The Effects of a Severe Fire on the Steel Frame of an Office Building

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ABSTRACT

This paper discusses the effects a severe fire had on a steel high-rise frame. The damage to this frame was no worse than in other fire-damaged steel-framed buildings such as the Alexis Nihon hotel in Montreal and the Broadgate in London. The yield strengths of the columns were not significantly decreased. There were locked-in residual moments and forces. However 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 residual moments, as would be expected from the principles of plastic analysis. The steel frame and floor system of this building could have been reinstated relatively quickly, as has been done in comparable fire-damaged steel-framed high-rises.

INTRODUCTION

A high-rise office building (Figure 1) suffered a fire that burned for more than 18 hours and involved nine stories near the top of the building, arguably one of the worst high-rise fires in U.S. history. Typically, the fire was burning in as many as three floors at a time as it progressively moved up the building. Based on observations, it is estimated that the most intense part of the fire at each floor occurred over a two-hour period.

The beams and columns had typical spray on insulation for fireproofing. Also, the building was in the middle of being retrofitted for sprinklers at the time of the fire. Unfortunately, the work had just begun at the top of the building and had only progressed down several floors. Firefighters had abandoned fighting this fire due to several fatalities. However, the fire was eventually brought under control when it reached the floors that had sprinklers, illustrating the effectiveness of sprinkler systems.

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. Many beams and secondary members of the floor system on some of the fire floors were buckled and required replacement. However, a majority of the steel members of the floor system were unaffected or could be straightened relatively easily. Among the various reinstatement plans that were considered, 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.

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. Despite the fact that the visual evidence would suggest that the columns were not heated above the annealing temperature of steel, there was concern that the yield strength of the steel members may have been reduced. However, as discussed later in this paper, the distribution of yield strength values was the same as would be expected for shapes unaffected by fire.

Figure 2 shows the results of a survey of the positions of the columns after the fire. Figure 2 shows that the building leans toward the Column-A side, a trend that begins well below the fire floors, indicating that possibly the building was erected out-of-tolerance or had displaced due to eccentric gravity load. However, it is also apparent that the floor

Fig. 1. High-rise steel framed building showing boarded up floors near the top where a fire occurred.

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systems of the fire-affected floors had contracted and were pulling the outer columns inward on these floors.

Many of the columns on the fire floors were well outside the erection tolerances of the AISC *Code of Standard Practice*. Typically, the columns were within an envelope equal to twice the AISC erection tolerances, with one exception at about 380 ft. The displacement at this location was 5.6 in., while the erection tolerance of the AISC *Code of Standard Practice* is 2.4 in. However, more than 8 percent of the columns on the floors well below the fire were also out of tolerance. For example, 65 percent of the columns on floor 13 were found to be out of tolerance, nine stories below the fire.

The interstory displacements on the fire floors were typically less than 1 in., although one column was displaced 1.5 in. in one story, which is much larger than the 0.28-in. (*L*/500) tolerance. (Each story height was 140 in.).

These 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 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.

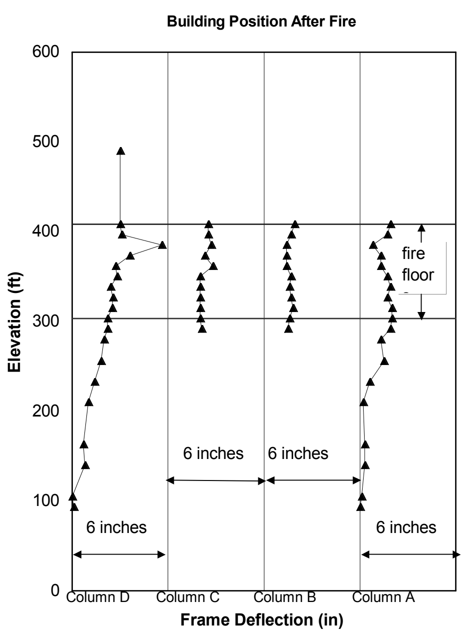

Fig. 2. Surveyed position of the four major columns in one frame.

