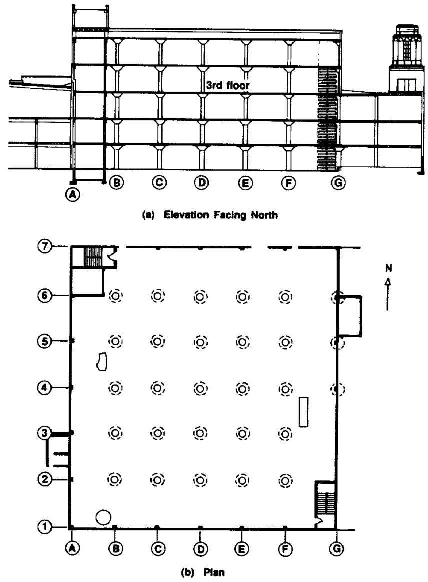


Fig. 1 — Details of Kimberley-Clark warehouse structure.

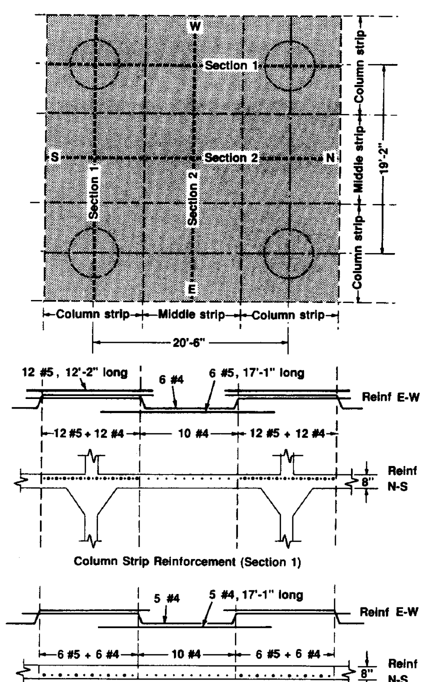


Fig. 2 — Reinforcement details for third story floor slab.

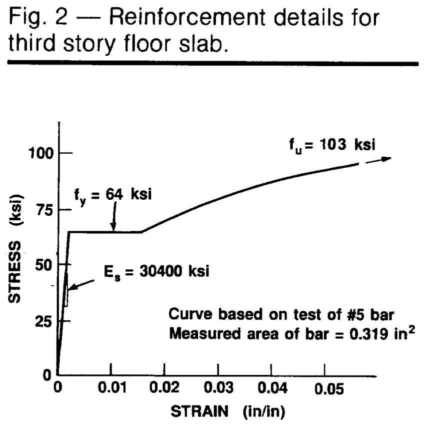


Fig. 3 — Reinforcement stress-strain properties.

# Investigating the Collapse of a Warehouse

by F. J. Vecchio and M. P. Collins

**A reinforced concrete warehouse structure was subjected to floor loads considerably greater than design-specified values when a major collapse occurred at about the same time. Investigations were undertaken to estimate floor load capacity and to determine the effects of other factors so the cause of the collapse could be established. Details of the structure's design and construction and of the loads imposed are presented. Nonlinear analyses are described in which calculations were made of the floor's theoretical load-deformation response and of its response under fire conditions. Results indicate that nonlinear effects in reinforced concrete structures, most notably membrane action, can result in floor load capacities surprisingly larger than the design values.**

The ability of a well designed and well constructed reinforced concrete building to resist extremely high overloads prior to collapse was demonstrated by an incident in Niagara Falls, Ontario, Canada.

The third floor of a four-story flat slab building was being used to store drums of nickel pellets. Although the floor had been designed for a live load of only 125 psf (6.0 kPa), it supported a load of nearly 900 psf (43.1 kPa) prior to collapse.

Further, it was not clear whether the excessive load was the sole cause of the collapse since at about the same time the floor collapsed an explosion and a major fire occurred in the lower stories of the structure. Whether the collapse of the floor triggered the explosion or the explosion triggered the collapse became the issue of a lengthy legal dispute.

From a structural engineer's point of view, it is of interest to understand how this structure could resist a superimposed load seven times greater than the load for which it was designed. An analytical investigation of the collapse focused on some fundamental aspects of reinforced concrete behavior, often overlooked in conventional analyses, that can greatly increase a structure's load-carrying capacity.

## Details of the structure

The Kimberley-Clark building was built in 1944, in Niagara Falls, Ontario, Canada, in accordance with then current building codes. The building was a simple four-story structure with basement, having plan dimensions of approximately 125 x 119 ft (38 x 36 m) (Fig. 1).

