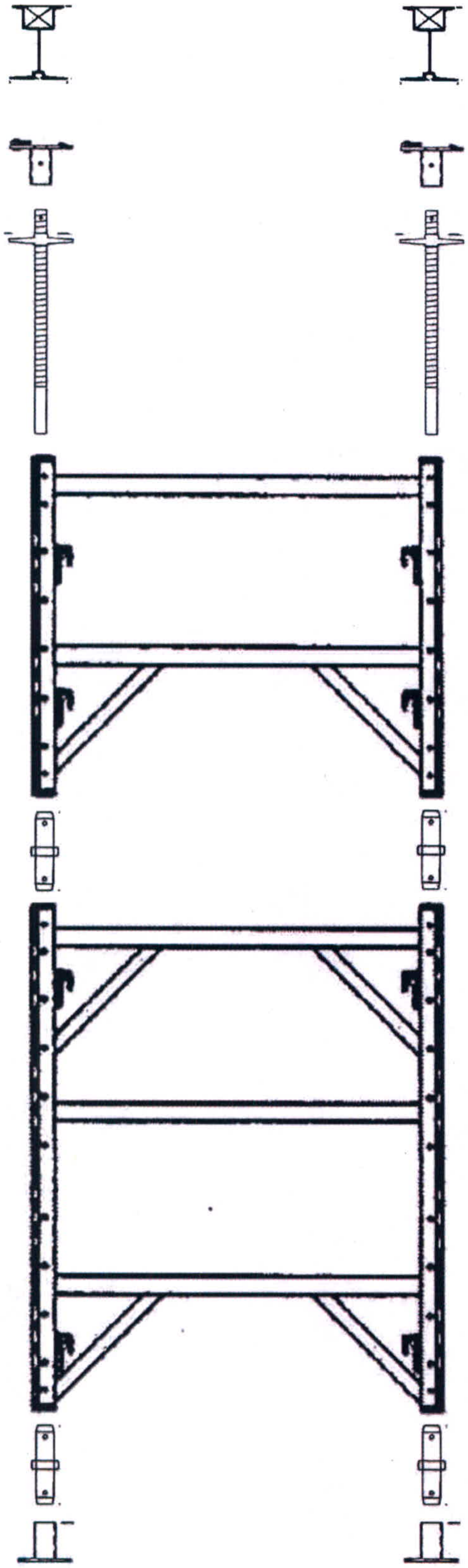
Folder: 08-02-15; SH 41190





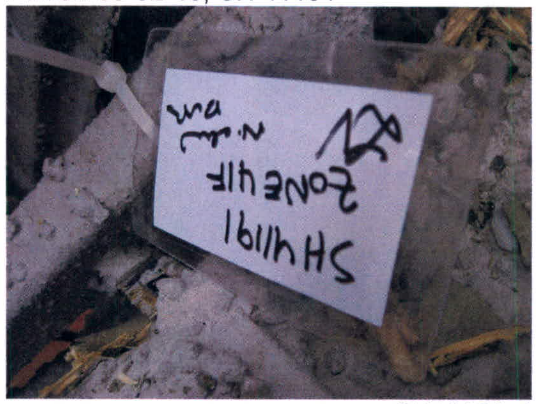
SH41190_1

SH41190_2



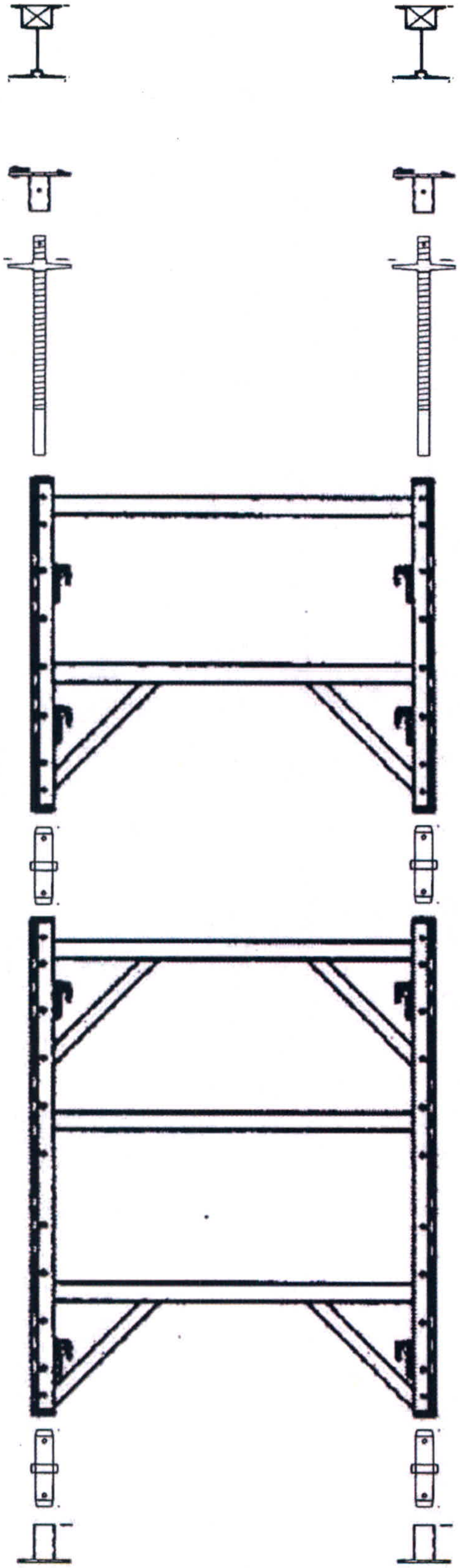
SH 411191
Part of SH 41190
Zone 41F

Folder: 08-02-15; SH 41191



