EVALUATING CONSTRUCTION FAILURES

By Harvey A. Kagan,[1] M. ASCE

ABSTRACT: Four of the construction failures the writer has investigated are reviewed. Two failures involved light framed timber trusses, one failure involved precast tees, and one involved overloaded steel floor joists. Each collapse centered around a failure to recognize the weakness of a partly completed structure. Human failure was the heart of the problem. The writer suggests greater attention to details by those responsible for construction procedures.

INTRODUCTION

This paper reviews four of the construction failures that the writer has investigated. In addition, the paper reviews appropriate details of the structure under construction and describes the cause of collapse. Each failure was unique yet there was a common feature: human error. The paper notes that construction professionals should not allow time pressures to allow them to overlook critical construction details. Design professionals should also consider the loads imposed during construction and forewarn contractors of potential hazards.

INCREASING NUMBER OF RECENT CONSTRUCTION FAILURES

For most of us in the design and construction professions, failure or collapse will be an event that we read about in the newspapers or magazines. While a student, the writer was shown movies of the collapse of the Tacoma Narrows Bridge, which he thought to be a freak occurrence. Old bridges might fall down but not modern structures. The Hartford Coliseum and the Kemper Arena collapses have changed perceptions of the "fail-safeness" of modern construction.

Collapse of structures during construction is, unfortunately, much more common. These collapses usually do not make the headlines. However, the cooling tower collapse at Willow Island, West Virginia, and the apartment tower collapse at Bailey's Crossing, Virginia, are major failures that have made news.

During construction a partly completed structure does not yet have its full design strength. Thus, loads imposed during construction can often be greater than the final service loads. The nature of loads may be different from any that the completed structure might experience. The structure is often quite unstable during construction, yet it is most likely at this point to receive heavy temporary loads and even unforeseen loads.

Books and articles have been published and seminars have been held on methods to avoid construction failures through lessons learned from past failures. However, failures continue and will continue to occur. From

[1]Pres., Construction Advisory Group, Inc., New York, N.Y.

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a technical viewpoint most construction failures are preventable; the human factor is the weak point.

The writer's work has brought him into contact with various construction failures that have resulted in property damage, personal injury, and even death. In these failures the writer was retained by attorneys representing one of the parties to the ensuing litigation. In his capacity as a construction expert, he was called upon to determine the cause of the collapse. Once the probable cause of collapse was determined, contracts were reviewed to determine the duties owed by the various parties to each other. This assisted the attorneys in preparing their cases proving or defending issues of negligence.

This paper describes four failures which the writer has personally investigated. Some are relatively small cases, some larger; however, to the parties involved, the cases were not small. Each case points out some of the aspects of human failure involved in construction failures.

Wood Roof Truss Collapses

Wooden trusses are commonly used to support the roofs of light industrial and institutional buildings and offer considerable economies in fabrication and erection. The trusses are usually built from 2 by 4 and 2 by 6 or 2 by 8 members and are connected by special pronged plates. Since the trusses are only 1-3/4 in. (45 mm) in lateral dimension they are extremely unstable and flexible without lateral bracing. It is during the construction process that the trusses are most susceptible to failure. They are shipped to the site in bundles and then raised individually to their position on the roof. The Truss Plate Institute (TPI), a trade association of roof truss manufacturers, publishes a detailed guideline for installation of their product. A contractor cannot be too careful in erecting these trusses.

Despite warnings and erection guidelines there are many failures of partly completed roofs each year. Many of the collapses occur to experienced contractors, showing that previous success is no reason to be lax in precautionary measures.

The reasons for truss collapses during construction can be any one or a combination of factors: inadequate lateral bracing, overloading, improper lifting by the construction crane, defects in the material, defects in the fabrication, improper handling by workmen, inadequate design, and damage during construction.

Church Roof Collapse.—The case deals with a church building whose roof trusses had just been installed on the day of the collapse. The wooden trusses spanned 80 ft (25 m) between concrete masonry exterior walls. Fig. 1 is a sketch of the truss. The contractor finished his installation of the trusses and then lifted bundles of plywood sheathing to the top chords of the trusses. These were to be left in place overnight so that the carpenters would have the material ready when they started in the morning. The job was then shut down for the night.