The structural system employed was primarily a reinforced concrete

**Keywords: collapse**; concrete slabs; deformation; fires; **floors**; frames; **loads (forces)**; reinforced concrete; structural analysis; **warehouses.**

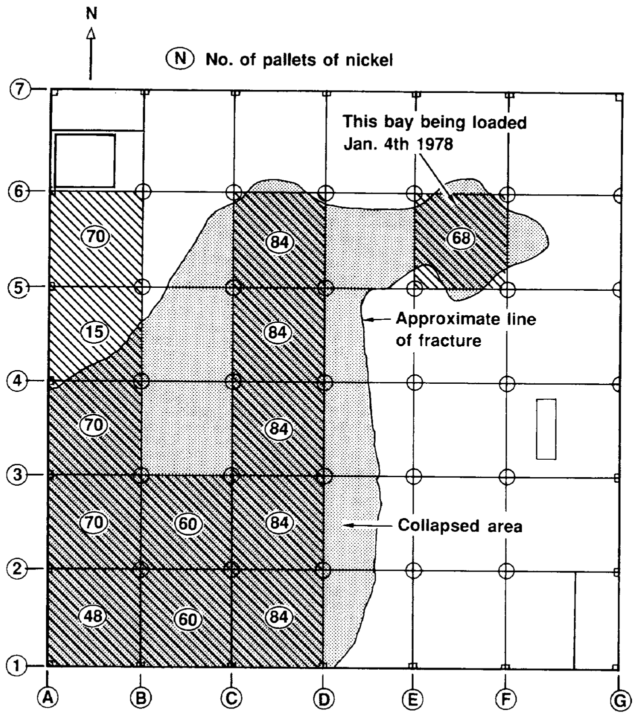



Fig. 4 — Loading and collapse patterns.

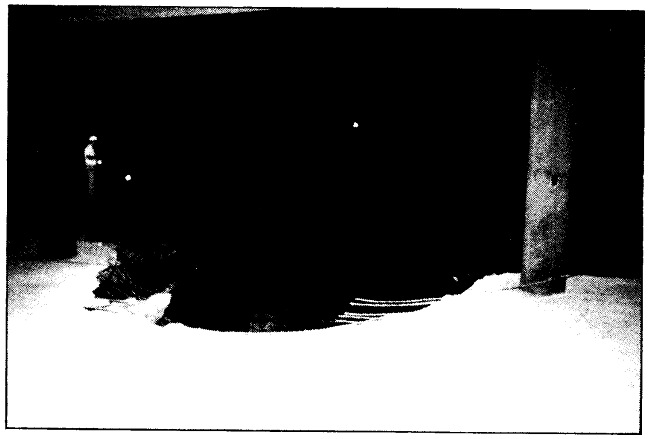


Fig. 5 — Portion of collapsed floor.

flat slab system with six bays in each direction. The center-to-center column spacing was approximately 20′-6″ (6.25 m) in the N-S direction, and 19′-2″ (5.85 m) in the E-W direction. The exterior columns were rectangular with haunches, while the interior columns were circular with capitals. Column and capital diameters decreased with elevation. The columns supporting the third floor were 18 inches (450 mm) in diameter with 5′-0″ (1.52 m) diameter capitals.

The floor slabs were 8 in. (200 mm) thick, with the first and second floors having an additional 1½ in. (40 mm) concrete topping. At the first floor level, the floor slab was thickened by 4 in. (100 mm) drop panels at the columns. At higher floor levels, the slab was thickened by 6 in. (150 mm) around the perimeter, over a width of 4′-3″ (1.3 m).

The floor to floor height ranged from 11′-0″ (3.35 m) to 12′-0″ (3.65 m). The exterior walls were brick masonry, and stair/elevator shafts were located at various points around the perimeter of the structure.

The floor slabs were reinforced with No. 4 and No. 5 deformed bars. The reinforcement details, shown in Fig. 2, were consistent with a column-strip/middle-strip design method. Similar patterns were used in both directions.

The original design assumptions for the structure specified design live loads of 250 psf (12.0 kPa) for the basement and first floor, 125 psf (6.0 kPa) for the remaining floors, and 40 psf (1.9 kPa) for the roof. The specified concrete strength was 3000 psi (20 MPa), and the allowable tensile stress in the reinforcement was 20,000 psi (138 MPa).

The as-built details of the structure were found to essentially conform to the original design drawings and specifications.

## Material properties

After collapse, the constituent materials of the then 34-year old structure were sampled and tested. The compressive strength of the concrete, determined from cores taken at various locations, ranged from 4240 to 6110 psi (29.2 to 42.1 MPa). The average compressive strength was 5400 psi (37.2 MPa); considerably higher than the specified 3000 psi (20.7 MPa).

Coupons were taken from No. 5 reinforcing bars at the third floor level. The steel was found to have a yield stress of 64,100 psi (442 MPa) and an ultimate strength of 103,000 psi (710 MPa), with a modulus of elasticity of 30,400 ksi (210,000 MPa). As can be seen from the stress-strain relationship shown in Fig. 3, the steel exhibited a short yield plateau.

The effective depth of the slab reinforcement, as placed on the third floor of the structure, ranged from 5½ to 7¼ in. (140 to 185 mm).

## Loading and collapse

The third floor of the warehouse became the site for storage of drums of nickel beginning in mid-December 1977. The drums, on wooden pallets, were transported to the third floor by a freight elevator at the northwest corner of the building. From the elevator, the pallets were transported to their storage location by forklift truck.

