

INVESTIGATION OF THE NOVEMBER 8, 2011 PARTIAL COLLAPSE OF A BUILDING UNDER CONSTRUCTION AT 2929 BRIGHTON 5th STREET, BROOKLYN, NY

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

May 2012



This photograph was taken by Paul Martinka on November 8, 2011 at 3:13:12 PM

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OF A BUILDING UNDER CONSTRUCTION
AT 2929 BRIGHTON 5th STREET,
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1. Executive Summary

On November 8, 2011, at approximately 2:30 p.m., the front bay (west side) of the five-story building under construction at 2929 Brighton 5th Street in Brooklyn, NY suddenly collapsed killing one employee and seriously injuring four others. At the time of the incident, concrete was being pumped to the third floor. Hours earlier, concrete had been poured on the fourth floor without any apparent difficulty, although workers noticed swaying and vibrations of the floor. The building was a hybrid construction of load-bearing light metal framing with interior steel rolled shaped beams and columns, and masonry walls at the core.

Personnel from the Manhattan Area Office of the Occupational Safety and Health Administration, Region II, arrived at the scene within hours of the incident. The OSHA investigation began soon thereafter; it included gathering facts, interviewing witnesses, observing the failures, obtaining contract documents, and conducting structural analyses. The Regional Administrator, Region II, asked the Directorate of Construction (DOC), OSHA National Office, Washington, DC to provide technical assistance in this matter. A structural engineer from DOC visited the incident site and provided engineering assistance.

Based on the findings of the investigation, it is concluded that:

1. The building under construction partially collapsed because the contractor failed to maintain the stability of the front bay of the structure. All joints of the front bay (west side) at the third, fourth and the fifth floors were “hinged”, and had minimal flexural rigidity. Under lateral load, the front bay separated from the rest of the structure, and collapsed in the south-west direction.
2. The quality of construction was poor, and contributed significantly to the collapse.
 - A telephone pole was left standing in the middle of the building to be removed at a later date. Concrete for the floors was poured around the telephone pole.

- The metal deck at the floors was changed from 1½" to 9/16". The contractor claimed that the change was approved by the structural engineer, who denied doing so.
 - The thickness of the concrete slab over the deck was reduced to 2½" from 3¼". The contractor claimed that the structural engineer approved the change, but the structural engineer denied this.
 - Some of the floors C-joists were not connected to the bridle hangers as per the manufacturer's recommendations.
 - In some locations solid blockings for C-joists were not installed.
 - The exterior studs were not properly aligned and plumbed.
 - At some locations, nails were used to secure the metal deck to the C-joists instead of screws.
 - The contractor substituted exterior heavier studs on second and third floors with lighter studs, 16 gage instead of the specified 14 gage as indicated on the structural drawings.
 - Masonry walls in the core were neither reinforced nor grouted, in violation of the drawings.
 - At some locations, the exterior studs were connected to the top tracks only on one face instead of on both faces.
 - Diagonal bracings at the exterior of the studs framing were not provided, in violation of the standard industry practice.
3. The contractor did not prepare any shop drawings for structural steel, metal deck, C-joists, and reinforcing steel in violation of standard industry practice and contract drawing S001.00, Note 7.

4. The contractor decided to pump the concrete through a metal hose which was fastened to the exterior metal studs at each floor during placement of concrete on the third and fourth floors. Using a boom to pour concrete was considered impractical because framing of the floors was already completed. Fastening the metal hose to the exterior bay could have induced undue vibrations to the already laterally fragile frame of the front bay.
5. Structural drawings prepared by the structural engineer lacked clarity.
 - Different thicknesses of concrete floors were indicated at different locations.
 - C-1 and C-5 columns were omitted on the first floor structural plan.

2. Description of the Project

The architectural drawings (Ref. 1) of the building were prepared by Bricolage Architecture & Design, PLLC of 6321 Utrecht Ave., Brooklyn, NY 11219. Schneider Associates of 866 Johnston Drive, Watchung, NJ 07069 prepared the structural drawings (Ref. 2). The drawings were reviewed by the City of New York, and a permit was issued for construction.