SH41191_1

SH41191_2

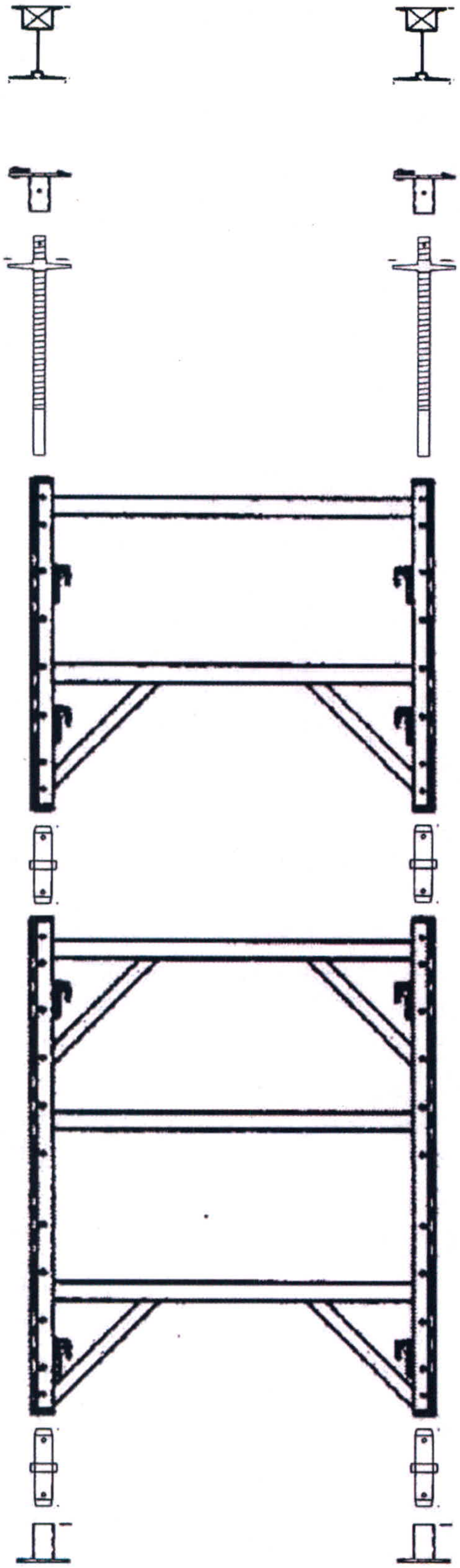


SH 41192
Coupler
Zone 41F

Folder: 08-02-15; SH 41192

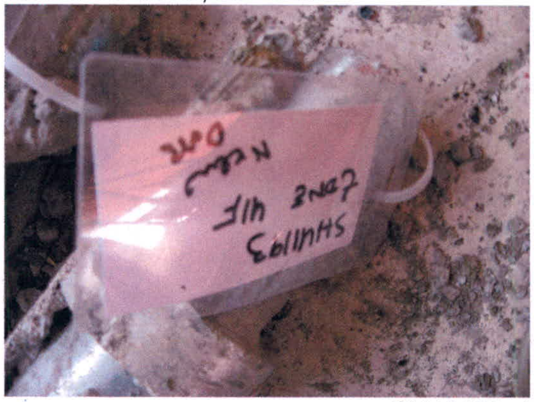
SH41192_1

SH41192_2

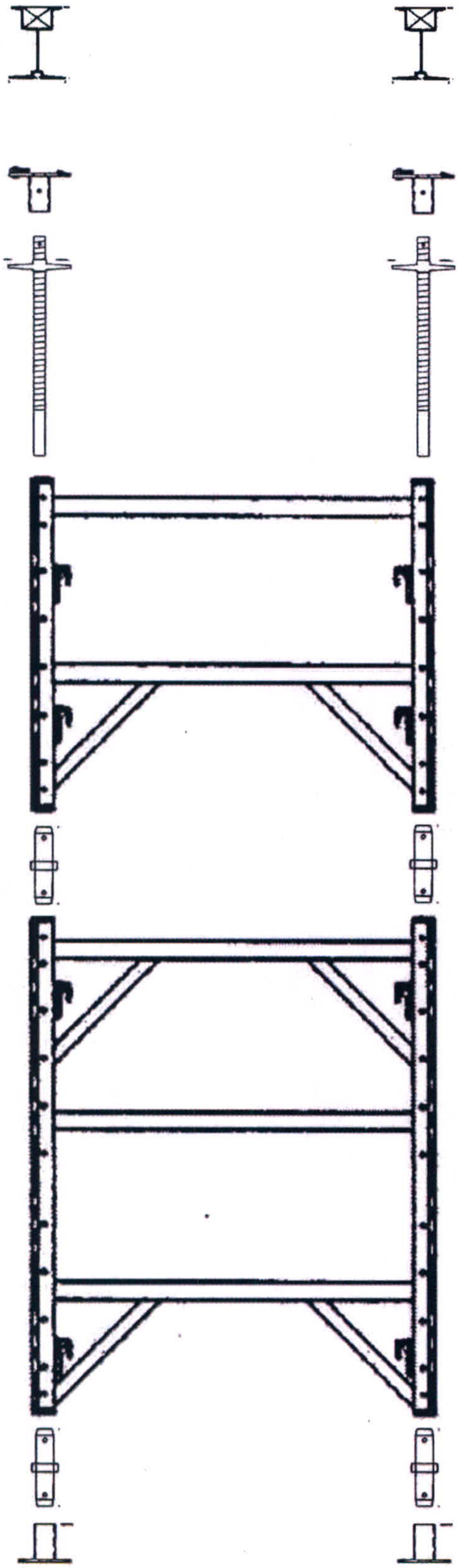


SH 41193