The nickel located near the collapsed area consisted of nickel pellets encased in 500 lb (225 kg) drums, with eight drums to a pallet. The total weight of each pallet was about 4160 lb (1890 kg). The pallets measured 3′-0″ (915 mm) square, thus each resulting in a floor load of approximately 450 psf (21.5 kPa). In general, the pallets were stacked two high, giving a total floor live load of about 900 psf (43.1 kPa).

The exact number and arrangement of pallets stored on the third floor in early January 1978 is a point of contention. However, records of the amount of material received and post-collapse inspections suggest that the loading pattern was close to that indicated in Fig. 4, which shows the number of pallets per bay. Bays containing 84 pallets had an average superimposed load of 890 psf (42.6 kPa). Note that this high load extended over almost the entire north-south length of the structure.

The floors below that used for storing the nickel contained a paper-products manufacturing plant. With such operations, the dangers of dust explosions and fire are an ever-present concern.

On January 4, 1978, large sections of the third floor collapsed (Fig. 4 and 5). The collapse encompassed approximately 14 bays of the structure, with the columns, slabs, and drums of nickel crashing down to the basement level. At about the time of the collapse, a large explosion was heard. As emergency equipment and personnel arrived, intense fire was gutting the structure. The fire raged for 48 hours before it could be brought under control. Two men died in the incident.

The contention of the paper manufacturing firm was that the stored nickel overloaded the floor, resulting in its collapse and consequently causing an explosion and fire. The proprietors of the nickel operation argued that an explosion and fire in the paper plant was the primary cause of collapse, with the intense heat below subsequently weakening the floor and leading to its failure. Legal proceedings were launched, culminating in an out-of-court settlement 10 years later.

## Analysis of ultimate load capacity

To obtain an estimate of the load-bearing capacity of the slab at the third floor level, the structure was analyzed using computer program TEMPEST.[1] This program can perform nonlinear structural analyses of reinforced concrete plane frames subjected to thermal and/or mechanical loads. It takes into account material nonlinearities (both concrete and reinforcement), geometric nonlinearities, membrane action, temperature degradation of material strength, time-related effects, and various other influencing factors.

The most critically loaded portion of the structure was assumed to be along Column Line 4, at the third floor level. For analysis purposes, four bays extending from Column Lines A through E were considered. Fig. 6 gives details of the frame model chosen. The model was one-bay wide and had 36 joints and 37 member segments, with 7 different member types (i.e., varying in section details).

The columns framing into the floor from below, given as 18 in. (450 mm) in diameter, were taken as equivalent to 16 in. (400 mm) square columns. The columns framing into the floor from above were taken as equivalent to 14 in. (350 mm) square. The reinforcement in the columns, unknown at the time of the analysis, was assumed to be 4 percent for all columns.

To take into account the influence of the column capitals and reduced clear spans, beam elements representing drop panels were used. These beam elements were of 14 in. (350 mm) depth and 18 in. (450 mm) length, measured from the column centerlines, and were included at each end of Interior Span CD. Details of all member cross sections are given in Fig. 6.

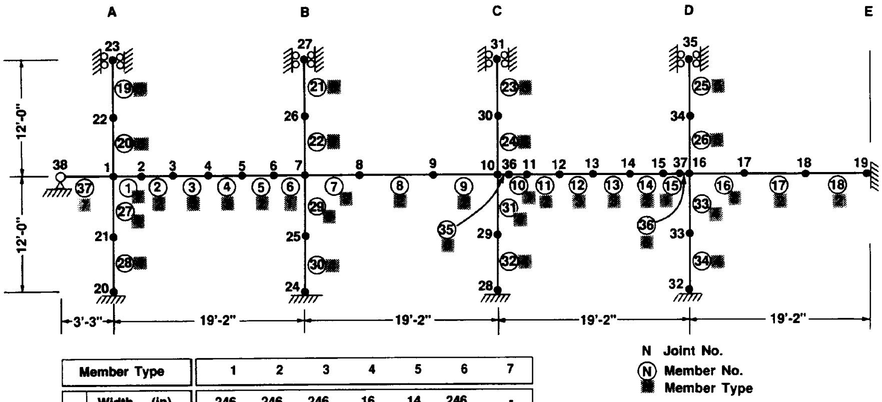
The four stairwells, located roughly at the corners of the warehouse floor plan (Fig. 1), would likely contribute to the ultimate capacity of the floor slab by restricting outward expansion (i.e., enhancing membrane action). To estimate their stiffness, the stairwells were considered to be cantilevered from the basement floor. A nominal cross section corresponding to a box section 120 x 168 in. (3.0 x 4.25 m) and 6 in. (150 mm) thick, with bending about the weak axis, was assumed.

A reduction of 75 percent in the effective stiffness was assumed to account for cracking and shear-lag effects. Thus, at the third floor level, the stairwells were estimated to add a lateral stiffness of 314 kip/in. (55 kN/mm) to the system. A spring member of equivalent axial stiffness was added to the frame model at the location of the exterior column-slab joint, i.e., Member 37 in Fig. 6.