The building under construction was a five-story structure, approximately 80' by 48', intended for residential use. The height of the building was approximately 50 ft. It had a floor area of 3,840 sq. ft. for the first through fourth floors. The fifth floor had an area of 2,690 sq. ft. In the middle of the building, a stairway and an elevator were located.

Reproduced below are the first and second floor plans taken from the structural drawings, see Figs. 1 & 2. A cross-section of the building taken from the architectural drawing is also reproduced below, see Fig. 3. The building was divided into three bays in the north-south direction, and four bays in the east-west direction.

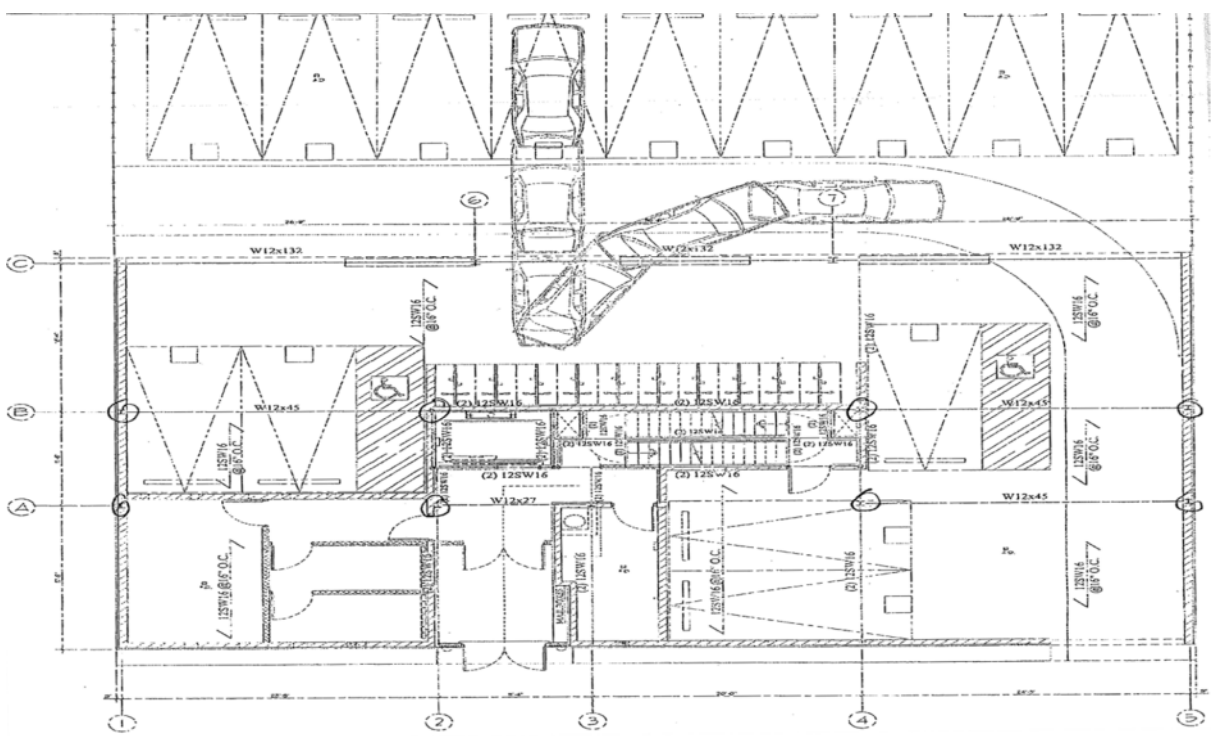


Fig. 1 First Floor Structural Plan.