The church was located near a small municipal airport. At about 5:15 p.m. a plane passed over the uncompleted building. Eye witnesses stated that about half the trusses "disappeared" as the plane passed over the building.

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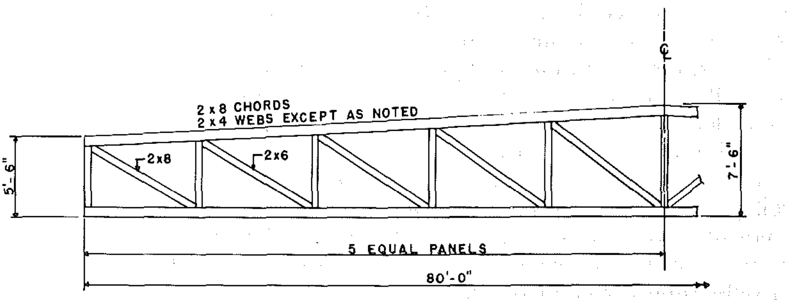

FIG. 1.—Church Roof Truss (1 In. = 25.4 mm)

The contractor sought to recover the cost of the damage from the company that had issued his construction insurance. The contractor's insurance company refused to pay any claim on the building, arguing that the collapse was not caused by wind or natural elements. The weather records for the area indicated a maximum wind speed of only 10 knots at the time of collapse. The problem was resolved immediately by having the roof fabricator refabricate trusses to replace those which fell. The old trusses were removed and destroyed.

Over a year after the collapse the writer was consulted on the matter. There were only photographs, testimony, and engineering reports to work with. No physical evidence was available.

The system of lateral bracing was examined first, with reports and testimony reviewed to recreate the placement of the bracing. This information was correlated with bracing shown in post-collapse photographs. The contractor had only partially followed the TPI bracing recommendations. Cross bracing had not been installed for all vertical members as suggested by the TPI. Top chords had unbraced lengths of about 10 ft (3 m). The bracing would have been adequate had the contractor not made one further deviation from the TPI recommendations. Stacks of plywood 50–55 sheets high had been placed on the trusses prior to collapse [55 sheets of plywood would weigh about 2,640 lb (11.75 kN)]. The contractor had placed individual loads of over a ton each on the unfinished truss system. The TPI recommendations state: "Full bundles of plywood should not be placed on trusses. This construction load should be limited to 8 sheets of plywood on any pair of trusses and should be located adjacent to the supports." Eight sheets would weigh only 384 lb (1.71 kN).

The photographs showed the bundles were relatively undisturbed and appear to have dropped more or less vertically. Two stacks were placed transversely to the trusses approximately over the second panel point from the support. These were supported by 3 trusses. A third stack had been placed with its long axis parallel to the trusses over the third panel point from the support. This bundle was supported by 2 trusses. Therefore, excessively heavy bundles placed out on the span had been installed in violation of recommended procedures.

Applicable truss design codes indicated that a maximum stress of 106 psi (730 kPa) was allowable in the top chord of the truss at the location

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of the third bundle of plywood. The stress was limited by the lateral supports of the top chord. An analysis of stress in the roof truss caused by various positions of the plywood bundles *only* (no truss dead load), indicated a stress of 143 psi (985 kPa) for the laterally placed stacks, and a stress of 382 psi (263 kPa) for the bundle spanning only two trusses. The top chord of the truss was overstressed even under the most favorable conditions.

We thus had the following conditions:

1. Stacks of plywood 50–55 sheets high placed on the top chords of trusses.
2. The top chords were insufficiently braced in the lateral direction and were overstressed. The trusses did not collapse immediately but remained standing in unstable equilibrium.
3. The vibration caused by the passing airplane was sufficient to provide the force needed to start the collapse chain. One or more truss chords buckled and pulled down the other trusses.

Had the contractor not been so careless in placing the bundles of plywood the collapse would not have occurred. He should have taken the time to break the bundles into piles of 8 sheets at the maximum and placed these on the trusses around the perimeter of the building. The carpenters would have had to break the bundles up later in any case.

It is possible that at the end of the work day someone was in a hurry to go home and became careless. Instead of taking the time to divide the bundles he had the crane lift them up on the trusses, leaving the distribution of plywood to the carpenters on the roof. The result was limited to property damage. What would have happened if the carpenters were working on the trusses at 5:15 that evening?

