



COLLAPSE OF STRUCTURAL STEEL FRAMING DURING CONSTRUCTION

Figure 1 shows the building where the collapse occurred, after reconstruction and the completion of the erection of the steel framing. The building is a five story office building, 221 feet by 271 feet in plan, in a suburban Boston location. The floor deck consists of concrete filled steel deck which acts compositely with the supporting steel beams. The roof deck is an unfilled steel deck covered with insulation and roofing.

The lateral load resistance and stability of the building is provided by a series of *rigid frames* running across the entire width and length of the building, four running east-west, and three running north-south. A rigid frame is one where the members (beams and columns) are rigidly connected to each other so that there can be no relative rotation between the ends of the members, thus restraining the frame from swaying under lateral load. The structural force that this rotational restraint resists is called a *moment*; the rotationally restrained connections are called *moment connections*. All the other rows of beams and columns have *simple connections*, which essentially do not resist rotation; these rows of beams and columns have no sway resistance and rely on the rigid frames for lateral stability.

A major architectural feature of the building is a 4 foot projection of the second to fifth stories beyond the first story, along most of periphery of the building. This 4 foot projection is accomplished by setting the steel columns back from the perimeter, cantilevering beams from the columns at the second through fifth floors, connecting the cantilever beams to the columns with moment connections, and cantilevering the roof beams over the tops of the columns.

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Figure 1. View of the northwest corner of the building, showing the area where the steel framing collapsed during erection.



Figure 2. View of collapse at northwest corner of the building, taken prior to the removal of debris.

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The free ends of the cantilever beams support principal members of the structural system. For stability, a cantilever beam must have a moment connection at its supported end, to resist the rotational effect of the loads on the beam about the supported end.

Figure 3 is a schematic plan showing the floor framing in the area of the collapse. The figure shows beam and column sizes and the steel fabricators identification numbers for framing referred to in this narrative. The symbols *MC1* and *MC2* denote moment connections between beams and columns; all other connections are simple connections. Figure 4 is a section showing the cantilever framing along the west side of the building.

The collapse occurred in the northwest corner of the building. It involved only the first tier of columns, second and third floor beams and girders, and the area north of Grid Line C and west of Grid Line 15.

The contract drawings and the approved shop drawings indicated that the flanges and webs of cantilever beams 41B1 and 50B1 (see Figure 3) were to be welded to the flanges of columns 41C1 and 50C1, respectively, in the fabricating shop, with full penetration welds (i.e. welds through the full thickness of the steel of the web and flanges of the beam). However, the fabricator revised the detail before fabrication to that shown on Figure 4, so that an erection angle would be welded to the column flange in the shop, the cantilever beam would be shipped separately, the cantilever beam web would be temporarily bolted to the erection angle in the field, and the cantilever beam flanges

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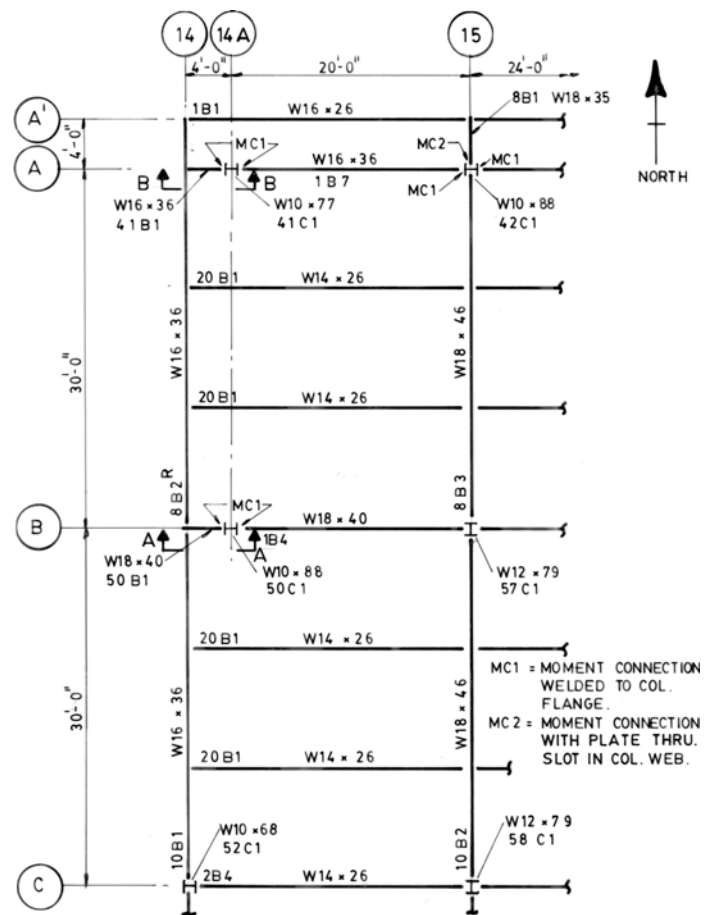


Figure 3. Schematic framing plan of 2nd, 3rd, and 4th floors, in the area of the collapse.

