

# INVESTIGATION OF THE MARCH 22, 2012 PARTIAL BUILDING COLLAPSE DURING DEMOLITION AT 604-606 WEST 131<sup>ST</sup> STREET, NEW YORK, NY

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

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DEMOLITION AT 604-606 WEST 131<sup>ST</sup> STREET,  
NEW YORK, NY**

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## 1. Introduction

The Directorate of Construction (DOC), OSHA National Office, was asked to provide assistance in the investigation and causal determination of the March 22, 2012 partial building collapse during demolition of a building at 606 West 131<sup>st</sup> Street, New York, NY (see Figure 1 for location). As a result of the partial building collapse at the second floor, one employee was killed and two employees were injured. Our investigation and evaluation were based on the information provided by OSHA's Manhattan Area Office, and field observation made by a structural engineer from DOC.



Figure 1 Aerial View of the Building

## 2. Incident

The project consisted of demolition of a century old (1920), two-story warehouse building as part of Columbia University's expansion. Columbia University recently acquired the building. The original construction consisted of structural steel framing members with beams and girders encased in ash concrete for fireproofing, concrete masonry unit (CMU) bearing walls, and concrete slab on the second floor. The building was used as a warehouse and a commercial parking garage. See Figure 2 for second floor framing plan, drawn as part of the demolition plan.