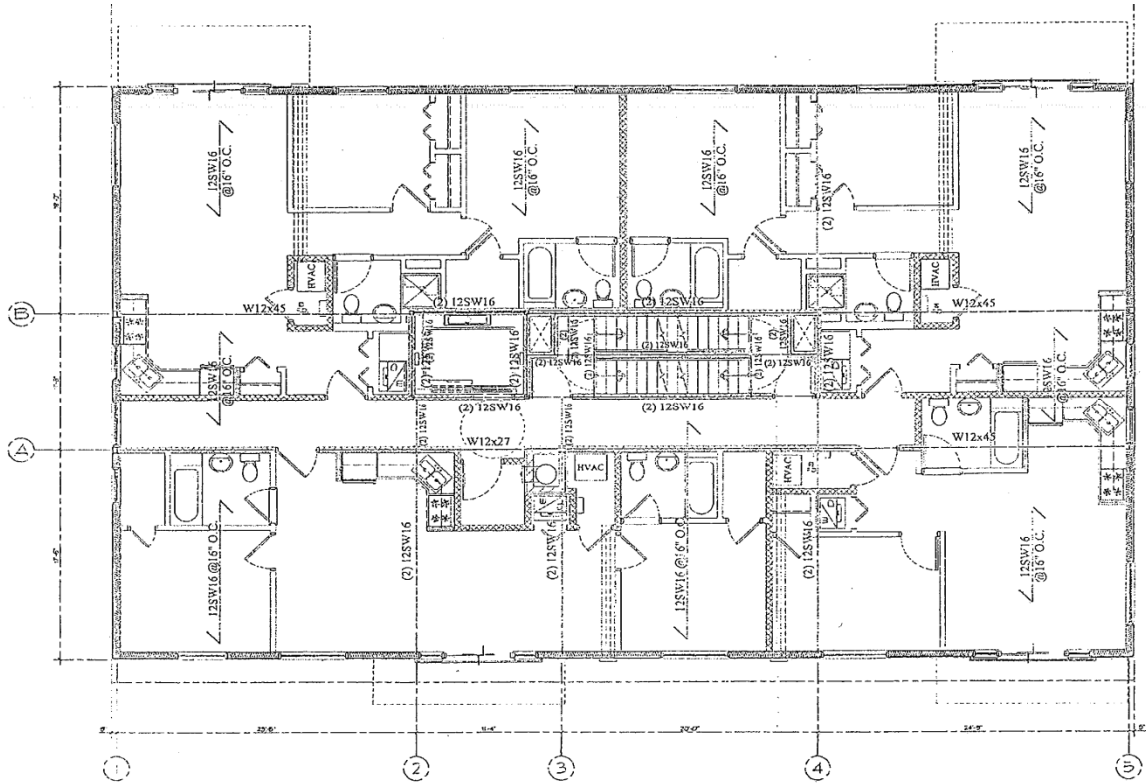


Fig. 2 Second Floor Structural Plan.