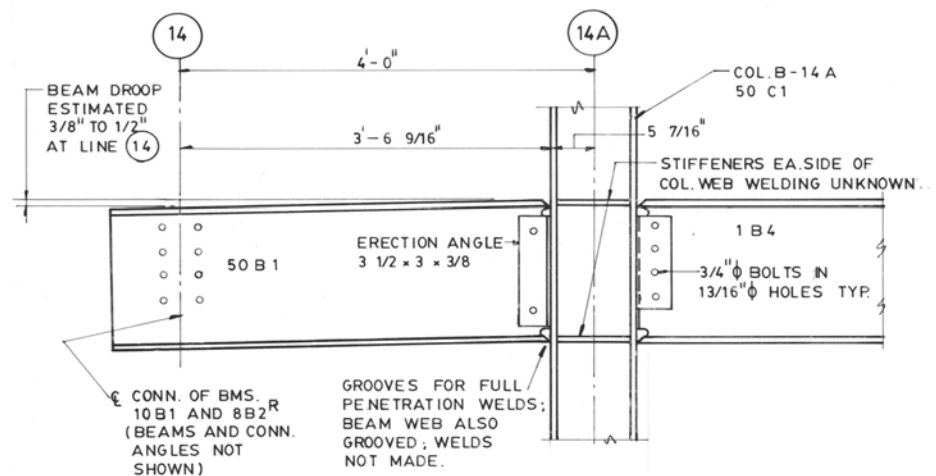


Figure 4. Section A-A, showing cantilever framing along the west side of the building at the time of the collapse.

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and web would thereafter be connected to the column flange by the required full penetration welds. Figure 5 shows the details of the erection angles at Column 50C1

The cantilever beams and the columns to which they were to be attached were fabricated and shipped, as just described. The erector erected the columns and beams in the northwest corner section of the building, attached the subject cantilevers to the erection angles, and then attached framing to the free ends of the cantilevers. However, he did not make the permanent (full penetration) welds between the cantilever beams and the column flanges, and he did not provide any temporary support at the free ends of the cantilevers.

The erector placed bundles of steel floor deck on the floors after each floor was framed in, before erecting the beams and girders of the floors above - a customary erection practice. In the course of this procedure, bundles of steel deck were placed on framing that loaded the free ends of the cantilevers.

The collapse occurred on the morning after the steel deck bundles were placed on the third floor, when two ironworkers were on the third floor framing near cantilever 50B1. The (shop) welds connecting the temporary erection angles to the column flanges, where cantilever beams 41B1 and 50B1 project from columns 41C1 and 50C1, on both the 2nd and 3rd floors (four locations), failed, leaving the cantilever beams and the framing attached thereto without support.

Technical Cause of Collapse

The permanent, full penetration, welded connections of the ends of the cantilevers to the faces of the flanges of Columns 41C1 and 50C1, which were designed to resist moment, were not made. Therefore, the moment and shear due to the weight of the deck bundles and steel framing had to be resisted by the erection angles and the welds connecting them to the faces of the column flanges.

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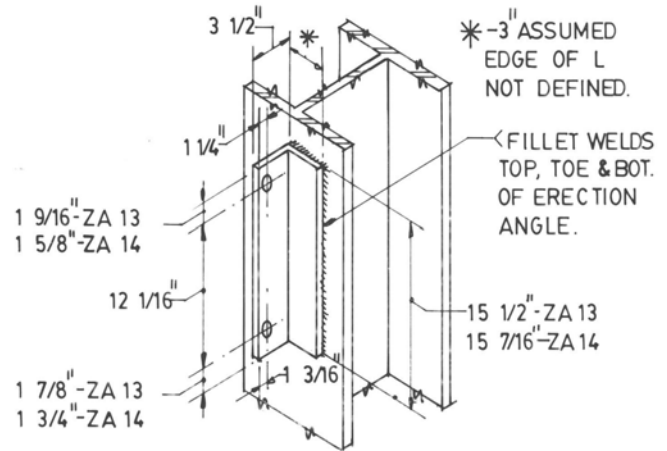


Figure 5. Detail of the erection angles at Column 50C1.

