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IIIb

Calcul et conception des structures métalliques ou mixtes en vue de leur résistance à l'incendie

Bemessung von Stahl und Verbundbauwerken gegen Brandeinwirkungen

Design of Steel and Composite Structures for Fire Resistance

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Creep Buckling of Steel Column at Elevated Temperatures

Flambage par fluage d'un poteau en acier aux températures élevées

Kriechknicken von Stahlstützen bei hohen Temperaturen

SIGGE EGGWERTZ
The Aeronautical Research Institute of Sweden
Stockholm, Sweden

1. INTRODUCTION

The buckling strength of a steel column may be considerably reduced due to exposure to elevated temperatures during a fire. This reduction is now taken into account by use of a chart where the buckling stress for the steel material is plotted versus the slenderness ratio for each temperature considered [1]. Such curves have been obtained by introducing in conventional room-temperature buckling formulas the mechanical properties determined from standard material tests at the various temperatures. There is a large variation between curves for temperatures exceeding 500°C published by different authors. The reason is probably that the creep rate of ordinary structural carbon steels increases rapidly at this temperature. The material tests rather arbitrarily include creep during the time taken to increase the load to ultimate failure. During a fire the column is usually subjected to constant load during the whole heating period implying a larger creep deformation. Furthermore, creep buckling has a non-linear course, rendering the present design procedure an unconservative approximation.

In order to establish the basis of a more reliable method, including the time parameter, for determining the collapse load of a column in a fire, a study has been made of a hinged steel column of I-section with an initial deflection, subjected to elevated temperatures, mainly 600°C but also 550 and 650°C. Material creep tests were carried out at 600°C, the results being extrapolated to other temperatures by use of the Dorn-theory. Creep constants were determined and introduced into a computer programme providing the creep life at given constant stress and temperature. By performing a large number of such calculations at different stress levels a diagram was obtained giving the buckling stress versus the slenderness ratio for various times of exposure to the temperature considered. The computer programme was also modified to allow a realistic variation of temperature history and computations were run to determine the critical stresses corresponding to maximum temperatures of 600 and 650°C.

2. CREEP LAW AND CREEP TESTS

Standard creep tests are performed on material coupons at constant load and temperature, giving a relationship between strain ε and time t.

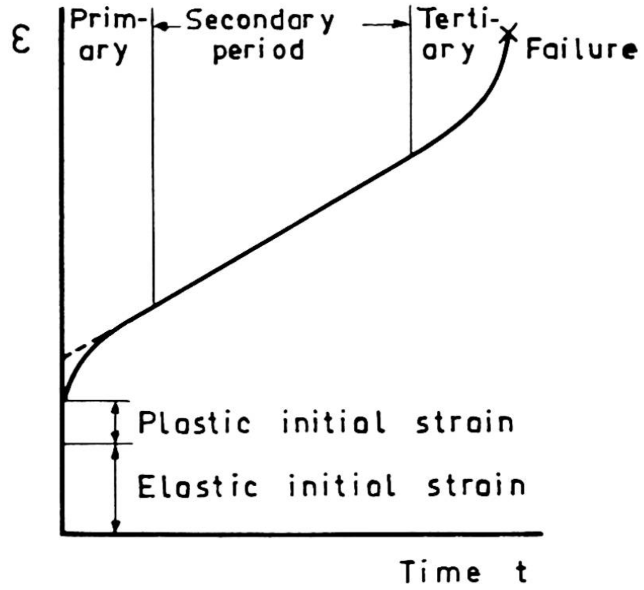


Fig 1

Creep curve for metal at constant load and constant temperature

A creep curve typical for metals at elevated temperatures, Fig 1, includes three phases of which the secondary creep is dominating. In modern metal creep research the creep law of Norton-Odqvist is normally used

$$d\varepsilon/dt = \dot{\varepsilon} = k\sigma^n \qquad (1)$$

where σ is the constant stress and k and n are creep constants belonging to the temperature applied. It was found by Dorn that the creep rate $\dot{\varepsilon}$ may be determined for other elevated temperatures by introducing a temperature compensated time parameter

$$\theta = \int_0^t \exp(-\Delta H/RT)dt \qquad (2)$$

where ΔH and R are constants and T the temperature in $^{\circ}$K. Harmathy [2] carried out several creep tests and established a generalized creep curve, based on Dorn's theory, for ASTM A36 steel valid for temperatures of 400-700°C. Results of creep tests within the same temperature range have also been published by Thor[3].

To obtain creep data for calculations of critical times to buckling, creep tests were run in tension at 600°C with four constant stress levels σ = 30, 40, 50 and 60 MPa. The material coupons were made of a carbon steel with yield strength 300 MPa and ultimate strength 460 MPa, i.e. rather similar to A36. The creep rates determined gave the creep constants

$$k = 1.88 \times 10^{-11} \qquad n = 4.9$$

For other temperatures the Dorn theory was used to obtain creep rates, introducing $(\Delta H/R) = 39000$ $^{\circ}$K as found by Harmathy for A36. This gives a constant value of n for all temperatures, while

$$k = 1.88 \times 10^{-11} \exp(44.7 - 39000/T) \qquad (3)$$

3. THEORY OF CREEP BUCKLING

In a column having an initial maximum deviation w_o from a straight line and subjected to an axial load P with an excentricity a, Fig 2,

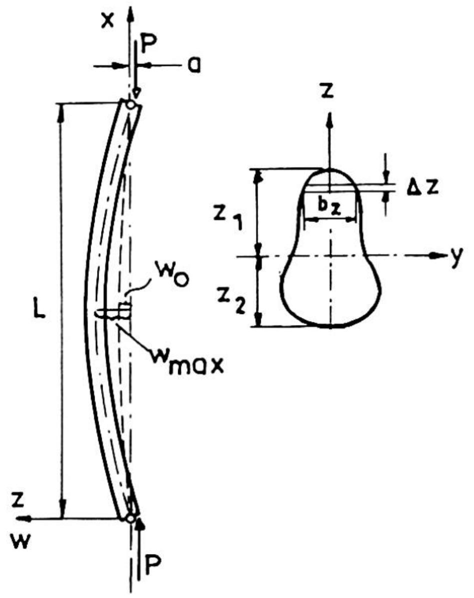


Fig 2 Initially curved column hinged in both ends, with cross-section of single symmetry and eccentric axial load P

a bending moment will occur increasing the deflection to w_{max}. If the load is much smaller than the short-time buckling load, and no bending out of the xz-plane can take place, the increase is rather small but if the load is kept constant and creep sets in, the deflection increases with time. The creep strain rate at a distance z from the CG-axis of a section, Fig 2, may be written

$$\dot{\varepsilon}_x = \dot{\sigma}_x / E_o + k\sigma_x^{\,n} \qquad (4)$$

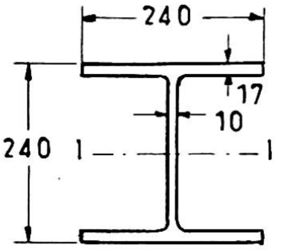
where σ_x is the compression stress which is continuously growing with increasing deflection. E_o is a modulus taking elastic and plastic deformation, and possibly also primary creep, into account. The second term represents the secondary creep. A constant n considerably larger than one will obviously cause a fast acceleration of the deflection w_{max} with growing stress.

Creep buckling theories and approximate solutions of the creep buckling life t_k for metal struts were first published by Hoff and Hult. Closed solutions for more general cases present considerable mathematical difficulties. Samuelson [4] developed a computer programme for a hinged column of singly symmetrical constant section subjected to constant load and temperature. The cross section was divided into thin layers of thickness Δz and width b_z, while the length L was split up into elements Δx and time into intervals Δt. This programme was used for evaluating the critical time for different loads on a column with varying slenderness ratios, and also modified to allow a variation of the temperature between time intervals.

