Critical Temperature and Fire Resistance of Steel Columns

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submitted in partial fulfillment of the

Master of Applied Science Degree in Civil Engineering

at the

University of Ottawa

Ottawa, Canada

April 20, 1984

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UNIVERSITY OF OTTAWA

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Acknowledgements: The author would like to thank Prof. C. Berwanger, her thesis advisor; T.T. Lie, National Research Council of Canada; Dr. D.C. Stringer, AMCA International; M.I. Gilmour, CISC; Dr. N.J. Gardner, University of Ottawa; and her husband Charly. Without the advice and encouragement of these gentlemen, this thesis would not be complete.

1. BACKGROUND and LITERATURE SURVEY

Provision of structural fire resistance is an important component of the design of modern buildings. Although loss of life in building fires over the past 50 years has been more often due to factors other than structural collapse (for example, the spread of combustion products), assemblies must provide some resistance to fire to allow adequate occupant escape time and to minimize property damage.

Little was known about the behaviour of structural assemblies in fire before the advent of the standard fire resistance test. Massive protection, consisting of concrete, masonry or tile was very common. The standard fire test (1) provided the benchmark against which the performance of various assemblies could be measured. In such a test, a life-sized structural element is loaded and supported in a manner similar to its projected use in a structure and then exposed to a standard furnace fire. This fire, prescribed by a time temperature curve (1), (Figure 1) arbitrarily resembles a fire that might occur in one room of an office building and is the international benchmark against which all fire rated structural assemblies have been tested over the past 60 years.

The time during exposure to this fire at which a given assembly fails by collapse, loss of integrity or excessive transfer of heat is termed its "fire resistance rating." It is on the basis of test fire resistance rating that code requirements for fire safety design have been based.

A typical fire resistance test costs on the order of $10,000. It yields one result: the fire resistance rating assigned to one particular assembly. The accepted source of fire rated assemblies is the List of Equipment and Materials, Vol. II, Building Construction (2), and similar American and international publications. Through experience gained from years of testing, rules of thumb have been developed to help ensure the successful rating of proposed test assemblies. This process has led to the development of calculation procedures to predict fire resistance, to reduce the cost of testing and to provide a rational basis for the prediction of the behavior of actual building components in real building fires.

As mentioned above, there are three fire test failure criteria. Any one of these three may govern the fire resistance of an assembly, with the exception of columns and beams in which only the last criteria applies.

The following is a review of sources of information and calculation procedures predicting the fire performance of structural assemblies with reference to each failure criteria. The focus of this review, reflecting the thesis topic, is steel assemblies and, in particular, steel columns.

The rise of temperature on the unexposed surface, which may govern the fire resistance of steel assemblies acting as barriers to fire, such as walls, roofs and floors, is generally a function of the insulative value of the materials protecting the steel. Steel has a very high thermal conductivity and specific heat compared with most common building materials (3) and thus does not provide good insulation value. Supplement No. 2 to the National Building Code of Canada (4) lists the contribution of various protection materials to the fire resistance of steel walls, floors and roofs.

Calculation procedures developed to predict fire resistance governed by temperature rise on the unexposed surface are naturally based on heat transfer principals. Since the fire resistance test is a non-steady state problem (the fire temperature varies with time) simple heat transfer equations don't describe the process very well. Therefore, simple scaling formulas have been developed relating the fire resistances of two assemblies which differ in one or more respects(5). Reference (6) describes an example of this type of formula called the 1.7 power law, which relates the fire resistances of slab or wall assemblies with different thicknesses and thermal conductivities of insulation.

When no reference tested assembly is available, or if more accuracy is required, the heat transfer equations can be represented by analytical charts (7), which give the unexposed surface temperature of slab type constructions, given the thermal conductivity and thickness of the slab. Finally, if a more complex assembly is under study, or if the variation of thermal conductivity with temperature is important, finite difference computerized heat transfer analysis can be used (7). For walls and slabs, the elements are one dimensional.

Structural steel, like all metals, expands when heated (approximately 1% over a 1000^{o}C change in temperature). This expansion can be beneficial; for example, if floor assemblies are restrained from axial expansion during fire, lateral deflection can be reduced. However, in other situations, lack of expansion joints and other detailing can cause cracks and openings in protection materials leading to loss of integrity. For example, in steel wall systems, studs may buckle and board protection may fall off if gaps are not provided between the top of the studs and the test frame. Another factor to consider in assessing the potential for loss of structural integrity in a fire test assembly is the behaviour of the protection materials themselves at elevated temperature. Temperatures rise sharply in fire and can cause shrinkage and thermal shock in some materials. Presence of moisture in materials can also affect integrity. Too much moisture, reflected in high pore pressures, can result in destructive spalling.

Thus, as calculation procedures gain acceptance, there will always be a need to carry out small scale and large scale tests to evaluate the performance of new materials and systems in fire, in particular with regards to their ability to maintain structural integrity.

The third fire test failure criteria is maintenance of load without structural collapse. The fundamental principal behind all methods designed to predict the structural failure of steel assemblies in fire is the fact that steel gradually loses strength and stiffness at elevated temperatures (8). Room temperature structural steel design procedures, in Canada CSA S16.1M-1980(9), dictate the allowable load for steel members, given the mechanical properties of the steel and incorporating a certain safety factor on strength and load against collapse. As a steel member is heated in fire, it may reach the temperature at which its strength and stiffness decrease to the point that this safety factor is reduced to zero. This temperature, which is a function of the type of member, its end support conditions and the load applied, is known as the critical temperature of the member. The critical temperatures of structural steel assemblies under different conditions has been the subject of much research and many fire resistance tests and has resulted in, for example, the failure criteria for beams and columns in the fire test standard, ULC S101 (1). Thus for common steel asssemblies under full design load, calculating fire resistance means calculating the time at which the members will reach their critical temperature. Thus structural considerations are reduced to simpler temperature predictions and heat transfer methods can once again be used. One caution: the critical temperatures in ULC S101 refer to common but specific loading and boundary conditions. Critical temperatures do vary with these parameters.

1.1 The Fire Resistance of Structural Steel Columns

Since columns do not act in themselves as barriers to the spread of fire, the only failure criterion applicable to them in a fire test is structural collapse under load. The first standardized column fire tests were carried out in 1920 at the U.S. National Bureau of Standards (10). Results indicated that most steel columns collapsed under design load at average temperatures above 538°C. Thus, in 1947 an alternate standard fire test method for steel columns was adopted which has been in universal use since that time (1). It prescribes an unloaded test where the failure criterion is the attainment of the "critical temperature" of 538°C. Supplement No. 2 to the NBCC, pages 44-48 (4), lists the thicknesses of various protection materials required to prevent the attainment of 538°C in columns for various fire resistance periods.

Procedures that have since been developed to calculate steel column fire resistance are based purely on thermal analysis: predicting the time of exposure to the standard fire when the average steel temperature will reach 538°C. This analysis is based on the fact that the column is exposed on all four sides and that all heat transferred is absorbed by the mass of the column and by its protection. These principals have resulted in the M/D concept for steel columns (11) which states that the fire resistance of a column is proportional to its mass and inversely proportional to its heated perimeter. The determination of column heated perimeter is illustrated in Figure 3.

Based on M/D concepts, scaling formulas similar to the wall formula referred to earlier, have been developed relating the fire resistances of columns of different sizes with the same protection material(6). These have served as a base for the development of various semi-empirical formulas predicting fire resistance of columns with specific protection materials which have been verified with many fire resistance tests (7),(11).

Analytical charts can be used to solve the column heat transfer problem if the above formulas do not apply or if more accuracy is required (7). And again, if variations in thermal conductivity are important and further refinement required, finite difference computer programs have been developed to solve the problem (12). In this case they are two-dimensional in nature.

All the prediction formulas are based on the concept that the attainment of the critical temperature of 538°C signifies collapse. As mentioned before, this temperature is actually a function of the load applied to the column and its end restraint conditions.

In certain situations, steel columns need no protection; the mass of steel in itself is sufficient to absorb the heat transferred to it. Calculation procedures have been developed (13) to calculate the mass of steel required to provide resistance to collapse for various fire resistance periods. Here the critical temperature concept is clearly not appropriate.

Since the 1940s our knowledge of the factors affecting column behaviour, both at room and elevated temperatures, has increased to the point where the gross simplification of a universal critical temperature is no longer sufficient or necessary. Calculation procedures for predicting column failure at elevated temperatures are now under development and verification in Europe using recently constructed loaded column fire test facilities (14)(15). In addition, for the first time in 50 years, a North American column fire testing facility has been constructed with loading capabilities. This facility, at the National Research Council in Ottawa, provides an opportunity to evaluate the behaviour of loaded columns exposed to the standard fire.

2. OBJECTIVE

The objectives of this study are, first of all, to develop a computer analysis technique designed to study the influence of load and slenderness ratio on the critical temperature and fire resistance of wide flange steel columns and secondly, to verify the technique with a full scale fire test.

3. PREDICTION OF FIRE RESISTANCE

The prediction of the behaviour of a steel column (or any structural assembly) in fire is a two-step process. First, a thermal analysis must be carried out to determine the temperatures reached by the column at any time during the fire. Then the structural response of the column to this temperature must be assessed. Normally, these two steps are performed separately; in other words, it is assumed that structural behaviour does not influence heat transfer to the column during the fire. (This implies that the protection materials are not dislodged by deformation of the column they are protecting.) A further assumption that is commonly made is that structural behaviour is time independent.

3.1 Thermal Analysis

Lie and Harmathy have developed a two-dimensional numerical heat flow analysis technique for predicting the temperature a protected steel column will reach during a simulated fire test (12). This computerized finite difference method makes the following assumptions about heat transfer mechanisms:

- o Heat transfer from fire to column is radiative,
- o Heat transmission through protection material is conductive,
- o Thermal resistance between protection and steel is negligible, and
- o Steel temperature is uniform over the cross-section.

Thermal properties of the protection materials, which vary with temperature, and the geometry of the steel section are required input for the analysis. Output is the predicted average steel temperature history. The technique is explained in detail in reference (12).

3.1.1 Thermal Properties of Protection Materials

Thermal conductivity and specific heat are the two properties of relevance to this heat transfer analysis problem. The former reflects a material's ability to transmit heat; the latter indicates its ability to store heat.

These properties of building materials and their variation with temperature have been extensively researched (3) but there is information lacking on the properties of common fire protection materials, especially at high temperatures.

The National Research Council of Canada has developed equipment and procedures to measure these properties at elevated temperatures (16). A typical dataset is shown in Figure 2 (17). A limitation of the method is that the maximum temperature of measurement is 600°C.

3.1.2 Steel Geometry

The properties of a steel section which influence heat transfer to it are its mass and its heated perimeter, the perimeter of steel enclosed by a protecting material (11). Figure 3 demonstrates the heated perimeter concept.

3.2 Structural Analysis

A finite element computer program designed to analyse the behaviour of a pin-ended wide flange steel column under eccentric axial loading was developed by the author. Based on a program written earlier by Lie (18) to analyse the behaviour of reinforced concrete columns, it follows the following procedure for a column of a given wide-flange geometry, length, initial imperfection at midheight, made from a steel with given stress-strain properties, at a given temperature.

1. An initial midheight lateral deflection is assumed.
2. An axial strain is assumed.
3. Thermal, axial and bending strains are summed for each element.*

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*Thermal strains are those resulting from the expansion of the column and are a function of its temperature. The program assumes that this expansion is completely externally unrestrained. Thus, stresses will result only if there are thermal gradients across the section.

4. Elemental stresses are calculated from stress-strain relationships.
5. Total section load and moment are calculated (stresses x areas and moment arms).
6. The value of load (deflection + initial imperfection) is compared with the value of moment and steps 2-5 are repeated with a new axial strain until equilibrium is achieved.
7. Steps 1-6 are repeated, incrementing the midheight lateral deflection until the total load drops off, signifying failure.
8. Output from the program includes elemental stresses and strains, total axial strain and lateral deflection as a function of load.

Appendix A is a description and listing of the program and a sample input. Reference (19) describes a similar program developed at the Column Research Council in the early 1970s.

3.2.1 Mechanical Properties of Steel at Elevated Temperatures

The elastic and yield properties of steel and their reduction with increasing temperature have been well documented (3),(8),(20). In general, there is a wide spread in this data (11). Traditionally, this information has been obtained through small scale high-strain-rate tensile tests carried out at constant temperature. Methods of predicting structural behaviour in fire (including those described in this thesis) are generally time independent; yet creep deformation can be significant at the temperatures reached by steel in fire.

Recently, small scale, constant-load, compression tests were carried out by European researchers (14) wherein the specimens were heated at 5-25°C/min (a typical heating rate for a full size protected steel column in a standard fire test) until failure. The resulting yield strength and elastic modulus versus temperature relationships (Figures 4 and 5) thus implicitly include the influence of creep deformations likely to occur during the fire test period.

Lie in his work on reinforced concrete columns presented stress-strain curves for steel based on small scale high temperature compression tests (21) which provide for stiffness above the yield point (Figure 6).

3.2.2 Verification of Analyses Results

A sample output of computer analysis in the form of load versus lateral deflection behaviour of a column of intermediate slenderness (KL/r = 52) is shown in Figure 7. Input to the analysis was the room-temperature stress-strain curves of Figure 6. This output is compared in the figure with elastic analysis and plastic theory predictions of column behaviour. There is close correlation between them. Appendix B outlines the elastic and plastic theory calculations.

3.3 Influence of Residual Stress

It has been well documented (21) that residual stresses induced in the flange tips of wide flange steel column shapes during the hot rolling process have a pronounced effect on beam-column strength. In order to examine the influence of residual stress on high temperature column behaviour, the residual stress pattern shown in Figure 8 was imposed, through computer analysis, on a column of intermediate slenderness in the form of initial residual elemental strains. Figure 9 shows the predicted influence of these stresses on the load deformation behaviour of the column at room temperature and at 350°C. It is evident that although the load-deformation path is affected, the maximum load achieved by the column is minimally so. This can be understood by the small lateral deflections at which this column reached failure (less than 10 mm). Residual stress only affects the bending behaviour of the column: this column is almost under pure axial load.

Stress relieving of steel members is usually carried out at temperatures greater than 400°C. Therefore, the influence of residual stress is expected to be small for columns of critical temperatures above this value.

3.4 Prediction of Critical Temperature

The critical temperature of a column is that temperature at which its capacity is exceeded by the load applied. The capacity of a given column at any given temperature may be calculated using the numerical technique described above with stress-strain curves (Figure 6) (page11) as input. It may also be calculated using room temperature column design resistance equations, such as those in CSA S16.1M (9) (page 11) (excluding the resistance factor), using the relationships between FY, E and temperature shown in Figures 4 and 5. Figures 10 and 11 shows results of both these calculations in the form of elevated temperature buckling curves. The stress strain curves from Lie's work (18), (Figure 6), a design out-of-straightness of L/1000 and a uniform temperature over the cross- section were assumed in the analysis for the purpose of these curves.

Examining Figures 10 and 11 it is apparent that the two sets of curves bear little resemblance to one another, particularly in the low KL/R range. There are two apparent major reasons for this discrepancy:

(1) The S16.1 buckling curves are based on elastic-plastic stress-strain assumptions whereas the elevated temperature stress-strain curves shown in Figure 5 have an upward slope beyond the elastic limit; and

(2) The S16.1 buckling curves include the influence of residual stress which is particularly significant for columns of low slenderness ratio.

3.4.1 Influence of Initial Eccentricity

Figure 12 shows the influence of assumed initial column eccentricity on analysis derived buckling curves at room temperature and at 600^{o}C. It appears that increasing the initial eccentricity from 2.5 to 10 mm (over a base length of 2 m used to derive the curves) decreases strength by approximately 10 percent at both temperatures.

3.5 Prediction of Fire Resistance

Using either of the sets of elevated temperature buckling curves shown in Figures 10 and 11, the temperature at which a given column will fail under a given stress may be determined. Carrying out a thermal analysis of the column will determine the time during the fire test that the column will reach this temperature: its fire resistance.