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. Nevertheless, the existence of the locked-in forces raised concerns about the lateral-load capacity of the frame.

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.

Locked-in forces must be in self-equilibrium within the building, and therefore these locked-in forces will not affect the limit load for gravity- or lateral-load carrying capacity of the steel frame. 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, 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 W14×314, 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 the gravity or lateral load capacity of the steel frame.

COMPARISON TO OTHER MAJOR FIRES IN STEEL FRAMED BUILDINGS

The fire, the resultant damage, and the reinstatement work are compared to other steel-framed high-rise fires (Isner, 1988; SCI, 1991; Klem, 1989; Anon., 1982) such as the Alexis Nihon hotel in Montreal (Isner, 1988) and the

Table 1. Summary of Some Severe Fires in High-Rise Steel-Framed Buildings.

Fire	Number of Stories	Duration (Hours)	Number 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

Broadgate building in London (SCI, 1991) in Table 1. The intensity of the Broadgate fire was probably greater, since a great deal of construction debris and a trailer burned 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 (Kirby, Lapwood, and Thomason, 1986; Wildt, 1972; Tide, 1998; Smith et al., 1981). 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 AFTER THE FIRE

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 (Tebedge and Tall, 1973; Bjorhovde, 1972; Tall and Alpsten, 1969). Figure 3 shows the histograms of the distribution of yield strength values from the fire-affected columns compared to the Lehigh database.

In this figure, the bars representing the fire-affected columns are on the left while the data from Lehigh database (labeled "flange static") are on the right. The very low values of 24 ksi and 30 ksi are from the unaffected columns. The yield strengths of a few of the columns that had been exposed to the fire were below the specified minimum yield strength of the A36 specification. However, it can be seen that 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 fire did not reduce the yield strength of the columns significantly.

Figure 4 shows a similar comparison for the samples taken from fire-affected beams. The beams were obviously heated to greater temperatures than the columns and suffered a great deal of damage, but even these beams do not show a distribution of yield strength values that would indicate that the yield strength was reduced significantly by the fire.

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.

For example, Figure 5 compares the static test data from the flanges of many A36 column sections tested at Lehigh University (the same data shown in Figures 3 and 4) to the corresponding mill test data for these same shapes. It can be seen that the mean of the static test data is about 36 ksi whereas there are no mill test values less than 36 ksi, and the mean from the mill test data is about 43 ksi.

MEASURED RESIDUAL STRESS AFTER THE FIRE

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.

Figure 6 compares the residual stress in the beams from the fire floors to the residual stresses in beams from non-fire floors. It can be seen that the residual stress distributions are not that much different. 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.

Figure 7 compares 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 (Tebedge and Tall, 1973; Bjorhovde, 1972; Tall and Alp-

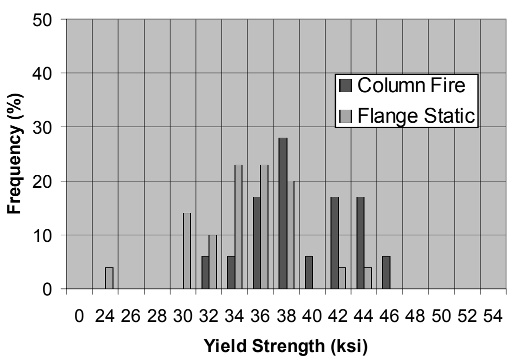

Fig. 3. Comparison of yield strength of flanges of column sections after the fire (bars on the left) to distribution of yield strength from flanges of a large number of A36 structural steel sections unaffected by fire (bars on the right).

