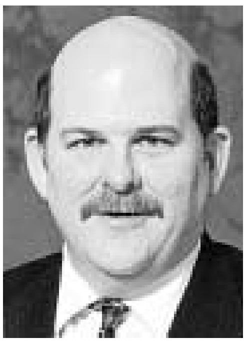
North American Steel Construction Conference

Steel High-Rise Outlasts Blaze



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For more than 18-hours, fire raged through nine floors of a 38-story building, burning the contents of each floor in about two hours and then jumping to the floor above. While the building was in the midst of a sprinkler retrofit, the work had only just begun on the top floors of the building. However, the fire was eventually brought under control when it reached the floors that had sprinklers.

A survey of the positions of the exterior columns after the fire showed that the fire had displaced many columns on the fire floors—with many of the columns on the fire floors outside the erection tolerances of the AISC Code of Standard Practice. However, more than 8% of the columns on the floors well below the fire were also out of tolerance, indicating that possibly the building was erected out-of-tolerance or had displaced due to eccentric gravity load. For example, 65 percent of the columns on floor 13 (nine stories below the fire) were found to be out of tolerance. The interstory displacements on these floors were also large, often larger than the twice the L/500 tolerance. (Each story height was 140”). The largest interstory displacement on a fire floor was 1.5 inches. Other than this column, almost all interstory displacements on the fire floors were less than L/150 or 0.93 inches.

An examination of the displacements as a function of the height of the building shows that the building leans to one side, a trend that begins well below the fire floors (which start at about 250’). This lean of the building was believed to be due to eccentric gravity loading. However, it is also apparent that the floor systems of the fire-affected floors had contracted and were pulling the outer columns inward on these floors, especially at about 380’. The largest absolute displacement was 5.6” at this location while the erection tolerance of the AISC Code of Standard Practice is 2.4”. Other than this location, the displacements were all within an envelope equal to two times the erection tolerances of the AISC *Code of Standard Practice*.

These and other permanent distortions observed in the structure after the fire are indications that it had experienced inelastic deformations during the fire. Accompanying the distortions are locked-in forces present in the members located within and near the fire-damaged areas. The locked-in forces were induced by the changes in the length of beam members.

Beam members are partially constrained by neighboring parts of the frame that prevent free expansion or contraction of the members during and after the fire. The beam members buckle at high temperatures when the modulus of elasticity and yield strength are very

Table 1: Summary of largest fires in high-rise steel-framed buildings					
Fire	**# of stories**	**Duration (hours)**	**# of fire floors**	**Duration per floor (hours)**	**Extent of Repair**
This building	38	18	9	2	$24M + cost of replacement, Building was completely dismantled.
Alexis Nihon Plaza	15	14	4	3.5	$80M, Replace all walls, deck, and beams on all fire floors and roof in the corner of the building affected by the fire
One New York Plaza	50	5	3	1.7	$10M, 24,000 sq.ft. of floor and 100 beams replaced
Broadgate - Phase 8	13	5	2	2.5	$40M, 16,000 sq. ft. of floor, 51 beams, and 5 columns replaced
First Interstate Bank	62	4	5	0.8	Minor damage to decking, fireproofing replaced
Westvaco Office Bldg.	42	2	2	1	$15M, Replace deck and 40% of floor framing on fire floor
Bally's (was MGM Grand)	26	2	3	0.7	Minor damage to structure

low. After buckling, the members eventually cool and want to contract, but at this point the yield strength is restored. The contraction is resisted by the constraints and therefore, locked-in forces develop.

Because of the severity of the fire, concerns were expressed about the effect it had on the load-carrying capacity of the steel frame of the building. There was concern that the yield strength of the steel members may have been significantly reduced. However, as discussed later, the distribution of yield strength values was the same as would be expected for shapes unaffected by fire. The existence of the locked-in forces was also a concern.

Among the various reinstatement plans, at least one third of the girders and floor beams were to have been replaced on the fire floors, and many more were to have been straightened. On one floor, it was planned to replace at least half of the girders and 70 percent of the floor beams. Replacing and straightening the damaged beams would be expected to relieve some of the contraction and locked-in forces and allow the columns to move back at least part of the way toward their original configuration.