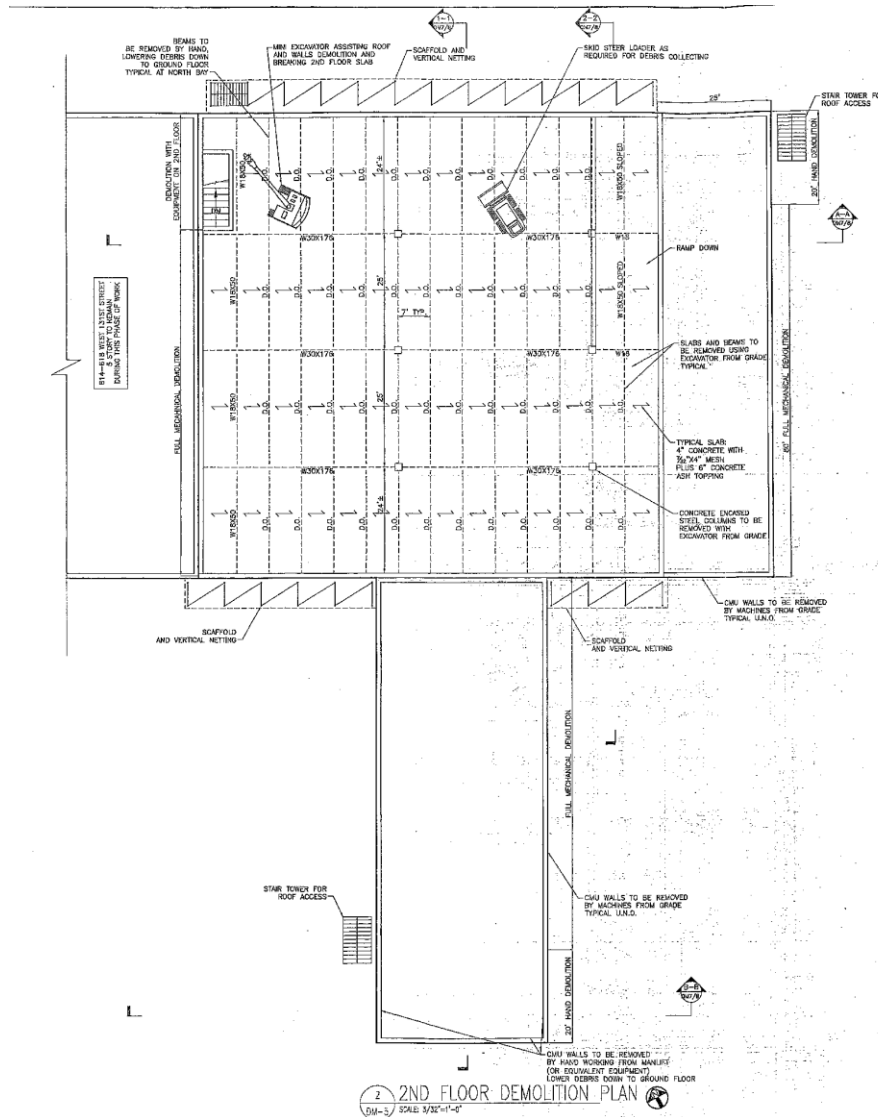


Figure 2 Second Floor Demolition Plan (Mr. Shapiro's drawing DM-005.00)

See Figure 3 for the original second floor plan, reproduced from the archives at the New York City Department of Buildings (NYCDOB).

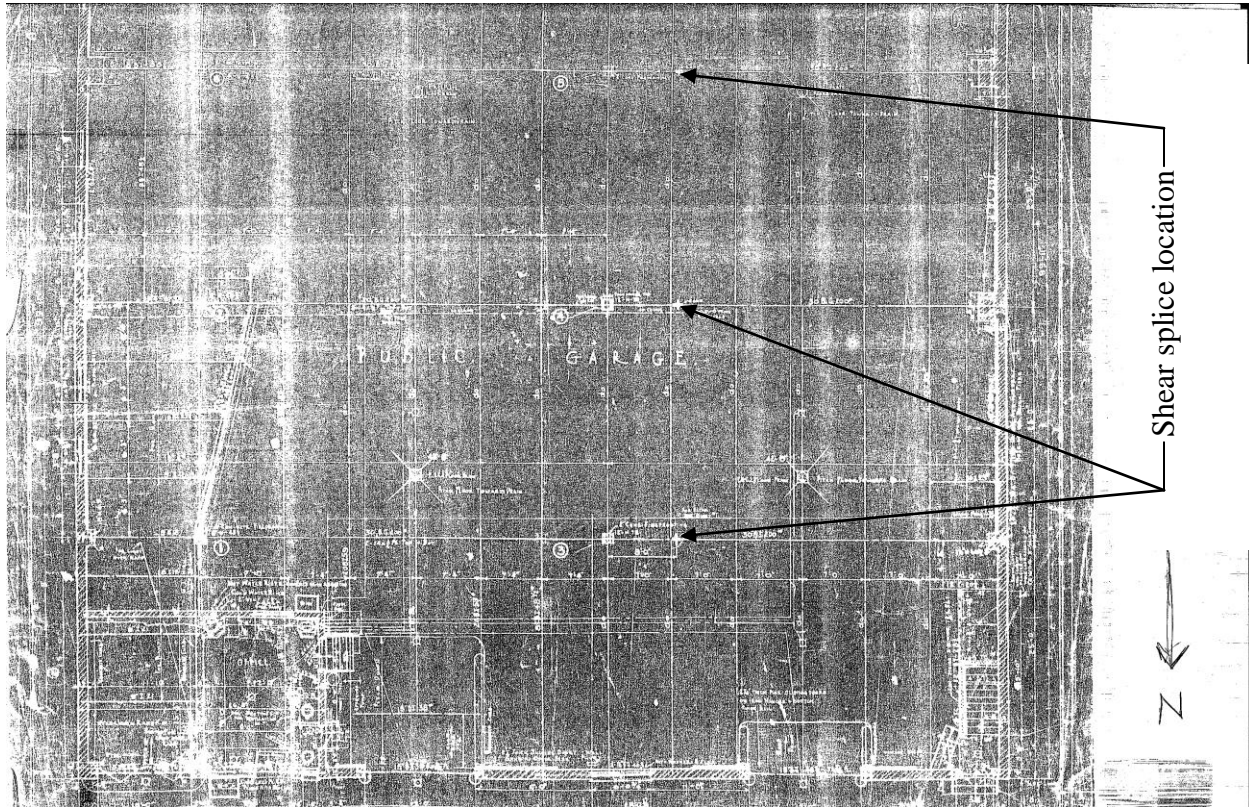


Figure 3 Original Second Floor Framing Plan (from NYCDOB)