Equations of Stress-Strain Curves

yield strain = $4\times10^{-12}\times F_y = 1.2\times10^{-3}$

stress = E×strain if strain ∠ yield strain

$E = 6.9\times10^{9}\times(50-0.04\times\text{Temp})\times(1-e^{(-30+0.03\times\text{Temp})(\sqrt{0.001})})$

if strain > yield strain

$\text{stress} = E\times0.002 + 6.9\times10^{6}\times(50-0.04\times\text{Temp})\times(1-e^{(-30+0.03\times\text{Temp})(\sqrt{\text{strain}-0.0002})})$

where Temp = temperature of steel

CSA S16.1 Buckling Curve Equations

(a) $0 \leq \lambda \leq 0.15$, $C_r = \phi A F_y$

(b) $0.15 < \lambda \leq 1.0$, $C_r = \phi A F_y\,(1.035 - 0.202\lambda - 0.222\lambda^2)$

(c) $1.0 < \lambda \leq 2.0$, $C_r = \phi A F_y\,(-0.111 + 0.636\lambda^{-1} + 0.087\lambda^{-2})$

(d) $2.0 < \lambda \leq 3.6$, $C_r = \phi A F_y\,(0.009 + 0.877\lambda^{-2})$

(e) $3.6 < \lambda$, $C_r = \phi A F_y\,\lambda^{-2} = \phi A\left[\frac{1\,970\,000}{(KL/r)^2}\right]$

where

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$$

4. TESTING

A full scale loaded column fire test was carried out at the Fire Research Laboratory of the National Research Council of Canada.

4.1 Test Specimen

The test specimen was a 3.8 meter long, W150 by 37 wide flange steel column, obtained from a local fabricator. Column dimensions, specified (22) and measured are listed in Table 1.

The steel was specified CSA G40.21M 300 MPA. Appendix C is a copy of the Mill Test Report. Standard tensile tests were carried out according to CSA G40.20 (23) on coupons cut from the web and flange of a stub of the column. The test results are listed in Table 1 with specified (22) minimums. It appears that the column was understrength.

The ends of the column were cut and milled by NRC staff and 400 by 610mm, 25mm thick top and bottom cap plates were welded to them. Figure 13 shows the column before protection was applied.

4.1.1 Thermal Protection

"Vicuclad," an inorganic bonded vermiculite board, 25 mm thick, was selected for the column fire protection. Although the product is not widely used in Canada, it is uniform, durable and has stable thermal properties at elevated temperatures. Figure 2 shows the variation of its thermal conductivity and specific heat with temperature. It is applied to wide flange steel columns in a box shape using noggings and a special high temperature glue. Figure 14 shows the protected column.

4.1.2 Instrumentation

Thermocouples were peened onto the surface of the column at locations shown in Figure 15 to measure the variation of steel temperature with time over the column height. (A uniform steel temperature is a common assumption in predicting fire resistance.) In order to measure the column's strain response to loading at room temperature, strain gauges were welded to the cross-section of the column at midheight at locations shown in Figure 15.

4.2 Determination of Effective Length and Selection of Test Load

The unsupported column length in the test furnace was 3.8m. The column ends were rigidly restrained against rotation in the loading facility by the furnace frame (Figure 16). A calculation of the relative stiffness of the column and the furnace restraining beams indicates a K factor of close to 0.5 (Appendix E). However, in reality, there would be some movement in the column to beam bolted connections which would increase the theoretical K.

The heated length of the columns was approximately 3.2m. At high temperature, the stiffness of the unheated column ends relative to the heated portion of the column was great enough that the ends would contribute to a reduction in the column effective length. Lie in his studies on reinforced concrete columns (7), has found that selection of an effective length of 2m or 0.53 of the total length agrees with experimental behaviour.

The test column load was 742KN, factored room temperature capacity (S16.1 resistance equations (13)), divided by 1.4, a conservative load factor based on a live to dead load ratio of 1.5. Appendix D shows the load factor calculations.

4.3 Column Testing Facility

The test facility (24) is illustrated in Figures 17 and 18. It consists of a furnace chamber inside a steel framework supported by four columns. Loads are applied by a 9790KN capacity hydraulic jack mounted at the base of the column. The loading rate of the jack is manually controlled and its fluid pressure is periodically calibrated by a load cell. The travel of the jack is measured by an LVDT with an accuracy of 0.002 mm and a maximum range of 150 mm. Two other horizontal jacks prevent lateral movements of the top of the column. The top and base of the framework are heavily laterally braced.

The furnace chamber, 2.6m square by 4.3m high, is constructed with insulating fire brick and lined with ceramic blanket. The fire is fuelled by 12 propane gas burners symmetrically distributed along the chamber walls. Furnace temperature is measured by 9 chromel-alumel thermocouples installed 0.3m from the test specimen at 0.6m intervals in height. The feedback from the thermocouples is used to control the burners to guide the furnace temperature along the standard time temperature curve (Figure 1).

4.4 Test Procedure

The column was placed in the furnace on a plaster of paris levelling seat and levelled by plumbing and shimming as required. The top and bottom plates were bolted to furnace bolting and head plates (Figure 17). The column and furnace thermocouples were connected to a datalogger; the travel of the loading jack, measured by the LVDT, was recorded by the datalogger every minute. Visual observations of column behaviour were made periodically through furnace portholes.

The column was loaded to test load at room temperature; loading was controlled manually and averaged 4.7 KN/min. The test load was held for an initial 45 minute period at room temperature. The furnace was then turned on and the standard time-gas temperature curve was followed. The test was terminated when the test load on the column could no longer be maintained.

5. COMPARISON OF TEST RESULTS AND ANALYSIS PREDICTIONS

The column failed at 90 minutes at an average steel temperature of 563°C and a total axial expansion of 17 mm. Figure 19 shows the column after test.

5.1 Thermal Behaviour

The thermal analysis technique described above was used to predict the time-steel temperature history of the test column exposed to the standard fire. Input to the analysis was the thermal properties of the "Vicuclad" protection, shown in Figure 2, and the heated perimeter value shown in Figure 3. The results of the analysis in comparison with test results are shown in Figure 20. Computer analysis predicted that the column would reach the failure temperature (563°C) 20 minutes before the actual test failure. This conservatism may be due to the thermal property data used for analysis input, obtained using the thermal property measuring technique (5) described in section 3.1. Fire temperature reached 1000°C; properties of the protection material above the 600°C measurement limit were linearly extrapolated for input.

Table 3 lists the average thermocouple readings at each measurement height (Figure 15). Temperatures varied with height by a maximum of 20°C.

5.2 Structural Behaviour

The structural analysis technique described above was used to predict test column behaviour. In carrying out the structural analysis, Lie's stress-strain curves (Figure 6), a uniform temperature over the cross-section, an effective length of 2m and an initial eccentricity of 2.5mm (L/1500) were used as input. The lower initial eccenticity reflects the accuracy of the test furnace compared with field construction.

Figure 21 shows typical outputs from the structural analysis program: the load deflection history of the column at various temperatures. For all analyses run, failure (load decrease) occurred at or near the 10mm lateral deflection point.

Figure 22 shows the predicted axial strain versus average steel temperature curve, compared with the equivalent test measurement, vertical travel of the loading head. The initial negative strain results from compressive loading;

the positive strain results from the sum of loading strain and thermal expansion. It is apparent that the column sustained much higher axial strains than predicted.

Table 4 lists the strains recorded across the column cross-section at midheight during loading prior to the fire test.

5.3 Critical Temperature

Both structural analyses and the elevated temperature buckling curves derived from the S16.1 resistance equations were used to predict the critical temperature of the test column under the test load. These values are listed in Table 2 in comparison with the test critical temperature. Note that the two predictions differ substantially from one another. The test column is in the low KL/R range, where the two sets of buckling curves, Figures 10 and 11, differ.

Both predictions are very conservative in comparison with test results. Since both are based on the same elevated temperature steel property data, it is reasonable to assume that this data does not reflect full size column behaviour.

5.4 Fire Resistance

Combining the prediction of critical temperature with the thermal behaviour prediction in Figure 20 yields the predictions of fire resistance listed in Table 2. Some of the lack of correspondence between test and predicted results may be due to the furnace design. Recent round-robin testing in Europe (10) has shown a wide variability in results of loaded column tests on identical specimens. Some of the conditions contributing to this variability which are difficult to measure are: variations in heated column length, uneven heating over the column length, heat transfer to the column from the furnace, variability in column end fixity and degree of initial eccentricity in loading. Some of these conditions also occur in column testing at room temperature. However, measurement of lateral displacement and strain gauging can determine the magnitude of their influence on test results. Strain gauges which are accurate in 1000^{o}C environments are available but cost on the order of $600 each. The measurement of lateral displacement cannot be carried out by conventional plumb line techniques or transit measurements in hot environments. A laser displacement measuring device is currently under development (16).

Because of the variability in all the above factors in what might be assumed to be identical tests, a true evaluation of a predictive model should consist of a series of fire resistance tests. An extension of this test series is planned for the future and includes five more column tests in which column slenderness and load applied will be varied.

6. SUMMARY AND CONCLUSIONS

The structural behaviour of building elements exposed to fire has been the subject of much research, largely in the form of full scale performance tests. Recently, calculation procedures have been developed to predict this behaviour. The subject of this study was the fire resistance of structural steel columns as determined by thermal and structural analysis and verified by fire test.

In the study, a two dimensional finite difference type thermal analysis is described which predicts the temperature history of a steel column exposed to a standard fire. A structural analysis technique was adapted from a previously developed reinforced concrete model to predict the structural behaviour of a wide flange steel column's behaviour under load at elevated temperatures. The technique is a finite element model which assumes a given geometry, pin-ended length, initial imperfection at midheight and includes the effect of initial eccentricity, thermal and bending strains and optional residual strains. The inputs to both thermal and structural analysis techniques (thermal and structural properties of the protection and steel) are discussed. Results compare closely with conventional theory at room temperature.

The models were used to predict the results of one loaded fire resistance test on an insulated wide flange steel column, carried out as part of the study. The temperature of the column, its strain under initial loading, and its overall axial displacement and fire resistance were measured. Assumptions were made regarding its effective length and resulting design load. There was poor agreement between test results and model predictions. In order to accurately predict fire test behaviour, the elevated temperature properties of steel and protection materials, and furnace loading and heating conditions must be better understood.

This study recommends a further series of tests to determine the influence of the above factors on the fire resistance of steel columns.

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18. Lie, T.T., "A Procedure to Calculate Fire Resistance", International Seminar, Three Decades of Structural Fire Safety, Borehamwood, England, 1983.

19. Ossenbruggen, P.J., V. Aggarwaal and C.G. Culver, "Steel Column Failure Under Thermal Gradients", American Society of Civil Engineers, NSE Meeting, Preprint 1643, Cleveland, April 1972.

20. Ingberg, S. and T. Sale, "Compressive Strength and Deformation of Structural and Cast Iron Shapes at Temperatures up to 950°C", American Society for Testing and Materials Proceedings Vol. 26, 1926.

21. Huber, A.W. and L.S. Beedle, "Residual Stress and the Compressive Strength of Steel", Welding Research Supplement, December, 1954.

22. Handbook of Steel Construction, December 1980, Willowdale, Ont.

23. CSA G40.20-M1978, "General Requirements for Rolled or Welded Structural Quality Steel".

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Column Dimensions

	w, mm	t, mm	b, mm	d, mm
W150 by 37 specified	8.1	11.6	154	162
test column	8.2	11.6	152	162

Results of Coupon Tests

Coupon	area, mm^2	Yield Strength MPa	Modulus of Elasticity MPa	Ultimate Strength MPa
web	104	284	183000	445
flange	149	282	149000	470
specified minimum		300		450

Table 1

Steel Data

Test Column Area = 4722mm^2,	kl/r = 52, Test Load = 742 KN

Predicted Performance

Prediction Method	Critical Temperature °C	Fire Resistance minutes
Computer Analysis	325	42
Buckling Curves	425	52
Test Results	563	90

Table 2

Comparison of Predicted and Test Results

Table 3

Variation of Column Temperature with Height

time min	furnace average °C	1 meter average °C	2 meter average °C	3 meter average °C	overall average °C
0	63	25	25	25	25
15	748	71	74	71	72
30	837	166	170	155	164
45	890	287	288	273	283
60	926	395	396	382	391
75	948	488	492	475	485
90	974	566	570	553	563

Table 4

Strain Gauge Measurements

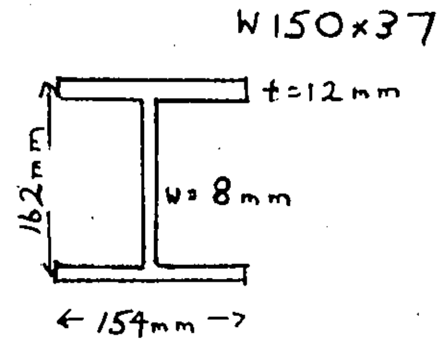
Load KN	Stress Mpa	Modulus of Elasticity Avg, Mpa$\times10^6$	Microstrains Corresponding to Each Strain Gauge Number				Time min,sec
			1	* 3	4	* 6	
0	0	0	0	0	0	0	0
180	38	19.4	1.22	4.39	0.8	1.39	0,55
250	53	20.7	1.43	4.08	1.99	2.75	3,15
321	68	18.6	2.68	5.58	2.77	3.56	5,30
392	83	22.4	3.18	3.95	3.49	4.21	7,30
461	98	27.8	3.7	1.79	3.88	4.76	9,10
532	113	26.2	4.76	2.18	4.61	5.74	10,55
602	127	24.7	5.45	3.82	5.06	6.27	12,30
672	142	27.7	6.03	2.13	5.12	7.18	13,55
742	158	22.8	6.62	6.84	6.36	7.92	15,40

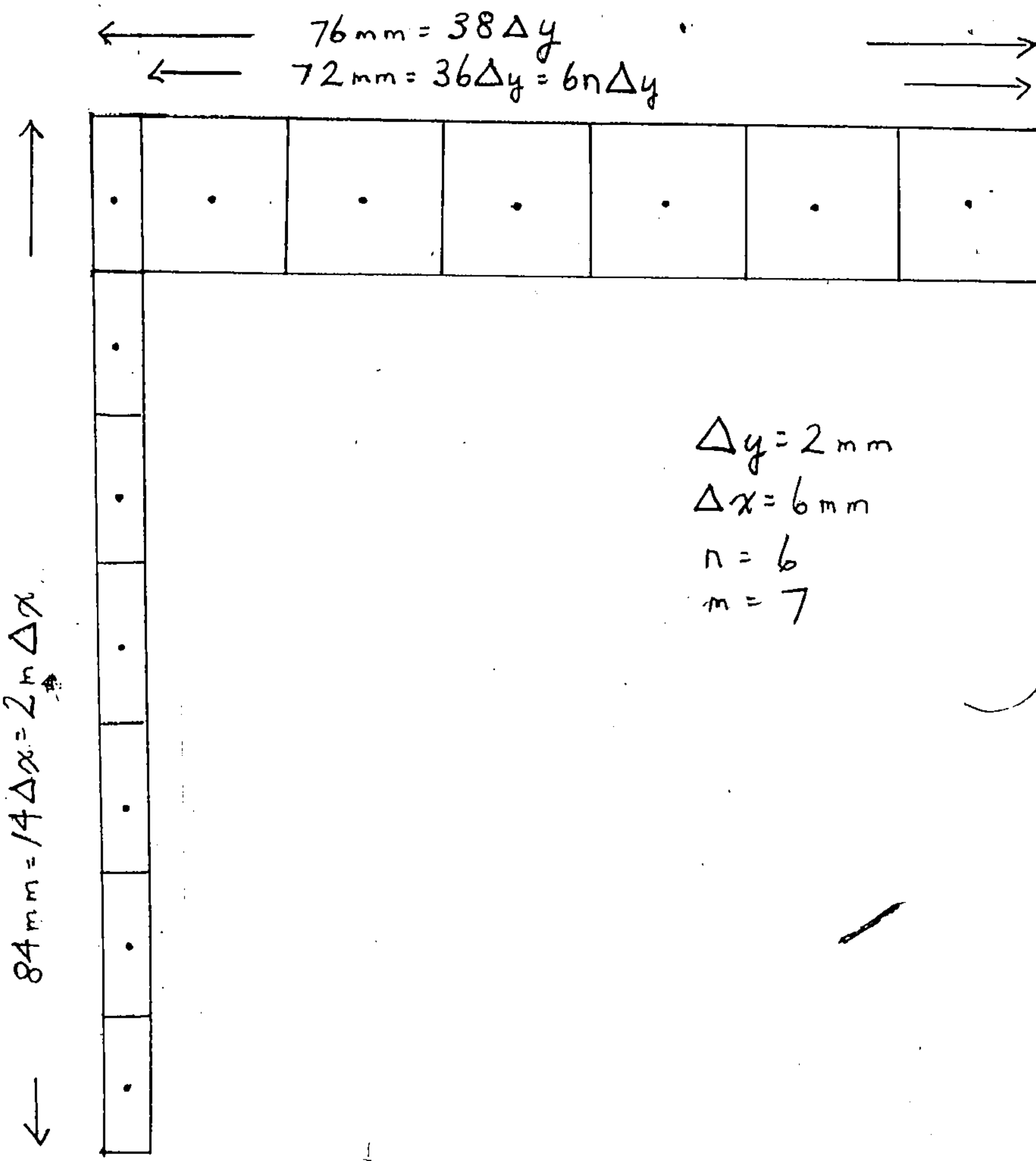
Average Modulus of Elasticity = 23.4×10^6 mPA