Warehouse Roof Collapse.—When wooden roof truss collapses occur, the resulting lawsuits can be as complex as for any major structural collapse that makes the headlines. Experts for each interested party can view the same event and derive divergent opinions. Because wooden trusses fabricated of 2-in. (50-mm) thick lumber are highly unstable until laterally braced, construction failures customarily exhibit lateral buckling as part of the collapse mode. The erector blames the manufacturer, the manufacturer the erector, and the owner blames both. In the case to be considered, weather was a factor. The insurance carrier denied weather as a contributing factor and blamed the erector and manufacturer. There was an element of truth to all the arguments.

A plumbing contractor in a small Kentucky town had a new warehouse building constructed for him. The building was of concrete masonry with wooden roof trusses spanning 80 ft (24.4 m). The trusses were made of 2 by 8 and 2 by 4 lumber connected by nail plates (Fig. 2 is a sketch of a typical truss).

There were no formal plans or contracts for the building. This was a small town where the various parties had been dealing with each other on an informal basis for many years. The plumbing contractor (owner) paid for the materials and hired the contractor to provide the labor to build the building. The owner furnished the builder a scaled outline of the building. Construction details were to be handled by the contractor.

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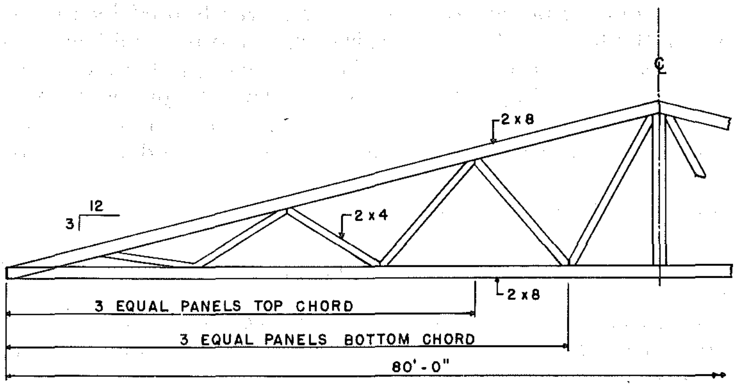

FIG. 2.—Warehouse Roof Truss (1 in. = 25.4 mm)

The contractor purchased the fabricated trusses from a local truss manufacturer, which was a franchisee of a larger designer-manufacturer. The franchisor had designed and put its engineer's stamp on the shop drawings. The shop drawings contained disclaimers as to responsibility for erection, bracing, and assembly of the trusses. Use of bracing was recommended but bracing was not shown on the drawings.

The local truss fabricator provided the contractor with sheets copied from the Truss Plate Institute erection recommendations. These were the only instructions furnished as to bracing of the trusses and were more schematic than detailed. No engineer or architect supervised the design or construction of the overall building.

These were the largest trusses ever erected by the contractor. He had had no formal technical training but had been a framing contractor for many years and had erected wooden trusses for private homes and apartments.

Erection of trusses commenced in June of 1975. On the first day, six trusses were set. During the night, there were heavy showers and, in the morning, the trusses were severely warped or bowed out of plane. The contractor then spent the day straightening the trusses. He also contacted the local fabricator who suggested use of additional lateral bracing for the trusses. The contractor then spent three more days completing the truss erection. He installed continuous lateral X bracing between the center verticals, and straight bracing between top and bottom chords. This was acknowledged to be temporary bracing. His intent was then to go back and install a complete system of permanent X bracing between other vertical truss members. After completion of erection and temporary bracing, piles of plywood sheathing were left on the trusses for installation the next day. The contractor claimed these were 8–10 sheets high.

There were winds and thunderstorms again the night after truss erection was completed. Winds of over 40 mph (64.4 km/h) were recorded at a nearby recording station. The windstorm was in a direction perpendicular to the trusses. In the morning, the center section of trusses had warped out of plane. The contractor and his workmen proceeded to straighten the trusses one at a time by loosening portions of the lateral bracing. During the operation, one truss fell over and collapsed the middle third of the trusses in a domino fashion. By good fortune, the workmen were all on the trusses which did not collapse. Damage was confined to the collapsed trusses.

After inspection by interested parties, as described later, the trusses were removed and replaced with new trusses. The second time, only five trusses were set at a time and then soundly braced before additional trusses were placed.