The second floor consisted of 6" thick lightweight concrete slab spanning 7'-0" in an east-west direction and was supported by W18 filler beams, approximately 25' long, spanning in a north-south direction. The 18" deep filler beams (from grid line 1 to 3 and A to D) were supported by 30" deep girders, approximately 42' long, in east-west directions. The 18" deep filler beams from grid line 3 to 4 were supported by shallower girders (18" or 15" deep). The girders were supported over steel columns H 8x43½, and H 12x78 with cap plates and 8 bolts. The 30" girders at the west end and 18"/15" girders at the east end were supported by CMU pilasters/steel columns.



The general contractor of the project was Lend Lease of New York, NY who subcontracted to Breeze National Inc. (Breeze) of Brooklyn, NY for the demolition of the building. Construction management was included in Lend Lease's contract. Breeze retained a structural consultant, Howard I. Shapiro & Associates PC (Shapiro) of Lynbrook, NY, who prepared the demolition plans on November 14 & 15, 2011, later approved by the NY City Department of Buildings (NYCDOB) on January 31, 2012. Breeze started the demolition project on March 1. The contractor made good progress prior to the incident. The entire roof was demolished. CMU wall at the entire east end on grid line 4 and the south wall at grid line E from column lines 2 to 4 were demolished. Demolition of the second floor then began in earnest. Second floor filler beams (but not main girders) located east of grid line 2 from column line A to E were removed. The framing left in place two days prior to the demolition is shown in Figures 4 & 5.