* no readings from these strain gauges

APPENDIX A

Finite Element Matrix





$\Delta y = 2$ mm

$\Delta x = 6$ mm

$n = 6$

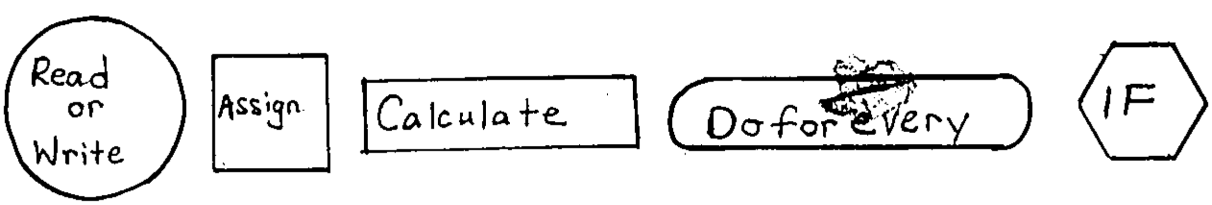
$m = 7$

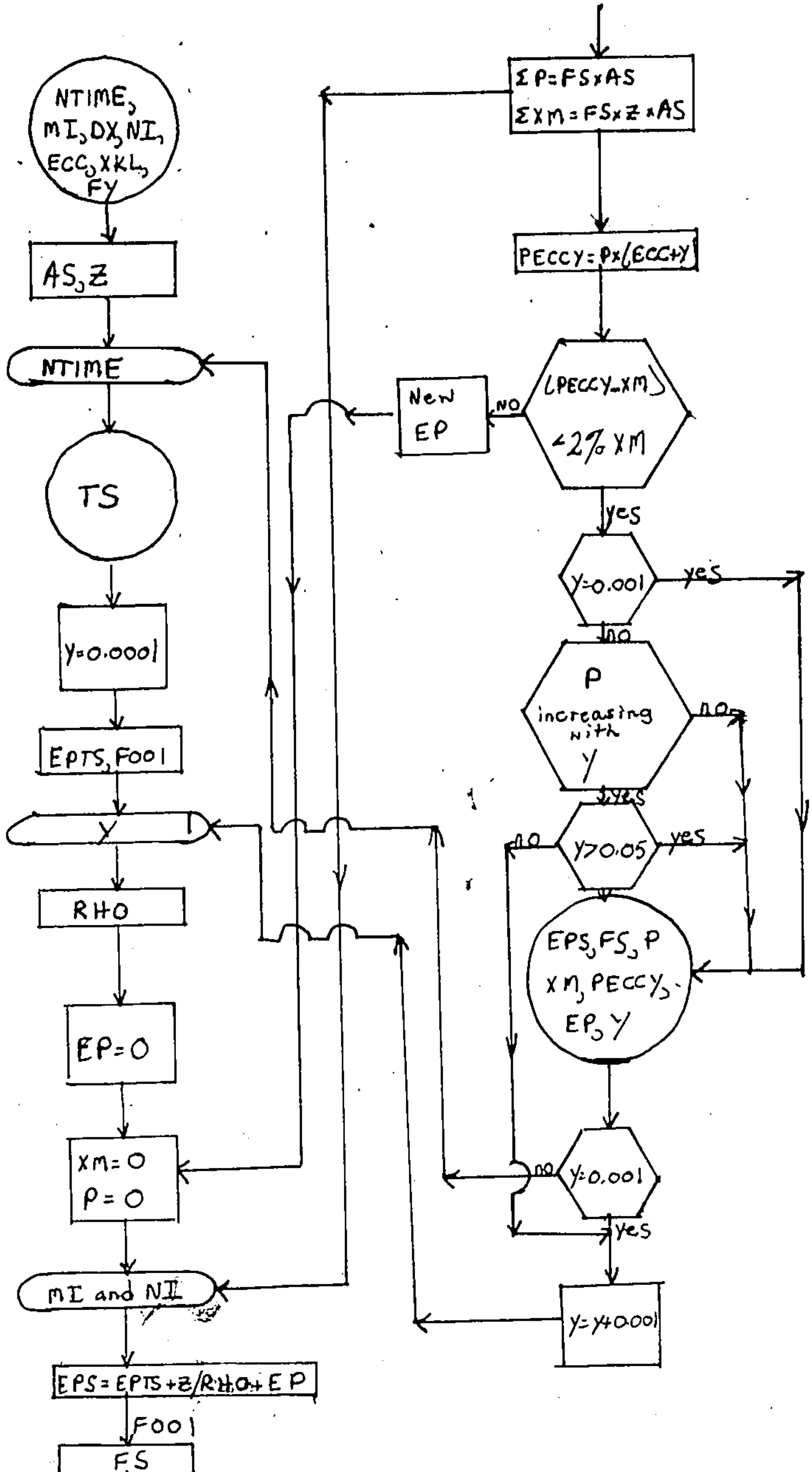
FLOWCHART

PROGRAM WIDEF

Legend

NTIME	no of temperatures at which analysis is carried out
TS	temperature of steel ^{o}C
MI	no of elements in flange/2
NI	no of elements in web/4
DX	½element width m
ECC	initial out of straightness of column m
XKL	effective length of column m
FY	yield strength of steel N/m^2 (MPA$\times 10^6$)
AS	element area m^2
Z	distance of element from neutral axis m
Y	assumed lateral deflection m at column midheight
EPTS	thermal strain temperature
F001	relationship between stress and strain temperature N/m^2
RHO	curvature at column midheight(function of Y)
EP	axial strain
FS	elemental stress N/m^2
EPS	elemental strain
P	axial load N
XM	internal moment N•m
PECCY	external moment N•m




NTIME, MI, DX, NI, ECC, XKL, FY
AS, Z
NTIME
TS
Y=0.0001
EPTS, F001
Y
RHO
EP=0
XM=0
P=0
MI and NI
EPS=EPTS+Z/RHO+EP
F001
FS
ΣP=FS×AS
ΣXM=FS×Z×AS
PECCY=P×(ECC+Y)
(PECCY-XM) <2% XM
NO
New EP
yes
Y=0.001
yes
no
P increasing with Y
no
yes
Y>0.05
no
yes
EPS, FS, P XM, PECCY, EP, Y
Y=0.001
no
yes
Y=Y+0.001
cont'd above

PROGRAM WIDEF

Fire Resistance of Wide Flange Steel Columns

Read NTIME, MI, NI, DX, ECC, XKL, FY
I5 , I5, I5, 3F10.5 , F20.5

NTIME = number of analyses (number of temperatures)

MI = number of elements in web half, including corner , L.E. 10

NI = number of elements in half flange, L.E. 10

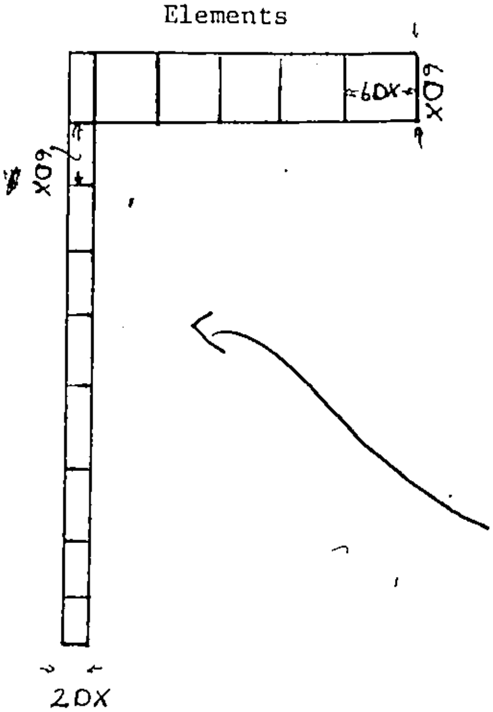
DX= 1/2 element width = 1/4 web width

ECC = assumed eccentricity in applied load - meters, typically 0.0025 (weak axis bending) .

XKL = effective length = 2 meters for column furnace

FY = yield strength of steel in N/m^2 = MPa$\times10^6$

Elements



(1/4 of section analysed then symmetry)

centroids of elements used to calculate moment arms to bending axis

NOTE : flange depth must = 3xweb depth

web and flange length must be roughly even multiples of flange depth

This area is dimensioned but no stresses, strains or areas are assigned

Summary of Analysis Process

Given an eccentricity
Guess at curvature (centerline deflection) and an axial strain
Calculate strain (including thermal, axial and bending)
Calculate stresses
Calculate load and moment (sum over elements times areas and moment arms)
Does load × (deflection + eccentricity) = moment ??

- No – then guess a new axial strain and repeat
- Yes – then guess a new curvature (centerline deflection) and repeat

Initial lateral deflection at midheight assumed y = 1mm

EY = $4\times10^{-12}\times F_y$ (proportional limit strain)

The following analysis is carried out at midheight at each temperature (TS)

Read steel temperature and time Format 2F6.1

ALPHAS (expansion) = $0.004\times10^{-6}\times TS + 12\times10^{-6}$

EPTS (thermal strain) = ALPHAS × (TS−20)

F001 (tangent E) = $6.9\times10^{6}\times(50-0.04TS)\times(1-e^{-30+0.03TS\times\sqrt{0.001}})$
(from experimental curve fitting)

RHO (curvature) = $1/y\times XKL^{2}/12$

The initial deflection assumed is

The following analysis is carried out on each element

Strains

Strain on each element on right (EPSR(I,J)) = − EPTS+EP+Z(I,J)/RHO
left EPSL −

where EP= strain(compressive) due to applied load (initial = 0)
Z= distance to bending axis

Stress-Strain Relationship

If strain is less than EY Stress= F001×strain/0.001 = FSR(I,J) or FSL(I,J)

If strain is greater than EY Stress = $6.9\times10^{6}\times(50-0.04TS)\times(1-e^{-30+0.03TS\times\sqrt{strain-EY+0.001}})$
$+(F001/0.001\times EY-F001)$
from experimental curve fitting

Calculating load and bending moment for each element

```
(load)   PS = (stress left + stress right)x area x 2
         P  = sum of loads on all elements
(moment)XMS= (-stress left + stress right)x area x z x 2
         XM = sum of moment on all elements
```

Calculating moment generated by calculated P

```
    PECCY = P x (ECC+Y)
```

If (PECCY-XM) GT 2% of XM, EP is incremented and the whole process begins again (stress-strain calculation)

When XM and PECCY are balanced for a given Y, then a new Y is assumed (up to 10 mm) and the process begins again.

When several Y's are attempted and the axial load remains the same, then a new temperature is input and the process begins again.

```
[illegible] (Y).[illegible] MAP(N),LIST DATA SET(Y).

      C     [illegible] FLANGE [illegible] COLUMNS
            [illegible] (A-[illegible])
            [illegible] (10,10),EPS (10,10),FSR(10,10),FSL(10,10),Z(10,10),AS(10,10)),EPSMM(10,10),-
           1[illegible] (10,10),[illegible](10,10),[illegible](10,10),RESID(10,10)),RESTRA(10,10))
            [illegible] CC,XKL,FY
          1 [illegible]
            [illegible] CC,XKL,FY
      C     [illegible]
            [illegible]
            [illegible] I=1,[illegible]
            [illegible] J=1,[illegible]
      500   [illegible]
            [illegible]
      C     CALCULATI[illegible] CO-ORDINATES OF STEEL ELEMENTS
            DO [illegible] I=1,[illegible]
            Z(I,1)= [illegible]
            [illegible]
      [illegible] [illegible]
            [illegible] J=2,NII
            Z(I,J)-[illegible]-7.0)
            [illegible](I,J)=([illegible]+Z(I,J)/((NII*6-7.0)*DY))
      501   A[illegible](I,J)[illegible]
      C     [illegible] STEEL TEMPERATURES AND CALCULATION OF STEEL PROPERTIES
            [illegible] II=1,NII[illegible]
            [illegible]
            [illegible] TS(II)),II=[illegible]
        3   [illegible]
            [illegible]TS(II)+12.0-[illegible]
            [illegible]
            [illegible]+(TS(IT)-20.00)
            [illegible]*TS(IT))*(1.00-DEXP((-30.00+0.0300*TS(IT))*DSQRT(0.00100)))
      C     [illegible] INITIALIZATION
      1500  [illegible]ZY*SKL**2/12.E0
            [illegible]
            [illegible]
            [illegible]
            [illegible]
            [illegible]
            [illegible]
            [illegible]
      1510  [illegible]
            [illegible]
      C     CALCULATION OF STRAINS
            DO [illegible] I=1,[illegible]
            DO [illegible] J=1,NII
            IF (AS(I,J)-0.0)[illegible]
      600   [illegible]STRA(I,J)=[illegible]SIG(I,J)/(E001/0.00100)
            [illegible](I,J)=-EPTS+EP+Z(I,J)/RHO+RESTRA(I,J)
            [illegible](I,J)=-EPTS+EP-Z(I,J)/RHO+RESTRA(I,J)
      C     CALCULATION OF STRESS-STRAIN RELATIONS AND STRESSES IN STEEL
```

```
      IF( [illegible](I,J).[illegible])GO TO 6
      [illegible]
      [illegible]
      [illegible]=-1.[illegible]
    5 IF(DABS( [illegible](I,J))-EPY)6,6,7
    6 [illegible](I,J)=([illegible]/0.001)*DABS(EPS[illegible](I,J))
      [illegible]
    7 [illegible](I,J)=[illegible]*([illegible]-0.[illegible]*TS(IT))*(1.D0-DEXP((-30.D0+0.03*TS(IT))*DSQRT(DABS(EPSR(I,J))-EPY+0.001D0)))-
     1+C*([illegible]/0.001D0*[illegible]Y-F[illegible])
    8 IF([illegible](I,J).[illegible])GO TO [illegible]
      [illegible]=1.[illegible]
      GO TO [illegible]
    9 [illegible]=-1.D0
   10 IF(DABS(EPSL(I,J))-EPY)11,11,12
   11 FSL(I,J)=(F[illegible]/0.001D0)*DABS(EPSL(I,J))
      GO TO 13
   12 FSL(I,J)=[illegible]*([illegible]-0.0430*TS(IT))*(1.D0-DEXP((-30.D0+0.03*TS(IT))*DSQRT-
     1(DABS(EPSL(I,J))-EPY+0.001D0)))+C*(F[illegible]/0.001D0*EPY-F00[illegible])
C     CALCULATION OF LOAD
   13 [illegible]=[illegible](F[illegible](I,J)+FSL(I,J))*AS(I,J)*2.D0
      [illegible]
C     CALCULATION OF BENDING MOMENT
      [illegible]=[illegible](-F[illegible](I,J)+FSL(I,J))*AS(I,J)*Z(I,J)*2.))
      [illegible]
  [illegible] CONTINUE
C     CALCULATION OF MOMENT GENERATED BY AXIAL LOAD
      [illegible]=[illegible]*([illegible]+Y)
      IF([illegible]-0.[illegible])[illegible],2002,2003
 2002 FACT=[illegible]
      IF([illegible].LT.[illegible])FACT=20.D0
      GO TO [illegible]
 2003 FACT=[illegible]
 2004 IF(DABS(X[illegible]-[illegible](Y)-0.[illegible]*X[illegible])2303,2303,2005
 2005 IF([illegible]-1)[illegible],[illegible],2110
 2006 IF([illegible]-1)[illegible],[illegible],[illegible]
 [illegible] IF(DABS(X[illegible]-[illegible](Y)-0.[illegible]*XM)2303,2303,2110
 2010 IF([illegible])2015,2015,2020
 2015 [illegible]=[illegible]-0.001/FACT
      GO TO 1[illegible]
 2020 IF(X[illegible]-[illegible])[illegible],[illegible],2030
 2030 IF(DA[illegible]([illegible](Y)-0.[illegible]*X[illegible])2303,2303,2040
 2040 IF(X[illegible]-[illegible](Y))[illegible],[illegible],2050
 2050 IF(I[illegible]-[illegible])[illegible],2060,2070
 2060 [illegible]+0.001/FACT
      IN[illegible]=IN[illegible]+1
      GO TO 1[illegible]
 2070 [illegible]+0.[illegible]/FACT
      [illegible]=[illegible]+1
      GO TO 1[illegible]
 2080 IF(I[illegible]-[illegible])[illegible],2090,2100
 2090 [illegible]-0.001/FACT
      INDIC=INDIC+1
      GO TO 1[illegible]
```

```
 2110  [illegible]/FACT
       [illegible]
       [illegible]
 2115  IF([illegible] GGY)2120,2125,2130
 2120  IF([illegible])GO TO 2300
       [illegible]0.001/FACT
       [illegible]=NO[illegible]+1
       [illegible]=1
       GO TO [illegible]
 2130  IF([illegible])GO TO 2300
       [illegible]=[illegible]/FACT
       [illegible]
       [illegible]=1
       GO TO [illegible]
 2300  WRITE([illegible])
 2302  FORMAT(////'[illegible]',T20,'MOMENTS NOT BALANCED WITHIN 2 PERCENT; NUMBER=',I3)
 2303  IF([illegible]0.001)GO TO 2304
       IF([illegible])GO TO 2304
       [illegible]
       IF([illegible]0.05[illegible])GO TO 2304
       [illegible]
       GO TO [illegible]
 2304  WRITE([illegible])[illegible]
 2305  FORMAT('[illegible]=',1X,F7.1,1X,'MIN')
       WRITE([illegible])[illegible](II)
   14  FORMAT('[illegible] (C) ',T2[illegible],F6.1)
       DO [illegible]=1,[illegible]
       DO [illegible]=1,[illegible]
       [illegible](M,N)=1.0[illegible]
       [illegible](M,N)[illegible]
       [illegible](M,N)[illegible]1.0E-[illegible]
       [illegible](M,N)[illegible]1.0E-[illegible]
       WRITE([illegible])([illegible](M,N),N=1,NII)
       WRITE([illegible])([illegible](M,N),N=1,NII)
       WRITE([illegible])([illegible](M,N),N=1,NII)
       WRITE([illegible])([illegible](M,N),N=1,NII)
   15  FORMAT('[illegible] ON RIGHT  [illegible]',T26,16F6.1)
   16  FORMAT('  [illegible]AIN ON LEFT  [illegible]',T26,16F6.1)
   17  FORMAT('  [illegible] ON RIGHT  [illegible] ',T26,16F6.1)
       WRITE([illegible])([illegible] A(M,N),N=1,NII)
       FORMAT([illegible])
   18  FORMAT('  STRESS ON LEFT  [illegible]',T26,16F6.1)
  [illegible]  [illegible]
       [illegible]1.0E-[illegible]
       [illegible]1.0E-[illegible]
       [illegible]Y+1.0E-3
       WRITE([illegible])[illegible]
       WRITE([illegible])[illegible]
       WRITE([illegible])[illegible]
   1[illegible]  FORMAT(1X,' TOTAL LOAD = ',F10.2,1X,' KN')
   1[illegible]  FORMAT(1X,' TOTAL MOMENT (SUM OF MOMENTS OF ELEMENTS) = ',F10.2,1X,'KN-M')
   1[illegible]  FORMAT('[illegible] (LOAD*(ECCENTRICITY+D:FLECTION)) =',F10.2,1X,'KN-M')
       [illegible]
```