The owner's insurance policy covered wind damage during construction. He submitted a claim for the replacement cost of the trusses. However, the insurance carrier denied the claim, blaming the contractor for causing the collapse.

After the collapse, engineers were retained by the insurance company, by the franchisor of the trusses, and by the owner. These engineers viewed the collapse before trusses were removed. All three reports emphasized the obvious fact that the trusses were highly unstable because of inadequate lateral bracing. The owner's engineer criticized the lack of specific erection instructions accompanying the delivered trusses. The truss manufacturer's engineer blamed inadequate bracing of trusses during erection. He claimed plywood had been piled 40 sheets high. His contention was that the contractor should have done a better job of bracing. Failure, he claimed, may have started at the point of application of the plywood loadings. The insurance company's engineer claimed the design was inadequate for the finished roof system as well as for erection and handling, and also stated that the trusses were improperly fabricated. He specifically considered wind loadings and dismissed these as not having had any contribution to the collapse.

The owner instituted suit against the insurance company to recover damages which he held were due to the windstorm. The writer was retained by the owner to determine probable cause of collapse.

There was no question that the trusses required considerable lateral bracing in order to function as load carrying members. None of the trusses could stand freely by itself. This was recognized by the manufacturer and the contractor. The insurance company emphasized this fact to further its contention that the trusses were unsuitable for the long span of the roof. In the opinion of the writer the trusses, when installed with a complete system of bracing and roof sheathing, were capable of carrying the design loads. The issue was one of proper care during erection and the effect of wind.

In the writer's opinion, the temporary bracing system was adequate for low wind velocities. A more substantial and permanent system was needed to handle vertical and lateral loads during the service life of the building.

During the windstorms, high dynamic forces were developed against the sides of the trusses. The temporary lateral bracing system was adequate enough to prevent collapse of the trusses but was inadequate to hold the trusses in place. The result was the shifting and distortion of the trusses. The accompanying rainstorm soaked the wood and possibly decreased the holding power of the nail plates. The net effect was the development of abnormally high stresses in the connector plates and in the members.

During the straightening operation, one of two possibilities occurred: the contractor removed too many lateral supports and a truss collapsed, or a weakened truss connector or a lateral bracing connection failed and caused the collapse.

Should the temporary bracing have been adequate to withstand wind gusts in excess of 40 mph (64.4 km/h) without any shifting of trusses? The answer depends on one's point of view. To the contractor, the answer was no; temporary bracing was only intended for a few days. The day of collapse would normally have been spent installing permanent bracing and roof sheathing. Thus, possible exposure to wind was for less than a work week. To the insurance company, the answer was yes. The company was asked to cover the risk taken by the contractor. Thunderstorms and high winds were not uncommon in Kentucky in June. Bracing should have been adequate for any possible condition foreseen.

Who was right? As previously mentioned, after the collapse, the contractor placed a few trusses at a time and braced them completely as he went. Thus, *his* concept of adequate bracing certainly changed as a result of the collapse.

The case was heard before a judge who decided in favor of the insurance company. The judge ruled that the roof trusses should have been more heavily braced. The insurance company was, thus, not held to be responsible for the contractor's gamble.

Roof Tee Collapse

Precast, prestressed concrete tees are another commonly used roof structure system. More design and fabrication effort is generally involved with this product than with wooden roof trusses, but construction failures occur also. Precast tees, while not as fragile during erection as wooden trusses, are unstable and require adequate temporary support until construction is completed. The case under review involved a precast member that was particularly unstable and which collapsed when the supporting devices failed.

During the summer of 1975, construction was progressing on a school in New York state. One wing of the building, called the natatorium, was to house an indoor swimming pool. The roof of the building consisted of precast single tees spanning 60 ft (18.3 m). The end members were half tees, or ell shaped members, supported vertically at their stems by columns. The columns were located at the ends and third points of the members (Fig. 3). These members were unstable against rotation. Column supports at the stems were not designed to withstand an overturning effect due to the unbalanced weight of the half tees. Flange connections, as shown in Fig. 4, had been designed by the precast concrete contractor to support the members until a 1-1/2 in. (38.1 mm) composite concrete topping had been poured. The connections, designated as "standard weld plates" by the contractor, consisted of No. 3 reinforcing bars embedded in the tips of the full-tee and half-tee flanges. Each "weld plate" consisted of a 24-in. (609.6-mm) long bar bent at third points to create a section 8-in. (203.2-mm) long parallel to the joint between tees.