4. DISCUSSION OF COMPUTED CREEP LIVES

The numerical analysis was carried out for a column section HE240B, Fig 3, assuming no excentricity, but an initial deviation according to Dutheil

$$w_o = 4.8 \times 10^{-5}\, L^2/d = 4.8 \times 10^{-5} L^2/0.12 = 4 \times 10^{-4}\, L^2$$



$A = 1.06 \times 10^{-2}\ m^2$

$I_1 = 1.13 \times 10^{-4}\ m^4$

$i_1 = 0.103\ m$

$d = H/2 = 0.12\ m$

Fig 3

Column section HE 240B used in computations

The modulus of elasticity E_o of Eq(4) was determinad from a formula proposed by Thor [3].

$$E_o = 325000 - 404\ \vartheta_s \quad \text{MPa} \tag{5}$$

The creep buckling was defined as the moment when w_{max} exceeded twice the height of the section in the buckling direction giving a very high creep rate. Critical creep times at 600 °C were determined for columns of four different lengths L = 3, 4.5, 6 and 9 m, yielding slenderness ratios $\lambda = L/i_1 = 30, 45, 60$ and 90. A number of different mean stresses were treated for each column length. The results of the computations are presented in Fig 4, where the creep buckling time is plotted versus the compression stress of the column for each slenderness ratio. Creep lives were also obtained for the steel temperatures $\vartheta_s = 550$ and 650 °C, assuming in both cases $\lambda = 45$, while σ was 70 and 35 MPa respectively. These results are entered into Fig 4 as isolated

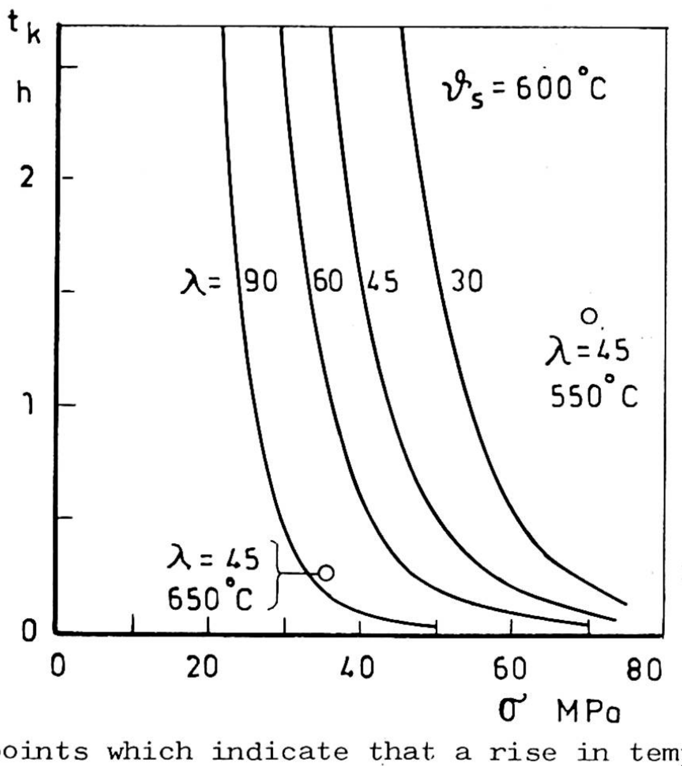


Fig 4

Creep buckling time versus compression stress for various slenderness ratios at 600 °C

points which indicate that a rise in temperature of 50 °C corresponds to a shortening of life by a factor of 10, or a decrease in stress by about 40 per cent.

The curves of Fig 4 are replotted in Fig 5, giving the buckling stress versus the slenderness ratio for exposures to 600 °C from 0.2 to 2 h. The buckling curves presented by Kawagoe-Saito [1a] and Sfintesco [1b] are introduced for comparison. While the former is extremely conservative, corresponding to several hours of heat exposure, the latter seems to be

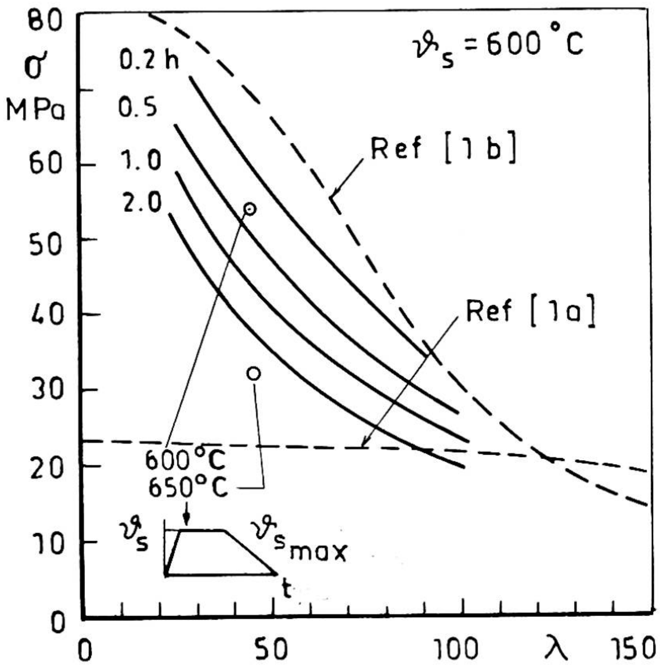


Fig 5

Buckling stress versus slenderness ratio for various times of exposure to 600°C

unsafe even for a few minutes of 600°C.

In a fire the temperature of the steel structure is normally gradually rised from room temperature to a maximum determined e.g. by the fire load, after which the cooling starts. A temperature-time history according to Fig 6 was introduced in the computer programme assuming $\vartheta_{s\ max} = 600$

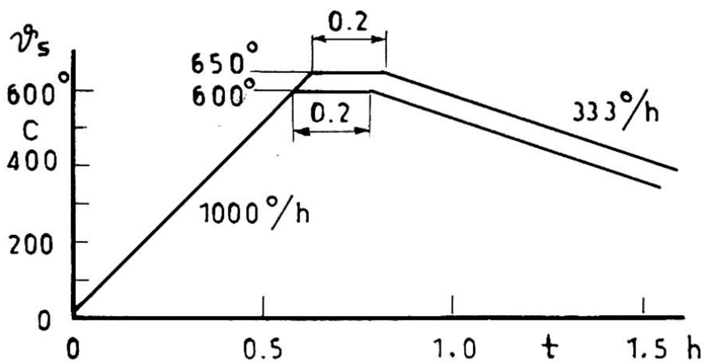


Fig 6

Temperature-time history introduced into computer programme

and 650°C. Using the same column section as before, the slenderness ratio λ = 45, the stress was varied to allow interpolation of the value just causing collapse during a temperature cycle. These stresses are also plotted in the diagram, Fig 5.

Although it may be objected that still a number of factors remain to be considered in a realistic analysis of the behaviour of a steel column during fire, the results of the calculations clearly show that an analysis of creep buckling is worth-while.

REFERENCES

1. IABSE 10th Congress, Introductory Report, Zürich 1975 - a. Kawagoe, K - Saito, H: Thermal effects of fires in buildings. - b. Sfintesco, D: Calcul et conception des structures métalliques ou mixtes en vue de leur résistance à l'incendie.

2. Harmathy, T Z: A compressive creep model. J Basic Eng, Trans ASME, Vol 89, Series D, Sept 1967, p 496-502.

3. Thor, J: Deflection and strength of statically determined steel beams under fire conditions (in Swedish). Jernkontorets forskning. Serie D, Nr 54, Stockholm 1972.

4. Samuelson, Å: Creep deformation and buckling of column with an arbitrary cross section. FFA Report 107, Stockholm 1967.