Coupler
Zone 41F

Folder: 08-02-15; SH 41193



SH41193_1



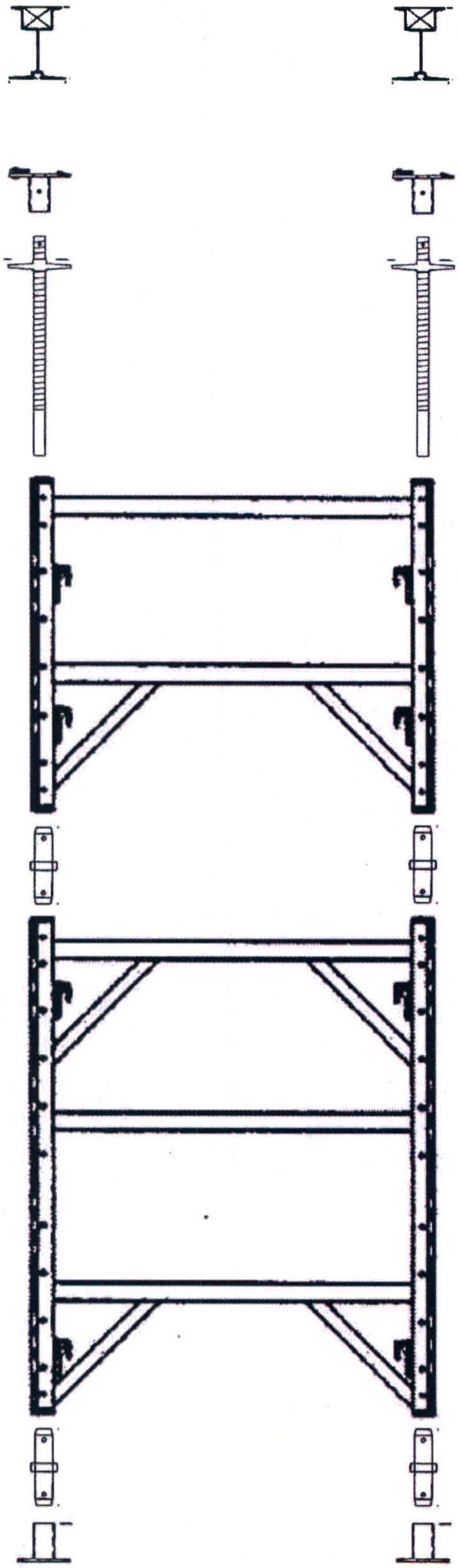
SH 41194
Bottom plate
Zone 41B

Folder: 08-02-15; SH 41194

SH41194_1

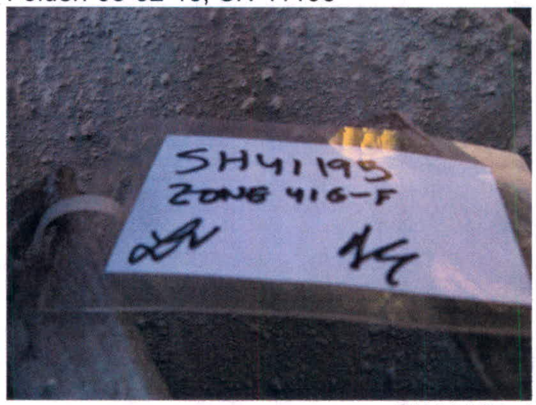
SH41194_2

SH41194_3



SH 41195