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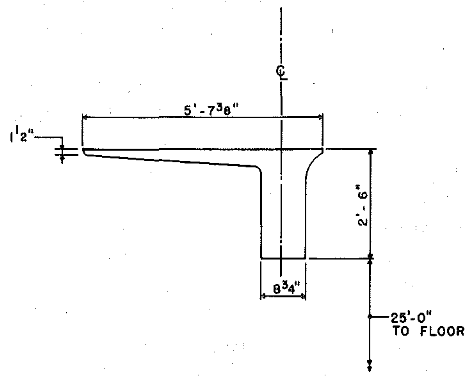

FIG. 3.—Dimensions of Half Tees (1 in. = 25.4 mm)

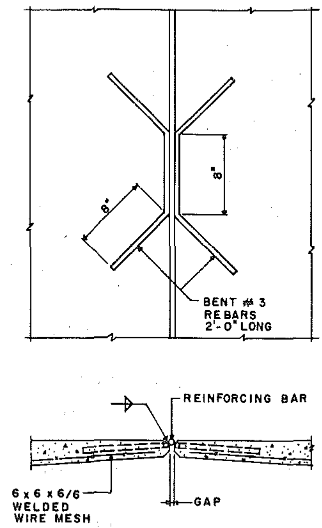

FIG. 4.—As-Built Tee Connection (1 in. = 25.4 mm)

The bent bars were not fastened in any manner to wire mesh, which constituted the flange reinforcement. The embedded bars were placed at 6 points along the flange tips, 10 ft (3.05 m) apart. As constructed, the "weld plates" were not connected to each other because of gaps between members in excess of 1/4 in. (6.4 mm). Instead, a large diameter reinforcing bar was welded between the two tips. These "weld plates" served as the only flange supports during construction.

The roof deck was completed by pouring a 1-1/2-in. (38.1-mm) concrete topping that was to act compositely with the tee flanges. A concrete floor subcontractor was employed for this operation. The subcontractor employed motorized concrete buggies to transport the wet concrete to the area being topped. During concreting, a load of wet concrete was dumped on a half tee by a buggy, which was then parked on another portion of the roof. As the workmen proceeded to spread and finish the concrete, the half tee rolled over and fell 25 ft (7.63 m) to the ground. Two workmen were killed, and a third workman was permanently disabled.

The ensuing lawsuit named as defendants the prestressed concrete contractor, the prestresser's structural engineer, the architect, the school board, the municipality's urban renewal agency, the general contractor, and the construction manager. The concrete flooring subcontractor was not sued by the workmen because of Workmens' Compensation liability limitations. The flooring subcontractor was eventually brought into the suit by the various defendants.

The writer was retained by the insurance carrier for the flooring subcontractor, and was provided with photographs taken of the collapse, along with reports by other experts and with the original design calculations for the tees. The investigative reports indicated that for 5 of

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the 6 "weld plates," the welded connection had held, but the embedded bars had either broken or pulled out of the tee flanges. Only one of the welded connections failed. There was no failure of the half tee itself.

From the photographs and descriptions of the collapse, it was evident that collapse was initiated by a loss of the flange support offered by the "weld plates." This was followed by an initial rotation of the half tee, a failure of the stem supports, a complete rotation, and then a drop of the half tee to the ground.

The investigation then concentrated on the strength of the "weld plates," particularly the minimal cover of the embedded steel bars and the distance that the bars had been embedded into the concrete. Required cover and development length depend on the size of reinforcing bars. The American Concrete Institute (ACI) Building Code sets minimum criteria for cover and development length. ACI 318-71, in effect at the time of design, called for a minimum cover of 5/8 in. (15.9 mm), and a minimum development length of 12 in. (304.8 mm) for No. 3 bars. Had the bars forming the "weld plates" been placed perfectly at the center of the 1-1/2-in. (38.1-mm) thickness of flange tip, the 5/8-in. (15.9-mm) cover requirement would have been met. The wire mesh used for flange reinforcement was approximately 1 in. (25.4-mm) below the top surface at this point, possibly interfering with positioning of the No. 3 bars. The 12-in. (304.8-mm) development length requirement was definitely not met by the 8-in. (203.3-mm) long bent legs. The "weld plates" would thus not be able to develop the full capacity of the reinforcing bars.