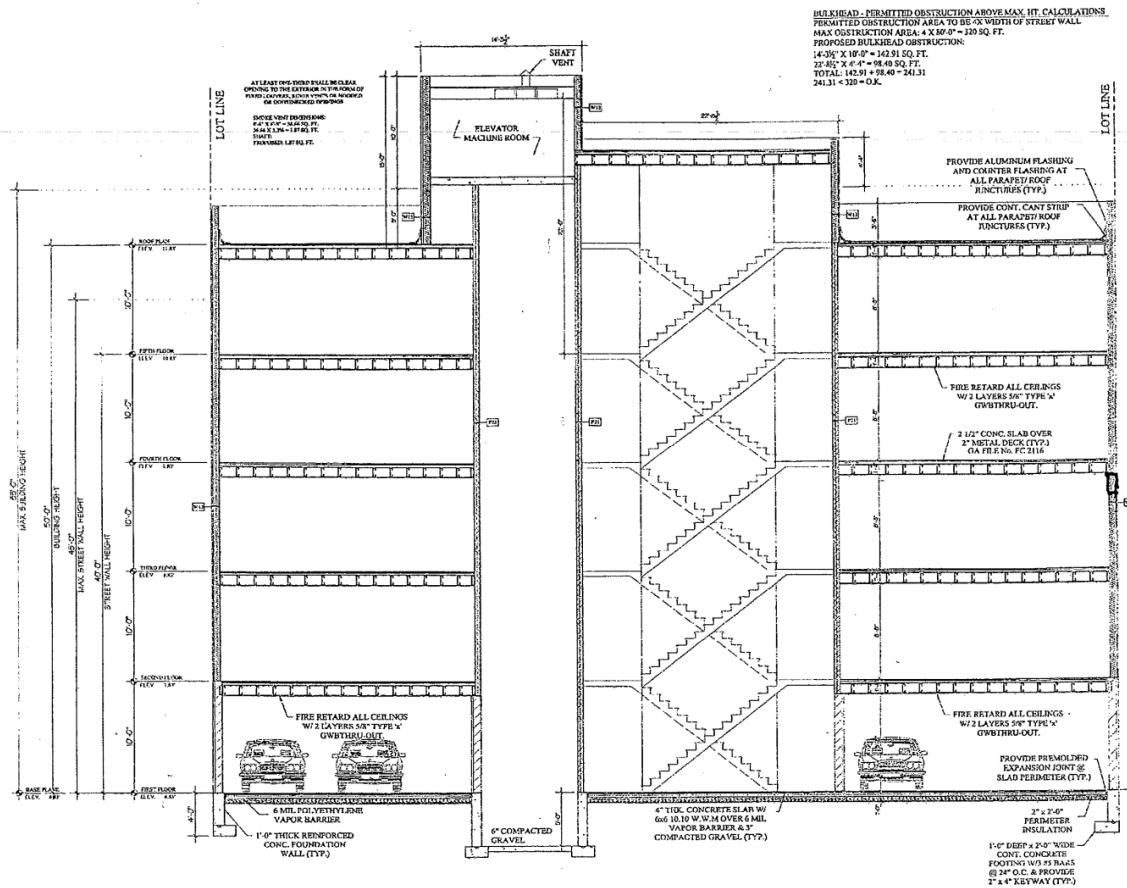


Fig. 3 Vertical Section of the Building.

The foundation consisted of a series of 2' x 3' poured in place grade beams supported by spread footings over helical piles. The first floor was designed to be 6" thick slab on grade reinforced by welded wire fabric. The upper floors were designed to be poured in place slabs with lightweight concrete over metal deck or slabform. The floors were supported by 12" deep 16 gage cold formed C-joists, designated as 12SW16, placed at 16" o.c. The C-joists were supported on bridle metal hangers welded to the interior steel beams, see Fig 4 and supported on exterior load-bearing walls. The C-joists were manufactured by Marino Ware, see Appendix. The C-joists were designed to be braced by light metal blockings at 7 ft. on centers.



Fig. 4 Bridle hangers welded to a wide-flange steel beam and supporting floor C-joists.

The exterior load-bearing 8" thick masonry walls stopped at the second floor, above which were light metal stud load-bearing walls at the perimeter of the building. Steel columns were located at the two interior bays and extended up to the roof.

At the location of the elevator was four-sided reinforced grouted 10" thick masonry wall designed to act as shear walls to resist the lateral loads. Above the second floor, the core masonry wall was 8" thick. All window openings headers were framed by double 12" deep cold-formed metal studs. The exterior of the building was designed to consist of 3/4" thick stucco over 5/8" exterior sheathing attached to exterior stud walls.

3. The Progress of Construction Leading to the Incident

The developer of the project was RS Condo LLC of Brooklyn. SP&K construction Inc., (SP&K) also of Brooklyn was the general contractor. SP&K was also an investment partner of RS Condo. The following were the main participants in the construction phase:

1. Architect: Bricolage Architecture & Design, Brooklyn, NY
2. Structural Engineer: Steven Schneider, PE of Schneider Associates, Watchung, NJ
3. General Contractor: Myroslav Khymych of SP&K, Brooklyn, NY
4. Concrete Pump: Cova Concrete Corporation, Queens, NY
5. Structural Steel: Vladimir Lekhnovskiy of Vlad Iron Work, Brooklyn, NY
6. Concrete Supplier: Stillwell Ready Mix, LLC of Brooklyn, NY

The construction began in August 2011. From the very beginning, there were some basic issues which were not properly addressed. The structural engineer was concerned about the existence of a sewer line crossing the property below the foundation level. There were additional questions raised at the time of pouring of the foundation regarding the helical piles which were addressed by the structural engineer during his visit to the site. A telephone pole existed in the middle of the structure which was left as it was, and the structure was erected around it. The structural engineer objected to the telephone pole, but was ignored.