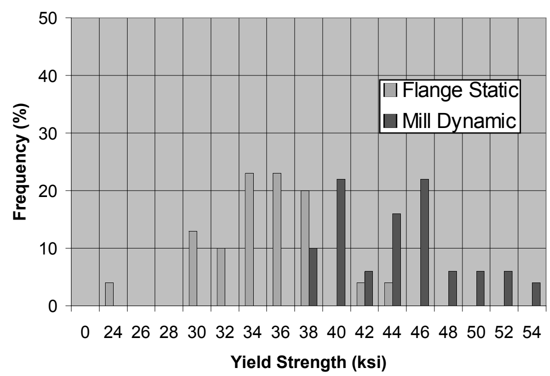

Fig. 5. Comparison of distribution of quasi-static yield strength values from flanges of a large number of A36 structural steel sections unaffected by fire (bars on the left) to the corresponding mill test data (bars on the right).

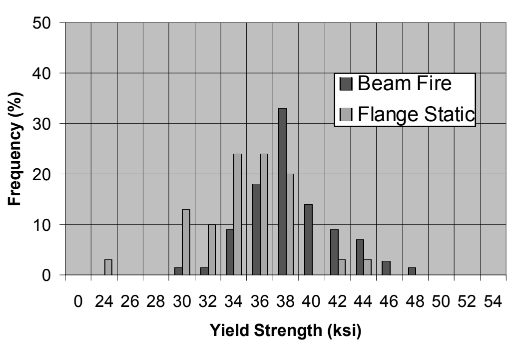

Fig. 4. Comparison of yield strength of flanges of beam sections after the fire (bars on the left) to distribution of yield strength from flanges of a large number of A36 structural steel sections unaffected by fire (bars on the right).

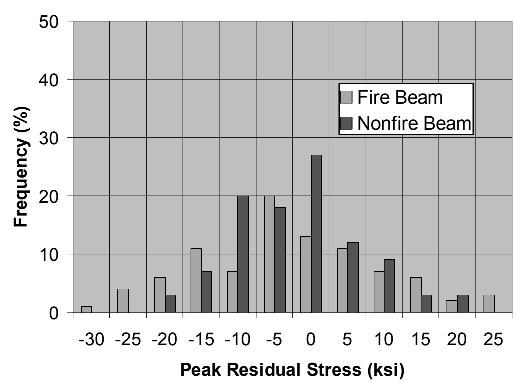

Fig. 6. Comparison of residual stresses measured at the tips of beam flanges on the fire floors (bars on the left) compared to data from the non-fire floors (bars on the right).

sten, 1969). It is expected that the residual stresses in the members in the building would be similar. 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, residual stress data from columns on the fire floors is compared to non-fire floor column data in Figure 8. Again, the columns on the fire floors show somewhat greater compression, and much greater values of tension. The data from the fire floor columns are compared to the Lehigh database of flange tip data in Figure 9. The results of this comparison are similar; 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.

FRAME ANALYSES TO ILLUSTRATE THE 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. This principle holds true if the distortions are negligibly small and if instability or second-order effects are insignificant.

This principle can be demonstrated by a simple example. Consider the fixed-base portal frame shown in Figure 10, which has a span length of 25 ft and a height of 12 ft. The girder is W27×94, and the columns are W14×176, all made of A36 steel. The effect of expansion or contraction of the girder due to fire is simulated by inserting, at its mid-span, a special thermal element. The amount of thermal expansion (or contraction) can be adjusted to produce a desired "locked-in" bending moment distribution in the members as well as some rotation at a plastic hinge. In this example, the plastic hinges develop at the base of the columns.