[illegible]

```
      [illegible]
      WRITE([illegible]
      WRITE([illegible]
      WRITE([illegible])Y[illegible]
  200 FORMAT('  RELATIVE AXIAL STRAIN =',F10.7)
  201 FORMAT('  TOTAL AXIAL STRAIN =',F10.4,1X,' MM')
  202 FORMAT('  LATERAL DEFLECTION AT MIDHEIGHT =',F10.2,1X,' MM')
      IF(Y.NE.0.001)GO TO 502
      Y=Y+0.001
      GO TO 1500
  502 CONTINUE
  400 CONTINUE
      STOP
      END
```

APPENDIX B

Elastic Theory

reference: Timoshenko and Gere
"Theory of Elastic Stability"

Pin-ended strut with initial curvature defined by

$$y_o = a_1 \sin \pi x/L \qquad \text{(at } x=L/2,\ y_o=a_1\text{)}$$

Load struct to P, deflection becomes y

Curvature equation $\qquad \dfrac{d^2(y-y_o)}{dx^2} + P/EI \,.y_o = 0$

integrate twice, substitute for y_o and $P_E = \pi^2 EI/L^2$

$$(1) \qquad y = \frac{a_1 \sin \pi x/L}{1-P/P_E}$$

Load to cause first yield $\qquad P_y \qquad$?

$$y \text{ at } P_y = \frac{a_1 \sin \pi x/L}{1-P_y/P_E}$$

Moment at x=L/2, differentiate twice and substitute

$$M = \frac{P_E P_y a_1}{(P_E-P_y)}$$

Let distance to extreme fiber = c

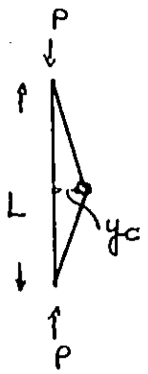
Maximum fiber stress = yield stress σ_y when

$$\sigma_y = P_y/A + \frac{P_E P_y a_1 c}{(P_E-P_y) I}$$

Let $I = Ar^2$; $P_y = AP_y$, $P_E = AP_E$, $\eta = a_1 c/r^2$

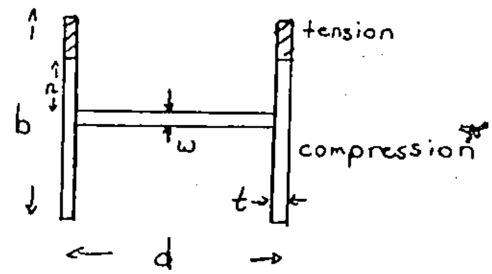
$$(2) \text{ then } \quad P_y = 1/2 \left\{ \sigma_y + (1+\eta) P_E - \sqrt{(\sigma_y + (1+\eta) P_E)^2 - 4\sigma_y P_E} \right\}$$

Plastic Theory



reference: course notes CVG 4142
private communication:
D.C. Stringer

Assume a perfectly plastic material



Plastic moment of resistance about neutral axis

$M_p = 2(b/2-n)(b/2+n)t\sigma_y$

Axial thrust $P = [4nt+(d-2t)w]\sigma_y$

At equilibrium $Py_c = M_p$

Equate and eliminate n

$y_c = 2/P\left\{b^2/4 - [1/4t(p/\sigma_y - (d-2t)w)]^2\right\}\sigma_y t$

W150 by 37

$L=2m$
$a_1=2.5mm$
$A=4730mm^2$
$I_y=7.07\ 10^6 mm^4$
$r_y=38.7mm$
$d=162mm$
$b=154mm$
$t=11.6mm$
$w=8.1mm$

$\sigma_y=300MPa$
$E=200000MPa$
$P_E=\pi^2 ET/L^2=3490KN$

Plastic Theory

$$y_c=2/P\left\{5930-[0.022(P/300-1124)8.1]^2\right\}3480$$

Elastic Theory

From 2 $P_y=251MPa \Rightarrow 1188KN$

From 1 at $x=L/2$

$$y=\frac{a_1}{1-P/P_E}=\frac{2.5}{1-P/3490}$$

$$y-y_o=\frac{2.5}{1-P/3490}-2.5$$

APPENDIX C

[illegible]

Sault Ste. Marie, Ontario, Canada

PURCHASE ORDER NUMBER	ENTRY DATE	SHIP DATE	SHIPPERS NUMBER	CARRIER	PAGE NO.	MILL ORDER
5417	MAY 05/81	81-07-07	05 5547	C.P.R. - DIRECT CP 337218	1	27475

TEST REPORTS

WE HEREBY CERTIFY THAT THE MATERIAL HEREIN DESCRIBED WAS MADE AND TESTED IN ACCORDANCE WITH THE RULES OF THE SPECIFICATION SHOWN HEREIN AND AS CONTAINED IN THE CORPORATION RECORDS.

TO CUSTOMER NAME AND ADDRESS

SERVAN AND CO LTD
STAR TOP ROAD BOX 9208
TERMINAL OTTAWA ONT
3T9

SHIP TO CUSTOMER NAME AND ADDRESS

M ZAGERMAN AND CO LTD
1630 STAR TOP ROAD
OTTAWA ONTARIO

SPECIFICATION

CTURAL SHAPES - CARBON - CSA STD G40.21-44W - (1976)

NTARY INSTRUCTIONS

MILL SM/TR

RESALE

NN-NN-NN

SENIOR METALLURGIST

SHAPE PRODUCTS

HEAT NO INGOT NUMBER	IDENTIFICATION OF PIECE TESTED	WEIGHT	NUMBER OF PIECES	THICKNESS OF TEST	CONDITION OF TEST	TENSILE PROPERTIES YIELD PSI	TENSILE PSI	% ELONGATION 2"	4"	8"	BEND	MC QUAID	TYPE DIRECT	IMPACT PROPERTIES 1 FTLB	2 FTLB	3 FTLB	TEMP °F
ITEM	2	OUR ITEM	002	W 6	X 25		X	60									
45		19406	13	.245		47,640	66,380			28							
00		1493	1	.250		44,150	65,480			27							
41		28362	19	.290		44,237	65790			24							

345	C .20	MN 1.05	S .017	P .025	SI .31
300	C .19	MN 1.14	S .020	P .011	SI .01
341	C .20	MN 1.15	S .035	P .011	SI .01

APPENDIX D

Load Factor Calculation

Assume Live load = 1.5 Dead Load

C.SA S16.1 Factored Capacity $= 1.25\ D + 1.5\ L$
$= 1.25\ D + 1.5\ (1.5D)$
$= 3.5D$

$$\text{Load Factor} = \frac{1.25D + 1.5L}{D + L} = \frac{3.5D}{2.5D} = 1.4$$

APPENDIX E

Calculation of Effective Length

Column $I_c = 22.6\ mm^4 \times 10^6$, $L_c = 3.8m$ $I_c/L_c = 5.9\times 10^6 mm^4/m$

Cross Beams - assume only 2 acting

$$\text{each } I_g = 2\left(\frac{1.5^3 \times 16}{12} + 1.5\ 16\times 21.125^2\right) + \frac{(42.25-3)\times 1}{12}$$

$$= 26500\ in^4$$

$$L_g = 9'8'' \text{ max} = 2950mm$$

$$I_g/L_g = 2I_g/L_g = 7.5\times 10^9 mm^4/m$$

Thus $k \rightarrow 0.5$

source: nomograph
CSA S16.1 Appendix C

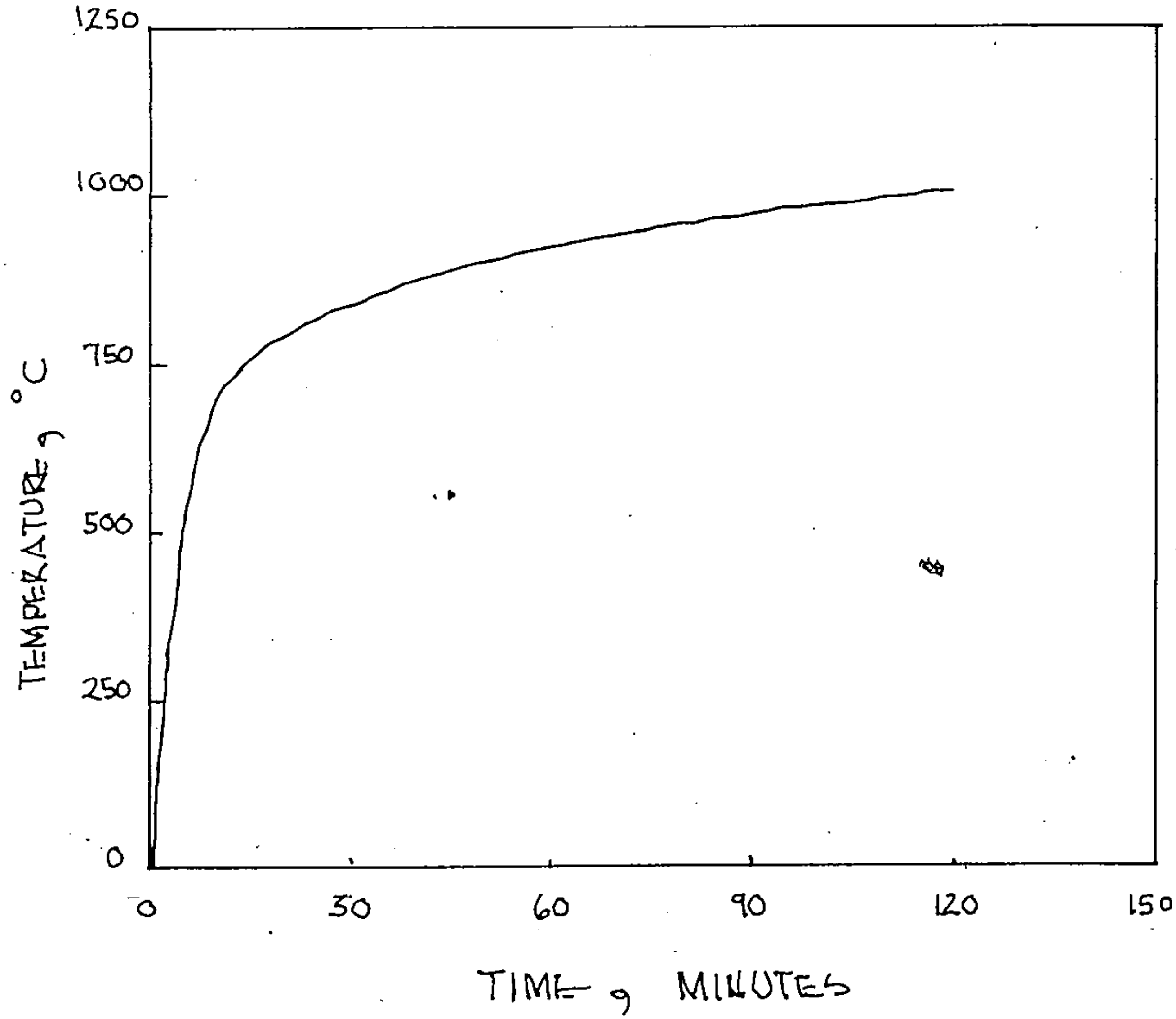
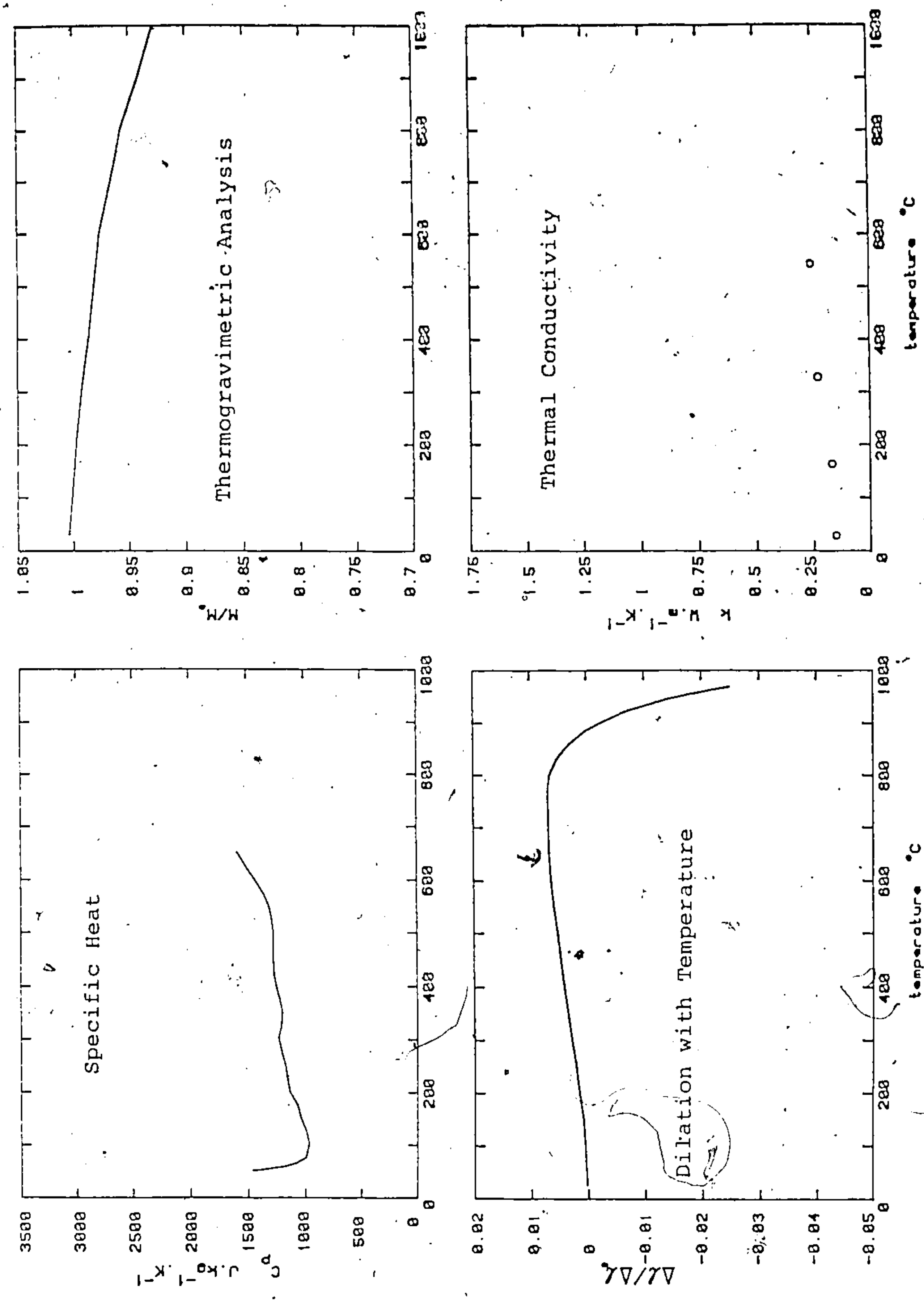