SUMMARY

The creep buckling life of a steel column is determined by feeding data from standard creep tests into a computer programme. It is shown that the effect of creep on the buckling strength is very important at temperatures around 600 °C.

RESUME

L'évolution du flambage par fluage d'un poteau en acier est déterminé au moyen d'un calcul numérique, dans lequel on introduit les résultats des essais de fluage standard. Il est montré que l'influence du fluage est très important aux températures de 600 °C environ.

ZUSAMMENFASSUNG

Die Belastungsdauer einer Stahlsäule bis zum Kriechknicken wird durch ein numerisches Programm bestimmt, wobei man Dehnungsmessungen von Standardkriechversuchen benutzt. Es wird gezeigt, dass dem Kriechen bei Temperaturen um 600 °C grosse Bedeutung zukommt.

Fire Resistance of Steel Structures in Neighbouring Fire

Résistance au feu des structures métalliques à proximité d'un incendie

Feuerwiderstand von Stahlkonstruktionen in benachbarten Brandstätten

F. ITOH
Professor
Tokyo Metropolitan University
Tokyo, Japan

T. WAKAMATSU
Chief of Fire Engineering Section
Building Research Inst., Min. of Construction
Tokyo, Japan

1. Thermal effects on bridge members.

It is not so usual to take so large thermal increment as to be produced by fire into considerations in design of bridges. But the anticipated high temperatures and long duration of fire in large scale oil plants forced us to do some studies on their effects against structural members of continuous truss bridges.

In our cases on preliminary design for Meiko Big Bridge and Bannosu Bridge, the effects of pressure by heated air enclosed in members with box type section, the effects of temperature differences between plates in a member, and the effects of temperature differences between members in three span continuous truss girder bridges were studied.

1-1. The effect of internal pressure.

The sections of the members were idealized as right square with width B, and the plate thicknesses are denoted as hf in cover plates and hw inweb plates. These are assumed to be made air-tight with diaphragms at the panel points, the both ends of these members. Then, the increment of inside pressure "P" must be

$$P = \frac{Po}{To}\, t$$

where Po is standard pressure of the air, "To" is the absolute value of normal temperature, and t is average temperature increment of the member.

Expected maximum stress σ_1 in transversal direction of the member can be estimated as

$$\sigma_1 = \frac{P}{2}\left(\frac{B}{h}\right)\left(1+\frac{B}{h}\right) \doteqdot \frac{P}{2}\left(\frac{B}{h}\right)^2$$

where h is smaller one of hf and hw, and the maximum stress will appear at inside of corners. If B equals to 900 mm, and h is 12mm,

$$\sigma_1 = 9.75\ t \quad (kg/cm^2).$$

1-2. The effects of temperature difference between plates in a member.

The temperature difference between plates will causes transversal and longitudinal stresses in a member.

Transversal maximum bending stress σ_2 will be estimated easily neglecting small amount of in-plane stress as

$$\sigma_2 = \frac{3E\alpha\Delta t}{2(1-\nu^2)\{1+3(Hf/hw)^3\} \quad (B/hf)},$$

where Δt denotes the temperature difference, E is Young's modulus, ν is thermal elongation coefficient, and α is Poisson's ratio. This stress appears also at member corners, and if B equals to 900 mm, and hf 45mm, hw45mm,

$$\sigma_2 = 0.52\,\Delta t \quad (kg/cm^2).$$

It seems not so dangerous, but must be added to σ_1 at corners of shaded/side of the members.

Longitudinal thermal stress σ_3 may appear by constraints against bending of the member, and they must be affected by boundary conditions at panel points. But we may assume safely that the panel points do not rotate at all. Then the stress σ_3 will be given as

$$\sigma_3 = \frac{E}{2}\alpha\Delta t = 12.6\,\Delta t \quad (kg/cm^2).$$

1-3. Effects of temperature difference between truss members.

The temperatures of each member by the fire had to be estimated, at first, and the following approximated assumptions were adopted to this end:

a) The temperature of a member depends on the view factor of the surface of plate facing to the fire, and the relation can be represented approximately linear in the region of present temperature under consideration.

b) View factor itself varies inversly as square of distance from the member to the center of fire, and is proportion to cosine of the angle between direction to the fire and the normal line of the plate.

The view factors of members shaded by another members or deck of the bridge are so smaller than the ones facing the fire. The results of more or less troublesome calculation considering shading effects on Bannosu bridge showed us very lower temperature increments in members at shaded side of the structure (Fig. 1). In this case, upper chords of shaded side of the bridge have been affected largely by the wide deck slab. As the Meiko Big Bridge have been designed as double deck bridge, behavior of the lower chord members will be almost the same to the upper chords of Bannosu Bridge.

In any case, no trouble will happen if the structure is statiscally determinate. But the continuous bridges are indeterminate, and the constraints at supports will make some effects on their stresses. Theoretical calculation of member stresses σ_T knowing each temperature increments is not so difficult one, and the maximum effect have been found at a chord member near the one of center piers in our cases. The estimated maximum stress due to the temperature differences reached $13.7\,\Delta T$ kg/cm^2 in Meiko Big Bridge and $17.4\,\Delta T$ kg/cm^2 in Bannosu Bridge, where ΔT denotes temperature difference at the nearest section of the bridge to the fire.

1-4. Permissible temperature conditions.

The permissible limit of the fire effects must be considered taking into account the resultant stresses of members and the buckling strength of compressed ones. But for buckling strength, the problem will be not so difficult if it is permitted to use increased allowable unit stresses of standard specifications.

For the stress limits, although local yield will occur at comparatively early stage of the fire because of the resultant effects of bending and in-plane stresses of the plate, the limit of the stresses plate may be considered as the instance when the one of principal stress on surface of the plate reaches yield point σ_Y member axes.

Standing on such considerations, the permissible limit of the temperature conditions can be decided from following three inequalities:

$$\frac{t}{t_0} + \frac{\Delta t}{\Delta t_1} \leq 1.0 \quad \text{(for transversal stress),} \qquad -(1)$$

$$\left| \frac{\sigma_T + \sigma_d}{\sigma_Y} \pm \frac{\Delta t}{\Delta t_2} \right| \leq 0 \quad \text{(for longitudinal stress),} \qquad -(2)$$

$$|\sigma_T| < \beta \sigma_{ca} \quad \text{(for buckling if } \sigma_T \text{ is negative),} \qquad -(3)$$

where

$$t_0 = \frac{577h^2 \sigma_Y}{B}$$

$$\Delta t_1 = \frac{\{1+3(hf/hw)^3\}\ (B/hf)\sigma_Y}{41.54\ (hf/hw)}$$

$$\Delta t_2 = \sigma_Y / 12.6$$

and β is a coefficient smaller than the safety factor, σ_d is dead load stress.