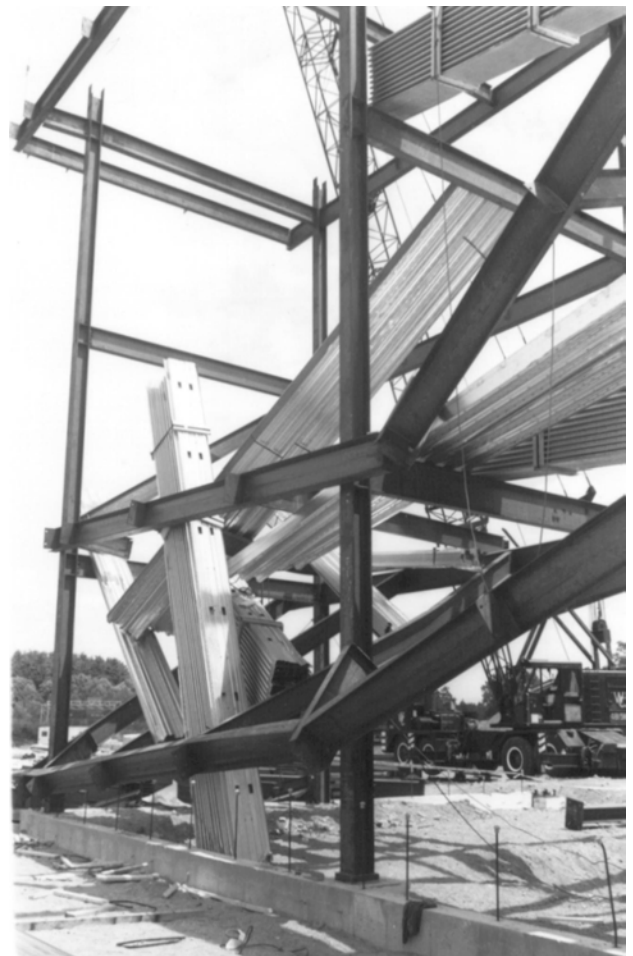


Figure 6. View of collapse looking northeast.

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A vertical connection angle similar to the erection angles is efficient for resisting the effect of the vertical load (called *shear*) and is commonly used for that purpose; however, it is grossly inefficient for resisting moment, and would not normally be used by structural engineers for resisting any significant moment. Because of the complexity of the structural action when the connection angle and its weld are subjected to moment, and because of the lack of test data in the engineering literature, Zallen Engineering performed tests on two full-scale mock-ups, each consisting of a cantilever, a connection angle, and a column. The estimated load on the third floor 50B1 cantilever, due to the steel framing and deck bundles, was similar in magnitude to the ultimate loads from the tests of the mock-ups.

Thus, the collapse was most probably initiated by the failure of the weld between Column 50C1 and the erection angle attached thereto at the third floor, due to the moment applied to it by the load on the cantilever; the failure propagated to the weld between Column 41C1 and the erection angle attached thereto at the third floor; the third floor collapsed on to the second floor; the similar welded connections at Columns 50C1 and 41C1 at the second floor failed; and the second floor then collapsed.

Procedural Causes of the Collapse

When the fabricator revised his shop details to change the cantilever connections from shop welds to field welds, he did not issue any revised drawings to the erector or to the general contractor. He also did not provide them with any other written instructions on erection procedure or limitations on the load that could be imposed on the cantilevers prior to making the permanent welded connections to the columns.



Figure 7. View of collapse looking northwest.

Although the fabricator did communicate verbally with the erector (who was his subcontractor) on the change in detail, there is no evidence of what they discussed. The accepted industry practice of informing the erector or other parties of erection procedures and requirements is to put the instructions on the erection drawings.

The erector's official in charge of steel erection and the erector's foreman both testified in their depositions that they did not understand the basic structural engineering concepts of moment or moment connection, how a cantilever needed to be supported, or the load carrying limitations of the temporary erection angles. They both assumed that the temporary erection angles would support the loads that were imposed on them until the permanent moment and

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shear connections (full penetration welds) were made between the cantilevers and the columns. An erector's supervisory personnel must understand basic structural concepts in order to safely erect steel framing.

The erector's official in charge and the erector's foreman both further testified that they were following normal erection practice in which the moment connections are not made in the section of steel frame being erected until the section is plumbed-up, because if they made the moment connections before plumbing, they could not plumb the structure. This reasoning is correct for the moment connections of the rigid frames that provide the lateral stability for the building, but it is incorrect for the cantilevers; the welding of the moment connections of the cantilevers could not in any way prevent the plumbing-up of the steel frame.

The general contractor's superintendent testified in his deposition that he knew that the change in the cantilever to column connections from shop welds to field welds did not conform to the structural engineer's design nor the approved shop drawings; that he knew that the permanent full penetration welds between the cantilevers and the column flanges had not been made; knew that framing had been attached to the free ends of the cantilevers and that bundles of steel floor decking had been placed

on that framing, with only the erection angles supporting the cantilevers; knew what a moment and a moment connection are; and knew that cantilevers had to be moment connected, but had not before in his experience seen the condition of a cantilever being supported on the face of a column flange. Nevertheless, the superintendent allowed erection to proceed without first consulting with his superiors or with the structural engineer, to whom he had ready access, to obtain an explanation and instructions on how to proceed.

Thus, the steel fabricator, the steel erector, and the general contractor's superintendent each were responsible for the collapse; had the fabricator or the superintendent done what he was supposed to do, or had the erector had the knowledge he was supposed to have, the collapse would have been averted.

Principal Rubin M. Zallen investigated this failure and provided technical assistance to the plaintiffs' attorney. See On-Line Issue No. 5 for a description of the testing of the mock-ups.

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