Figure 1

Standard Time Temperature Curve

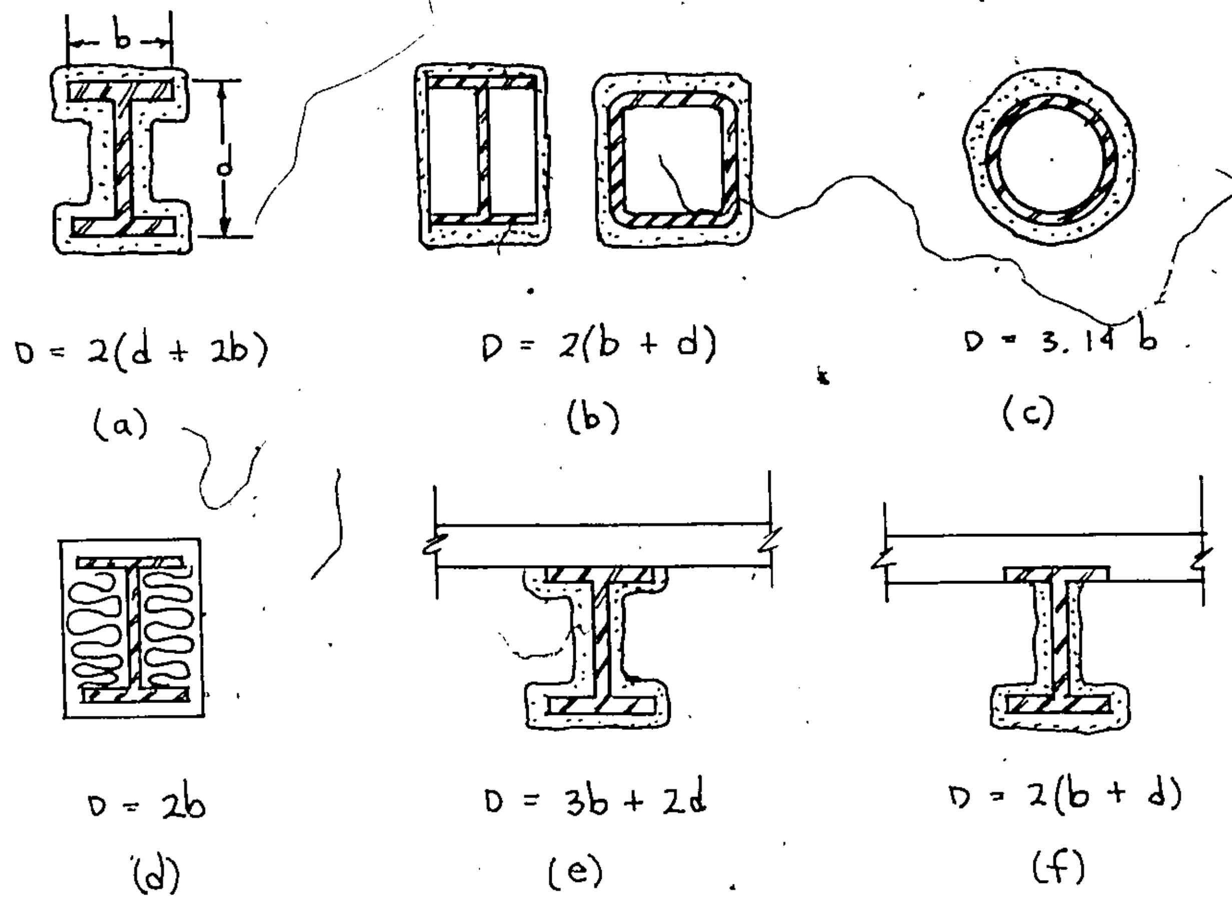
Fire Research Section
Division of Building Research
National Research Council of Canada
Ottawa, Ontario. K1A 0R6

