FIRE RESISTANCE OF STEEL FRAMES

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ABSTRACT

This paper verifies analytically the fire resistance of the World Trade Center's steel frames and also examines precautions to be taken in designing skyscrapers.

In the analysis, the temperature increase of steel members for the columns (perimeter and core columns) and the floor trusses of the World Trade Center, when subjected to heating by hydrocarbon fire and standard fire, was calculated in case the steel members were adequately fire-protected and in case they were not fire-protected (or in case the fire protection was blown off). The fire resistance of these steel members was also verified by thermal deformation analysis.

The analysis revealed that, compared with the columns (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, that the increase in steel temperature posed restraint on thermal deformation, thereby causing the steel members to buckle and leading eventually to the failure of the floor trusses at a relatively low temperature, and that a larger restraint acted on the end connections.

Based on the findings, this paper advances the following three suggestions:

(i) Elaborate care should be exercised in using light-gauge steel members for skyscrapers.

(ii) When hinged connections are to be used, the existence of any fractures at joints should be checked for especially at elevated temperatures and after cooling.

(iii) For designing skyscrapers, fire engineering is essential in addition to traditional structural calculations.

Further, the use of steels with superior heat resistance is recommended to improve fire resistance of steel frames. This does not mean that the fire protection can be deleted, but that given the same or similar protection, the length of time that the steel can tolerate fire is extended. Or, if the fire protection system is destroyed, the unprotected steel will have a longer survival time.

This paper will describe the characteristic of the steel and fire resistance of columns made by the steel compared to the conventional steel. In addition, some of the important major structures in which it has bee used in Japan, will be presented.

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

1. Foreword

The collapse of the World Trade Center Towers (hereafter referred to as "WTC," Photo 1 : at the time of construction") was caused by damage to the structures due to aircraft impact and ensuing fire exposure that led to 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. Its steel frames were protected from fire with insulating materials (hereafter referred to as "fire protection") to meet the requirements.

The lengths of time between the aircraft impact and total collapse were fifty-six minutes for WTC1 and one hour forty-three minutes for WTC2. Indeed, there is little point in discussing the precise 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 laws and regulations. But, to clarify 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, this 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: 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/m^2) 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] (the same as JIS A1304 [3]). The same may be said, within very close limits, of the U.S. specifications (ASTM E119 [4]).

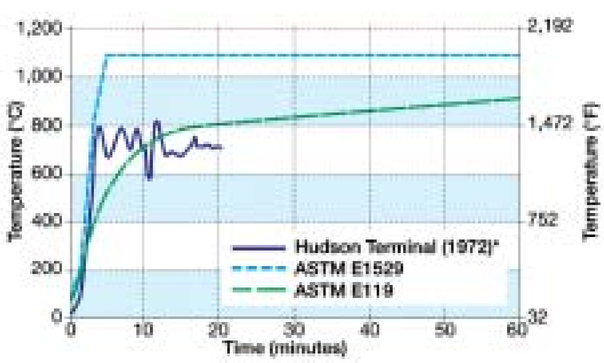

FIG. 1. Comparison between Standard Fire and Hydrocarbon Pool Fire

For buildings other than typical buildings, the gas temperature for a petrol spill fire is also specified in the United States (ASTM E1529 [5], hereafter 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 first rises more 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.

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 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. Their 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.

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	Hp / A = 35.7
Perimeter column	□-355.6 x 355.6 x 6.35 x 6.35	Hp / A = 173.4
Floor truss rod	ϕ-1.09” (27.7mm)	Hp / 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. Hp/A is the indicator of a member’s thermal capacity (the rate of a member’s temperature increase), with Hp = circumferential length and A = sectional area. The larger the Hp/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 (Hp/A), regardless of fire protection. In the case of the floor truss rod and the perimeter column with smaller thermal capacity (Hp/A is larger), the temperatures rise more rapidly, reaching as high as 900°C in five minutes especially when the y 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 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.