A concrete compressive strength of 5400 psi (37.2 MPa) and a tensile strength of 290 psi (2.0 MPa) were assumed. The strain at the peak compressive stress was taken to be 0.0025 in./in., giving an initial tangent modulus of elasticity of 4320 ksi (29,800 MPa). For the reinforcing steel, a yield stress of 64 ksi (440 MPa) and an ultimate stress of 103 ksi (710 MPa) were used. Further, a modulus of elasticity of 30,400 ksi (210,000 MPa) and a strain-hardening modulus of 320 ksi (2200 MPa) were deduced from Fig. 3 and employed in the analysis.

The loading pattern described in Fig. 7 was assumed, based on the floor loads given in Fig. 4. The dead loads primarily represented the self-weight of the structure. A superimposed uniformly distributed load acting on Spans AB and CD represented the live load from the stored nickel. Span AB was against the west wall and was obstructed by some openings and some overhead ducts, that prevented full loading of this area. Because of this, the maximum live load on Span AB was limited to 500 psf (23.9 kPa). The live load on Span CD was increased until failure.

The analyses indicated that the ultimate floor capacity of Span CD was approximately 950 psf (45.5 kPa) superimposed load; that is, including dead load, a total of 1050 psf (50.3 kPa). Fig. 8(a) shows the load-deformation response determined for the structure. A gradually softening response was observed. At a floor load of less than 400 psf (19.2 kPa), yielding occurred at the supports and at the midspan of Span AB. This resulted in a temporary increase in the stiffness of CD, arising from rapidly increasing axial forces.




| Member Type | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|---|
| | Width (in) | 246 | 246 | 246 | 16 | 14 | 246 | - |
| | Height (in) | 8 | 8 | 8 | 16 | 14 | 14 | - |
| Reinforcement | Area ($in^2$) | 8.23 | 8.23 | - | 3.75 | 2.90 | 8.23 | 0.43 |
| | Depth* (in) | 1.5 | 1.5 | - | 3.0 | 2.75 | 1.5 | - |
| | Area ($in^2$) | - | - | - | 2.50 | 1.90 | - | - |
| | Depth (in) | - | - | - | 8.0 | 7.0 | - | - |
| | Area ($in^2$) | - | 5.12 | 5.12 | 3.75 | 2.90 | - | - |
| | Depth (in) | - | 6.5 | 6.5 | 13.0 | 11.25 | - | - |

*Reinforcement depth relative to top surface of member

Fig. 6 — Plane frame computer modelling of structure.

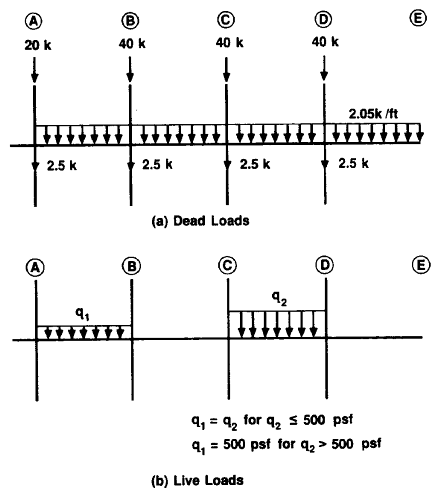



Fig. 7 — Loading conditions assumed in analysis.

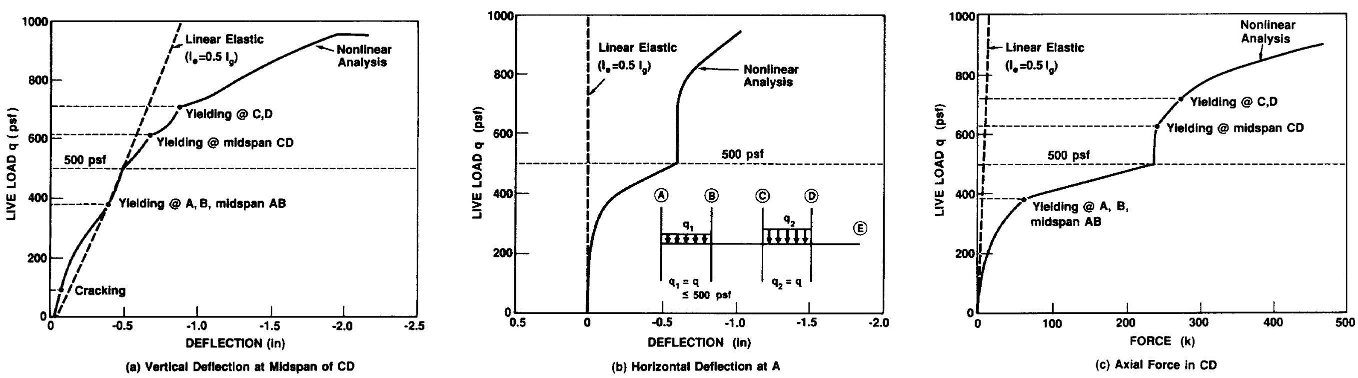



Fig. 8 — Computed response of structure.

