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FIRE RESISTANCE OF STEEL FRAMES

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Abstract

This paper presents an analysis on the fire resistance of the World Trade Center's steel frames and examines precautions to be taken in designing skyscrapers.

In the analysis, it was assumed that the temperature increase of steel members for the columns and the floor trusses of the World Trade Center, were subjected to hydrocarbon fire and standard fire, and for the cases of adequately fire-protected and non fire-protected. The results of the analyses were verified by thermal deformation analysis. The findings revealed that, compared with the columns (perimeter and core columns) of heavy sections, the floor trusses composed of light-gauge steel members, even if fire-protected or not, experienced a temperature increase at a more rapid pace. The rapid increase in steel temperature posed thermal deformation, thereby causing the steel members to buckle and leading eventually to the failure of the floor trusses at a relatively lower temperature, and large restrains on the end connections.

The findings concluded with some recommendations for care in using light-gauge steel members, usage of hinge connection and effects of elevated temperatures and fire-engineering aspects in skyscrapers.

Keyword : Steel, Safety, Structure, Fire-Safe Design, Failure

1. Introduction

The collapse of the World Trade Center Towers (WTC), ("WTC", Photo 1 : at the time of construction") was caused by damage to the structures due to aircraft impact and ensuing fire which led to the loss of the structures load-bearing capacity.

The WTC, like other high-rise buildings, was required to have three-hour fire resistance for its columns and two-hour resistance for its floor system members. The steel frames were protected from fire with insulating materials (hereafter referred to as "fire protection") to meet the fire protection.

The length of time between the aircraft impact and total collapse were fifty-six minutes for WTC1 and one hour forty-three minutes for WTC2.

There is little point in discussing details of the collapse time since damage to the structures due to both aircraft impact and the burning of the aviation fuel were extraordinary events-radical departures from the conditions expected by fire standard and regulations. However, clarifying the cause for the collapse can be considered extremely useful for designing high-rise buildings in the future



Photo 1 WTC under construction

From this point of view, therefore the paper examines the fire resistance of WTC's steel frames by referring to the report ^[1] made public by the Federal Emergency Management Agency (hereafter referred to as "FEMA Report").

2. Fire Exposure

The Boeing 767-200ER aircraft that impacted the WTC carried about 10,000 gallons (about 38,000 liters) of jet fuel. The FEMA report assumes: out of the total jet fuel carried, 1,000 to 3,000 gallons were exhausted as fire balls, 3,000 gallons flowed onto other floors and the remaining 4,000 gallons were burnt within the impact floors.

The calorific value of the jet fuel was 1 to 1.5 GW. If one-third or a half of that energy was released to the structure, gas temperatures reached 900 to 1,100°C around the ceilings and 400 to 800°C at the rest of indoor spaces according to the analysis of the FEMA Report. The jet fuel remaining on the floors burned within about five minutes, with the blaze engulfing combustible materials (4 to 12 psf: 20 to 60 kg/m²) in the floor areas.

The fire exposure assumed to occur in typical buildings is called the “standard fire” and the resulting gas temperature is specified in ISO835 [2] (similar to JIS A1304 [3]). This is an equivalent of the U.S. specifications ASTM E119 [4].

For buildings other than general buildings, the gas temperature for a petrol spill fire is also specified in the United States ASTM E1529 [5], also referred to as “hydrocarbon pool fire”. Fig. 1 shows the comparison of gas temperature between standard fire and hydrocarbon pool fire. In the case of a hydrocarbon pool fire, the temperature at the early stage rises rapidly to a high of about 1,100°C in the first five minutes than for the case of a standard fire and, afterwards, the temperature remains constant according to the U.S. specifications.