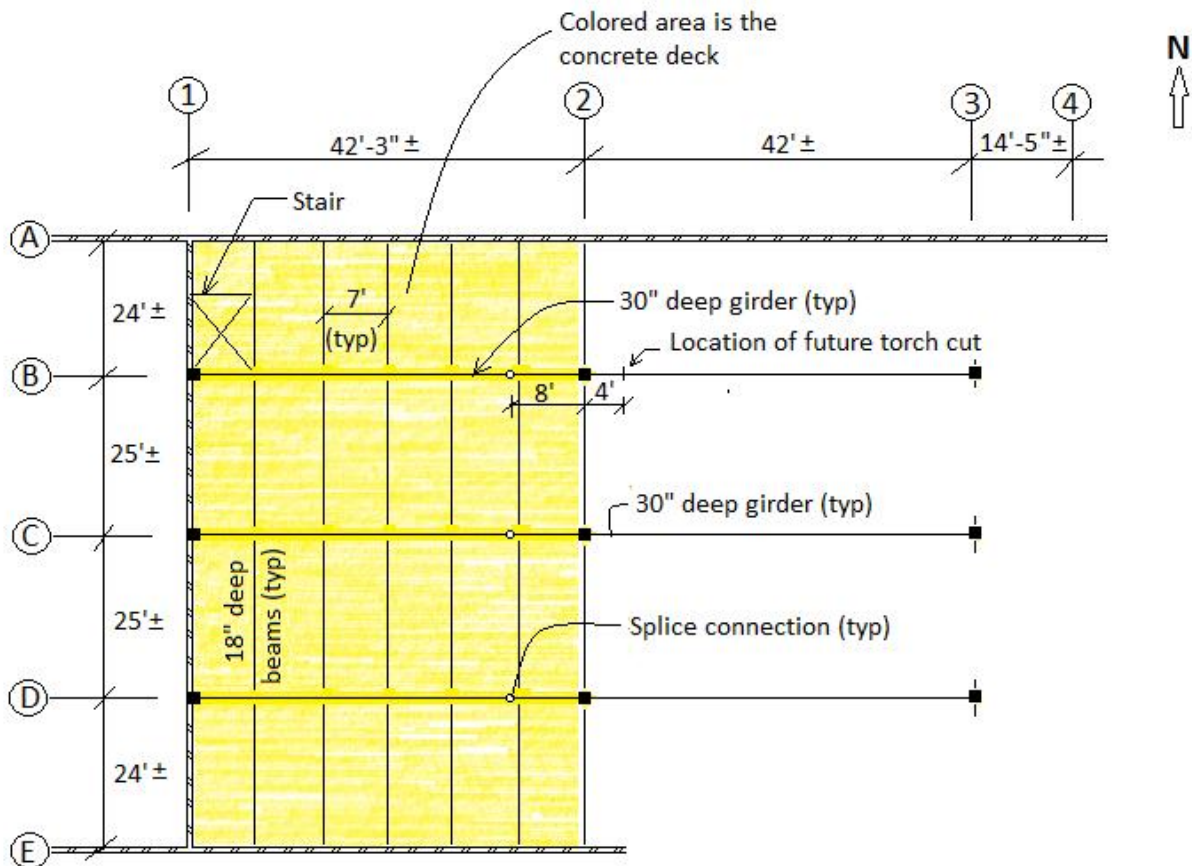


Figure 4 Second Floor Framing Plan (NTS) as it existed prior to the collapse



Figure 5 Second Floor Framing as it existed prior to the collapse

Framing to be demolished a day prior to the incident is shown in Figures 6 & 7.

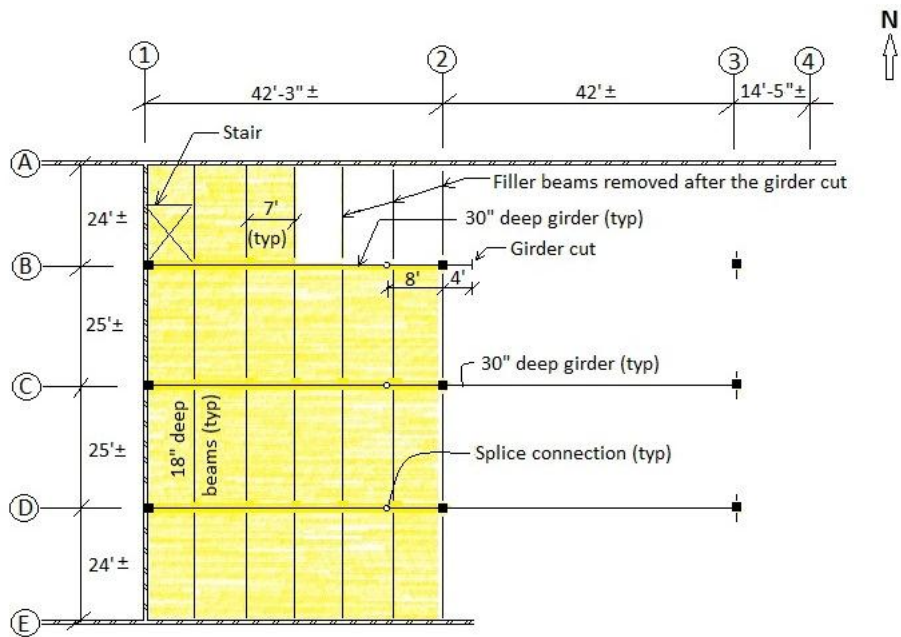


Figure 6 Second Floor Framing Plan (NTS) as it existed one day prior to the collapse



Figure 7 Second Floor Framing as it existed one day prior to the collapse

Framing to be demolished on the day of the incident is shown in Figure 8.