Figure 11 shows the bending moment distribution in the frame (plotted on the tension side of the members) when the amount of expansion introduced in the element is just sufficient to cause plastic hinges to form at these locations

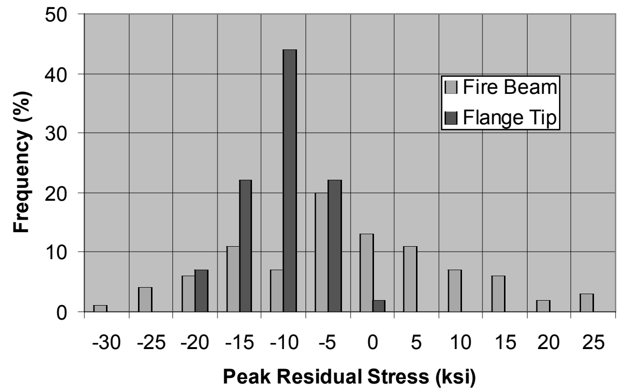

Fig. 7. Comparison of residual stresses measured at the tips of beam flanges on the fire floors (bars on the left) compared to data from the Lehigh database of residual stress measurements (bars on the right, labeled "flange tip").

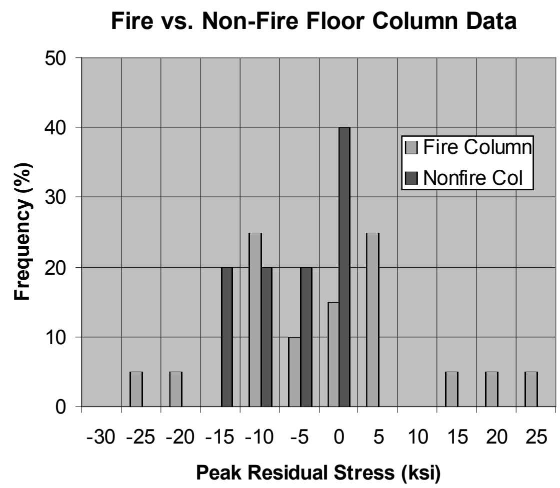

Fig. 8. Comparison of residual stresses measured at the tips of column flanges on the fire floors (bars on the left) compared to data from the non-fire floors (bars on the right).

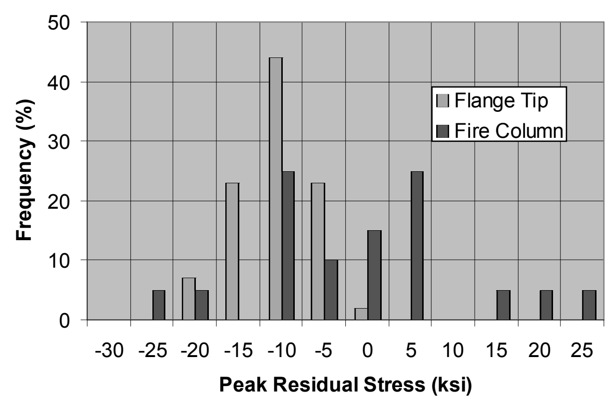

Fig. 9. Comparison of residual stresses measured at the tips of column flanges on the fire floors (bars on the right) compared to data from the Lehigh database of residual stress measurements (bars on the left, labeled "flange tip").

(bending moment equal to 11,520 kip-in). For any additional expansion, the columns behave as if they are hinged at their bases, as the resisting moment is unchanged. The bending moment diagram in the structure when the rotations of the lower ends of the columns reach 0.0029 radians is shown in Figure 12.

At this stage, a lateral load T is applied to the frame at the girder level. During the application of T, the bending moment at D remains at the plastic moment value. Figure 13 shows the bending moment diagram in the frame when a plastic hinge forms at C, the leeward end of the girder, at $T = 128$ kips. The bending moment at that location is 10,008 kip-in.

When T reaches 278 kips, a third plastic hinge re-forms at the base of the left column (point A). Figure 14 shows the bending moment distribution at this load. The structure attains its plastic limit load at $T = 299$ kips when a fourth plastic hinge forms at the windward end of the girder (point B). The limit-load moment diagram is given in Figure 15. The relationship between the lateral load T and the horizontal displacement at the column top is shown in Figure 16 [marked as Case (a)].