In contrast to the beams, the columns remained in good condition and required no straightening or replacement. In fact, the name of the steel mill was painted on the columns and was still readable, indicating that at least these columns had not even reached the temperature at which the paint would peel.

It is important to note that all buildings may have locked-in forces, and this building no doubt had some locked-in forces before the fire. Locked-in forces may be caused by differential settlement of the foundation and by forcing members into alignment during construction, among other causes. In addition to these long-range locked-in forces, each member has significant residual stresses. After all, steel shapes are manufactured at very high temperatures and are subjected to extensive deformation during rolling and straightening.

However, locked-in forces must be in self-equilibrium within the building, and therefore the limit load for gravity or lateral load carrying capacity of the steel frame will not be affected by these locked-in forces. A basic principle in the theory of plasticity states that "Initial stresses or deformations have no effect on the plastic limit load provided that the geometry is essentially unaltered" (ASCE, *Plastic design in steel, a guide and commentary*, 2nd edition, American Society of Civil Engineers, Reston, VA, 1971). In the case of a fire-damaged structure, the initial stresses are the stresses due to the locked-in forces and the initial deformations are the distortions observed in a post-fire inspection. According to this principle, the resistance of the structure to gravity and/or lateral loads should be the same whether or not locked-in stresses and initial distortions are present; if the distortions are negligibly small and if instability or second-order effects are insignificant.

In establishing this principle, it has been stipulated that no significant instability or second-order effects exist in the structure. The only potential effect that these residual stresses or locked-in forces could have is on the stability of slender columns. Because the columns of this frame were designed to reduce the drift due to wind loading, they were relatively stocky. (Typical columns on the fire floors were W14x314, for example.) Therefore, there are no significant instability or second-order effects, the conditions of the basic principle are met, and the residual stresses and locked-in forces would not be expected to have any effect on these gravity or lateral load capacity of the steel frame.

Fire Comparison

The fire, the resultant damage, and the reinstatement work are compared to other steel-framed high-rise fires such as the Alexis Nihon hotel in Montreal and the Broadgate building in London in

Table 1. The intensity of the Broadgate fire was probably greater, since a great deal of construction debris and a trailer burned up near some columns. Also, there was no fireproofing on the columns at the time of the fire. Damage to the columns was quite severe and five columns had to be replaced. The Alexis Nihon fire was also very severe, owing to an inability to fight the fire with adequate water supply. The Alexis Nihon also experienced a second fire during the construction to repair the damage from the first fire.

The building discussed in this paper was totally dismantled nine years after the fire for various reasons, including the concerns about the steel frame. However, Table 1 shows that other steel-framed buildings that experienced fires of greater severity were reinstated relatively quickly. In fact, it is considered standard practice to replace only members which cannot be straightened and quickly reinstate steel-framed buildings after a fire. In view of this experience, it is difficult to rationalize total demolition of the building because of concern about the load-carrying capacity.

Measured Yield Strength

Samples were removed from the columns and beams on the fire floors to investigate any potential reduction in yield strength due to the fire. The yield strength was measured in typical quasi-static tensile tests. The results are compared to a large database of yield strength values from quasi-static tests of flanges of A36 rolled shapes performed at Lehigh University in the 1960s and 1970s. The yield strengths of a few of the columns that had been exposed to the fire were below the minimum specified yield strength (MSYS) of the A36 specification. However, the rate of occurrence of these lower strength values was no greater than would be expected in the population of rolled shapes unaffected by fire. Therefore, it can be concluded that the yield strength of the columns was not reduced significantly by the fire. A similar conclusion can be reached from an examination of the beams, even though these were obviously heated to greater temperatures than the columns and suffered a great deal of damage.

It is well known that quasi-static tensile tests produce yield-strength values a few ksi lower than the typical dynamic tests performed at the steel mill and reported on the mill certificate. In addition, the yield strength of the flange is typically a few ksi less than the yield strength of the web, where the mill test specimen has traditionally been taken. Both of these factors conspire to give typical static yield-strength values more than five ksi less than the mill tests and many values less than the minimum specified yield strength values used in design. This phenomenon is taken into account in the safety factors used in design.