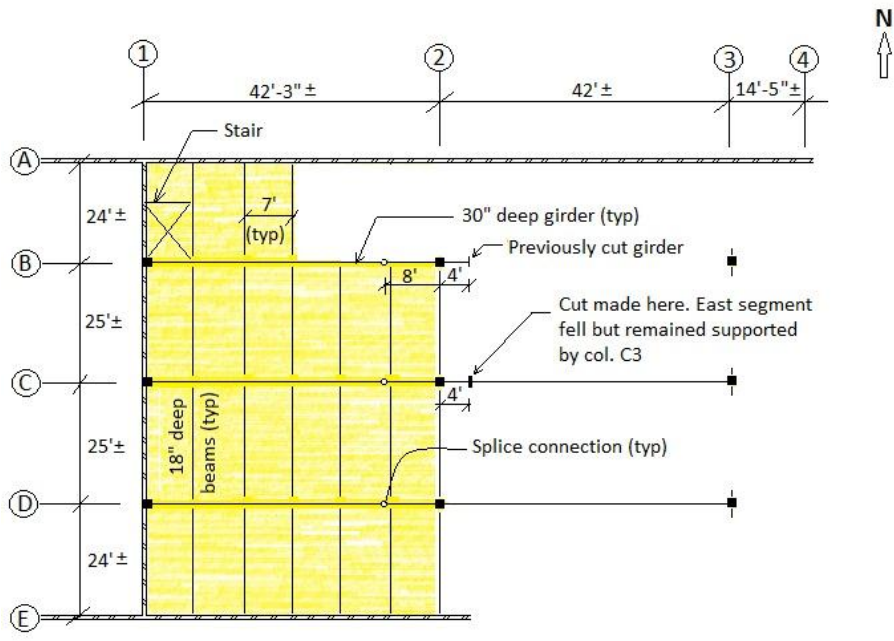


Figure 8 Second Floor Framing Plan (NTS) as it existed immediately prior to the collapse

Having removed some filler beams, the next step was to gradually remove girders on column lines B and C. First, the shallow girders from support column B3 to B4, and C3 and C4 were removed without any apparent difficulty. Next the girders from columns B3 and B2 and from C3 to C2 were to be removed. The contractor assumed that the girders on lines B and C spanned from column to column similar to the shallow girders they just removed. This assumption was incorrect because girders were spliced for shear connections 8'-0" away from the column. The contractor torch cut the girder between B3 and B2 some 4' away from column B2. The longer segment of the girder dropped on the ground leaving, 4' of overhang at column B2. After the cut was made, a wide crack developed at 8 ft. west of the column, which was later discovered to be the splice location, see Figure 10.

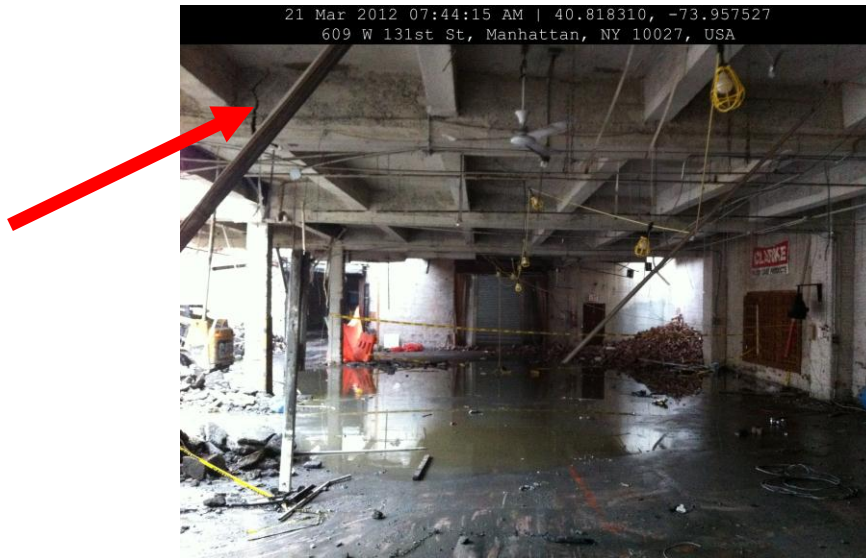
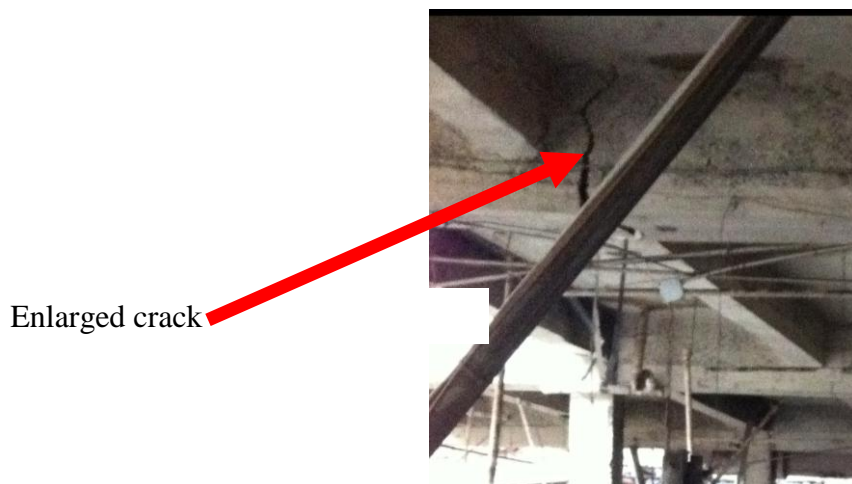