Photo 2. Core column (ultra-jambo H-shape)

Photo 3. Floor truss (before protected)

Photo 4. Fire Protection of floor truss

Photo 5. Fire protection at Bunker Trust Building

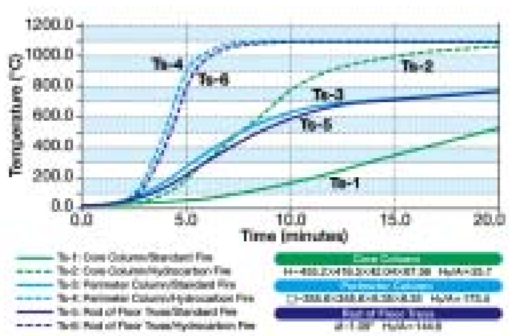

FIG. 2. Steel Temperature Rise Subjected to Heating (No fire protection)

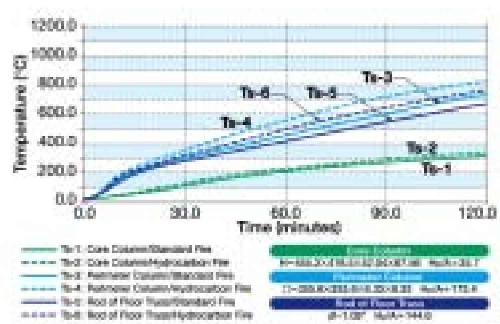

FIG. 3. Steel Temperature Rise Subjected to Heating (3/4-inch thick sprayed fire protection)

4. Fire Resistance of Steel Frames

Steel strength is reduced at high temperatures. A36 steel's high-temperature strength and its stress-strain relationship at high temperatures 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 hours) through loaded heating (the loaded heat test) 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 6. The proof stress of steel frames decreases with the increase of temperature, leading to the loss of the frame's load-bearing capacity.

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 (1,000F average, 1,100 maximum)
Beam:	593°C (1,100F average, 1,200 maximum)

As for fire testing in Japan [3], the heat test traditionally has been applied with the critical temperatures specified as follows:

Column:	350°C average, 450°C maximum
Beam:	350°C average, 450°C maximum

But, today, the loaded heat test 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 integration of the critical temperatures in the two countries with Fig. 4 (high-temperature strength of steel materials). The difference between the two countries is about twice in the critical temperature, eventually leading to a wide difference in the thickness of fire protection required.

This study obtained 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 to the columns' ratio of axial force in the FEMA Report. However, since the stress applied on the diagonal members of the floor trusses was estimated, on calculation, at about 50% of the yield point, the ratio was assumed to be the same.

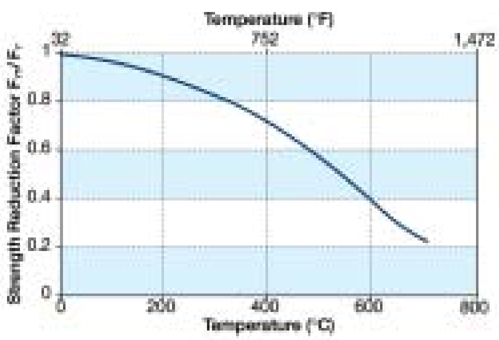

FIG. 4. High-temperature Strength of A36 Steel

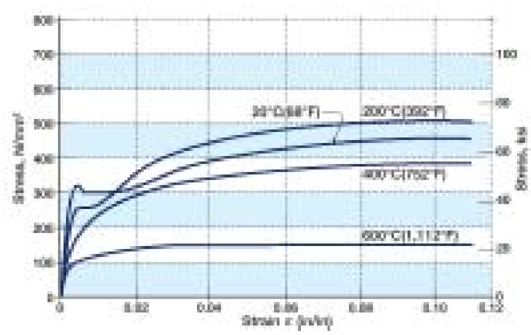

FIG. 5. Stress-strain Curves of A36 Steel At High Temperatures

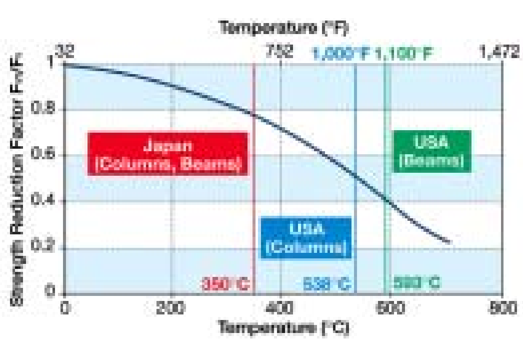

FIG. 6. Relationship between Strength of Steel Materials and Critical Temperature