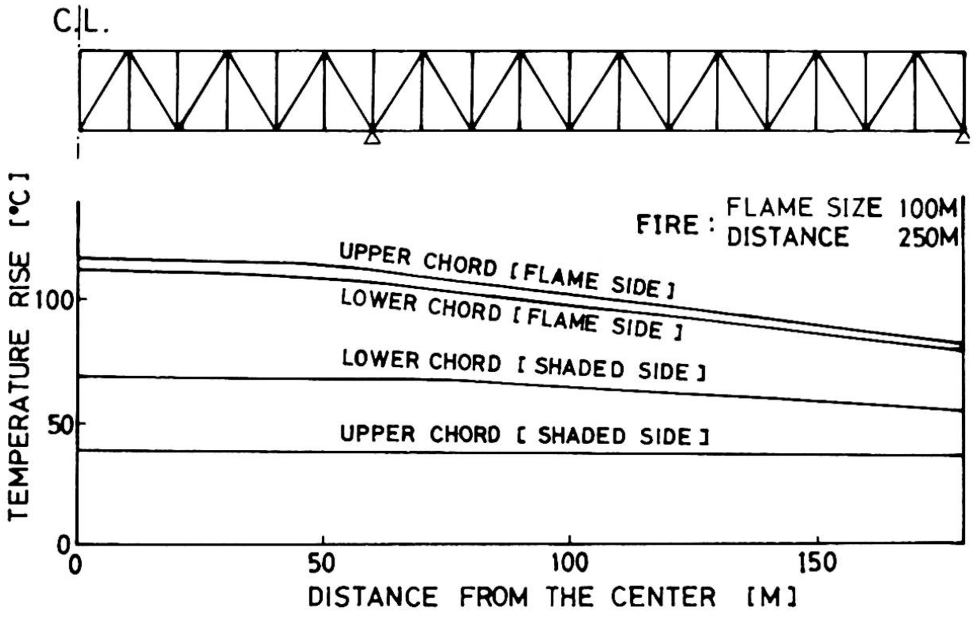


Fig. 1 Estimated temperature rises of cord members

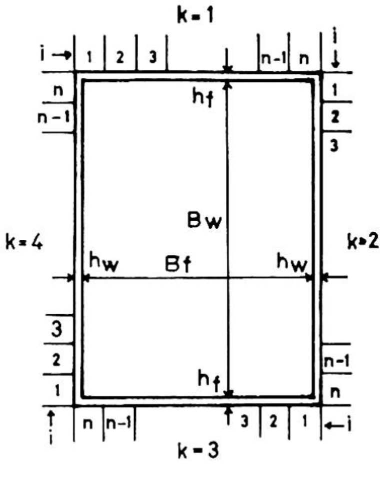


Fig. 2 Mathematical model of cross section of a member

2. Calculation of temperature of a bridge member.

In case of any fire occurring nearby a bridge, radiant heat from the fire should produce some temperature increase which has some harmful effects on bridge members as described above. The temperature increment depends on several conditions such as scale, severity and duration of the fire, distance from the bridge to the fire, and emissivity of surfaces of the member.

First of all an equation was introduced for estimating unsteady-state temperatures on different steel members having box type section. Then the equation was solved to obtain the temperature of a certain member to be used for a main truss girder of a bridge. The calculation was made by use of an electric computer for various values of the view factors for radiative heat exchange between the fire and the surfaces of the member. To examine the effects of paints applied on the surface on the temperature increase, we considered two kinds of paints, an ordinary paint with larger emissivity and an alumina paint with smaller one.

2-1. Equation for calculating the temperature.

Fig. 2 shows a cross section of a steel member as a mathematical model for deriving the equation which enables to calculate the unsteady-temperature distribution of the bridge member exposed to radiant heat from a fire. The equation was obtained from a heat balance on a small rectangular zone (k,i) in Fig. 2, by taking account of the following four heat exchanges;

(1) radiation between the external surface and the surroundings involving the fire
(2) radiation between any internal surfaces
(3) natural convection on the external and internal surfaces
(4) heat conduction in the solids of the member

The equation is written in finite-difference form as shown in Eg. (4), and the solution is accomplished by finite-time step advancement.

$$T(k,i,N+1) = T(k,i,N) + \frac{\Delta\tau}{c\rho h(k)}\Big[\sigma\varepsilon_s\Big\{FF(k)T_F(N)^4 - T(k,i,N)^4 - [1-FF(k)]T_0^4 - (\varepsilon_i/\varepsilon_s)\sum_{\eta=1}^{4}\sum_{j=1}^{n}F_{k\eta}(i.j)\,[T(k,i,N)^4 - T(\eta,j,N)^4]$$
$$- H_0(k,i,N)\,[T(k,i,N) - T_0] - H_I(k,i,N)\,[T(k,i,N) - \sum_{k=1}^{4}\sum_{i=1}^{n}T(k,i,)/4n] + \frac{\lambda n^2}{B(k)}\Big\{\frac{T(k,i-1,N)-T(k,i,N)}{\alpha(i-1)} + \frac{T(k,i+1,N)-T(k,i,N)}{\alpha(i+1)}\Big\}\Big] \quad ---(4)$$

where $\alpha(i-1) = \alpha(i+1) = B(k)/h(k)$, when $2 \leqq i \leqq n-1$;

$\alpha(i-1) = \alpha$, when $i = 1$; and $\alpha(i+1) = \alpha$, when $i = n$;

where $\alpha = (Bf + Bw)/(hf + hw)$

$B(k) = Bf$ and $h(k) = hf$, when $k = 1$ or 3;

$B(k) = Bw$ and $h(k) = hw$, when $k = 2$ or 4

$T(k,i,N)$: absolute temperature at the center of a zone (k,i) of a member at the time $\tau = N\Delta\tau$, where $\Delta\tau$ denotes a time interval

T_F, T_o : equivalent radiative temperature of a fire and temperature of the air in absolute value respectively

$\lambda, c, \rho,$: thermal conductivity, specific heat and density of

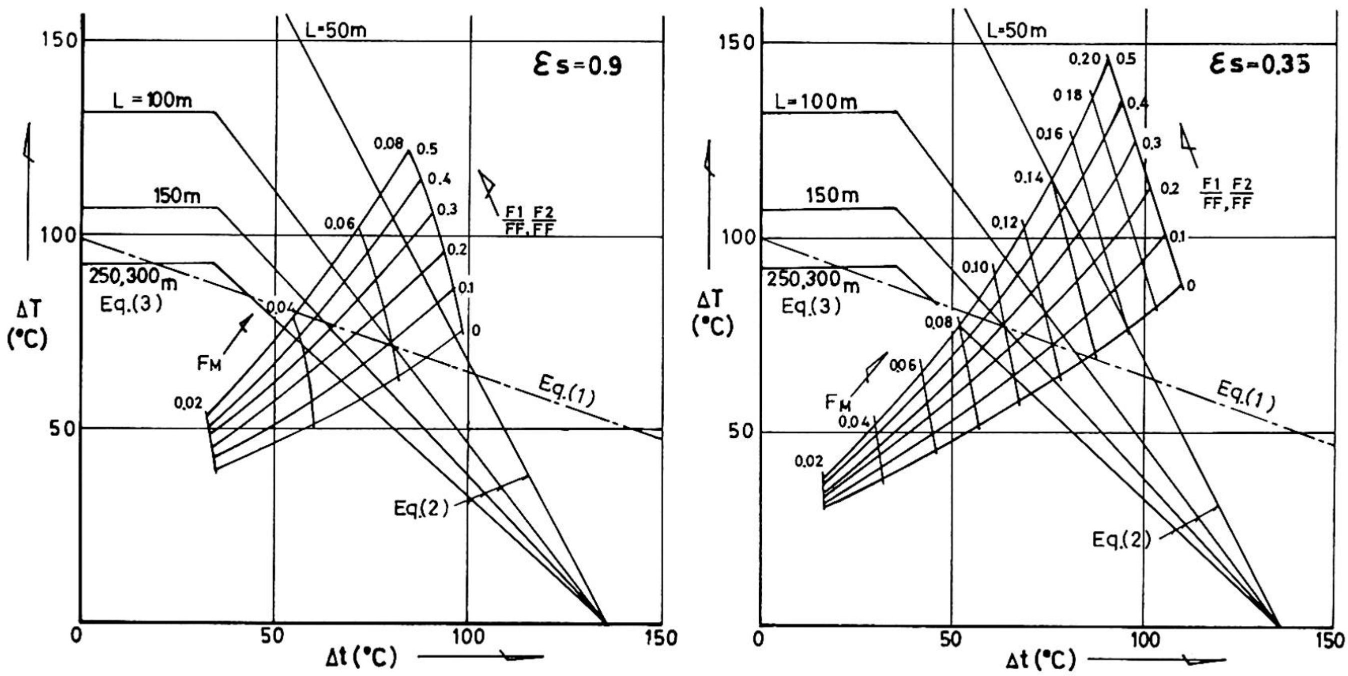
		steel respectively
σ	:	Stefan-Boltzmann constant
ε_s, ε_i	:	emissivity of external and internal surface of member respectively
FF(k)	:	view factor between internal surfaces of zone (k,i) and any zone
H_O, H_I	:	heat transfer coefficients by natural convection at an external and an internal surface of the zone respectively