The work progressed rather swiftly. Helical piles were installed; grade beams and spread footings were placed. Slab on grade and concrete on the second floor were cast. Steel columns and beams were erected, exterior masonry walls were completed up to the second floor, exterior metal stud walls were erected up to the 5th floor, and C-joists were placed on 3rd, 4th and 5th floors over interior steel beams or masonry walls, and on exterior stud walls. All this work was completed in just about three months. Unfortunately, no temporary diagonal braces were erected either in the north-south or east-west direction.

4. The Incident

Concrete was ordered by the contractor for delivery in the morning of November 8, 2011. Concrete arrived at approximately 8:30 a.m. A concrete pump was set up, and concrete pouring began on the 4th floor. Employees were spreading concrete over the 4th floor deck. Meanwhile, additional concrete was ordered for the 3rd floor which arrived shortly after lunch.

Concrete began to be pumped on the 3rd floor while workers were still spreading the 4th floor concrete. In addition, there were workers on the fifth floor engaged in masonry construction. The 5th floor had a bare deck, and concrete was not planned to be poured that day.

When the concrete work was almost finished on the rear (east side) of the 3rd floor, the front bay of the building suddenly collapsed towards the front, see Fig. 5. All C-joists from the 5th to the 3rd floor fell leaning towards the front. The entire exterior stud wall along with all header beams on the front collapsed. The rear of the building remained intact. The rolled steel beams and columns remained standing with little movement. The front of the shear core wall in the north-south direction failed, see Fig. 6. The steel stair partially collapsed. Fresh concrete on the 4th floor slid toward the front.



Fig. 5 Collapsed front bay of the building.



Fig. 6 Collapsed front face of the shear core wall.

As a result of the collapse, an employee working on the 3rd floor was crushed between the collapsing floors, and died. In addition, two employees on the 3rd floor, one on the 4th and one on the 5th floor were seriously injured.

5. Analysis and Discussion

We analyzed the structure to examine whether the structure as designed by the structural engineer would support the intended loads. We have concluded that although there are serious discrepancies in the design and in the preparation of the structural drawings, the structure would support the loads, if constructed in accord with the drawings and standard industry practice.

After the partial collapse of the front bay of the building, the structure below the second floor remained largely intact. Therefore, we did not consider in this report issues related to footings, helical piles, and subsurface drainage.

The discrepancies in the structural drawings relate to the lack of details of diaphragm connections to the masonry shear walls, lack of details on where the permanent diagonal bracings will be installed, and contradictory gage and thickness of the concrete deck. Additionally, the structural details do not clearly show masonry wall reinforcements.

As is standard in the construction industry, the structural engineer is responsible for the adequacy of the completed structure, while the contractor is responsible for its stability during construction (Refs. 3, 4, & 5). The methods and means of construction is the prime responsibility of the contractors, and not of the structural engineer.

The stability of the structural frame depends upon the support of the gravity loads and the resistance of lateral loads. Lateral loads are created by wind, seismic and dynamic movements of workers and materials. The lateral strength is derived by shear walls, bracings or rigidity of the joints of the horizontal and vertical members of the structure (Refs. 3 & 4). The lateral strength of the structure must be maintained during all phases of construction and at the completion of the project under service loads (Ref. 6). If shear wall is the primary source of lateral strength, then there must be distinct paths for the lateral forces to be transferred through the diaphragm to the shear walls. If bracings are the means to resist lateral loads, they must be placed at appropriate locations in both directions to resist such loads. Finally, if joints of horizontal and vertical members are to be the source of the lateral strength, such joints must have rotational and translational strength to resist lateral loads. In this case, the structure during construction was without any means to resist lateral loads.

The construction of the project began in an unconventional manner in that the contractor proceeded without preparing shop drawings for the structural steel, light metal framings, reinforcements, etc. The structural engineer neither asked for the shop drawings nor was he provided with any. Structural steel was purchased directly from a vendor and cut to lengths to suit the project without any shop drawings. The thickness and depth of the metal deck at all floors were changed from the original drawings without any documentation. The contractor claimed that he obtained the structural engineer's approval but the structural engineer during an interview with OSHA denied having approved any changes. The original drawing required

that 1½" deep metal deck, 18 gage be used at all floors for the concrete floors. Instead, 9/16" deep slabform was used.