To illustrate that the plastic limit load of 299 kips is unaffected by the presence or absence of the "locked-in" stresses, two additional analyses have been performed for the frame [Cases (b) and (c)]. In Case (b), a thermal contraction equal to the expansion assumed previously is introduced. The load-displacement relationship of the frame, also plotted in Figure 16, shows that the same plastic limit load is achieved. Case (c) assumes that there are no "locked-in" stresses anywhere in the structure, and the analysis again gives the same plastic limit load. All the calculations are based on the original geometry and dimensions of the frame.

The numerical study is next extended to one of the planar frames of the damaged building. To avoid excessive calculations, instead of analyzing a complete frame, a portion of the frame, where severe damage to beams and girders occurred, is selected for performing detailed analysis. It is a three-story sub-frame formed by placing points of contraflexure at the mid-heights of the columns between floor levels 25 and 26 and between 28 and 29. The dimensions and member sizes of the sub-frame and the working values of the gravity and lateral loads are given in Figure 17; they are obtained from the original design drawings. In the analysis, the gravity loads are maintained constant and lateral loads are increased or decreased proportionally with a factor α. The total working lateral load at the base is 86 kips (when $\alpha = 1.0$).

In multi-story structures, two instability effects are generally present: the member instability (or P-δ) effect and the frame instability (or P-Δ) effect. These effects may significantly alter the invariant nature of the plastic limit load. They should, therefore, be taken into account in the analysis of the sub-frame.

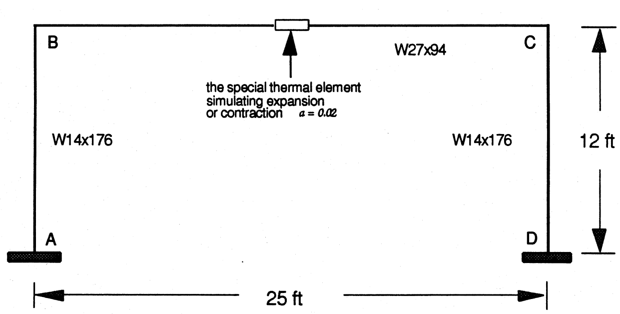

Fig. 10. Model for analyzing a steel portal frame with local expansion or contraction at center of girder.

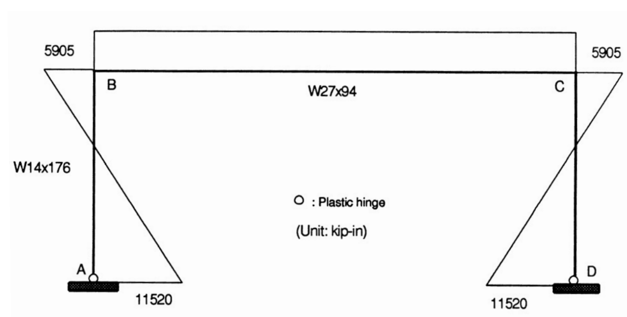

Fig. 12. Bending moment diagram when rotation at plastic hinges is 0.0029 radians.

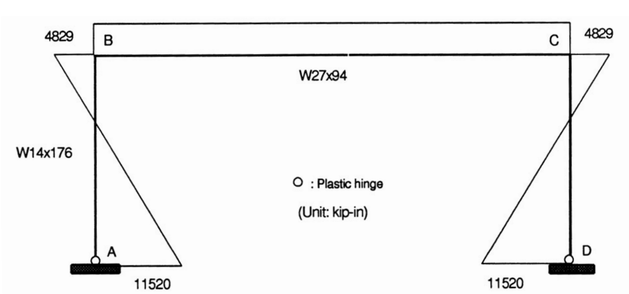

Fig. 11. Bending moment diagram when plastic hinges form at column bases due to expansion of girder.