2-2 Assumptions for calculation

Calculations were made on the following assumptions;

(1) cross section of the member: Bf = Bw = 900, hf = 22, hw = 25 (mm)
(2) number of zones (k,i): k = 4, n = 3, therefore 12 in total
(3) emissivity of the internal surfaces: $\varepsilon_i = 0.9$
(4) emissivity of the external surfaces: $\varepsilon_s = 0.9$ and ε_s 0.35 ($\varepsilon_s = 0.9$ for ordinary paint, $\varepsilon_s = 0.35$ for alumina paint)
(5) temperatures T_F, T_O: $T_F = 1000$, $T_O = 298$ (°K)
(6) duration of the fire: three hours
(7) view factors between the fire and three external surfaces of the member FM, F1 and F2: FM is for the surface receiving the largest amount of the radiative heat among the three surfaces. F1 and F2 are for the two surfaces normal to the surface concerned with FM, where F2 ≤ F1 ≤ FM.
FM: 21 steps in range from 0.02 to 0.92
F1: 6 steps, 0, 10, ..., 50 percent of FM
F2: 3 steps, 0, 50, 100 percent of F1

Therefore the number of cases for calculation amounts to 672 in total.



(a) for an ordinary paint (emissivity $\varepsilon_s = 0.9$)

(b) for an alumina paint (emissivity $\varepsilon_s = 0.35$)

Fig. 3 Relation between temperature differences (ΔT and Δt) and view factors (FM, F1 and F2) for the case of F1 = F2 = 0.5FM

2-3 Results of calculation

Through these calculations, many available data have been obtained to know various thermal conditions on the member under different situations of fire. But two examples of several diagrams synthesized from the calculated results only are shown in Fig. 3 for two cases of the emissivity $\varepsilon_s = 0.9$ and $\varepsilon_s = 0.35$ in each of which F1 = F2 = 0.5FM. The figure shows the relation between the temperature differences (ΔT and Δt) and the view factors (FM, F1 and F2) in range concerned with permissible temperature limits represented by inequalities (1), (2) and (3), where ΔT and Δt denote respectively the temperature difference between two truss members and one between two plates of a member.

3. Geometrical requirement for ensuring bridge against fire

As shown in Fig. 3, we have clarified the relation between the permissible thermal conditions and the view factors. They are given from the diameter "ϕ" and the height "H" of a fire, the distance "L" from the center of the fire to the surface of the bridge member and so on.

Thus we have obtained finally the following geometrical conditions represented by ϕ/L required for ensuring a bridge against fire, assuming to be H = 1.5 ϕ.

Emissivity of external surface of a member	$\frac{F1}{FM}$ (= $\frac{F2}{FM}$)	ϕ / L
0.9 (for ordinary paint)	0 0.5	$\leq$ 0.33 $\leq$ 0.29
0.35 (for alumina paint)	0 0.5	$\leq$ 0.46 $\leq$ 0.40

SUMMARY

A problem how to ensure elevated bridges against fires of oil plants situated near the bridges has been submitted recently in Japan. Thermal effects on steel members of bridges are discussed and some permissible thermal requirements for the members are derived. An equation for calculating temperatures of a member having a box section is introduced and solved for various conditions of radiation. A method for estimating necessary clearance between bridge and an oil plant according to probable scale of the fire is suggested.

RESUME

Le problème de la protection d'un pont contre l'incendie d'une grande raffinerie près du pont s'est posé récemment au Japon. La discussion de l'effet thermique sur les éléments métalliques du pont a permis de tirer quelques règles de résistance thermique des éléments. Une équation pour calculer la variation de température d'un élément en caisson a été développée, tenant compte de différentes conditions de radiation. Une méthode pour estimer l'éloignement nécessaire entre le pont et la raffinerie en fonction de l'importance de l'incendie a été proposée.

ZUSAMMENFASSUNG

In jüngster Zeit befasste man sich in Japan mit der Frage, wie Brücken vor Bränden in nahegelegenen Erdölraffinerien geschützt werden können. Man untersuchte die Wärmeeinflüsse auf Brückenglieder und gewann daraus einige zulässige Widerstandsbedingungen. Es wurde eine Gleichung zur Errechnung der Temperaturen für Stäbe mit Hohlquerschnitt abgeleitet und für verschiedene Strahlungsbedingungen gelöst, und eine Berechnungsmethode für die erforderliche Entfernung der Brücke von der Erdölraffinerie in Funktion der Ausdehnung des Brandes vorgeschlagen.

Probabilistic Analysis of Fire Exposed Steel Structures

Détermination probabilistique de la sécurité au feu des éléments de structure métallique

Wahrscheinlichkeits-theoretische Auswertung der Brandsicherheit von Stahlbauteilen

SVEN ERIK MAGNUSSON
Dr. Techn., Research Engineer
Lund Institute of Technology
Lund, Sweden

A large amount of work is presently in progress regarding the optimum level, in an economic sense, of the over-all fire protection of buildings. Structural damages can be prevented or limited by many measures, such as compartmentation, installation of detectors and sprinklers, reducing the attendance time of the fire brigade etc. Among those steps taken to reduce the fire damage, the oldest and most evident one is to increase the fire endurance of the individual structural member. For a high-rise building, the fire endurance must reach the level where the structural integrity of the building is maintained even during the most severe fire possible. For economic reasons, though, the fire endurance cannot be unlimitedly high. Some element of risk, however small, has to be accepted. Evidently, there is a need for a reliability analysis that makes it possible to identify this risk of structural collapse by fire and compare with the risks due to other kinds of catastrophic events.

This need has been accentuated by the different design rationales or systems put forward during the last few years. Particularly interesting in this connection is the differentiated Swedish method, see /1/, /2/. The special attention derives partly from the fact that for the first time the new developments have been transformed into a ready-to-use design manual. The manual permits, with the aid of charts, diagrams and tables, the practising engineer to make a rational design of fire-exposed steel structures. The method is based on the load factor concept, and as in any other design procedure, the choice of nominal loads (fire load density, live and dead load) and load factors will determine the final safety level.

The safety analysis of fire-exposed structures must begin with the procedure critical in every reliability evaluation; the assessment of underlying uncertainties.

Following the general outline of Fig. 1 in /1/, a general systematized scheme may be set up for the identification and evaluation of the various sources and kinds of uncertainty possible for a fire-exposed building component. Lack of space prohibits any attempt to account for the detailed process of data acquisition and evaluation, reference is made to /3/, where all particulars may be found. Here it can only be stated that, with Fig. 1 in /1/ as a functional basis and with the basic data variables selected (type of structural element, type of occupancy), the different uncertainty sources in the design procedure are identified and dissembled in such a way that available information from laboratory tests can be utilized in a manner as profitable as possible. The derivation of the total or system variance (R) in the load-carrying capacity R is divided into two main stages:

- variability $Var(T_{max})$ in maximal steel temperature T_{max} for a given design fire compartment
- variability in strength theory and material properties for known value of T_{max}.

Consecutively $Var(T_{max})$ is decomposed into three parts:

- equation error in the theory of compartment fires and heat transfer from fire process to structural component,
- variability in insulation material characteristics,
- possible difference between T_{max} obtained in laboratory test and in a real service condition.