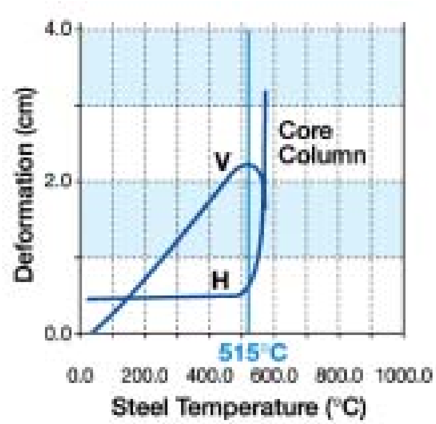

FIG. 7. Analysis of Fire Resistance of Column

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 calcula ted buckling temperatures were 515°C for a core column and 520°C for a perimeter column.

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

(i) The buckling temperature of the columns virtually corresponds to steel's high-temperature strength and the columns' ratio of axial force.

(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. It 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 the high-temperature strength of steel materials shown in Figs. 4 and 6 is the average. Fig. 8 shows the results of examining the high- temperature strength of steel materials, made by the Japan Iron and Steel Federation (JISF) [7]. The high-temperature strength varies widely, as shown in the figure. 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 in mind, the critical temperature of 350°C adopted in Japan secures a higher level of safety.

Photo 6. Buckling of Column after Loaded Heat Test

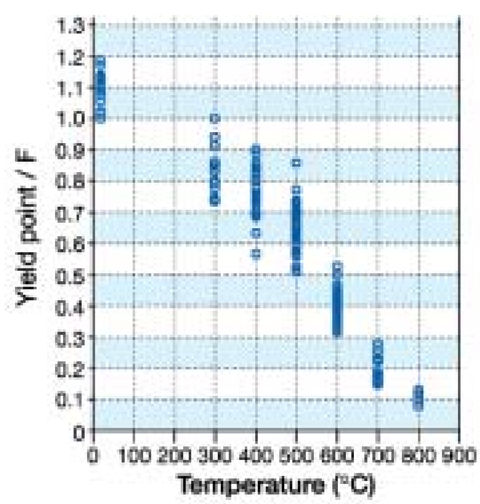

FIG. 8. Variations in High-Temperature Strength of Steel Materials

Fig. 9 shows the schematic diagram of the WTC's floor structure. The floor is of a structure 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). Their 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's diagonal member (Ø1.09 in.). Though the floor trusses are the composite structure with the concrete slabs, the concrete slabs were not considered in the analysis. The loads were design loads (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 truss's large deflection. 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. Besides, as the floor slabs serve to retard the thermal expansion of the trusses and to constrain the buckling of the upper chords, it can also be assumed that the actual failure temperatures must have been higher than those in the above analyses.