Brace
Connected to SH 41028
Zone 41G-F

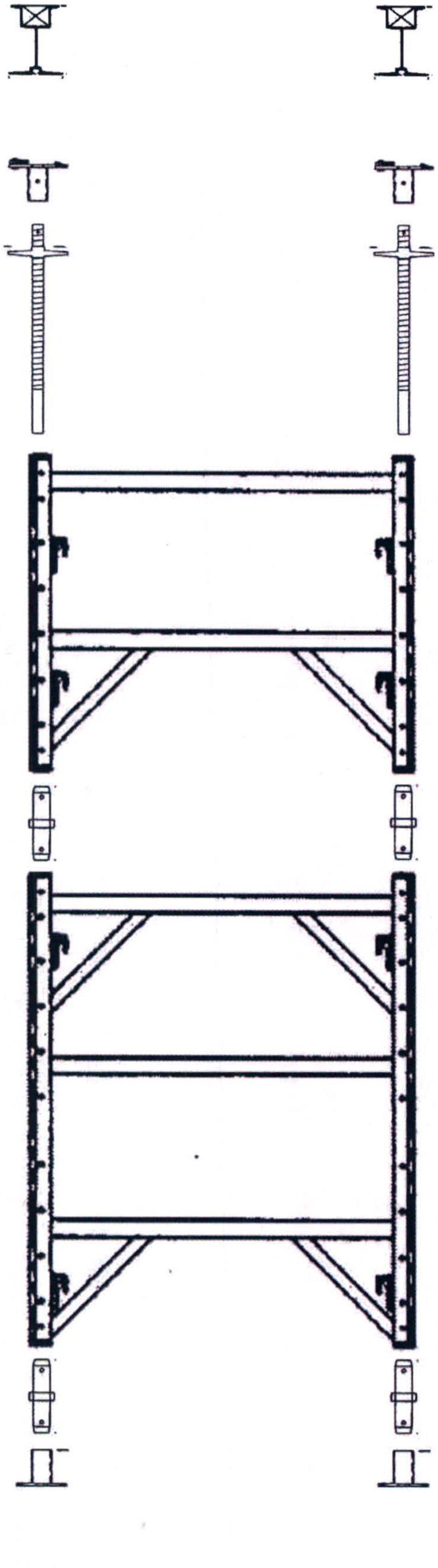
Folder: 08-02-16; SH 41195



SH41195_1

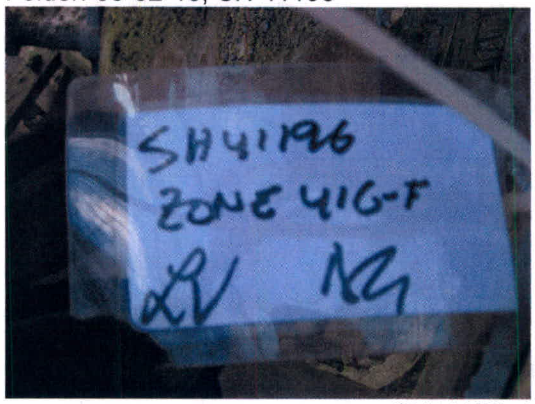
SH41195_6

SH41195_8



SH 41196
Top plate attached to timber beam on top of