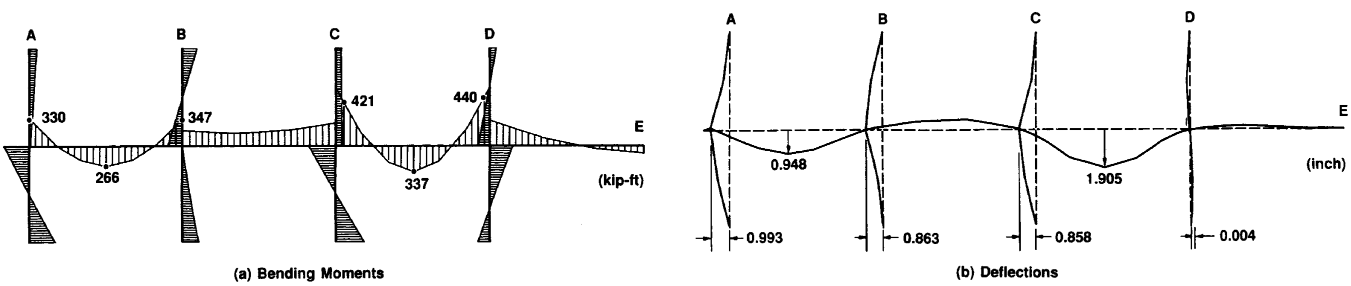



Fig. 9 — Analysis results for 950 psf superimposed load.

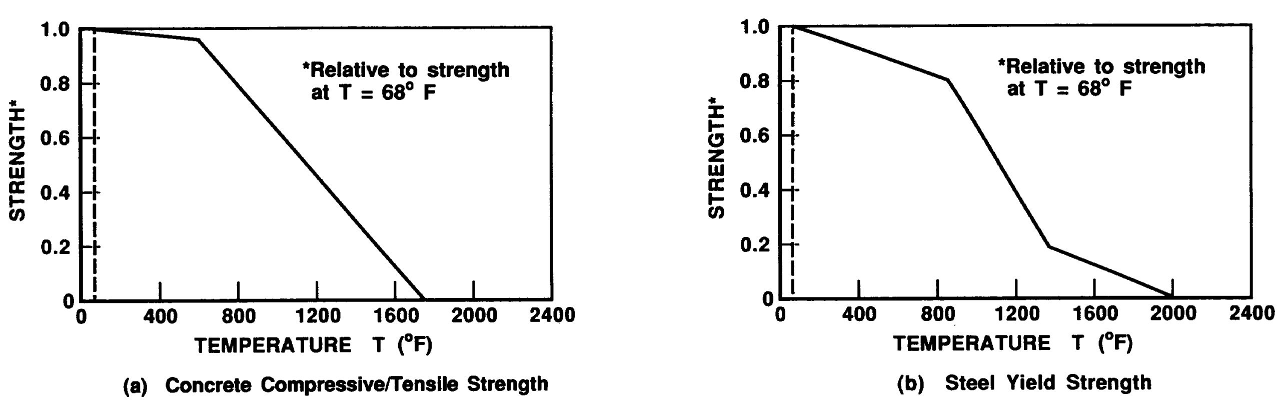



Fig. 10 — Temperature-strength degradation relationships used in analysis of response to fire.

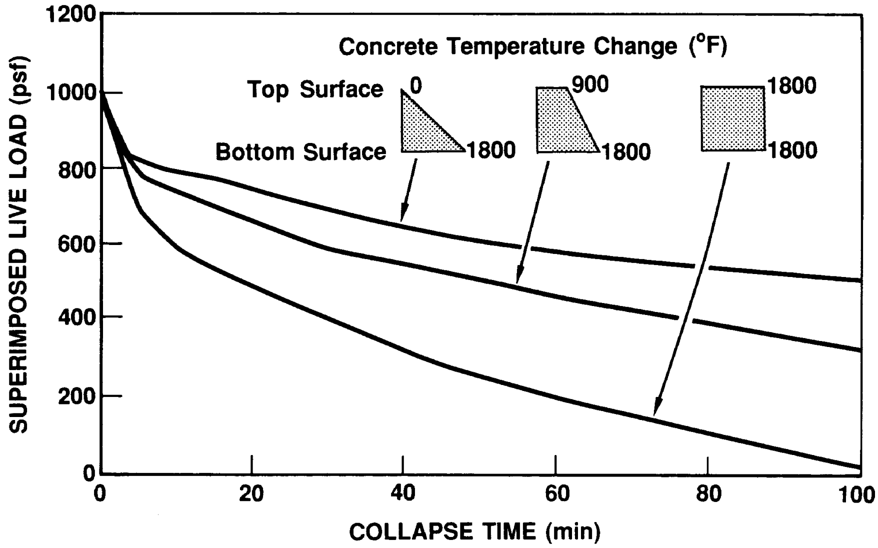



Fig. 11 — Computed collapse times of slab subjected to fire and live load.