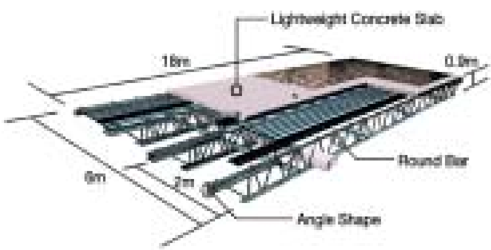

FIG. 9. Floor Structure

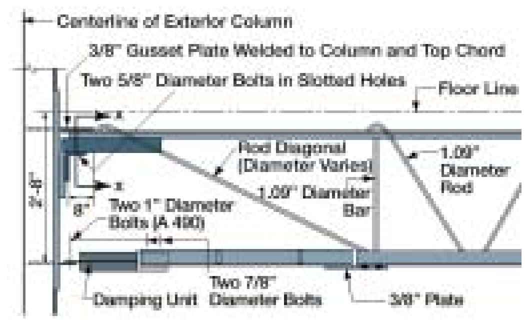

FIG. 10. Details of Floor Truss End

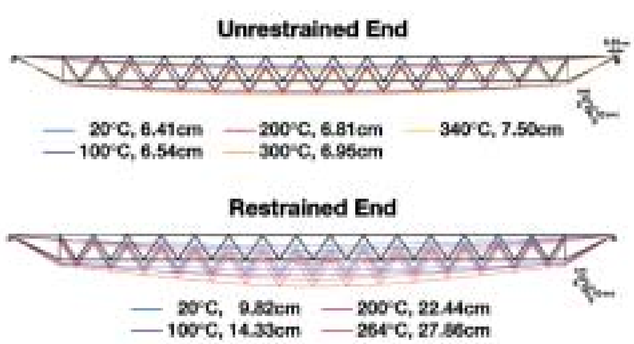

FIG. 11. Analysis of Thermal Deformation of Floor Truss

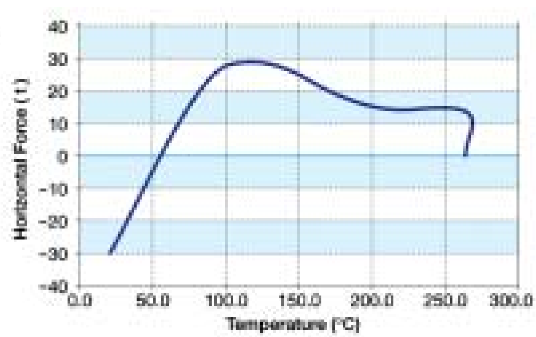

FIG. 12. Restraint Load at Floor Truss End

But, 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. In addition, a large shearing force acts on the end connections. From these, it is quite 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.

5. Proposition and the use of Fire Resistant Steel

From the above analytical study and examination of the fire resistance of the WTC's 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 steels excelling in high-temperature strength.

Fire resistant steel (Sakumoto et. al, 1992-1, hereinafter referred to as "FR Steel"), containing such alloying elements as molybdenum and chromium, is superior in high-temperature strength to ordinary steels. Fig. 13 shows the yield strength at high temperature of FR steel (Grade 325) and conventional steel. The yield strength at high temperature of conventional steel decreases to two-thirds the specified room-temperature value (217N/mm^2) at around 350°C. On the other hand, this specified value is guaranteed at 600°C for FR Steel.

A yield strength of two-thirds the specified room temperature value corresponds to allowable stress for sustained loads. Because fire and earthquake are not taken into consideration simultaneously, it can be said that yield strength able to support the dead weight of a building, allowable stress for sustained loads, is the strength required at the time of fire.

The elevated yield strength at high temperature is obtained from metallurgy, in which additive elements such as molybdenum precipitate to affect the molecular structure and, therefore, the yield strength.

Table 1 shows mechanical properties of FR Steel (Grade 325) with comparison of conventional steel ASTM A992 [9]. The performance of FR steel at room temperature corresponds to that specified in the standards for Japanese anti-seismic structural steel (JIS G3136 [10]), with yield ratio under 80%, Charpy absorption energy 27J or over at 0°C. Furthermore, it has the same weldability as that of conventional steel.

TABLE 1. Mechanical Properties of FR Steel

Standard	Grade	Yield strength		Tensile strength	Yield ratio	Elongation	Charpy
	N/mm^2 (ksi)	N/mm^2 (ksi)		N/mm^2 (ksi)	%	%	energy J
		at 0 C	at 600 C	at 0 C	at 0 C	at 0 C	at 0 C
		min / max	min	min / max	max	min	min
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
FR Steel	Grade 325	325/445	217 min	490 / 610	80	21	27
ASTM A992	Grade 345 (50)	345/450 (50/65)		450 min	85	18	1)

Note 1) : charpy absorption energy is a supplementary requirement for use at the option of the purchaser

Application of FR Steel

Whether FR Steel is used with or without protection is determined by building application and design concept.