Our analysis indicated that the shallower slabform would support the required loads, and the structural strength was not compromised. However, the manner in which it was attached to the C-joists would later contribute to the lateral instability, discussed below.

Another change was made in the thickness of the concrete on the floors. The structural drawings indicated that all floors should have a total thickness of concrete of 4½" including metal deck. The contractor arbitrarily reduced the thickness to 2½" without approval from the structural engineer, although the contractor claimed to have received approval from the engineer, who denied it. Our analysis indicated that the reduced thickness of the slab could support the gravity loads without compromising their strength.

In addition to the above, the gage of the exterior metal studs was changed from 14 gage to 16 gage, again without any approval from the structural engineer. While it reduced the load-carrying capacity, it was deemed adequate to carry the loads during construction and during the service loads after completion of the project. However, the means used to connect studs to the headers were highly questionable, and contributed to the lateral instability, discussed below.

The C-joists of the required depth and gage were generally placed at the required spacing of 16" o.c. and were supported on bridle hangers welded to the steel beams on the interior side, and placed over the header beams of the exterior wall. The C-joists were not required to be shored during placement of the wet concrete (Ref. 2).

The welds of the hangers did not fail during the collapse as the hangers remained connected to the steel beams. However, the connection of the C-joists to the hangers was made by only one screw at the bottom. At some locations, the C-joists were simply resting on the hangers without any screws. The standard industry practice is to have connections on the sides of the C-joists to prevent the joists from rolling and swaying sideways before the concrete above the metal deck hardens (Refs. 7, 8 & 9).

On the exterior side, the C-joists were supported over the headers supported on the stud walls. This connection was highly critical to maintain the lateral stability of the frame. This connection was made by only one screw at a number of locations instead of two screws as required. In addition, the exterior flange of the studs were not screwed to the header beams; only the inside flange was attached by screws.

The other area of concern was the lack of blocking between the C-joists. The blocking not only provides bracing for the compression flange of the C-joists, but also aids against lateral movement of the C-joists. The blocking, as per standard industry practice is to provide blockings at 12 ft. o.c (Ref. 9). The actual spacing was greater than that required, and at some locations, there were no blockings provided.

As discussed earlier, the metal deck was changed in thickness and depth from the original drawings, although in itself this did not present any hazard. However, the deck was at some places screwed to the top joists, and at some locations was nailed to the C-joists. Using nails to fasten the deck to the C-joists is unacceptable as it does not have the required withdrawal strength of screws. This contributed to the instability of the structure.

On the day of the incident, the contractor decided to pour concrete on the 4th floor first, and then onto the 3rd floor. Though contrary to the normal practice of pouring the lower floor before the higher floor, this by itself, did not present any hazard. Concrete trucks arrived in the morning, and instead of a boom truck, a concrete pump truck was brought in. Metal hoses were laid out from the pump truck on the ground, and then were taken vertically to the fourth floor by attaching the standpipes to the exterior stud walls. Concrete was poured on the 4th floor, and then the crew moved to the 3rd floor. Hoses for the concrete were removed from the 4th floor, and re-fastened on the exterior stud wall to facilitate pouring concrete on the 3rd floor. Thirty cubic yards of concrete was initially ordered, but as it fell short for the 3rd floor, additional trucks were ordered. Fresh concrete arrived at the site after lunch. The 3rd floor pouring started on the rear side, and progressed to the masonry core walls at which time the incident occurred. The front bay (west side) of the 5th, 4th and the 3rd floors collapsed toward the front.

At the time of the collapse, we examined the stability of the structure. There were no braces provided in the east-west direction. So, the structure's stability depended upon the rigidity of the joints of the horizontal and vertical members, or the ability of the diaphragm to transfer lateral loads to the masonry core walls. As explained earlier, the C-joists were simply resting on the hangers on one side where steel beams were located, and resting on the header beams of the exterior stud walls on the other side. The header beams consisted of light metal studs.