Figure 9 Photo showing crack on grid line B girder after torch cut was made two days before the incident



The existence of splices on column lines B and C were neither known to the contractor nor to the consultant, as they did not obtain the original drawings from the City of New York. The ensuing wide crack had afforded another opportunity to the consultant and contractor to discover the splice. This opportunity was missed. The contractor proceeded to pull down three more filler beams near the location of the girder splice. The operator then suddenly stopped and did not remove any other filler beams. It is highly likely that the crack that he would have noticed when he pulled down three beams prompted him to stop. That was an imminent danger signal, but it was ignored.

Despite the existence of the crack that developed after the girder on grid line B was cut, the next day, the contractor proceeded to torch cut the girder on grid line C some 4 feet away from column C2, similar to the cut made on grid line B. Soon after the cut was made, the collapse occurred, see Figure 10. Columns B2 and C2 remained standing upright but the segment of the girders on grid line B and C up to the location of the splice, along with two filler beams, fell in one piece towards the west. The longer girder segments on grid lines B and C, west of grid line 2, fell to the ground.



Figure 10 Second Floor Framing as it existed immediately prior to the collapse

After the incident (Figure 10), the following were observed:

- 30” girder at grid line C of the longer segment, east of grid line 2, fell but remained supported at column C3.
- The shorter segments of the 30” girder at grid lines B and C, from the torch cut location to the splice location, fell to the ground below along with two W18 filler beams and flipped over as one piece.
- Structural steel framing including concrete slab located between grid line 1 to 2 and A to D collapsed to the floor below.
- West CMU wall located on column line 1 between grid lines A to D collapsed.
- Of the three employees, one who was carrying scaffold frames across the floor near the west wall was killed and the other two employees, who were performing torch cutting, fell 13 feet below and were severely injured.

Post-incident observations from photographs revealed that the 30” girders located on grid lines C and B between column lines 1 to 2 failed at shear splice locations (Figures 11 to 14). The splice connection consisted of pairs of angles (5x4x5/16”x 1’-11½” long) riveted to the girder web and fastened back-to-back with 26-3/4” diameter square-head bolts (two rows of bolts at each angle, one row of 7 bolts and another row of six bolts), see figure below. Top and bottom flanges of the 30” girder were noncontiguous at the splice location.

### **3. Demolition Drawings**

The following observations were made concerning the demolition drawings DM-005.00 (Figure 2) prepared by Shapiro.

- Drawings were prepared without reviewing the original building drawings available at the New York City Department of Building (NYCDOB).
- The beam and girder sizes did not reflect the correct sizes as shown on the original document. The original girder size was 30BG200 against an indicated W30x176.

Similarly, the original filler beam size was 18BI48 ½ against indicated W18x50. The discrepancies in the sizes did not create any hazard.