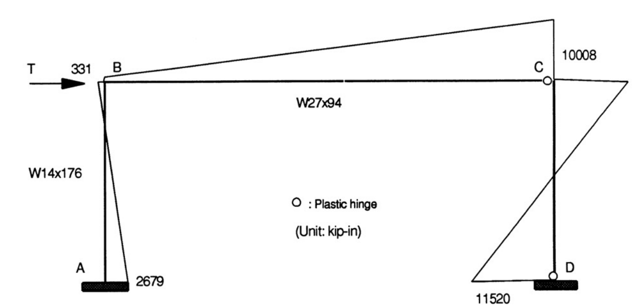

Fig. 13. Bending moment diagram when lateral load T = *128 kips.*

To study the influence of locked-in forces on lateral load-carrying capacity, the structure is analyzed once without locked-in forces and again with locked-in forces. An inelastic finite element program was used to perform the analysis. The locked-in forces (bending moment, axial and shear forces) are again introduced by using a thermal element placed at the mid-span of each girder. The introduction of the locked-in forces occurs when the structure is loaded only by gravity load. The amount of thermal contractions in the girders is adjusted so that the initial distortions of the columns are in approximate agreement with those found in the post-fire inspection (see Figure 2). The initial distortions and the locked-in bending moment distribution assumed in the analysis are shown, respectively, in Figures 18 and 19.

Figure 20 shows a comparison of the lateral load vs. lateral drift relationships obtained from the two analyses. There is a 2.4 percent difference in the maximum lateral load. It is interesting to note that the case with the locked-in forces achieves a slightly higher maximum load. This is most probably due to the different roles played by the instability effects (in one case the columns are essentially straight and in the other case the columns are initially bent). The maximum lateral loads that can be resisted by the structure are more than 9 times the working value. The overall lateral stiffnesses at the working load level of the two cases are very similar. These observations imply 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.

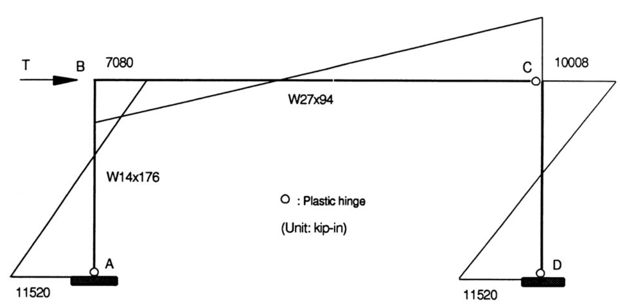

Fig. 14. Bending moment diagram when lateral load T = *278 kips.*

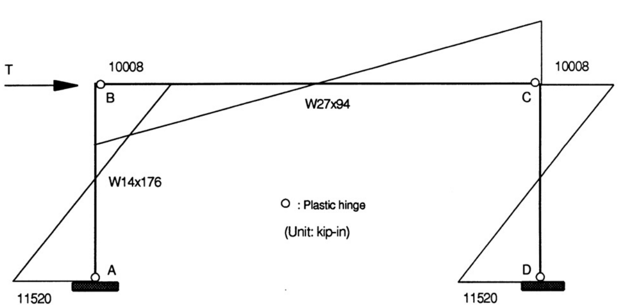

Fig. 15. Bending moment diagram when ultimate load T = *299 kips is reached.*

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.

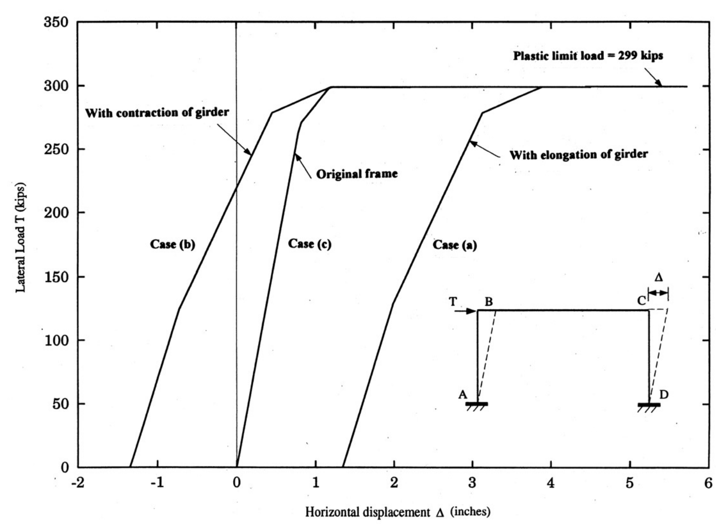

Fig. 16. Lateral load vs. displacement relationships of portal frame.