Fig. 14 shows numbers of buildings using FR Steel provided by Nippon Steel since 1989. Car parks, atriums, sport facilities, art galleries and a station building have been the major applications without protection. External steel-frame buildings also are a major application.

In case of buildings with high combustible loads, such as office buildings, warehouses, department stores and collective dwellings, thickness of protection can be reduced or the redundancy of a steel frame can be improved by extending fire resistance time with FR Steel protected with the same thickness as conventional steel.

Car park

Photos. 7 and 8 show the application of FR Steel to a multi-story car park. This 17-story car-park building, constructed close to Chiba Station, is one of the largest of its kind in Japan. It will accommodate 1,800 vehicles. Conventional steel-frame car parks present a dark and dirty appearance because the steel-frame protection blackens by exhaust gases and exfoliates when exposed to rain and wind. With FR Steel, no protection is required for the frame and making the columns slender enlarges the parking area.

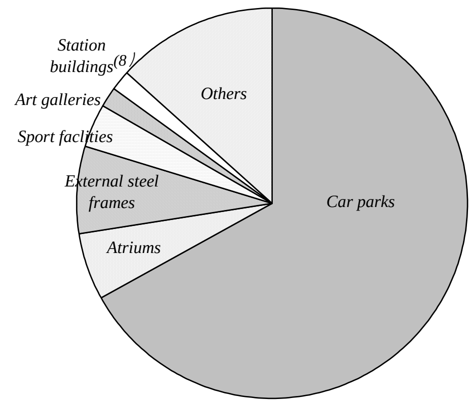

FIG. 14. Application of FR Steel

Sport facilities

For this gymnasium, constructed at Tokoname Park (Aichi Prefecture), a dynamic structure was adopted in which the roof, a space truss structure measuring 100.8 x 50.4 m, is supported on four sides by large truss structures (Photo 9). For the large truss structures, prestressed concrete was at first considered because protection is necessary when a steel-frame structure is adopted. However, FR Steel that matches the design intention was adopted as the material for this large truss structure.

Station building

East Japan Railway Company constructed a commercial building spanning the tracks of Oimachi Station in Tokyo (Photos. 10 and 11). In the case of constructing a large-scale building spanning a railway, it is necessary to provide protection for steel frames in the vicinity of the tracks. However, because of the exfoliation of maintenance, steel-frame construction without protection was required for this building.

Office building (external steel-frame building)

This 31-story building in Kobe houses offices and research laboratories (Photo 12). A suspended mega-structure, in which six stories are structured as one unit, was adopted for the construction of the building. It features a structural design in which the mega-structure is presented as the building's structural design element. In the building construction, FR Steel was adopted to give the impression of slenderness of the structural members through the application of lighter protection to columns and beams.

In addition, it is important to verify the fire resistance of steel frames by means of fire-safety design.

Fire tests address such structural members as columns and beams, as stated in section 4, and they 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 the like. Besides, load concentrates in connections, so hinged end and similar connections can fracture during the course of heating or cooling.

6. Summary

Photo. 7. View of Sen City Car Plaza

Photo. 8. Steel Frame of Sen City Car Plaza

Photo. 9. View of Tokoname Park Gymnasium

Photo. 10. Steel Frame of Sen City Car Plaza

Photo. 11. View of Oimachi Station Building

Photo. 12. View of P&G Building

5. Summary

In Japan, the number of such failure analyses have been increasing since around 1990, where fire-safety designs are used to verify the fire resistance of steel frames, targeting buildings adopting FR steel and concrete-filled steel tube columns. Henceforth, imperative is the active adoption of fire-safety designs particularly for high-rise buildings and publicly important buildings. The adoption of fire-safety design for high-rise steel buildings ensures greater safety in fire. FR Steel, used successfully in Japan for a decade, is an option to be considered.

ACKNOWLEDGMENT

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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