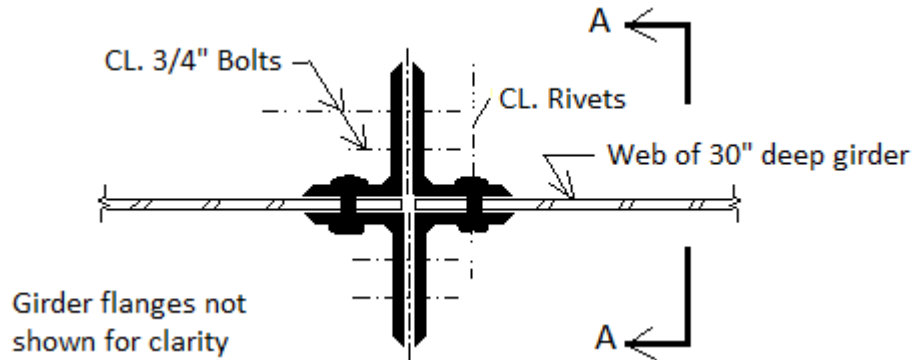
- The girders were shown from column to column. No splice connection on the girder was indicated. This was the major contributing factor to the collapse.
- The concrete floor slab thickness was shown as 4” plus 6” of ash concrete. No core drilling was performed to calculate the actual slab thickness.
- The demolition drawings did not provide specific instructions for removal of individual structural framing members such as beams, girders, and CMU walls. It only specified demolition tier-by-tier.

#### **4. Engineering Evaluation**

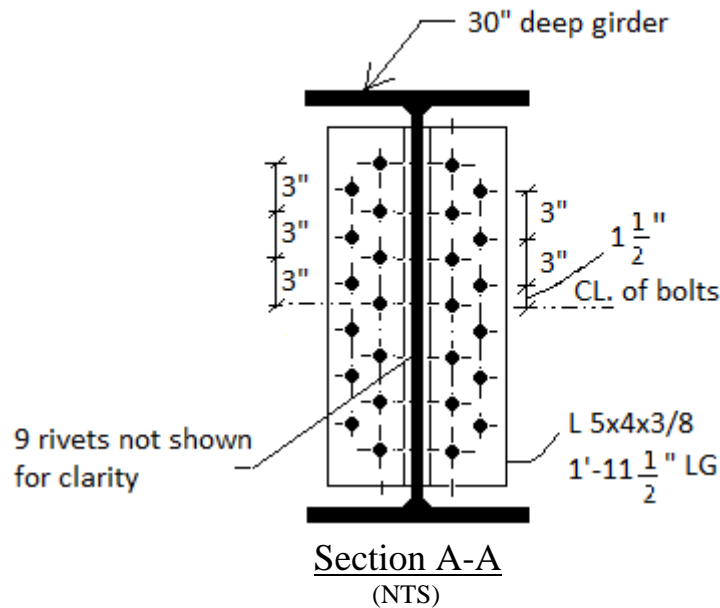
First, the structure as originally designed was evaluated to determine its adequacy. Beam sizes, spacings, and concrete thicknesses of the deck were taken from the existing structural drawings, prepared around 1914. The thickness of the fireproofing concrete around the girder and filler beams was assumed to be 2”. The drawings did not indicate the grade of steel and the ASTM classification of the bolts used in the connections. Steel Design Guide Series 15, published by the American Institute of Steel Construction (AISC), was used to obtain a reference for historic shapes and specifications. In accordance with the guide, the steel was assumed to have a yield strength of 33,000 psi. The ¾” bolts were assumed to have an ultimate tensile strength of 45,000 psi. Our computations generally indicated that the original design was satisfactory for the design loads. No reduction in cross sectional area was taken in the computations due to corrosion, which was observed to be minimal. Other assumptions made in our evaluation are listed below.

1. The thickness of the ash concrete slab was 6 inches, lightweight concrete weighing 90 pounds per cubic foot.
2. The average weight of the steel filler beams was considered to be 46 pounds per linear foot.
3. The weight of the girder was considered to be 200 pounds per linear foot.
4. The weight of the ash concrete as fireproofing around steel beam was considered using 2” concrete cover around the edges except on the top flange.
5. The live load for the warehouse and storage was considered as 150 pounds per square feet.