At 600 psf (28.7 kPa) loading, yielding was experienced at the midspan of Span CD. At 720 psf (34.5 kPa), yielding also occurred at the supports of CD (just beyond the drop panels). Nevertheless, membrane action and strain-hardening effects allowed a substantial further increase in load capacity to be realized. At 950 psf (45.5 kPa), hinging at the supports and midspan resulted in a collapse mechanism.

At 950 psf (45.5 kPa), the axial compressive force in Span CD was 475 kips (2115 kN). The bending moments at the midspan and supports were 340 kip-ft (460 kN-m) and 430 kip-ft (585 kN-m), respectively. The vertical deflection at the midspan was 1.90 in. (48.3 mm) and the horizontal deflection at the exterior slab-column joint was 0.99 in (25 mm). Moment and deflection diagrams are shown in Fig. 9.

A linear elastic analysis was also conducted, assuming cracked section stiffnesses equal to 50 percent of the uncracked stiffnesses. The resulting load-deformation response for the vertical deflection at the midspan of CD was found to be a fair approximation to that determined from the nonlinear analysis. [Fig. 8(a)].

However, the linear analysis was grossly in error in predicting the horizontal deflection at the exterior beam-column joint, shown in Fig. 8(b). It failed to account for the net axial elongation that occurs in a reinforced concrete slab, as cracking and yielding result in very high tensile strains. The axial elongation, of course, results in substantial axial compressive forces being developed in the slab, as seen in Fig. 8(c). It is this induced compression that results in an increased flexural capacity. Ignoring the beneficial influence of the membrane forces would reduce the calculated ultimate capacity of the floor to about 650 psf (31.1 kPa).

Recall that a spring element was introduced to account for the lateral stiffness of the stairwells. To investigate the sensitivity of the analysis to this assumption, the structure was re-analyzed with the spring stiffness doubled (i.e., to 630 kip/in. (110 kN/mm) ). It was found to result in an increase in floor capacity of less than 50 psf (2.4 kPa).

In all previous calculations, a patterned live load was assumed (i.e., alternate bays). A re-analysis was conducted to determine the influence of having each bay loaded. With superimposed live loads of 500 psf (23.9 kPa) acting on all other bays, the capacity of Bay CD was found to slightly exceed 1000 psf (47.9 kPa) live load. The increase was derived not from any redistribution of moments within the structure, but from further increases in axial compression induced in the slab.

The shear capacity of a slab, like the flexural capacity, is enhanced by an increase in membrane compressive stress. Assuming that the compressive force in the slab was 475 kips (2115 kN), the punching shear capacity around the column capital would have been approximately 525 kips (2340 kN). This would translate to a floor load in excess of 1300 psf (62.2 kPa); hence, this type of failure did not govern.

## Analysis of effects of fire

Fire, triggered by dust and fuelled by paper, could quickly result in temperatures exceeding 1800 F (1000 C).

Using standard heat-flow principles contained within program TEMPEST, time-dependent nonlinear thermal gradients developed within the slab were determined as functions of the imposed surface temperatures and elapsed time. A coefficient of thermal expansion of 5.56 x $10^{-6}$/F (10.0 x $10^{-6}$/C) and a thermal diffusivity of 1.86 x $10^{-3}$ in.$^2$/s (1.2 mm$^2$/s) were assumed for the concrete, and a coefficient of thermal expansion of 6.2 x $10^{-6}$/F (11.9 x $10^{-6}$/C) was assumed for the reinforcement. Strength and stiffness degradations as functions of temperature were assumed for the materials as shown in Fig. 10.

Analyses were conducted to determine the time required for Slab CD to collapse under various conditions of temperature and superimposed load. In the analyses, it was assumed that the surface tem-