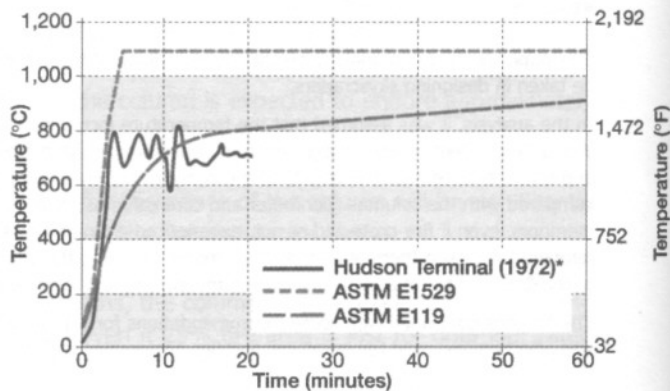


Fig. 1 Comparison between Standard Fire and Hydrocarbon Pool Fire

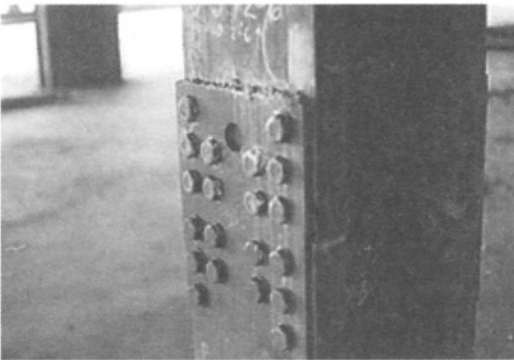


Photo 2 Core column (ultra-jumbo H-shape)

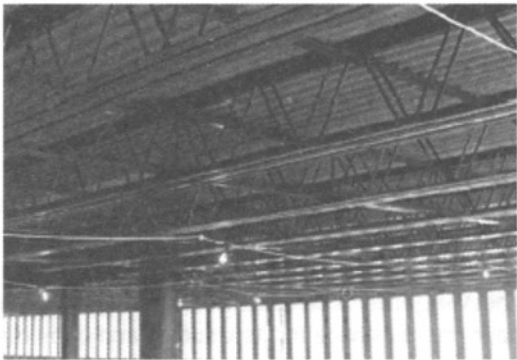


Photo 3 Floor truss (before protected)

3. Fire Protection and Steel-Frame Temperature

Fire protection for WTC1 was originally asbestos-containing spraying up to the 39th floor, but later it was changed to vermiculite plaster insulation. WTC2 was fire-protected wholly with vermiculite plaster insulation.

The fire-protection thickness for the floor trusses was originally 3/4 in. but, as a retrofit during tenant changes, was later redoubled to 1-1/2 in. For the columns and beams, the thickness was not specified because of variations in member sizes, but the thicknesses adequate to meet the specified two and three-hour protection requirements were adopted.

Photo 2 shows the core column of ultra-jumbo H-shape before fire protection and Photos 3 and 4 show the floor trusses before and after fire protection. The motion damper inserted between the outer wall and the end of the lower chord of the truss was not fire-protected, as it was not a load-carrying member of the main structure. It can be assumed from the photo that spraying, applied to such a small cross-section members posed great difficulties.

According to the FEMA Report, the aircraft incursion supposedly blasted away the vast majority of fire protection on the impact floors. Photo 5 shows the steel frames of adjoining Bankers Trust Building that was hit directly by the columns falling from WTC2. The fire protection was largely blown away, and it is not hard to imagine that the same phenomenon occurred at the impact floors of WTC1 and WTC2.



Photo 4 Fire protection of floor truss



Photo 5 Fire protection at Bunkers Trust Building

The temperature of heated steel frames varies, depending on heating conditions (standard fire or hydrocarbon pool fire and heating duration), existence of fire protection, insulative properties of protective materials and thermal capacity of steel members.

In this study, an analysis similar to that of the FEMA Report was made on the columns plus the diagonal members (rods) of the floor trusses. The analysis results are shown in Figs. 2 and 3. The sizes of the members adopted are as follows:

Core column	H-455.2 x 418.5 x 42.04 x 67.56	$H_p / A = 35.7$
Perimeter column	□-355.6 x 355.6 x 12.7 x 12.7	$H_p / A = 87.5$
Floor truss rod	□-1.09" (27.7mm)	$H_p / A = 144.6$

The sizes of the columns (core and perimeter) are only those assumed because the FEMA Report gives no specified dimensions for them at and around the impact floors. H_p/A is the indicator of member thermal capacity i.e. the rate of the member temperature increase, with H_p = circumferential length and A = sectional area. The larger the H_p/A , the higher the rate of steel-frame temperature increase.