In step number two, uncertainty in R for a given maximum steel temperature is, in the same way, broken down into three parts:

- variability in material strength,
- prediction error in strength theory,
- difference between laboratory test and a real life fire exposure.

These uncertainty terms must be superimposed upon the basic variability due to the stochastic character of fire load density. Mean and variance of load effect S are evaluated using results from publications covering the non-fire loading case.

To get appliable and efficient final safety measures, the reliability calculations are illustrated for the structural component, where the strength and deformation theories predicting the member performance under fire exposure seem most complete: an insulated simply supported steel beam of I-cross section as a part of a floor or roof assembly. The chosen statistics of dead and live load and fire load density are representative for office buildings.

The component variances are quantified, whenever possibel comparing the design theory with experiments. System variance is evaluated in two ways: by Monte Carlo simulation and by use of a truncated Taylor series expansion. Employing the Monte Carlo procedure, the mean and variance of R and S have been computed for different values of ventilation factor of fire compartment, insulation parameter κ and ratio D_n/L_n, where D_n = nominal dead and L_n = nominal live load used in the normal temperature design. The second moment reliability as a function of these design parameters is evaluated by the Cornell and Esteva-Rosenblueth safety index formulations /

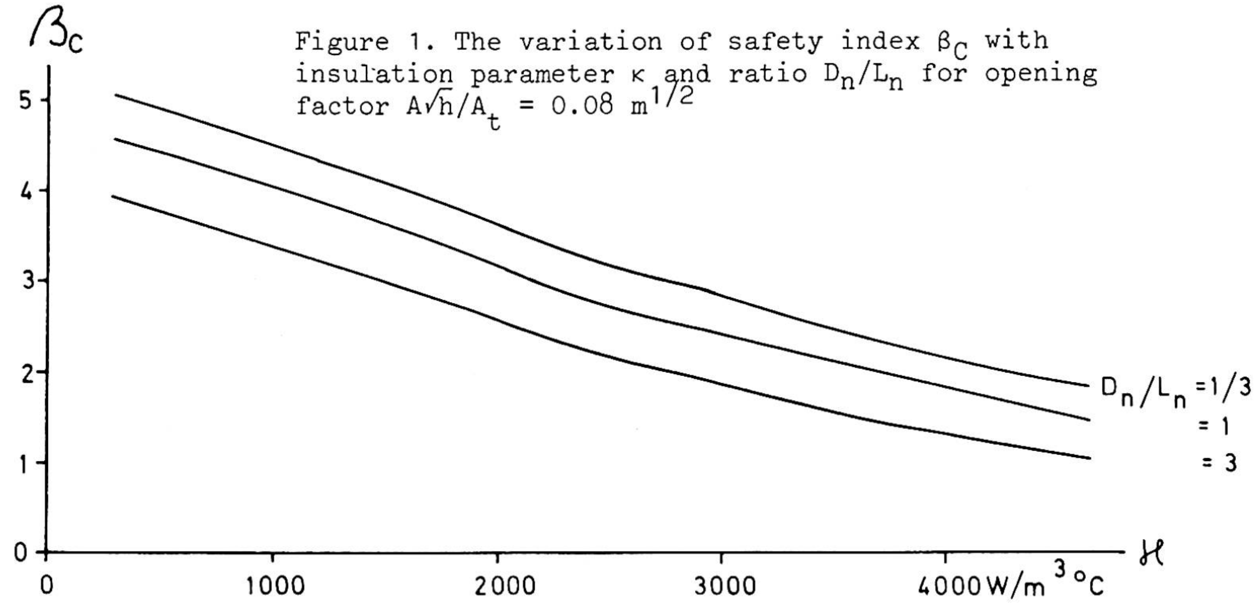


Figure 1. The variation of safety index β_C with insulation parameter κ and ratio D_n/L_n for opening factor $A\sqrt{h}/A_t$ = 0.08 $m^{1/2}$

Fig. 1 gives the safety index β_C,

$$\beta_C = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{1}$$

for an insulated, fire-exposed steel member as a function of κ. The insulation parameter κ is defined by

$$\kappa = \frac{A_i \cdot \lambda_i}{V_s \cdot d_i} \quad (W/m^3\ {}^oC) \tag{2}$$

where
A_i = fire exposed area of steel element (m^2/m)
V_s = volume of steel (m^3/m)
d_i = insulation thickness (m)
λ_i = thermal conductivity of insulation material (W/m ^oC)

Continuing the summary of /3/, the accuracy of the distribution-free second moment theories to uniquely define the reliability is touched upon, and the variation in safety-index value with varying uncertainty measures characterizing the insulation and the degree of complete combustion is examplified.

The Taylor series expansion method is compared with the Monte Carlo method and demonstrated to give surprisingly good agreement. The mathematical structure of the partial derivatives method makes it natural to use it as a basis for a closer investigation of how the total uncertainty in e.g. load-carrying capacity R varies with the uncertainties arising from different sources. Such information is necessary in a systematic study of how to economically optimize the avoidance of a structural failure.

Table 1 gives an example of such a decomposition. Of special interest is the variability inherent in the largely empirical design gastemperature-time curves, see /1/, /2/. The variance of these curves was measured by comparing design maximum steel temperatures with the corresponding experimental values for 97 natural fire-exposed insulated steel columns. The comparison was made for well-known thermal characteristics of the insulation material, but includes scatter due to the approximate heat transfer theory used in computing steel temperature values. From Table 1 it may be deduced that the uncertainties deriving from ventilation-controlled gas-temperature-time curves is of minor importance for the final safety index value.

The following section turns to the problem of comparing the reliability levels of the traditional and the new, differentiated design method. It is demonstrated how the flexibility of the new method results in drastically improved consistency for the failure probability P_f.

At the same time it is shown that the temporary nominal loads and load factors given by the manual /2/ do not result in reliability levels that are independent of the ratio D_n/L_n. Using the linearization factor defined by Lind, see /4/, it is examplified how statistically more consistent load factors easily may be derived. Finally it is pointed out how mathematical programming algorithms may be employed to obtain load factors or partial safety factors that for a broader range of design parameters minimizes the difference between the demanded, preselected and the actual reliability level.

These load factor evaluation studies underline a fundamental fact. In sharp contrast to the standard design procedure, the design model of Figure 1 in /1/ has the capability of being systematically and rationally improved as knowledge increases.

Summing up, this pilot study has demonstrated that a safety analysis, using probabilistic methods, of fire exposed structural steel components is today well within the bounds of possibility. The implication is that one of the main components in the over-all fire-safety problem for the first time has been rationally assessed, thus opening the way for an integrated system approach with a reliability optimization as final objective.

Table 1. Decomposition of the total variance of load-carrying capacity into a sum of component variances for an insulated steel beam designed according to the differentiated Swedish model

Variability in load-carrying capacity R due to	per cent of total variance
stochastic character of fire load density	36
uncertainty in insulation material properties	10
uncertainty in theory transforming fire load density into maximum steel temperature (theory of compartment fires and theory of heat transfer burning environment - structural steel component)	10
difference between laboratory test and an actual complete process of fire	2
uncertainty in yield strength of steel at room temperature	12
uncertainty in the deformation analysis giving the design capacity	11
difference between the impact of fire on R in laboratory test and under service conditions	19

References

/1/ PETTERSSON, O.: A Differentiated Approach to Structural Fire Engineering Design, IABSE 10th Congress, Tokyo, 1976.