Where there was interior masonry shear wall, the C-joists were resting on the top of the shear walls. In between the c-joists, the space was filled with brick masonry. The exterior stud wall in the north-south direction was over three stories high horizontally braced with C-joists at the 3rd, 4th and 5th floors with one or two screws, which must be considered as hinged. On the interior side, similar hinged conditions existed as the C-joists were attached to the hangers with one screw. Therefore the front bay was reduced to a situation where all joints were hinged for the bays on either side of the core masonry walls.

As a matter of fact, employees experienced shaking and vibrations of the floor while the 4th and the 3rd floors were being poured. In the area where the C-joists rested on interior masonry walls, the C-joists were blocked in by brick masonry, and had developed some fixity, see Fig. 7. But even in this area, the far end which rested on the exterior stud wall, the connection was hinged, see Fig 8.



Fig. 7 C-joists blocked in by brick masonry.



Fig. 8 C-joists resting on an exterior stud wall.

Light gage metal studs, headers, and C-joist were not properly connected using the type and number of fasteners necessary to transmit vertical and lateral loads. The improper connections of these members had compromised the structural stability of the building by creating hinges at the connections as loads were transferred from the floor decks to the exterior stud walls. Additionally, steel tie straps were not used to connect the light gage metal studs of adjacent floors to allow uplift load transfer through the floor system.

The rectangular core shear walls, though already completed at the time of the incident, were rendered ineffective because of lack of diaphragm action. The $\frac{9}{16}$ " deep slab form was not properly connected to the shear walls. For the slab form to be effective, it needed to be anchored to the four grouted walls. The general practice is to have a bond beam at each floor where the slab form could be connected, or to have dowels from the grouted shear walls into the floor slab. Even after the placement of concrete over the slab form, the diaphragm would not be capable of transmitting the lateral loads because of lack of positive connection i.e., rebar between the concrete deck and the grouted shear walls. The east and the west walls were unstable because of all hinged connections from second floor and up.

It is believed that the failure started in the two bays on either side of the core masonry wall and then spread to the bay in front of the masonry wall. It is probable that the actual collapse

might have been triggered by the dynamic forces imparted by the concrete pumping hose.
Below is a sketch of the unstable frame showing the hinged connections, see Fig 9.