Forces in the weld plates were computed under various postulated design and construction loads. None of the construction loadings would have induced a force in the weld plates in excess of the full service loading. Had the weld plates been sufficiently embedded in the concrete, they would have been strong enough to carry this full service load.

The following conclusions were drawn:

1. Collapse of the half tee resulted from a failure of flange support offered by the so-called "weld plates."
2. "Weld plates" as *designed* had insufficient embedment to develop the strength of the steel rod or weld.
3. No postulated construction load would have been of sufficient magnitude to cause failure of the "weld plates" had adequate embedment of the reinforcing bars been provided.

In the opinion of the writer this failure was the result of the negligence of the designer of "weld plates." The designer failed to observe minimum requirements for embedment length of reinforcing bars. The designer also failed to account for fabrication tolerances and allowed only a minimal amount of cover for the reinforcing steel. The result was a pull-out of the bars under construction loading and a tragic loss of life. More careful attention to detail could have prevented the failure.

The case was resolved by a settlement reached prior to the start of trial. The insurance carriers for the defendants contributed in various proportions to the claims of the plaintiffs.

During the construction of an urban redevelopment project in an eastern city in 1975, the floors of one of the buildings collapsed, injuring several workmen. The interesting feature of this event was that an unsafe construction procedure had been going on for months known to the architect, engineer, and general contractor, but no one had taken action to stop the construction. After the collapse, each party sought a defense by strict interpretation of its contractual responsibilities.

The project consisted of high rise and low rise residential units. The high rise units were constructed of reinforced concrete. The low rise units were two story townhouses having masonry party walls and steel stud and joist framing. The architect had been paid an additional fee for involvement in the construction phase of the project.

The architect was to make periodic visits to the site, not less than once a week. The structural, mechanical, electrical, and sanitary engineers were also obligated to visit the site periodically to observe their respective phases of the work.

The architect contracted with a structural engineer for design and construction inspection services. The structural engineer was responsible for direct inspection of the high rise units but had no inspection responsibility for the low rise units.

The steel framing was installed by a carpentry subcontractor, and the masonry work was performed by a mason subcontractor. For the first low rise units built, the masons worked from scaffolds placed on the ground and built up as the masonry work proceeded to the upper story. This procedure was changed as later units were constructed. For these units the masons built the party walls up to the first floor, had the carpenters set the joists and plywood subflooring, and then set up their scaffolds on the partly finished floors.

The steel joists were light gage steel, cold formed into a "Cee" section. Plywood subflooring was nailed to the flanges of the joists. As with wooden joists, bridging was required to provide lateral stability for the joists. Without the bridging, the joists could twist and collapse laterally at loads well below the design loads.

"V bridging" was manufactured for these steel joists. The joist manufacturer required that bridging must be in place prior to placing any construction or other live loads on the joists. The bridging had to be installed in continuous runs and be solidly anchored to end walls. Shop drawings submitted by the contractor and approved by the architect had stated that no loading of joists was to be permitted until the proper attachment of plywood subfloor and bridging.

In addition to lateral instability, another potential source of joist failure was web crippling. Since the joists were formed from light gage steel, a heavy concentrated load along the joist span or a large reaction at the end bearing could cause the steel to buckle or crush at the point of loading. This condition could be avoided by either limiting the loading or reinforcing the joist at the point of loading.

The joists used in the building floors were limited in their load carrying capacity by web crippling at the supports. However, when the joists were provided with web stiffeners, as well as lateral bridging, full

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bending stress capacity could be developed. The web stiffening at the end bearings was to be accomplished by filling the masonry bearing at the joist end solidly with grout. The solid grouting was essential to the development of the load carrying capacity of the joists.

The joist system was thus dependent on complete assembly before being capable of sustaining the design loads. The joists required:

1. Lateral support as provided by "V bridging" and subflooring to prevent lateral buckling.
2. Solid grouting at end bearings to prevent web crippling.

Deletion of one or both conditions for the longer spans would have dangerously reduced the joist load capacity.