Figs. 2 and 3 clearly reveal the following:

- (i) The increase of the steel-frame temperature corresponds to thermal capacity (H_p/A), regardless of fire protection. In the case of the floor truss rod and the perimeter column with smaller thermal capacity (H_p/A is larger), the temperature rise more rapidly, reaching as high as 900°C in five minutes when the members are not fire-protected.
- (ii) By comparison, in the case of the core column that has extremely large thermal capacity, the speed at which the temperature rises is moderate even when it is not fire-protected (or when the fire protection is blown off). The temperature increases to about 600°C in approximately 20 minutes.
- (iii) In the case of a hydrocarbon pool fire, the temperature of steel frames (fire-protected or unprotected) increases faster than in the case of standard fire, especially in the first five minutes. A hydrocarbon pool fire causes the gas temperature to rise rapidly and that is why, when steel frames are not fire-protected, the difference between a hydrocarbon pool fire and a standard fire is more pronounced in terms of the rate of the steel-frame temperature increase.

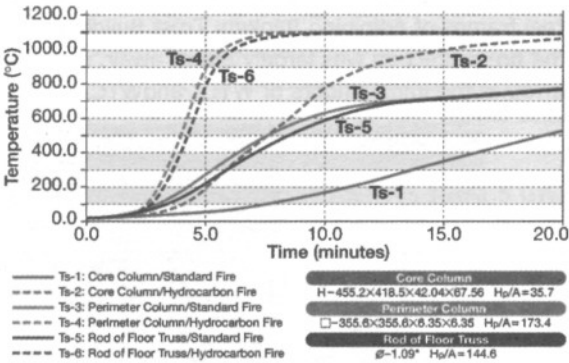


Fig. 2 Steel Temperature Rise Subjected to Heating (No fire protection)

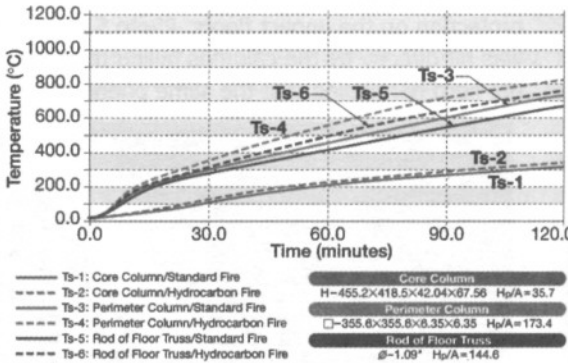


Fig. 3 Steel Temperature Rise Subjected to Heating (3/4-inch thick sprayed fire protection)

4. Fire Resistance of Steel Frames

Steel strength is reduced at high temperatures. The strength and stress-strain relationship of A36 steel at high-temperature are shown in Figs. 4 and 5, respectively (FEMA Report). A36 steel's yield point lowers by nearly half at 550°C, as shown in Fig. 4 (A36 steel [6]).

The fire resistance of columns and beams can be verified by a fire test. There are two methods for fire test: one is to obtain the time of failure (fire resistance) for the case of load and subjected to heating and the other is to obtain the period of fire resistance from the temperature of steel materials only through heating (the heat test). The condition of failure immediately after a loaded heat test made on a column is shown in Photo 5. The proof stress of steel frames decreases with the increase of temperature, leading to the loss of the frame's load-bearing capacity.