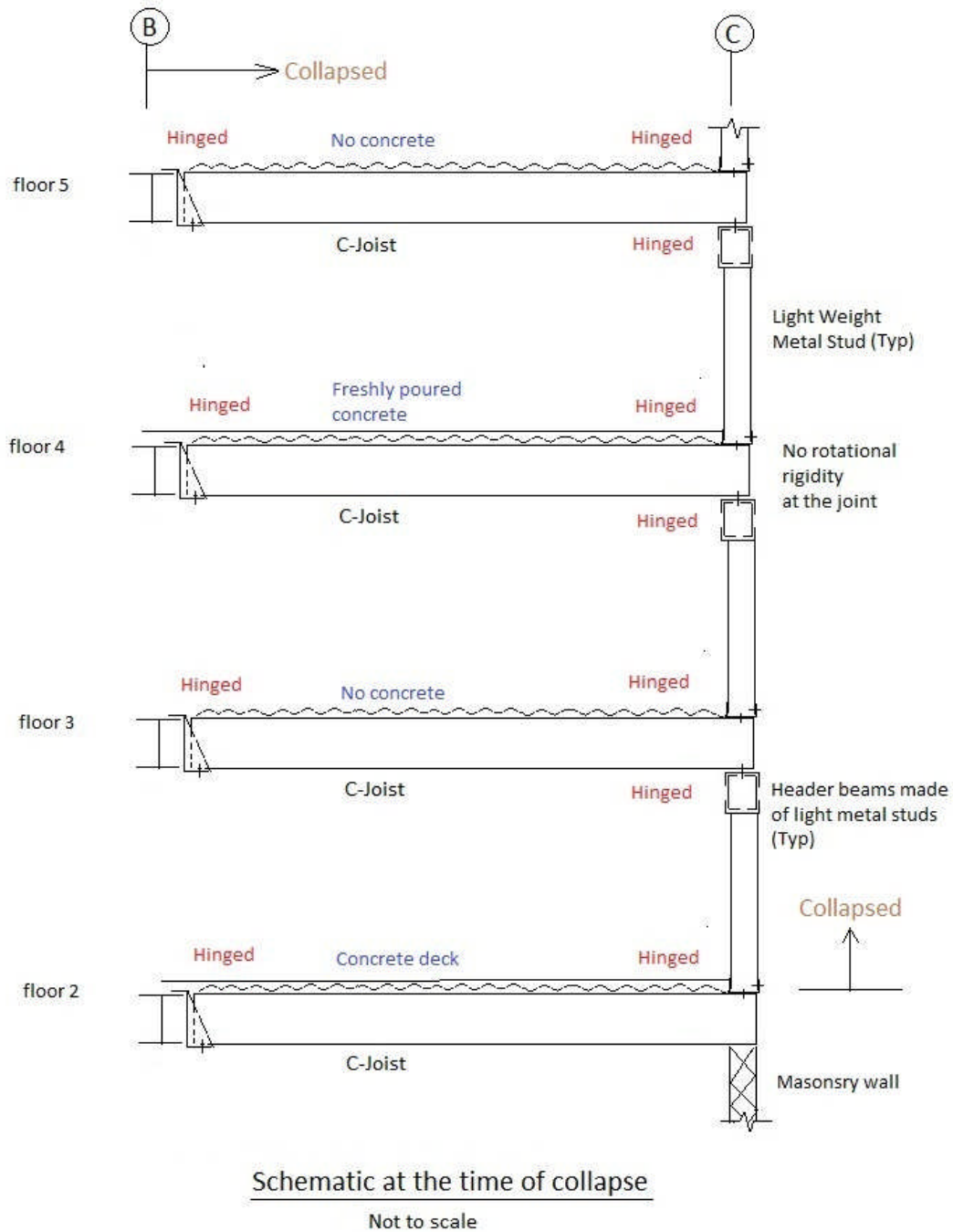


Fig. 9 Unstable frame showing the hinged connections.

6. Conclusions

Based upon the above, it is concluded that:

1. The building under construction partially collapsed because the contractor failed to maintain the stability of the front bay of the structure. All joints of the front bay (west side) at the third, fourth and the fifth floors were “hinged”, and had minimal flexural rigidity. Under lateral load, the front bay separated from the rest of the structure, and collapsed in the south-west direction.

2. The quality of construction was poor, and contributed significantly to the collapse.
 - A telephone pole was left standing in the middle of the building to be removed at a later date. Concrete for the floors was poured around the telephone pole.
 - The metal deck at the floors was changed from 1½" to 9/16". The contractor claimed that the change was approved by the structural engineer, who denied doing so.
 - The thickness of the concrete slab over the deck was reduced to 2½" from 3¼". The contractor claimed that the structural engineer approved the change, but the structural engineer denied this.
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 - The contractor substituted exterior heavier studs on second and third floors with lighter studs, 16 gage instead of the specified 14 gage as indicated on the structural drawings.
 - Masonry walls in the core were neither reinforced nor grouted, in violation of the drawings.

- At some locations, the exterior studs were connected to the top tracks only on one face instead of on both faces.
 - Diagonal bracings at the exterior of the studs framing were not provided, in violation of the standard industry practice.
3. The contractor did not prepare any shop drawings for structural steel, metal deck, c-joists, and reinforcing steel in violation of standard industry practice and contract drawing S001.00, Note 7.
 4. The contractor decided to pump the concrete through a metal hose which was fastened to the exterior metal studs at each floor during placement of concrete on the third and fourth floors. Using a boom to pour concrete was considered impractical because framing of the floors was already completed. Fastening the metal hose to the exterior bay could have induced undue vibrations to the already laterally fragile frame of the front bay.
 5. Structural drawings prepared by the structural engineer lacked clarity.
 - Different thicknesses of concrete floors were indicated at different locations.
 - C-1 and C-5 columns were omitted on the first floor structural plan.

7. References

1. Architectural Drawings, Drawing No. A 000.00, A001.00 - A008.00, A100.00 - A104.00, A200.00, A300.00, A400.00 - A405.00, & A500.00.
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