1-IN-003



Insulating Board

Figure 2
Thermal Properties of Protection Material



TEST COLUMN

W 150 x 37 CASE (b) $D = 2(b + d) = 2(162 + 154) = 732$ mm

Figure 3

Heated Perimeter Concept

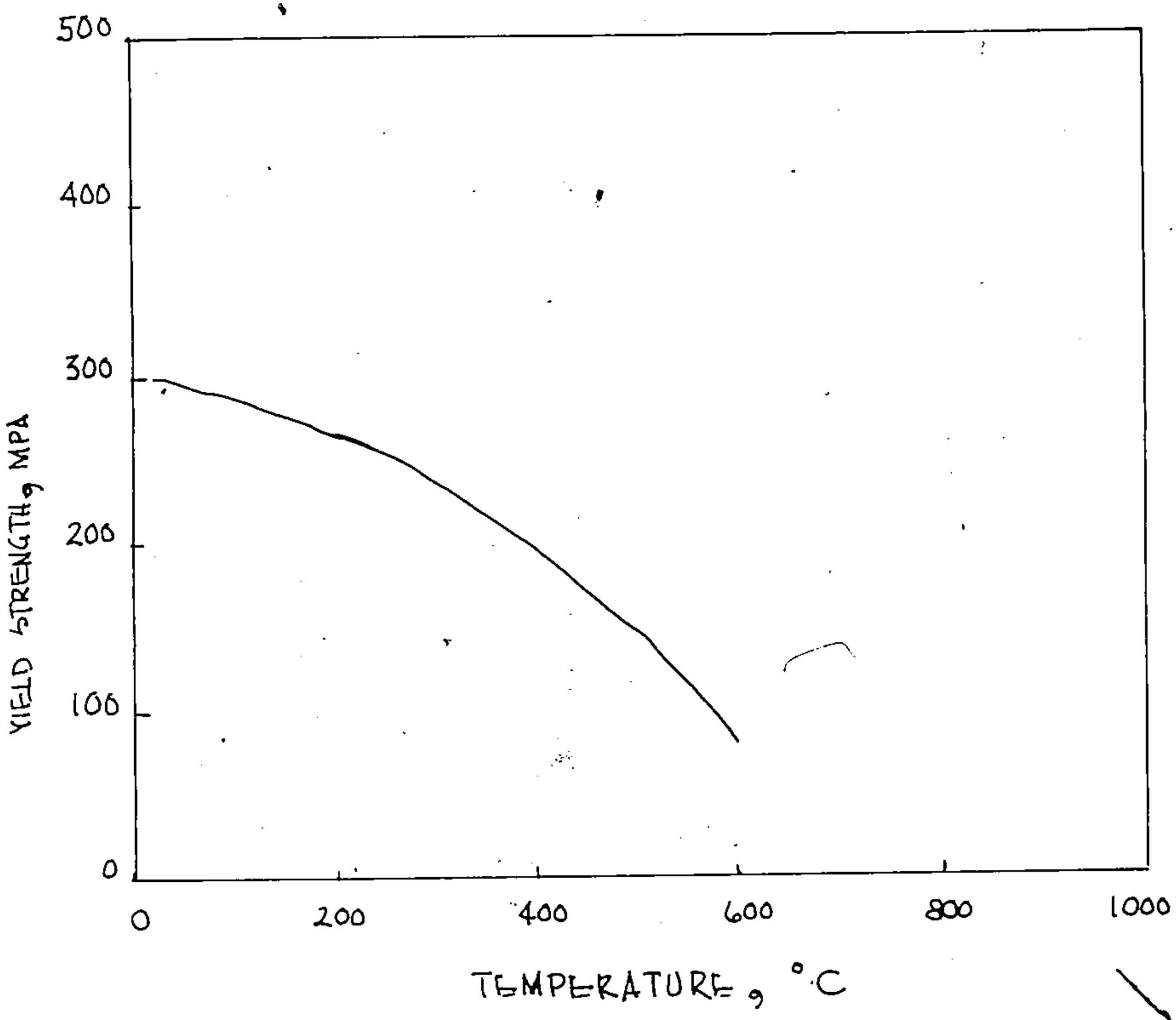


Figure 4

Yield Strength Versus Temperature

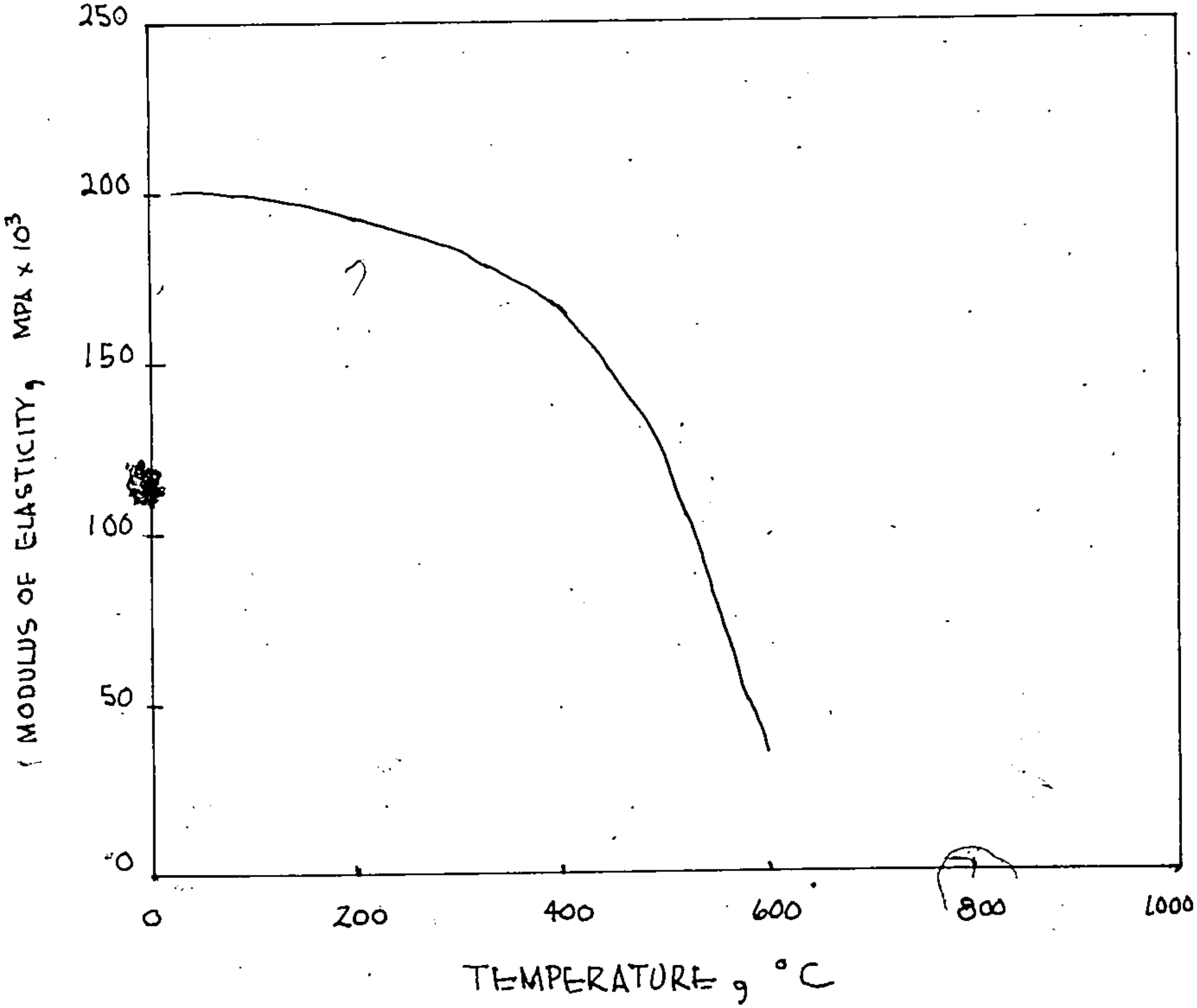


Figure 5

Modulus of Elasticity versus Temperature

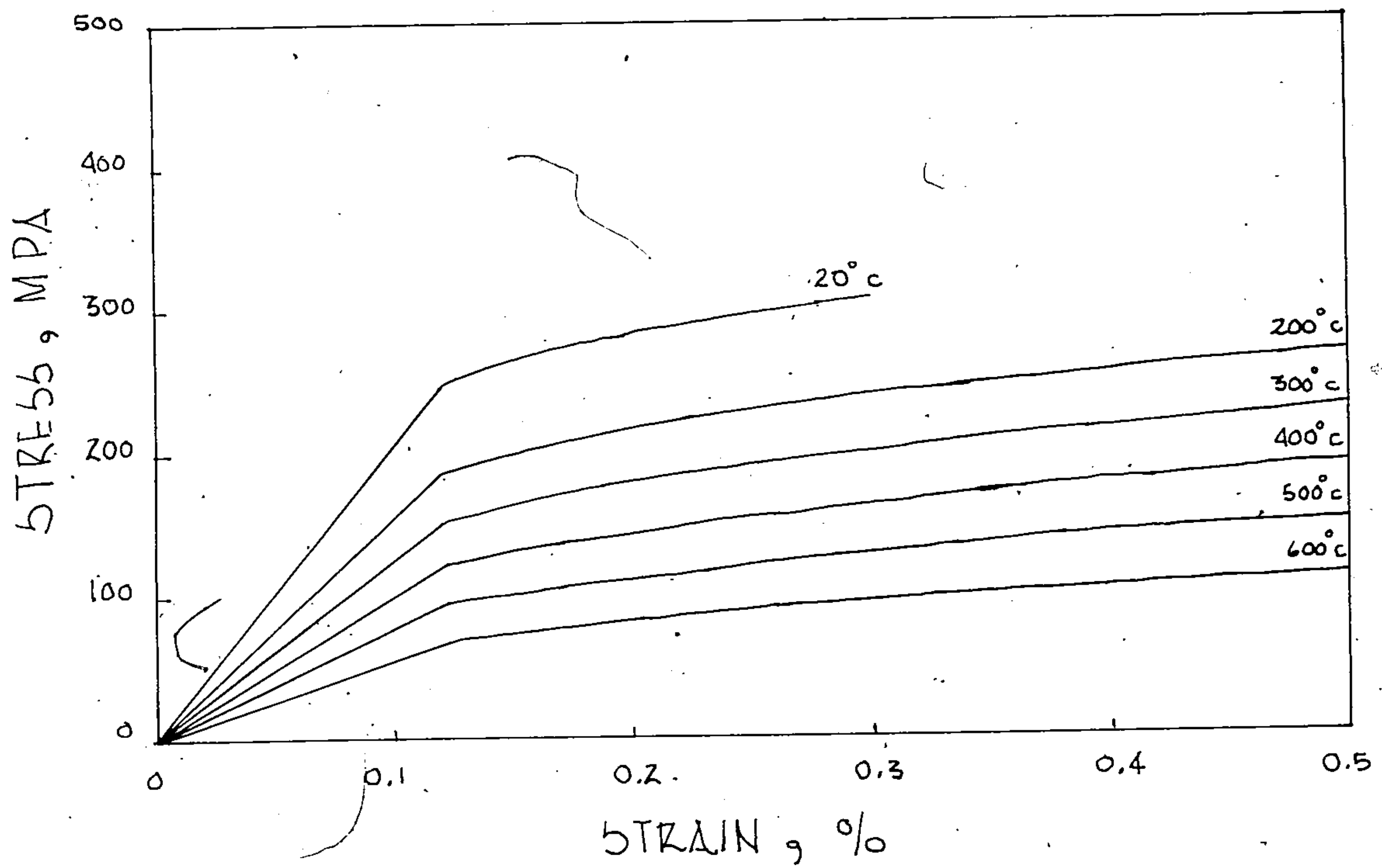


Figure 6

Stress Strain Curves

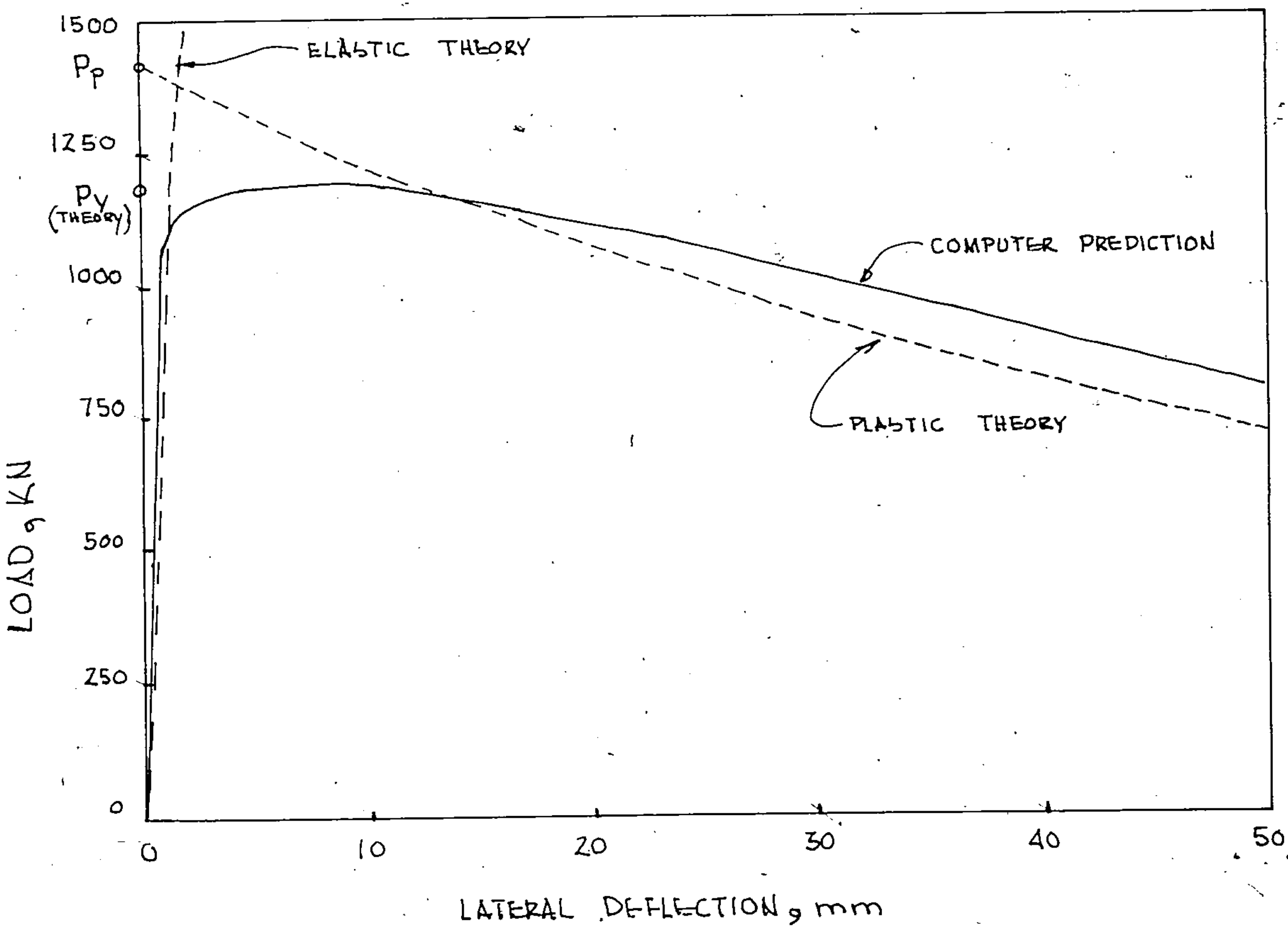
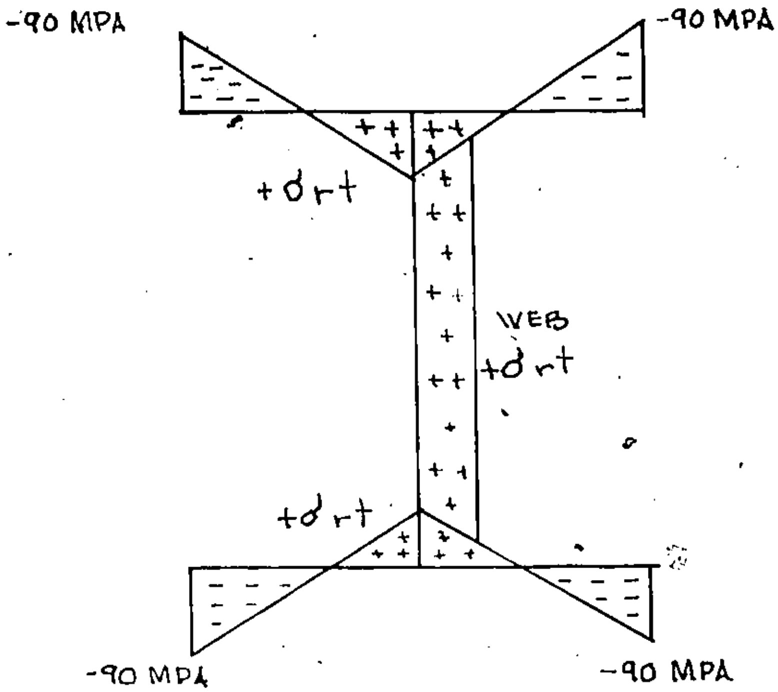