/2/ MAGNUSSON, S.E. - PETTERSSON, O. - THOR, J.: Brandteknisk dimensionering av stålkonstruktioner (Fire Engineering Design of Steel Structures). Manual, issued by the Swedish Institute of Steel Construction, Stockholm, 1974. See also Magnusson, S.E. - Pettersson, O. - Thor, J.: A Differentiated Design of Fire Exposed Steel Structures, Bulletin No. 44, Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology, Lund, 1974.

/3/ MAGNUSSON, S.E.: Probabilistic Analysis of Fire Exposed Steel Structures, Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology, Bulletin 27, Lund, 1974.

/4/ ANG, A.H.S. - CORNELL, C.A.: Reliability Bases of Structural Safety and Design, Journal of the Structural Division, Vol. 100, No. ST9, Proc. Paper 10777, September 1974.

SUMMARY - A first attempt has been made to assess the reliability of fire-exposed steel structural member, using the available tools of modern safety analysis.

RESUME - C'est une première tentative pour évaluer la probabilité de rupture d'une construction en acier exposée au feu, en appliquant les moyens disponibles de l'analyse de sécurité moderne.

ZUSAMMENFASSUNG - An diesem ersten Versuch wird gezeigt, dass eine wahrscheinlichkeitstheoretische Auswertung der Brandsicherheit von Stahlbauteilen entwickelt werden kann.

The Analysis, Design and Remedial Repairs for a Fire Damaged Two-Way Roof Truss Structure

Calcul, projet et réparations d'une charpente métallique endommagée par le feu

Berechnung, Entwurf und Überholungsarbeiten an einer brandgeschädigten Dachkonstruktion

THOMAS S. TARPY, Jr.
Structural Engineer

STANLEY D. LINDSEY
President

JACK R. HORNER
Structural Engineer

Stanley D. Lindsey & Associates, Ltd.
Nashville, USA

1. Introduction

The structural problems presented by fire damage to a structure are numerous in that many effects not normally considered in the design of structures must be taken into account. These problems are especially acute with steel structures where the tensile strength and yield strength of the material decreases drastically at temperatures above 370°C. This temperature is easily reached in a building fire of short duration.

Several publications (1,2) and textbooks are available to assist the structural engineer in the consideration of thermal effects on various grades of steel at high temperatures. However, these references are primarily concerned with the ability of steel to withstand continuous sustained high temperatures and not to assess the performance of steel under continuous high temperatures for relatively short periods of time.

A major consideration which the structural engineer must take into account is when steel is subjected to high temperature, it expands, reducing the modulus of elasticity of the steel. As a result of expansion, additional forces are applied to adjacent restraint points located in cooler parts of the structure. This additional force can result in increases in stress or stress reversals in adjacent areas of the building.

To determine the full effect of a fire on structural steel, one must have a fairly good idea of what happens to the steel during such an exposure. Complicating the problem of determining the effects are numerous uncertainties such as:

a. Temperature attained by the steel is hard to determine and can only be estimated.

b. Time of exposure at a given temperature is unknown.

c. Heating is uneven.

d. Cooling rates vary and are often subjected to sudden quenching through contact with water as the fire is extinguished.

e. Steel is usually under load and restrained from normal expansion.

f. Microstructural changes in material properties are often uneven throughout a particular member.

2. Structural System

It is the intent herein to describe one approach to the analysis, design, and remedial repairs to a 106 meter by 106 meter roof structure due to a sudden, intense fire of short duration in the roof area. The structure is the physical education, athletic and convocation center at Middle Tennessee State University in Murfreesboro, Tennessee. The architects for the project are Taylor & Crabtree of Nashville, and the structural engineers for the roof structure are Stanley D. Lindsey and Associates, Ltd., of

Nashville. The roof framework is a symmetrical two-way truss system supported on four columns as shown in Figure 1. The structural system is considered a space grid to obtain the local distribution on each truss. The four main support trusses spanning between the four columns serve to distribute the load equally to the support columns.

The roof structure was analyzed as a two-way grid system under transverse loading, with the member moments and shears being applied to the joints of the corresponding trusses. Due to the symmetry of the roof framework, only one-eighth of the total grid was analyzed using standard matrix methods of analysis. Numerous grid loading situations were considered to determine the maximum stress within each individual truss member. The resulting truss elevations are shown in Figure 2. Volunteer Structures, Inc. of Nashville fabricated the steel to form individual sections, some 8.84 meters and others 15.24 meters in length, and all 3.96 meters deep. The individual sections were joined at the site to form one large square. Extensive use of U. S. Steel's EX TEN 50 high strength steel was made throughout the structure. A490 high strength bolts were used for the main truss connections.

3. Structural Fire Damage

During the construction stage of the project (after all steel trusses, bar joists, and roof decking were in place and with the majority of the dead load present), a flash fire broke out on a scaffolding platform adjacent to and just below one of the mechanical rooms. This was just to the side and at midspan of one of the main support trusses. While the fire was under control within thirty minutes, the heat in the roof reached a minimum temperature of 540°C causing a major reduction in the strength of the steel and expansion of several of the truss members. As a result of these changes, several members deformed, thus weakening the structure and causing it to be on the verge of collapse. Immediate action was necessary.

Upon receiving proper authorization to save the structure, temporary shoring and bracing were placed in the area of greatest damage. Before installation of the shoring tower could be completed, the roof structure gave a loud "crack," and the main truss dropped 5 to 7.5 centimeters, as later verified by measurement. The structure remained standing; however, a considerable increase in deflections was apparent. As soon as the main shoring towers were in place under the main support truss, the truss was jacked back up 2.54-3.8 centimeters in an effort to eliminate the large deflections and to relieve stresses in the truss. The problem then became one of trying to assess the extent of the structural damage by deciding which members were no longer effective; the extent of the stress redistribution; and, ultimately the structural soundness of the roof once the full live load was placed on the structure.

An inspection of the damaged area revealed the following physical changes:

a. The top chord of the main support truss had major flange buckling and lateral deformations.

b. Virtually all bar joists and bridging were damaged beyond repair.

c. The top chords of several adjacent trusses had warped stems.

d. Several diagonals composed of double angles had buckled.

e. Virtually all the miscellaneous support steel for the mechanical equipment was deformed.

The above changes plus the large deflected positions of the trusses in the area resulted in a structural system substantially different from the original design.

Based on microstructural studies of A36 steel from the area of the fire excessive grain growth did not occur. Hardness measurements on damaged material indicated that the mechanical properties were still in the acceptable range, and the A490 bolts appeared to be undamaged and should not have to be replaced. The exact temperature reached was not known; however, cooling curves of material which had been partially melted indicated the temperature reached at least 540°C and the maximum temperature was probably below 650°C or of very short duration. The problem was one of trying to analyze and correct the structure as best one could due to large deformations present.

4. Structural Repairs

An extensive analytical investigation into the structural problems presented by the damage to the roof structure from the fire was undertaken. A structural model was formulated which predicted reasonably well the behavior of the structure as defined by inspection and displacement measurements. This model was based on the original design model with the panel that buckled (top chord of truss 3A) being zero effective. By modeling the structure this way, while not an exact solution, the analysis yielded a set of design parameters which were an upper bound for existing and future member loads and thus assured that all areas of stress redistribution were adequately anticipated.

Once the structure model was developed, modifications to this model were made to determine action necessary to correct the damaged zone. While many different modifications were considered, only three approaches seemed feasible. These approaches were:

a. The possibility of reshoring and jacking the entire structure back to its original elevation and replacing those members, joists, bridging, etc. which were damaged by the fire.

b. The possibility of reshoring and jacking a portion of the structure around the damaged zone to its original elevation. Once this was done, these members, joists, etc., which were damaged by the fire could be replaced.

c. The possibility of reinforcing the structure in its current condition (i.e., at some intermediate elevation and braced by cables and shores as mentioned). Those members and connections which received more than their design load with the addition of live load would be reinforced, and the joists. bridging, etc., which were damaged by the fire would be replaced.