SH 41195
Zone 41G-F

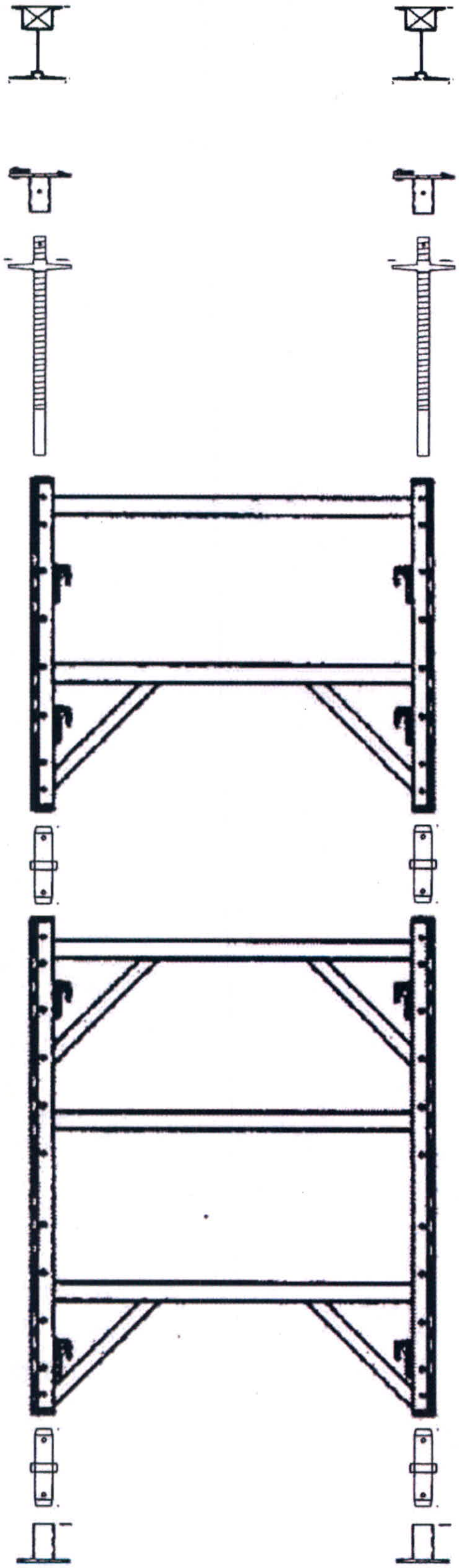
Folder: 08-02-16; SH 41196



SH41196_1

SH41196_2

SH41196_3

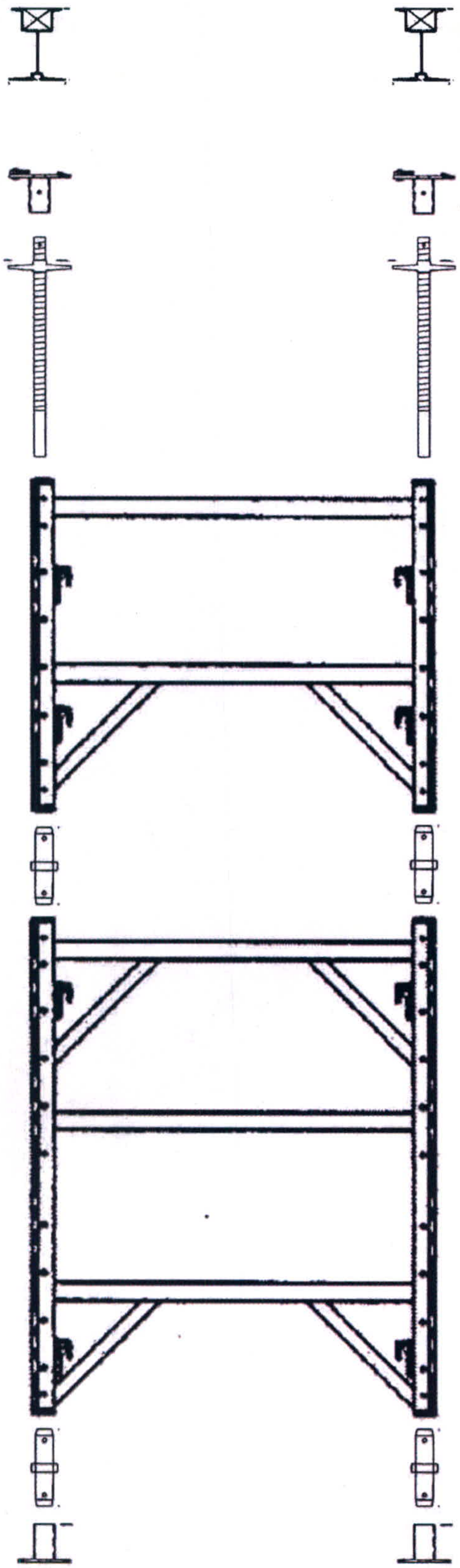


SH 41197
Bottom plate
41F

Folder: 08-02-16; SH 41197