Other steel-framed high-rises that have experienced similarly severe fires and damage have been 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.

ACKNOWLEDGMENTS

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.

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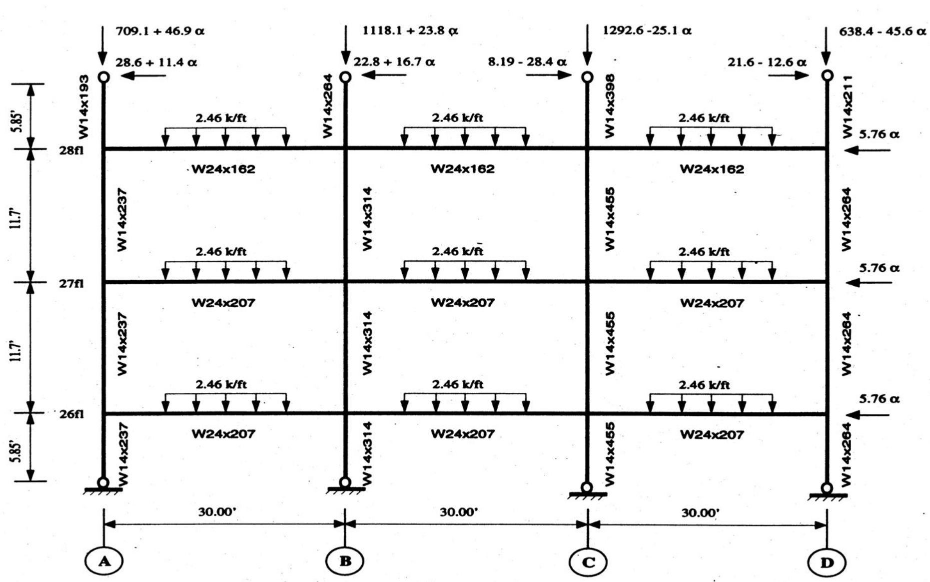

Fig. 17. Load arrangement for analysis without locked-in forces.

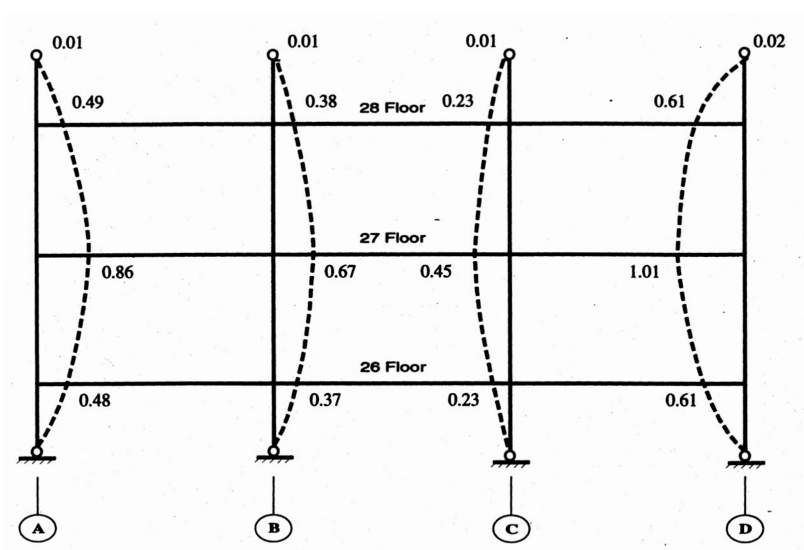

Fig. 18. Column distortions due to locked-in forces.