Measured Residual Stress

A large number of coupons were cut out from the beams and columns of both the fire floors and the non-fire floors in order to measure the residual stresses. Attempts were made to remove the effect of the gravity load from the residual stress, however this is not considered very accurate. Nevertheless, it is interesting to compare the distribution of residual stress measurements to the expected distributions.

Comparing the residual stress in the beams from the fire floors to the residual stresses in beams from non-fire floors reveals only a minimal difference. In the fire floors, the residual stresses of 25 to 30 ksi were measured in compression and values in tension were as high as 25 ksi, whereas on the non-fire floors the residual stresses did not exceed 20 ksi. Similar results occur when comparing the residual stresses from the fire floors to data from a large database of residual stress data from tests performed at Lehigh during the 1960s and 1970s. The Lehigh data are similar, but they also do not show values greater than 20 ksi in compression. The Lehigh data also do not show very much tension at all, not even as much as is evident on the non-fire floors. It is concluded that the residual stresses in the beams are slightly greater than would be expected for beams that had not been affected by fire, but this is not that surprising considering the level of damage of the beams.

Similarly, a comparison of residual stress data from columns on the fire floors to non-fire floor column data again shows somewhat greater compression on the fire floors, and much greater values of tension. Comparing the data to the Lehigh database yields similar results; the columns seem to have some residual tensile stress that is not expected. However, it is important to note that residual tensile stress is not detrimental to the compression strength of columns. Similar comparisons were made of residual stresses from the center of the flanges and from the center of the webs. In all cases, the residual stresses were not that much different from the expected residual stresses for new rolled shapes.

Effects of Locked-In Forces

Frame analyses were conducted to demonstrate the effect of the fire-induced distortion and locked-in forces on the overall strength (load-carrying capacity) of the structure. It is assumed that the structural steel is ductile and has an elastic-plastic stress-strain relationship. As mentioned in the introduction, a basic principle of the theory of plasticity indicates that the resistance of the structure to gravity and/or lateral loads should be the same whether or not locked-in stresses and initial distortions are present; if the distortions are negligibly small and if instability or second-order effects are insignificant. This can be demonstrated by a simple example. These analyses show that, if the heavily damaged girders in the building are repaired or replaced, the structure should perform satisfactorily in resisting severe wind storms in the future.

Summary

More than 30 percent of the girders and beams of the floor systems on the fire floors in this building were buckled and required replacement. However, most of the steel members of the floor system were unaffected or could be straightened relatively easily. The columns remained in good condition.

Inelastic deformations and associated locked-in forces were induced in the frame by the fire. Many columns on the fire floors were outside of the erection tolerances. However, more than 8 percent of the columns on the floors well below the fire were also out of tolerance, indicating that a large percentage of the out-of-tolerance displacement on the fire floors was due to being erected out of tolerance and gravity load displacement.

The distribution of quasi-static yield-strength values measured on specimens taken from the flanges of fire-affected columns and beams was no different from the expected distribution in A36 steel members.

The distribution of residual stresses in the fire-affected members showed slightly more extreme values than residual stress distributions in typical rolled shapes, although the measured values from the building are confounded by the gravity load stresses.

Pushover analyses of simple frames with and without residual moments show that the stability and lateral load carrying capacity of the frames were unaffected by large residual moments, as would be expected from the principles of plastic analysis. Therefore, the existence of residual stress in this building is not detrimental to the gravity or lateral load-carrying capacity.

Other steel-framed high-rises that experienced similarly severe fires and damage were reinstated relatively quickly, whereas this building was totally demolished.

The fire-affected floor systems of this building could have been reinstated relatively quickly. After reinstatement, the safety and performance of the building would have been expected to be as good as it was originally.

The authors appreciate the assistance of Ming Xue on the frame analyses. The measurements of column positions, yield strength, and residual stress after the fire were performed by others.