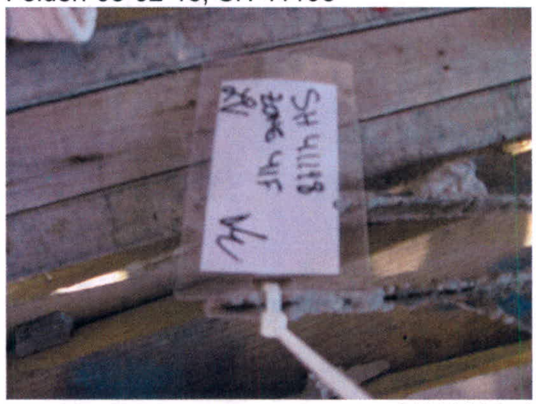
SH41197_1

SH41197_2



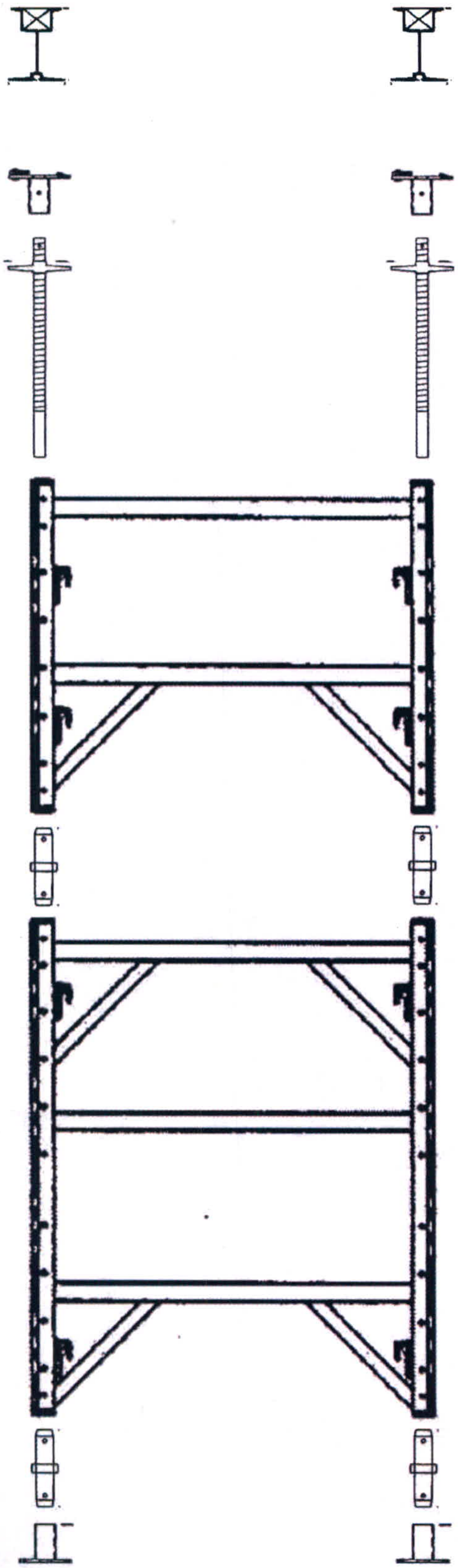
SH 41198
Brace
Zone 41F

Folder: 08-02-16; SH 41198



SH41198_1

SH41198_2



SH 41199
Brace
Zone 41F

Folder: 08-02-16; SH 41199

SH41199_1

SH41199_2