Our investigation showed that of the three different approaches, only the first and third approaches were feasible. These approaches, hereafter referred to as Option 1 and 2 respectively, are discussed herein. The second approach was not feasible due to the limiting capacity of commercial shores available to lift only a portion of the truss structure back to its original elevation and the serious stress reversal that would occur in adjacent truss members making reinforcing practically impossible.

The first option was that of reshoring and jacking the truss structure back to its original elevation and replacing those members, joists, etc., that were damaged by the fire. This procedure required the same shoring arrangements that were defined for the construction stage of the project. Once the truss was in its original elevation, the damaged members, joists, and bridging would have to be replaced. The buckled portion of the top chord truss 3A would have to be replaced while the top chord of truss 2 at mid-span of truss 6 and truss 7 would have to be reinforced. Also, it was decided that all high strength bolts in the top chord connections at the intersection of trusses 6 and 7 and truss 3 should be replaced individually once the roof structure is back to its zero elevation.

The apparent advantage of this option was that it returned the structural system to its original design before the fire with the exception of the reinforcement of the top chord of truss 2 and the two splices required to insert a new top chord section of truss 3. The deflections were then very close to the original design deflections. The apparent disadvantages were that the entire structure must be reshored back to its zero elevation and thus restrict work underneath the roof structure. This in turn could result in a delay in the project plan plus increase the labor involved in reshoring and jacking. Also, it should be noted that while the shoring pattern to raise the truss was defined, the jacking procedure was not well defined due to the new unsymmetrical deflection pattern. For this reason, the jacking procedure could result in stress reversals causing some tension members to buckle and as such must be monitored closely.

The second option was one of reinforcing the structure in its current state for live load and replacing the secondary members, joists, bridging, etc., which were damaged in the fire. Also, overstressed connections in both the top and bottom chord planes would need to be reinforced.

Since all members and connections must be reinforced to within the allowable stress for total design load, additional steel must be added to the truss in the overstressed areas. Likewise, the connections must be reinforced to carry the additional increase in force due to both the increase in dead load as well as the stress redistribution of the damaged structure. Once these corrections are made to the damaged structure, the system would be structurally sound. The only noticeable difference, in that the truss superstructure will be covered up, is that of an increase in deflection on the exterior facia under design loads.

The advantage of this option is that it results in a new structural system which is structurally safe without the addition of new shores. As such, work could continue underneath the roof structure. The disadvantages are that a large increase in pounds of steel would result in the damaged area in that 13 top chord members, 10 bottom chord members, 44 diagonal members, and 12 connections must be reinforced. This procedure results in the addition of approximately 25,373 kilograms of reinforcing members (plates, angles, and structural ties) plus the labor involved in this many corrections. Also, a non-symmetrical displacement pattern results.

Our investigation indicated that the two options to the correction of the damage to the truss structure as presented above were feasible and would result in a structural system which was structurally safe.

It was the concensus of all concerned that Option 2 would be the better of the two options. The corrections were made and the facility is now operational. The building has been subjected to approximately its full design load without unrealistic increases in deflections as predicted by the analytical model and based on later long term measurements.

REFERENCES

1. Fire-Resistant Steel-Frame Construction, 2nd Edition, American Iron and Steel Institute.
2. Manual of Steel Construction, 7th Edition, American Institute of Steel Construction.

SUMMARY

This paper describes the analysis, design and remedial work to a two-way steel roof truss damaged by a sudden, intense, flash fire of short duration in the main support area. As a result of the fire several members deformed, thus weakening the strucutre and causing the roof to be on the verge of collapse. The changes in the structural geometry due to permanent deformations, the resulting re-analysis of the roof frame, and the repairs required to return the structure to as close to the original design as possible are presented.

RESUME

Cet article décrit le calcul, le projet et les réparations d'une charpente métallique subitement endommagée par un feu violent et de courte durée dans la principale région d'appui. Le feu a déformé plusieurs membres et affaiblit l'ouvrage, de sorte que le toît s'est trouvé sur le point d'effondrement. L'étude traite les changements de géométrie de la structure dus aux déformations permanentes, l'analyse résultant de la charpente et les réparations à effectuer pour que l'ouvrage corresponde à nouveau, aussi fidèlement que possible, à sa conception d'origine.

ZUSAMMENFASSUNG

Diese Arbeit beschreibt die Berechnung, Entwurf und die nötigen Ueberholungsarbeiten an einer brandgeschädigten, dachtragenden Trägerkonstruktion. Die wichtigsten Tragelemente wurden durch kurzzeitige sehr intensive Feuereinwirkung geschädigt, so dass einige Tragelemente verformt wurden und die Gefahr des Einsturzes bestand. Die aus der Verformung resultierenden strukturellen Aenderungen wurden in die Neuberechnung aufgenommen; die nötigen Ausbesserungsarbeiten, um die Konstruktion der alten soweit als möglich anzugleichen, werden näher beschrieben.

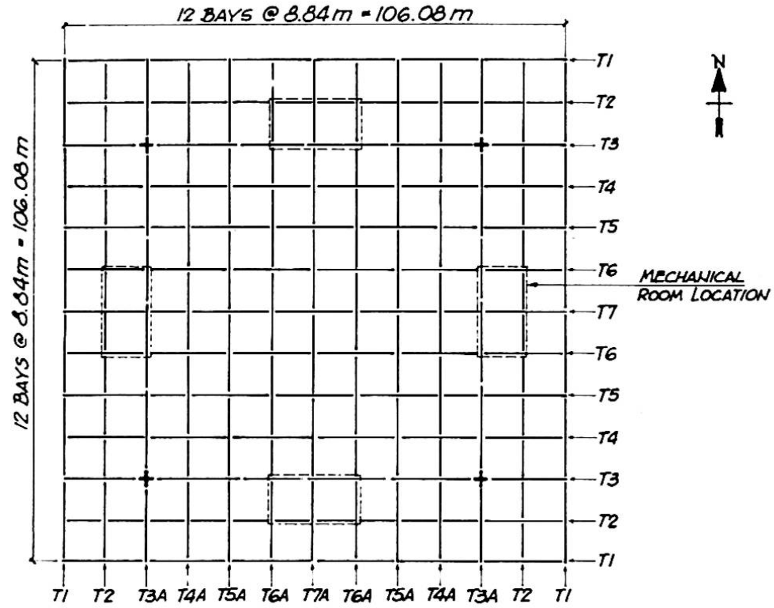


FIGURE 1. ROOF FRAMING PLAN

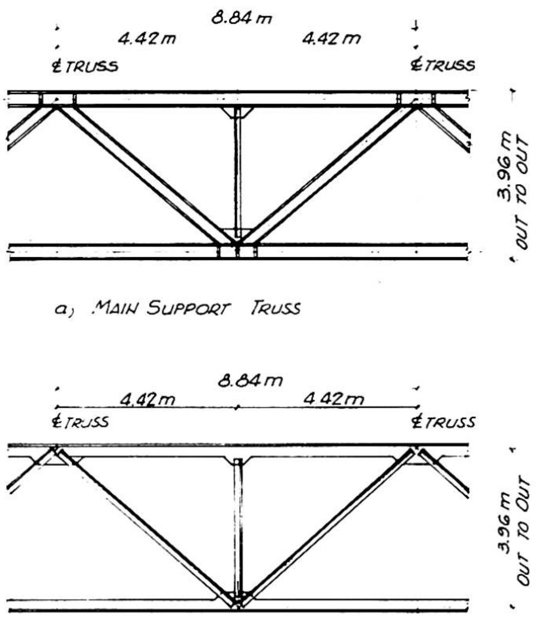


FIGURE 2. TRUSS ELEVATIONS