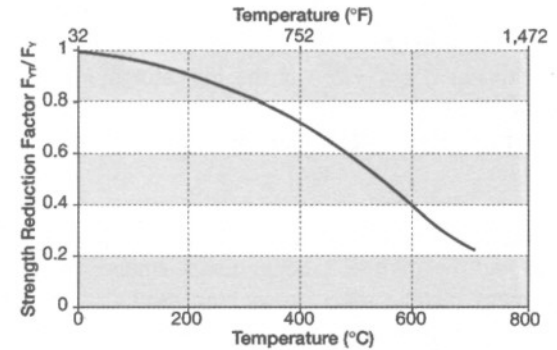


Fig. 4 Strength of A36 Steel at High Temperature

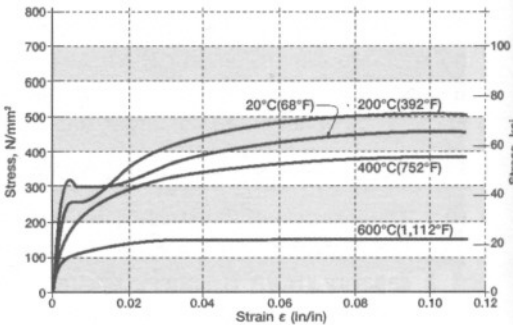


Fig. 5 Stress-Strain Curves of A36 Steel at High Temperatures

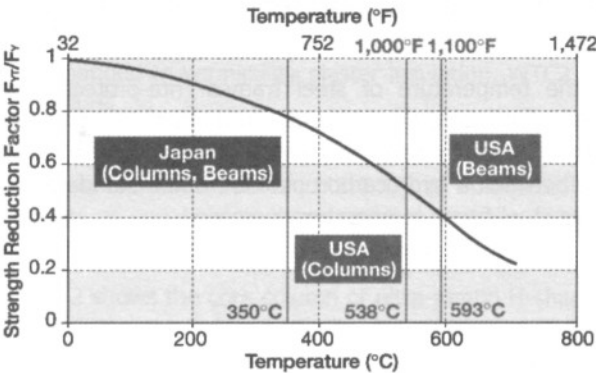


Fig. 6 Relationship between Strength of Steel Materials and Critical Temperatures

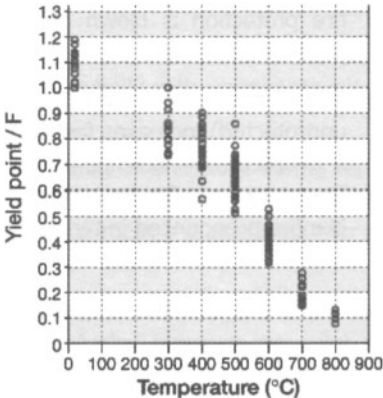


Fig. 8 Variation of Steel Strength at High Temperature.

For the ASTM E-119 fire test in the United States [4], the heat test is prevalent for columns, and the critical temperatures for steel members are specified at:
Column: 538°C (Average 1,000°F; Maximum 1,100°F)
Beam: 593°C (Average 1,100°F; Maximum 1,200°F)
As for fire testing in Japan [3], the heat test traditionally has been applied with the critical temperatures specified as follows:

Column: Average 350°C; Maximum 450°C
Beam: Average 350°C; Maximum 450°C
However, today, the applied load-heat is also in practice. The load applied is specified as a load equivalent to the long-term critical stress (2/3 of the yield point).

Fig. 6 shows the critical temperatures of steel between Japan and USA. The differences in temperature are about twice in the critical stage, eventually leading to a wide difference in the thickness of fire protection required.

This study derived analytically the fire resistance of the columns (core and perimeter) of the WTC at and around the impact floors. The sizes of the columns adopted were identical to those used for the analysis of steel-frame temperatures in Section 3. The steel grade was A36 and the ratio of axial force was set at 0.5, although no reference is made in the FEMA report regarding the ratio of axial force in the column. However, since the stress applied on the diagonal members of the floor trusses was estimated, through calculation at about 50% of the yield point, hence the ratio was assumed to be the same.

Fig. 7 shows the results of the analysis. The columns underwent thermal expansion as the temperature increased (vertical elongation: V), buckled due to the lowering of steel strength and the loss of their load-bearing capacity (horizontal deflection at the middle of the columns: H). The calculated buckling temperatures were 515°C for the core column and 520°C for a perimeter column.