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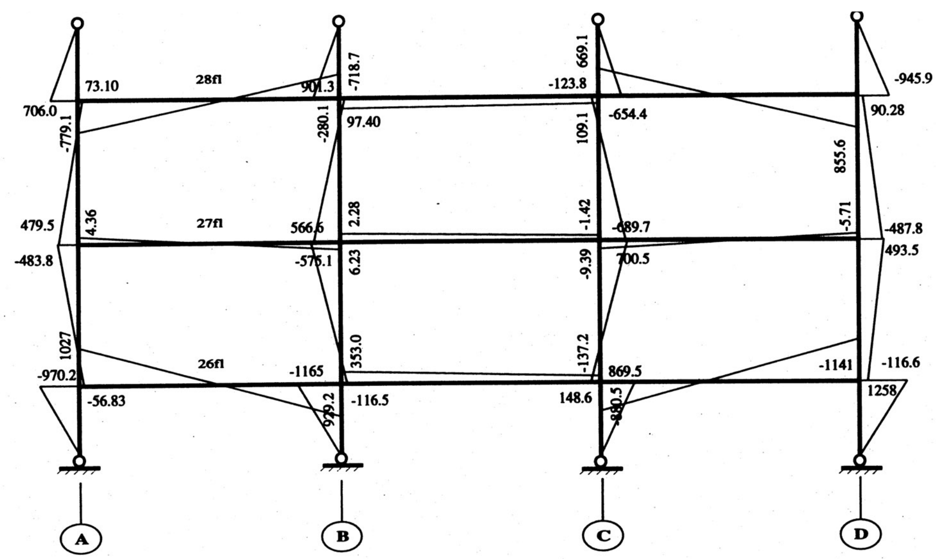

Fig. 19. Locked-in bending moment distribution.

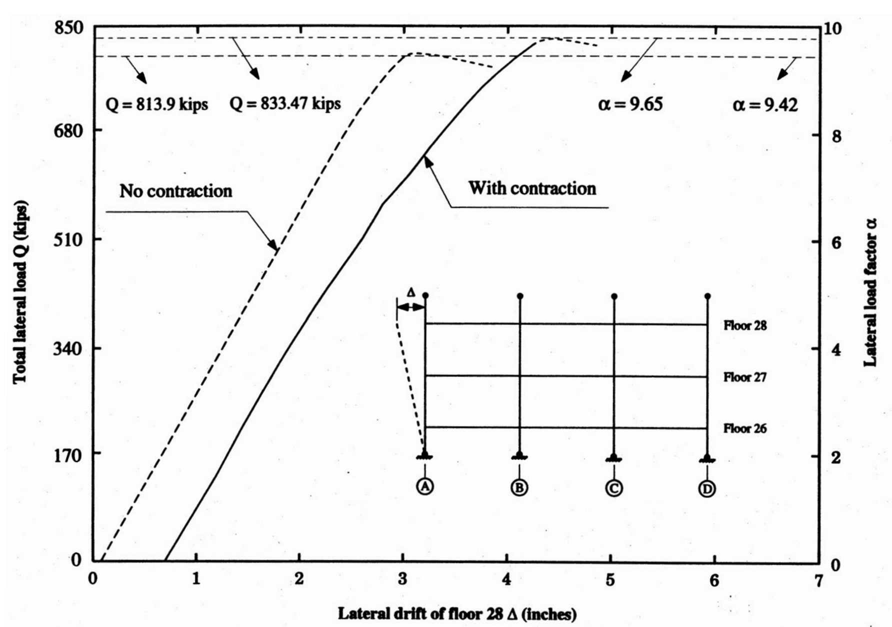

Fig. 20. Lateral load vs. drift relationships of sub-frame.