The carpentry subcontractor complained to the general contractor of overloading of floors by the mason subcontractor. A field report by the architect noted the presence on unbraced joists. Representatives of the structural engineer were notified of the condition. At a joint meeting, attended by representatives of the architect, as well as contractors and subcontractors, the deletion of bridging was discussed. Minutes of the meeting were distributed to all present and to the structural engineer. The minutes stated: "In reviewing the installation of bridging, it was determined that the bridging will be installed after the bricklayer leaves the floor due to popping of the bridge screwing." The bridging was overloaded, thus it was not to be installed!

The minutes of the following job meeting indicate that the minutes of the previous meeting were accepted. There were no comments about, or objections to, deletion of the installation of joist bridging from the architect or engineer. Though not present, the engineer had received a copy of the minutes.

As previously mentioned, the lateral support offered by the "V bridging" was essential for the development of the load capacity of the joists. Deletion of the bridging greatly reduced the capacity of the joists. This reduction in strength was obvious to the architect and the structural engineer, yet no action by either appears in the record after the statement in the job meeting.

The loadings placed on the incomplete floors by the scaffolding, stacks of masonry, mortar and workmen were of the magnitude of the design loadings for the joists. If the bridging had been in place, the joists would have developed their full design capacity.

The second structural requirement was for solid grouting at end bearings. Photographs taken by the structural engineer showed joists that remained in place after the collapse but which experienced web crippling at their bearings. The end bearings had not yet been filled and grouted. Other photographs taken by the engineer showed joist ends which did not appear to be completely grouted.

Scaffold legs were placed in proximity to the wall bearings of the joists, thereby introducing large concentrated forces in the joists supporting the legs. The induced forces were sufficient to cause web crippling of the joists if the end bearings were entirely ungrouted or were poorly grouted. Thus, the assembly was deficient with respect to the second requirement for stability.

Despite the knowledge of the unsafe condition by representatives of all the parties the work continued. In October 1975, while work was progressing on the top level of a townhouse, one of the floors supporting the scaffolding collapsed. The debris fell to a lower floor which partially collapsed. The workmen on the upper floor were injured in the fall.

The collapse was a predictable event. In the ensuing lawsuit, the various contractors accepted responsibility but sought to spread liability to the architect and engineer. In their defense, the architect and engineer looked at the terms of their contracts. They had had no contractual responsibility for methods of construction. Their responsibility was for quality of finished construction. The structural engineer's representative claimed he had never read the job meeting minutes. The architect asked what power it had to force compliance with safe practice.

This is a case where the issues seem clear. A subcontractor pursued an unmistakably unsafe construction practice. The practice was noted by the general contractor and the architect. The structural engineer would have seen the mason's scaffolding on the floors during periodic site visits. All knew what was going on but did nothing to stop the practice. Professional responsibility for the architect and engineer went beyond the terms of their contracts. They should have taken steps rather than await the future.

The case was settled out of court by contributions from the insurance carriers for all parties, including the architect and engineer.

Conclusions

The common feature in these cases has been human failure. In the church roof case, the contractor was in a hurry at the end of the day and piled the plywood sheets too high and too far out on the trusses. In the warehouse case, the contractor used what he considered as adequate temporary bracing. This would have been adequate had the windstorm not developed. He, too, was in a hurry. The roof tee collapse involved negligence by the designer of the flange connections. The designer did not pay adequate attention to detail and failed to provide enough embedment of the reinforcing bars. Finally, the low rise collapse is the most glaring human failure of all the cases. Contractors, architect, and engineer saw what was going on, knew the construction practice was improper, and took no direct action. Predictably, the building collapsed.

What can be done? Firstly, to keep in mind that the structure has not yet developed its full strength. Without careful planning the danger of collapse is very real. The partially completed structure will not be forgiving of a mistake. Secondly, to develop a safe erection plan and see that it is followed. Impress supervisors and workmen that the plan has been developed for their safety as well as logical construction sequence. The workers on the structure have the most to lose if it collapses. Thirdly, see that the erection plan is being followed. Delegate as many people as necessary to cover this aspect of the job.

Above all, under the ever present pressure of time in a construction job, do not neglect details. Do not try to gain time by omitting steps. A

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few more sheets of plywood in a bundle or a few less pieces of wood for bracing can gain time but can lose lives as well as money. An "obvious" support detail still requires checking.

Finally, view your responsibilities in a broad sense. If you see something wrong you should do all you can to see that it is done right. Do not be limited strictly by contractual language.