Figure 7

Comparison of Theory and Computer Prediction



$$\frac{\sigma_{rt}}{90} = \frac{bt}{bt + w(d-2t)}$$

TEST COLUMN

W 150 x37

$b = 154$
$t = 11.6$
$w = 8.1$
$d = 162$

$\sigma_{rt} = 55.2$ MPA

Figure 8

Residual Stress Pattern

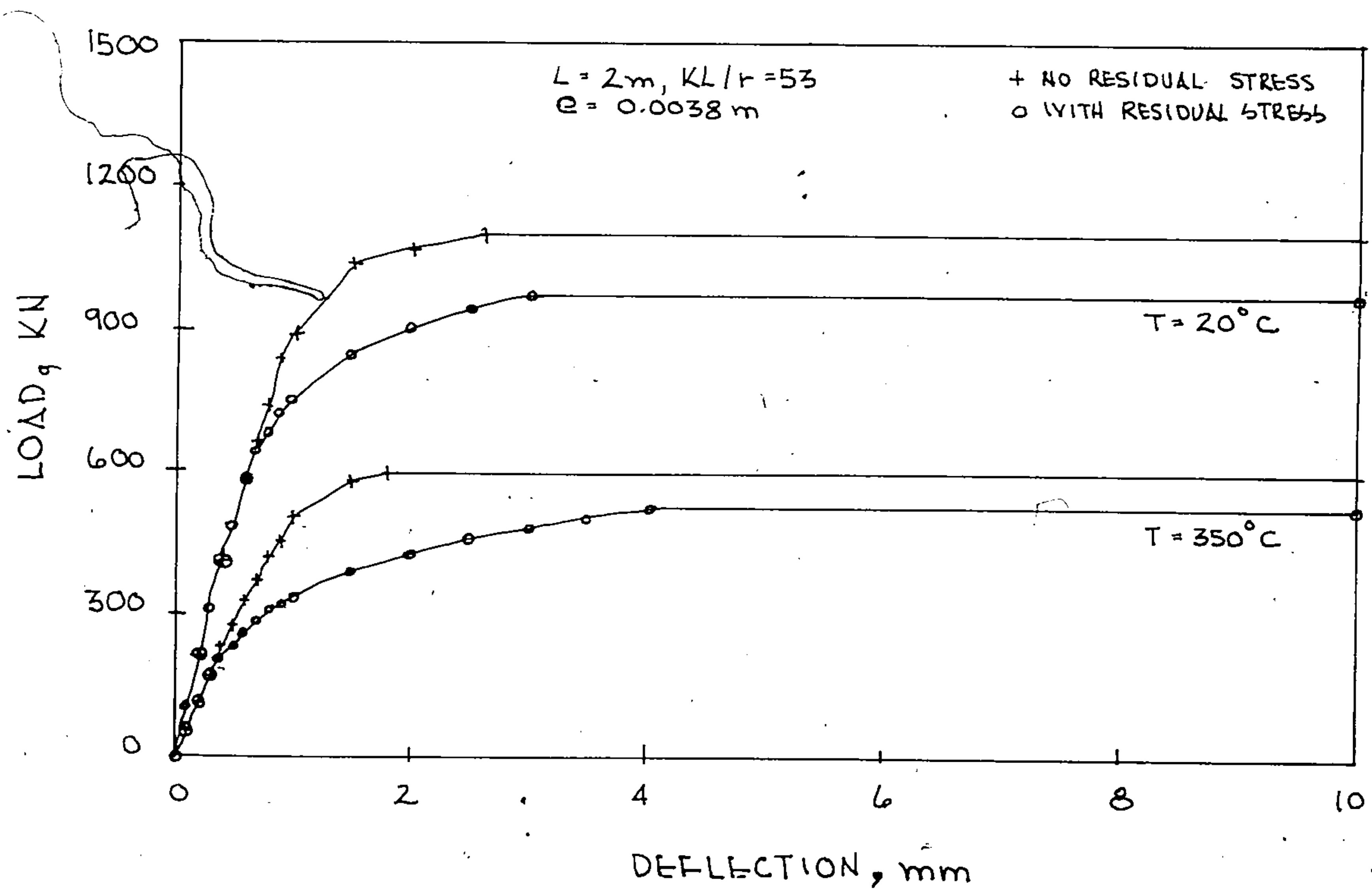


Figure 9

Influence of Residual Stress

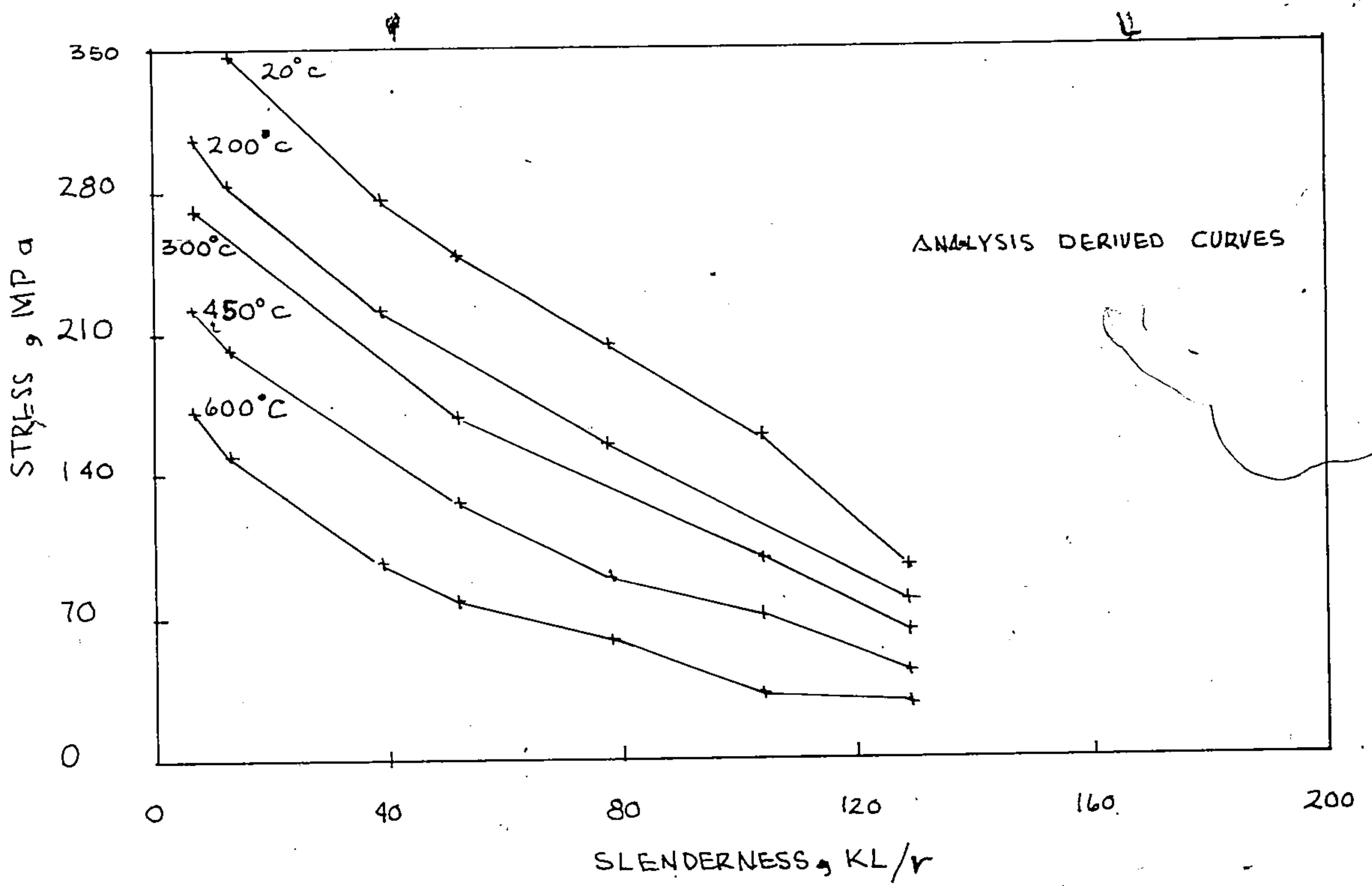


Figure 10

Elevated Temperature Buckling Curves

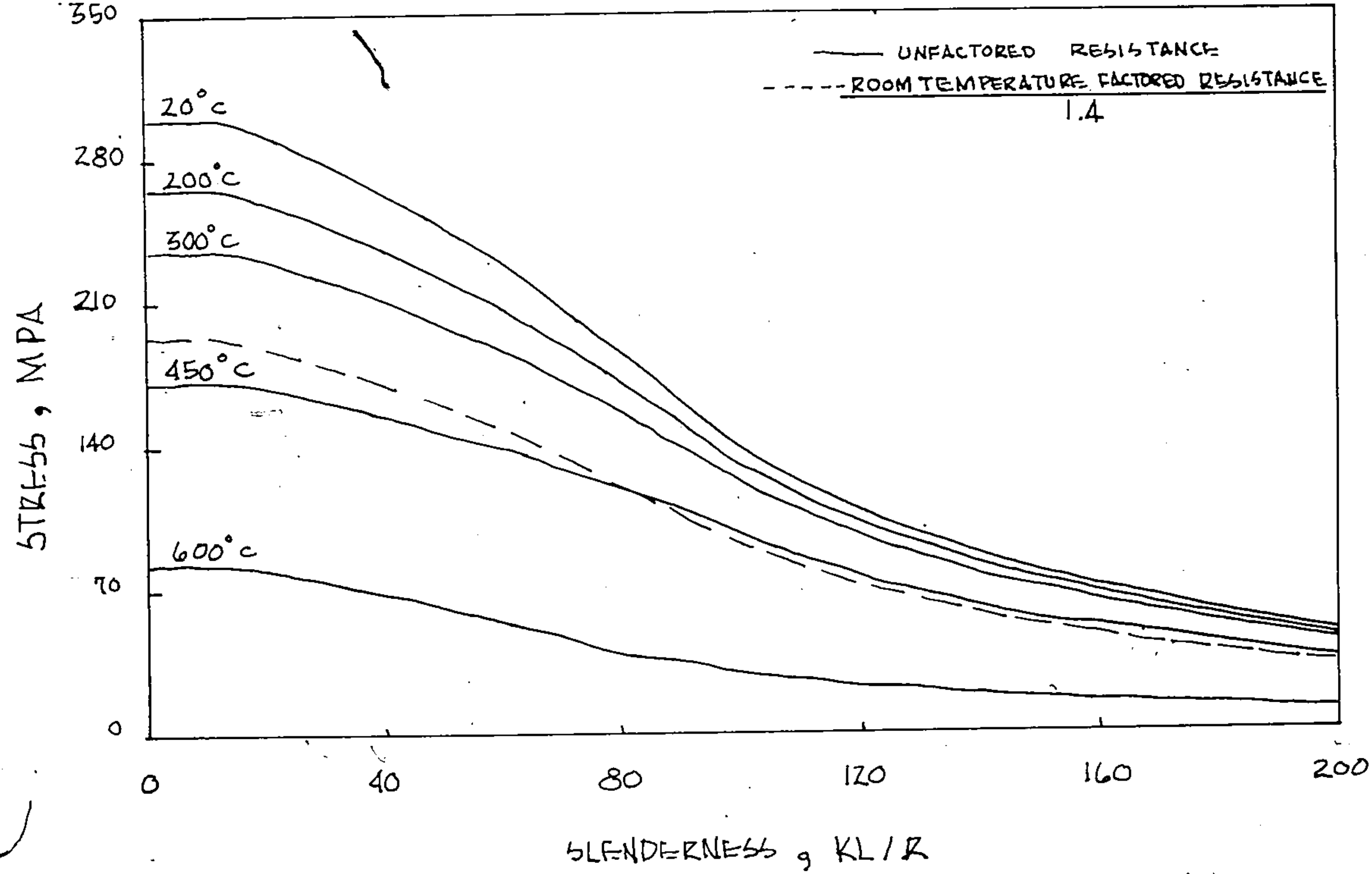


Figure 11

Elevated Temperature Buckling Curves

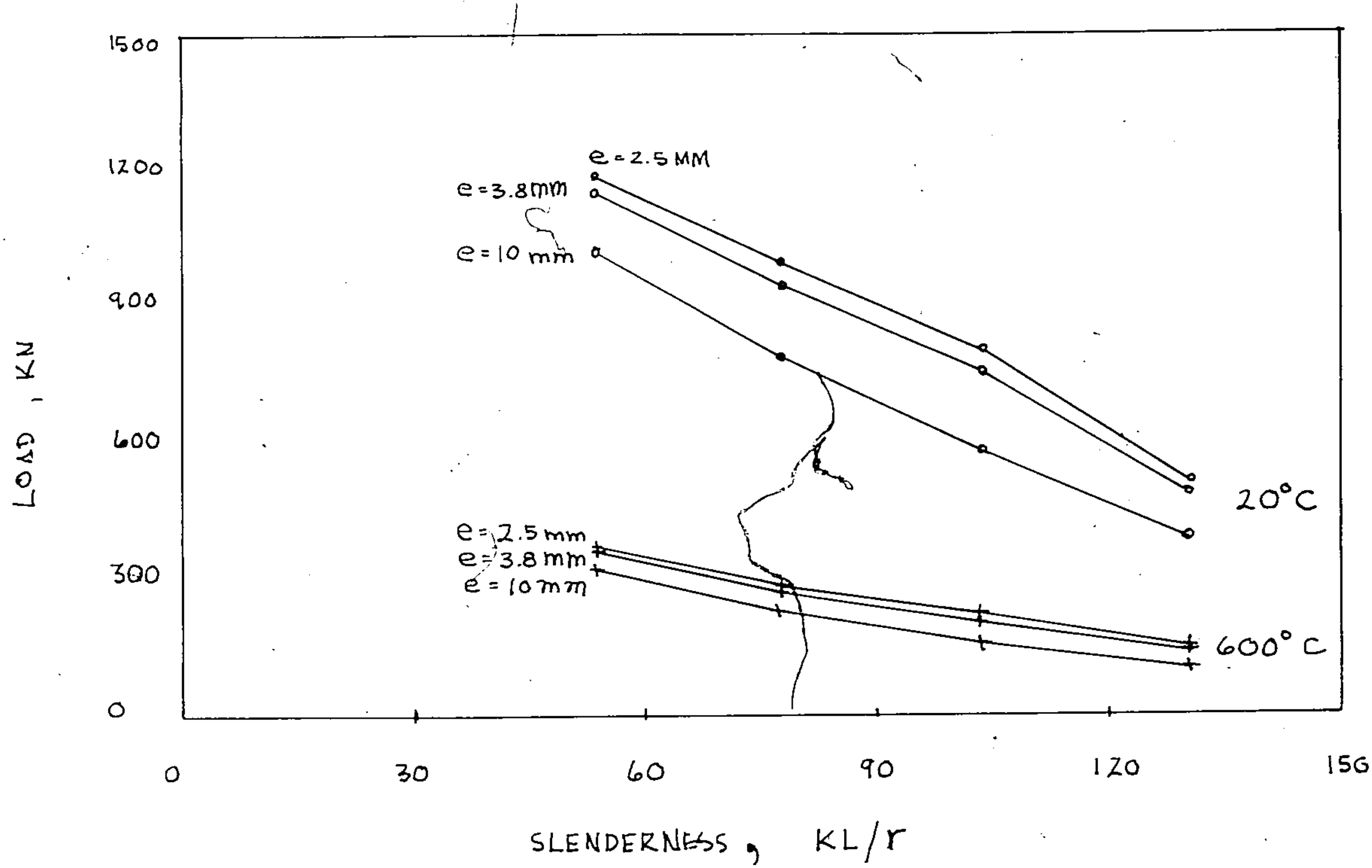


Figure 12

Influence of Initial Eccentricity

COLOURED PICTURES
Images en couleur

Figure 13

Test Column