A comparison between the above calculated results and critical temperature shown in Fig. 6 reveals:

- (i) The buckling temperature of the columns virtually corresponds to the steel strength at high temperature ratio of axial force in the column.
- (ii) The U.S. critical temperature, 538°C, is considered almost equivalent to the ratio of axial force of 0.5.

The failure temperature of beams is related to the presence of the floor. The failure temperature of beam is higher than that of the columns, because the temperature of the steel frames in contact with the floor increases at a slower rate and also because the floor itself shares the applied load. The critical temperature of 593°C for beams, as specified in the United States, is considered to put a restriction on the load applied.

Attention should also be paid to the fact that strength of steel materials at high temperature shown in Figs. 4 and 6 are average value Fig. 8 shows the results of examining [7] the strength of steel materials at high temperature undertaken by the Japan Iron and Steel Federation (JISF). As evident, the strength varies widely. In addition, there are cases where columns and beams are restrained by peripheral frames due to thermal expansion, resulting in failure at lower temperatures than expected, as discussed later. It is considered that, with such indefinite factors, the critical temperature of 350°C as adopted in Japan secures a relatively higher level of safety.

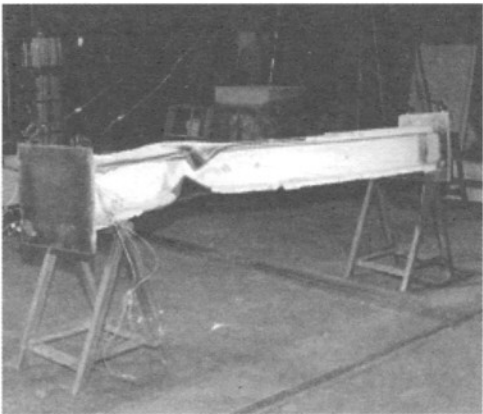


Photo 5 Buckling of Column due to applied load and subjected to heating (applied load-heat test)

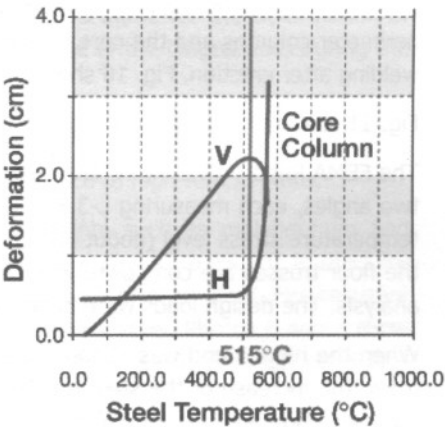


Fig. 7 Analysis of Fire Resistance of Column

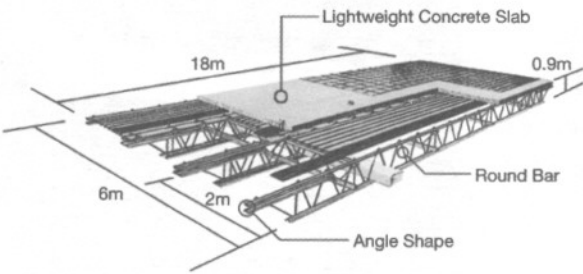


Fig. 9 Floor Structure

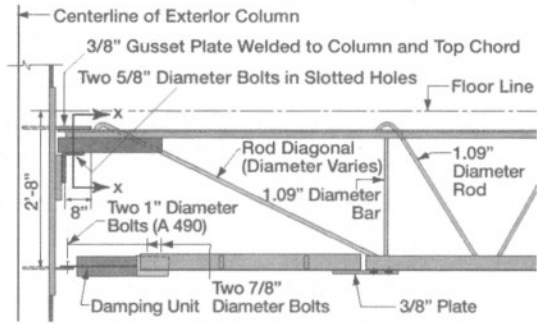


Fig. 10 Details of Floor Truss End

Fig. 9 shows the schematic diagram of the WTC floor structure. The floor was designed to support lightweight concrete (10 cm thick) on steel decking by trusses arranged at a spacing of 6 ft. 8 in. (about 2 m). The maximum length is 60 ft. (18.3 m). The trusses are lightweight, using angles for the upper and lower chords and round bars for the diagonal members. Their ends are linked to the perimeter columns and the core girders by two high-strength bolts (A325 bolt, 5/8 in. dia.) followed by welding after erection. Fig. 10 shows the details of the truss end (FEMA Report).