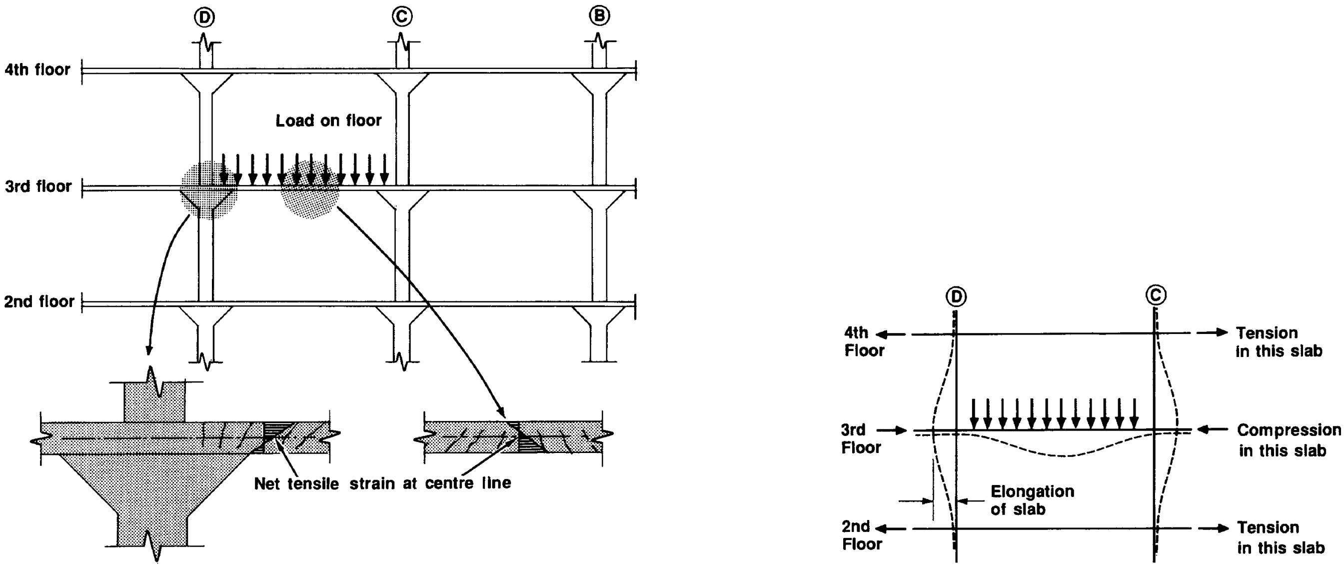



Fig. 12 — Membrane action in slabs.

perature changes occurred instantly, and that the axial force in the slab remained at 475 kip (2115 kN) as determined previously. The results of the analyses are summarized in Fig. 11.

An initial rapid drop in load capacity resulted, under all three fire conditions, from a crushing of concrete at the compression face due to the extreme thermal expansion. Steady decreases in capacity thereafter were primarily due to reductions in the yield strength and ultimate strength of the reinforcement.

It can be seen from Fig. 11 that if the superimposed load on the slab was 900 psf (43.1 kPa) (i.e., nickel pellet stacked two pallets high), collapse would have occurred in 2 to 3 minutes. With a superimposed load of 600 psf (28.7 kPa), the slab would have collapse after 10 minutes if the temperatures of the top and bottom surfaces were 1800 F (1000 C), while it would take about 25 minutes if the bottom temperature was 1800 F (1000 C) and the top temperature was 900 F (500 C).

## Membrane action

The greatest single difference between the linear and nonlinear analyses conducted lies in the fact that the nonlinear analysis accounts for the net elongation that typically occurs in a reinforced concrete member in flexure (Fig. 12). The tensile strains on the cracked face are normally much larger than the compressive strains on the opposite face, particularly if the reinforcement is yielding. Hence, the tendency is for the member ends to push out longitudinally as the member bends under the applied transverse loads.

In Slab Span CD at 950 psf (45.5 kPa) loading, the elongation over a 19′-2″ (5.85 m) span was 0.85 in. (21.6 mm). This translates to an average tensile strain of $3.7 \times 10^{-3}$ in./in., which is large despite the presence of a substantial axial compressive force of 475 kips (2115 kN).

When the ends are restricted from outward movements, in this case by the flexural stiffness of the columns and the axial stiffness of the floors above and below, a compressive thrust is induced in the member. For reinforced concrete members, axial compressive forces initially serve to increase flexural capacity. This behavior, known for many years, is commonly referred to as membrane action.

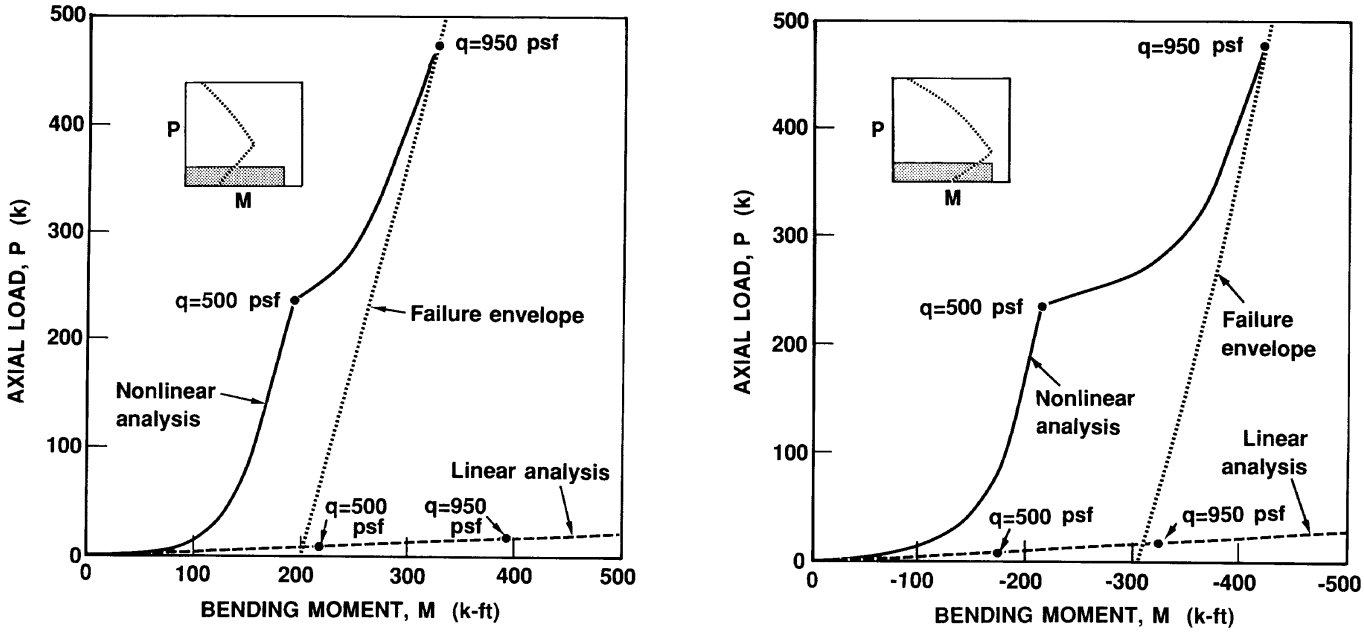
In Fig. 13, moment-axial load interaction diagrams are shown for Slab CD at the left support and at the midspan. Note that conditions at the two supports were virtually identical. The failure envelopes, identifying ultimate load combinations, show significant increases in moment capacity with increasing axial force. Also shown on the interaction diagrams are the force resultants determined from the linear and nonlinear analyses for increasing levels of superimposed live load.