Of greater interest to this investigation was the analysis of the girders on column lines B and C, between column lines 3 and 1. The girder on column line B was continuous from column 3 and extended approximately 8 feet beyond column 2, where it was connected to another girder that spanned up to column line 1. The girder on column line C had a similar arrangement. The existing structural drawings available did not contain the details of the splice connection between the two girders. However, the splices were preserved at the site and were examined, post-incident, to obtain relevant information. Two angles, 5x4x3/8 were riveted to the web of the girders with 9 rivets. The outstanding 5" legs were bolted back-to-back with 13 bolts, see Figure 10. There were no connections between the flanges of the two girders, rendering the connection devoid of any moment capability. Hence, it was a customary hinged connection.



Partial Plan (Existing shear splice connection detail)  
(NTS)



Section A-A  
(NTS)

Figure 11 Existing Shear Splice Connection Detail



Since the girder on column line B was cut some 4 ft. away from column B-2, the girder was analyzed for the loads that existed on the day the girder was cut. Only dead loads were considered. Live or construction loads were not considered. Two filler beams between column lines A and B which were already removed were not accounted for. A flexural moment of 231 ft. Kips was calculated at the splice location between B-2 and B-1. Similar analysis was done for the girder on column line C with due consideration of all loads present on the day of the collapse. Again, no live load or construction load was considered. A flexural moment of 306 ft. Kips was calculated.

Next, the capacity of the splice to resist flexure was evaluated. It was determined that although the splice was designed as a hinge, it could have had some capacity to resist flexural moment. It was determined that the splice could only resist approximately 50% of the imposed moment. Under the imposed load, the  $\frac{3}{4}$ " bolts were subjected to shear and tensile loads. The bolts on the column line C girder splice were subjected to stresses well beyond their ultimate capacity of 45 ksi, and hence contributed the failure.

There needed to be an explanation as to why the girder on line B did not collapse when a similar cut was made on column line B, a day earlier. The girder on line B had a relatively lesser load, approximately 80% of the load on girder C because at least three filler beams were removed prior to the cut. Besides, the ultimate tensile strength of the  $\frac{3}{4}$ " bolts could have been greater than the published value of 45 ksi. However, the splice on the column line B girder was in fact severely distressed, though not collapsed, as is evidenced by a photograph shown in Figure 9, taken on the day the cut was made on the column line B girder. As discussed earlier, a look at the distressed splice with a wide crack which was in plain sight of everyone should have alerted the contractor and the contractor's consultant of the imminent danger of a collapse.

## 5. Conclusions

1. Two primary steel girders of the main framing of the building were dismembered in a manner that overstressed the existing shear splice connections, leading to the collapse. This resulted in one fatality and injuries to two other workers. OSHA standard 1926.858(d) was violated.
2. The contractor retained an engineering consultant to prepare demolition drawings, but the consultant prepared the drawings without obtaining the original drawings of the structure available at the New York City Department of Buildings. This was a deviation from the standard industry practice, and contributed to the unplanned collapse. If the original drawing was obtained, the consultant/contractor would have become aware of the existence of the shear splice connections.
3. When the first primary girder was dismembered two days before the incident, a large crack developed in the adjoining span of the same girder in plain sight of everyone. The contractor and his consultant failed to notice the crack, and proceeded to dismember the next primary girder in a similar manner at which time both girders collapsed along with other filler beams. This was a serious lapse of judgment on the part of the contractor and his consultant.
4. If all the filler beams were removed sequentially before dismembering the primary girders, then the existing shear splice connections would not have been overstressed. In this manner, the incident could have been averted.
5. The consultant's drawings showed the framing of the building which did not represent the actual framing, thus misleading the contractor. This contributed to the collapse.

## 6. References

1. Original Architectural drawing of Second Floor Plan obtained from New York City Department of Building.
2. Original Structural drawing of First Floor Framing Plan obtained from New York City Department of Building.
3. Demolition drawings, DM-001.00 to DM-010.0 prepared by Howard J. Shapiro & Associates, dated 11/15/11.
4. Photographs of the incident taken by the OSHA's Manhattan Area Office Compliance Officer.
5. Steel Design Guides Series 15 "AISC Rehabilitation and Retrofit Guide" published by American Institute of Steel Construction, 2002.