Fig. 11 analyzes the failure process of the floor truss by raising the steel-frame temperature.

The FEMA Report gives no reference to the dimensions of the upper and lower chords. For this analysis, two angles, each measuring L-3.5 x 3/8 in. (88.9 x 9.525 mm), were overlapped to match the normal-temperature stress level (about 50% of yield point) of the truss diagonal member (Ø1.09 in.). Although the floor trusses are composite with the concrete slabs, the concrete slabs were not considered in the analysis. The design loads were dead load and service load.

When the hinged end was adopted for one end of the truss and the roller end for the other end of the truss, the increase of the steel-frame temperature caused the truss deformation to concentrate on the roller end side and the vertical members of the utmost end to buckle at 340°C. Deflection at the time of buckling was 7.50 cm in the middle of the truss, and horizontal deflection on the roll end side was 6.61 cm.

When each end of the truss was hinged, constraints on the thermal expansion of the truss resulted in the large deflection of the truss. Constraints concentrated on the upper chord, and the resultant buckling of the chord caused the failure of the truss at 264°C. The central deflection at the time of the failure was 27.9 cm. Fig. 12 shows the truss end's horizontal restraint load. The restraint load was pulled inward at normal temperature. Due to the thermal expansion of the truss, it then turned around but was eased by deformation of the upper chord. Maximum restraint load occurring outwards was about 30 tons. According to the FEMA Report, the shear strength of the high-strength bolts at each end (A325 bolt, 5/8 in. in inside dia. x 2) is as follows (the strength decreases by half at 550°C):

- Room temperature: Ru = 232 kips (105.2 tons)
- 550°C (1,022F): Ru = 116 kips (52.6 tons)

Since the trusses were connected to the perimeter columns and the core girders, it is assumed that approximately the central values of those given in the above two cases must have been applied for actual behavior. In addition to the floor slabs serving as a retard out to the thermal expansion of the trusses and constraint to the buckling of the upper chords, it can also be assumed that the actual failure temperatures to be higher than those in the above analyses.

When such a large-span truss is heated and undergoes thermal expansion, it deforms (deflects) due to constraints on its ends and its individual structural members are subjected to large compression stresses. Additionally, a large shearing force acts on the end connections. From these, it is likely that the failure temperature of the truss is far lower than the critical temperature for beams (593°C: 1,100F) in the fire test.

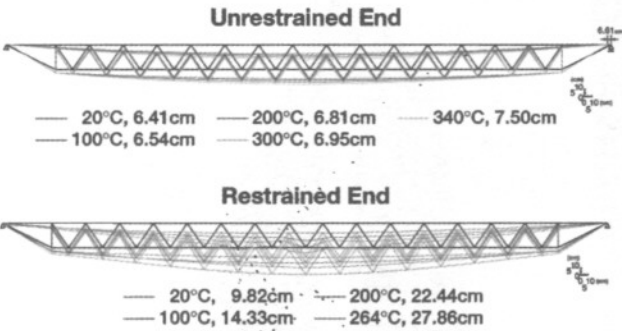


Fig. 11 Analysis of Thermal Deformation of Floor Truss

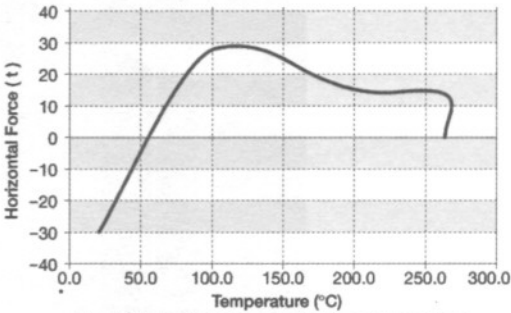


Fig. 12 Restraint Load at Floor Truss End

5. Recommendations and Summary

From the above analytical study and examination of the fire resistance of the WTC steel frames, the following precautions in the fire-safe design of high-rise buildings are proposed:

- (i) When lightweight steel frames are used, a thorough check should be made for the deformation of structural members and the failure of end supports (especially in the case of the hinged end) during the course of fire.
- (ii) Adequate fire-protection materials and construction methods should be selected.

Among other effective alternatives is the utilization of steel with resistance to high temperature.

Fire-resistant steel (Sakumoto et. al, 1992-1), containing alloy elements such as molybdenum and chromium, is superior in strength at high temperature components to ordinary steels. It is used for many buildings in Japan and Fig. 13 shows the high-temperature strength of each steel. Fire-resistant steel shows the excellent high-temperature strength, compared to ordinary steel. Photos 6 and 7 show the P&G building and Integral Tower Obayashi building (Sakumoto, et. al, 1992-2) which used fire-resistant steel for high-rise buildings.

It is important to verify the fire resistance of steel frames through fire-safety design.

Fire tests on structural members such as columns and beams, as stated in section 4, do not reflect the behavior of steel frames as a whole during a fire. Steel frames expand thermally due to heating by fire and the structural members are exposed to large additional stress by constraints. Especially lightweight structural members not only undergo rapid temperature increases within a short time, but also lose their load-bearing capacity even at a relatively low temperature due to buckling and alike. Besides the load is concentrated in connections, so hinged end and similar connections can fracture during the course of heating or cooling.

The number of similar failure analyses has been increase in Japan since around 1990, where fire-safety designs are used to verify the fire resistance of steel frames. The analyses are targeted for buildings adopting fire-resistant steel and concrete-filled steel tube columns. Hence, it is imperative that fire-safety designs are adopted particularly for high-rise buildings and public buildings. The adoption of fire-safety design for high-rise steel buildings ensures greater safety in fire. Fire-resistant steel, used successfully in Japan for a decade, is now an option to be considered for future construction.

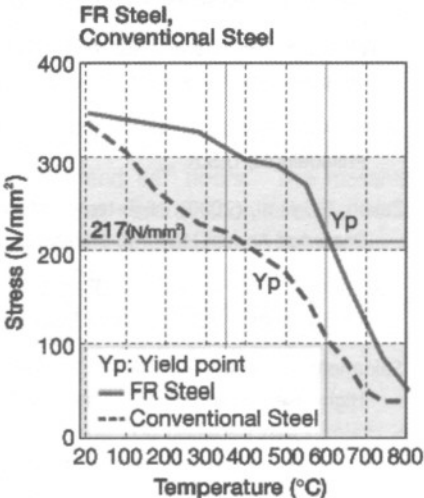


Fig. 13 Comparison of Strengths of Steel Products at High Temperature

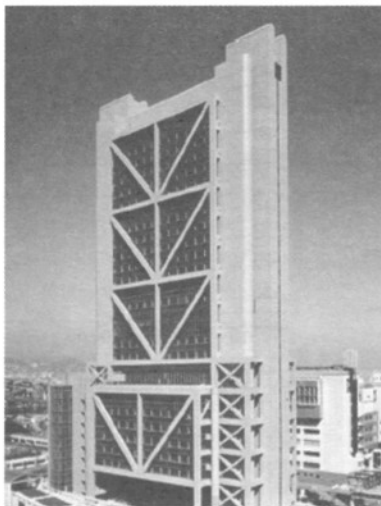


Photo 6 P&G buildings
(Fire Resistant Steel)



Photo 7 Integral Tower Obayashi Building
(Fire Resistant Steel)

Acknowledgements

The photos of the WTC at the time of construction, shown in this paper, were provided by Mikio Kasajima, former president of Daiken Sekkei Co. His cooperation is highly appreciated.

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