The linear analysis predicts a shallow linear gain in axial force as the applied load is increased. The flexural capacity at the midspan is exceeded at approximately 450 psf (21.5 kPa) live load. The capacity at the supports is not exceeded until the load reaches 900 psf (43.1 kPa), assuming that the elastic moment distribution is maintained. Failure, however, would be assumed at 450 psf (21.5 kPa).

Allowing for partial moment redistribution, as permitted by ACI 318,[2] the ultimate floor capacity would be calculated at 530 psf (25.4 kPa). If complete moment redistribution is considered, the floor capacity would increase to approximately 650 psf (31.1 kPa). The axial compressive force at this point would be in the range of 10 kips (45 kN).

The nonlinear analysis, on the other hand, shows a much more pronounced increase in axial force with increasing load. Note: the cusp point at 500 psf (23.9 kPa) results from the changing load conditions, i.e., Span AB is not subjected to loads higher than 500 psf (23.9 kPa). At the failure load of 950 psf (45.5 kPa), both the end and midspan sections reach the failure envelope simultaneously. By this point, the moment capacities at both sections have benefited significantly from the high induced axial compressive forces.

The slab at the third-floor level of the Kimberley-Clark Warehouse




(a) Interaction Diagram for Slab CD at Midspan

(b) Interaction Diagram for Slab CD at Support

Fig. 13 — Moment-axial load interaction curves for slab.

was designed for a superimposed load of 125 psf (6.0 kPa) plus a dead load of 100 psf (4.8 kPa), giving a total design load of 225 psf (10.8 kPa). The analysis described indicates that, with no fire, the floor could carry a total load of 1050 psf (50.3 kPa). This suggests that the structure had a factor of safety of 4.67 against collapse.

Ockelston[3] reported an experiment in which a 10-year-old reinforced concrete building was loaded to failure. The testing of the building occupied a full-time technical staff of eight and an unreported amount of unskilled labor for about four months. The building had been a dental hospital, built in 1942. It was designed for a superimposed floor load of 70 psf (3.35 kPa).

The 5.3 inch (135 mm) thick concrete slab floor weighed 66 psf (3.16 kPa), giving a total design load of 136 psf (6.51 kPa). When two bays of the floor were loaded with metal rail chairs, the floor did not collapse until the total floor load had reached 843 psf (40.4 kPa). The measured factor of safety was thus 6.20, which Ockelston partly attributed to membrane action.

## Conclusions

The investigation concluded that it is technically possible for the third floor of the Kimberley-Clark building, designed for 125 psf (6.0 kPa), to have sustained a short-term superimposed load of 950 psf (45.5 kPa). The collapse that ensued could as likely have been triggered by a fire below weakening the floor as by floor overload.

The investigation also showed that an appropriate nonlinear analysis can be a useful tool in understanding the behavior of reinforced concrete structures subjected to extreme conditions. It was found that before the reinforced concrete slab can collapse it must elongate considerably. The surrounding structure resists this elongation, which causes high axial compression to develop in the slab. This compression substantially increases the flexural capacity of the slab making it possible for the slab to carry a surprisingly high load prior to collapse.

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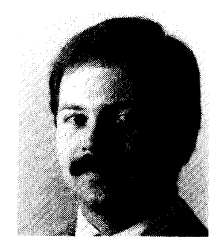

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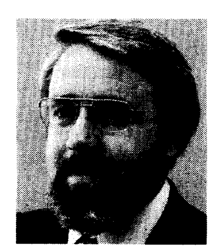

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