COLOURED PICTURES
Images en couleur

Figure 14

Protected Test Column

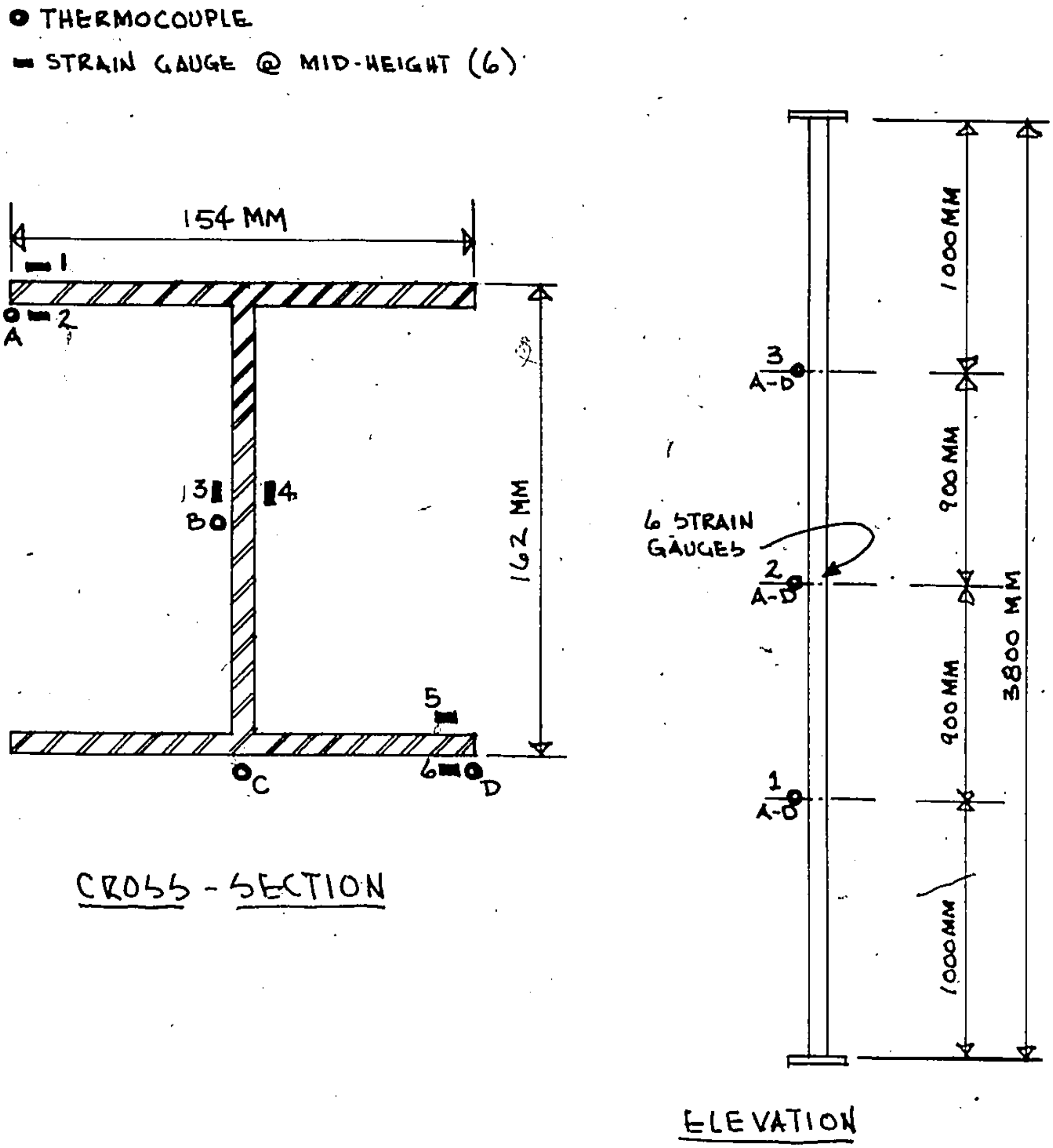


Figure 15

Thermocouple and Strain Gauge Locations

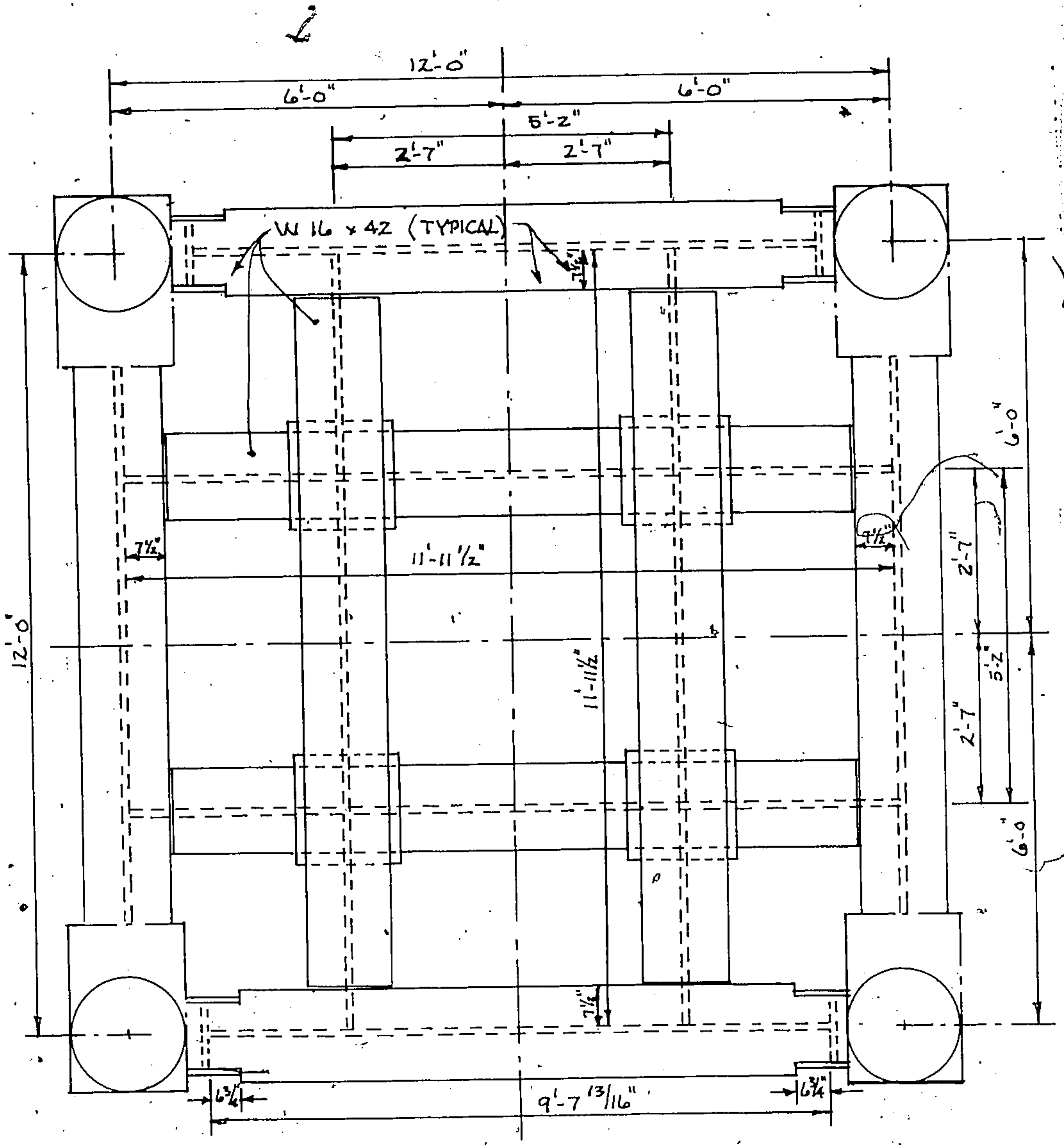


Figure 16

<u>Column Restraining Frame</u>

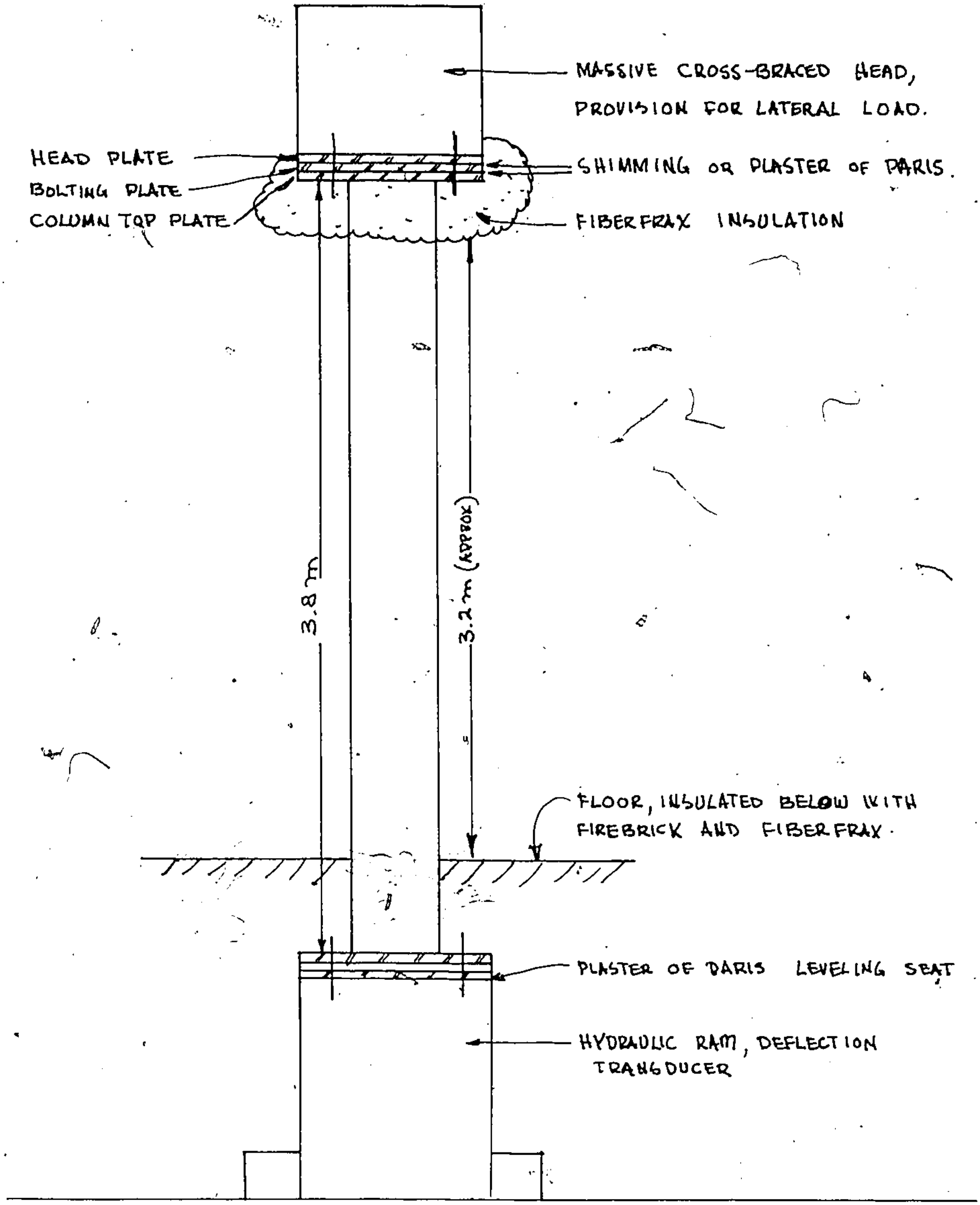


Figure 17

Schematic of Furnace Load Details

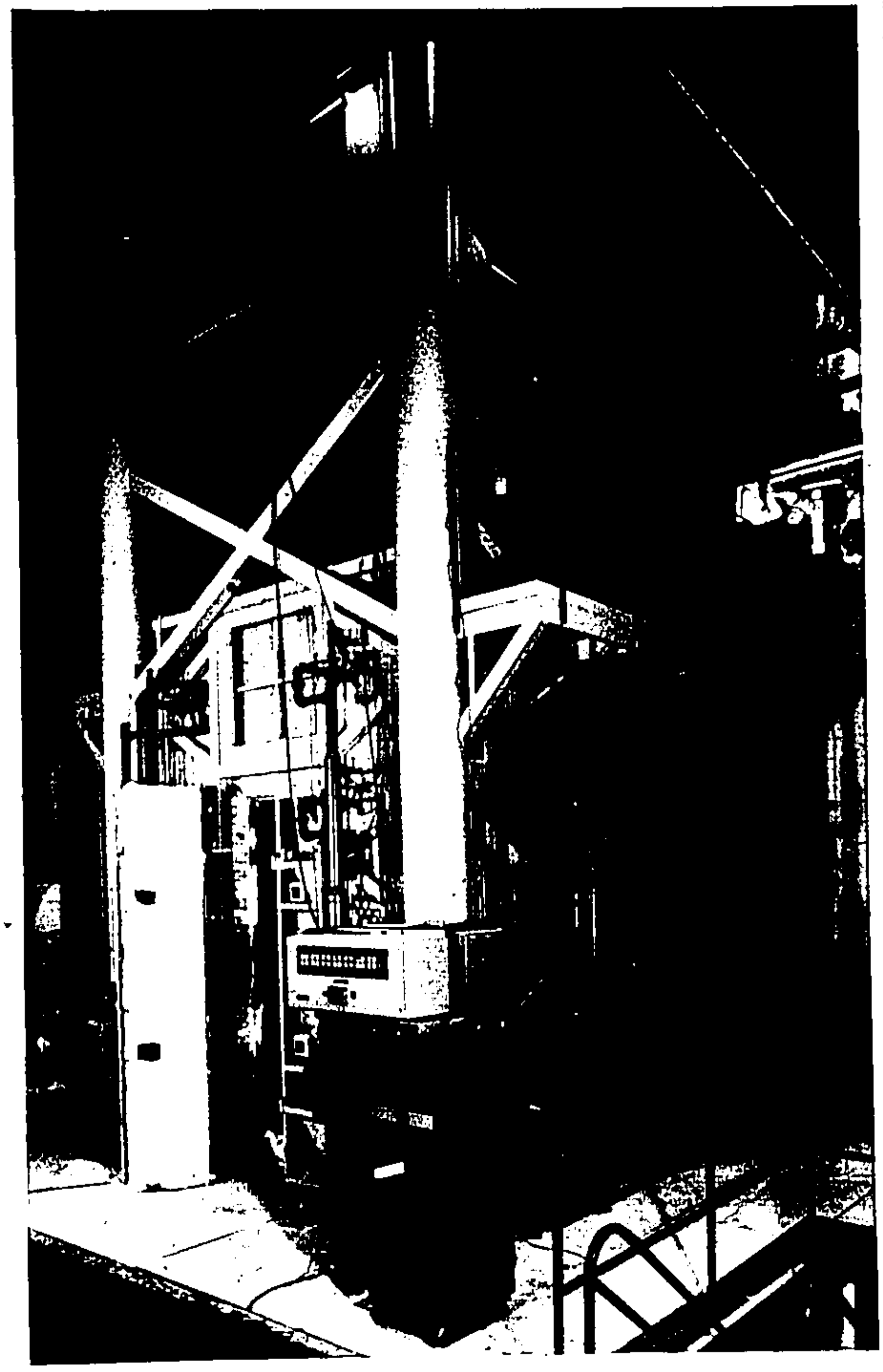


Figure 18

Column Furnace

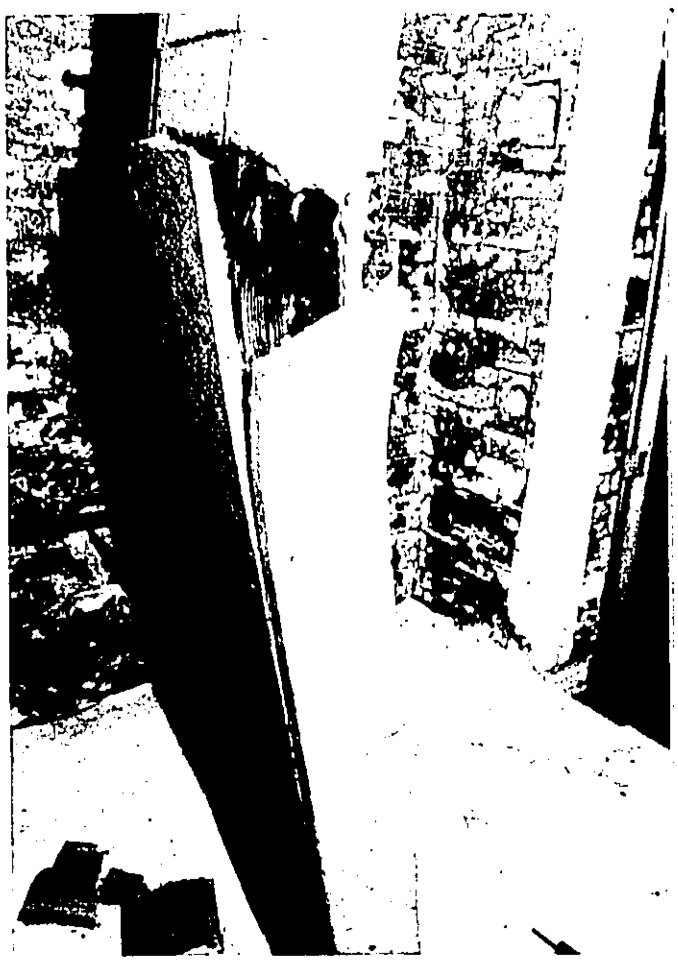


Figure 19

Column After Test

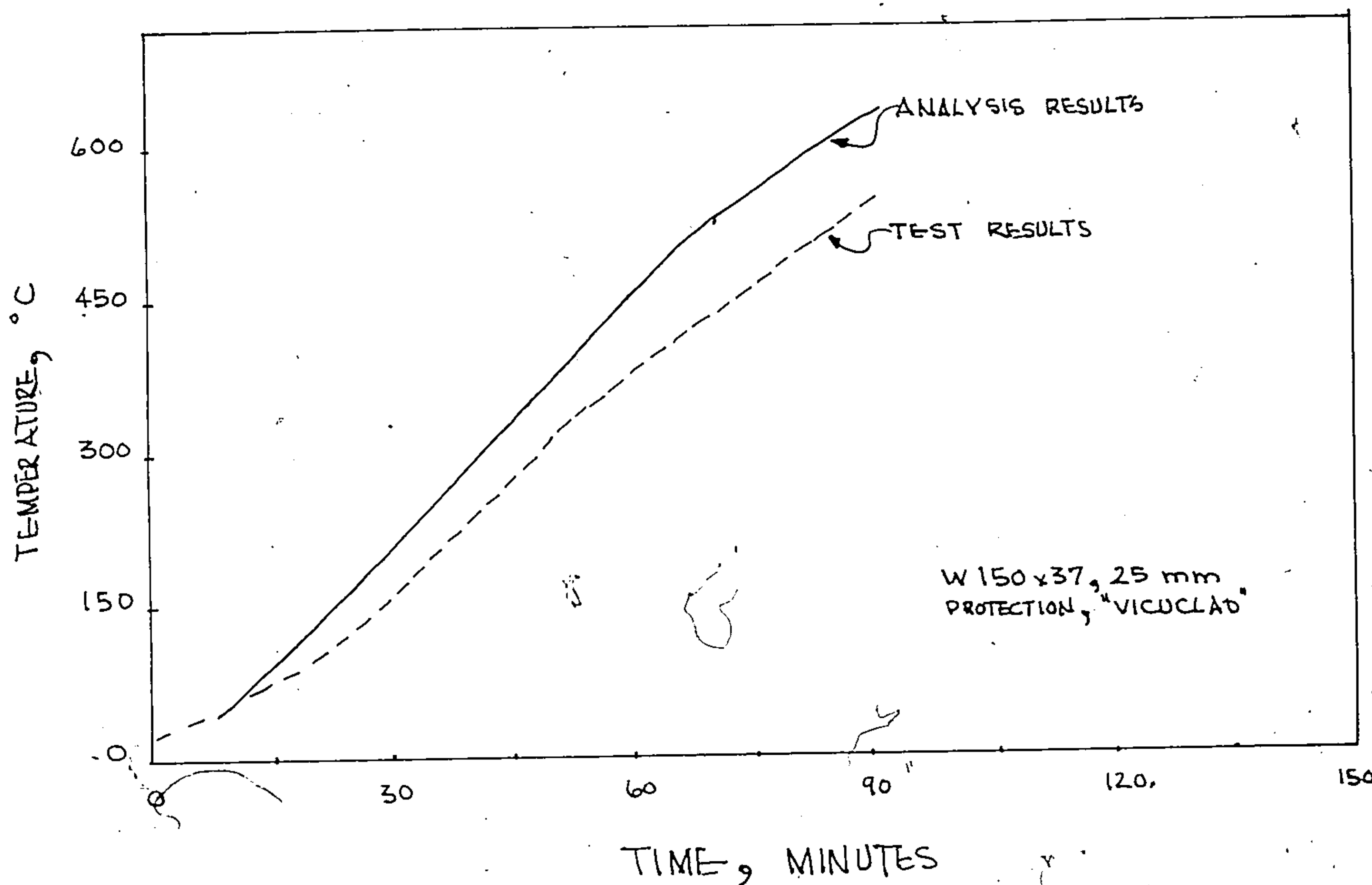


Figure 20

Steel Temperatures

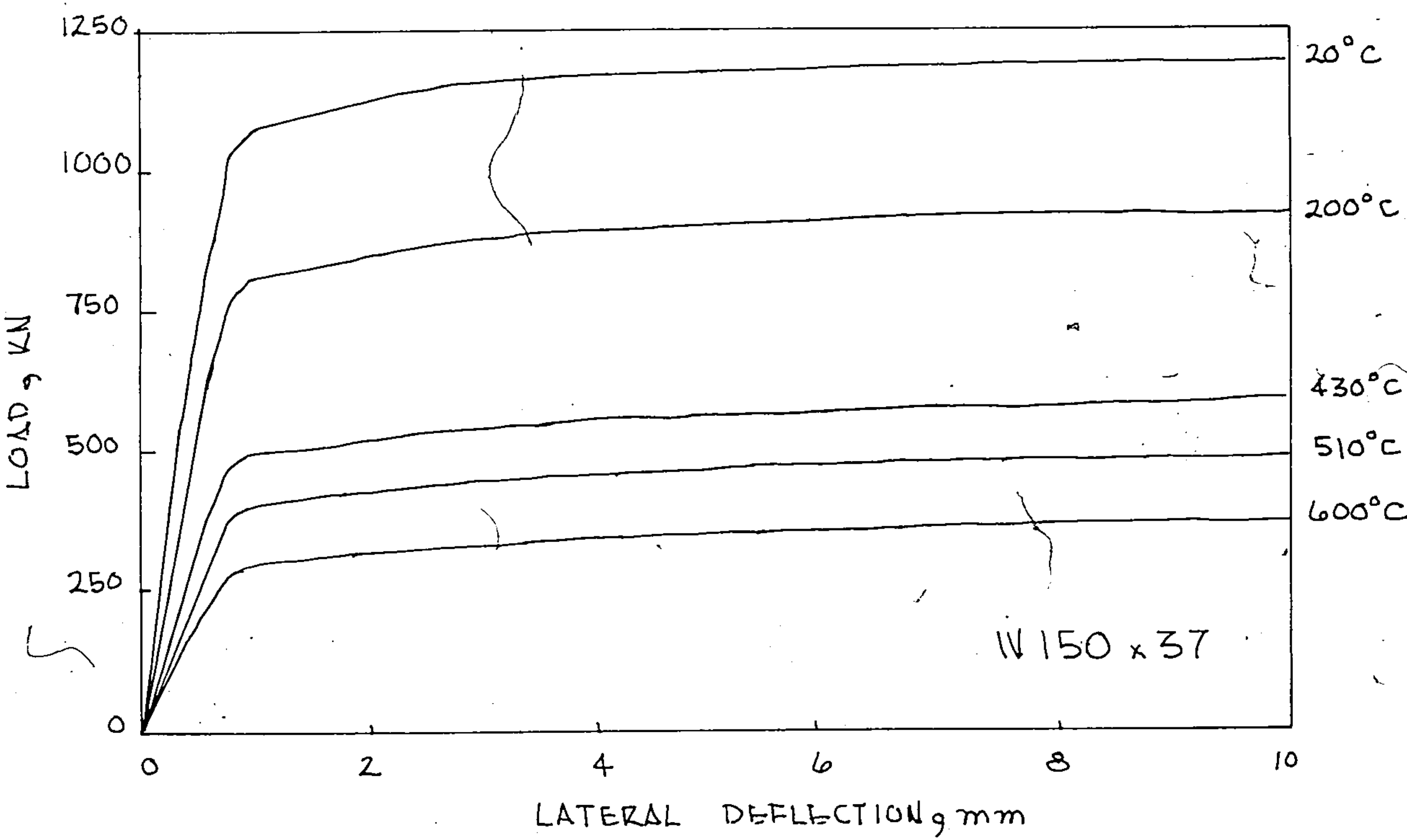


Figure 21

Load versus Lateral Deflection

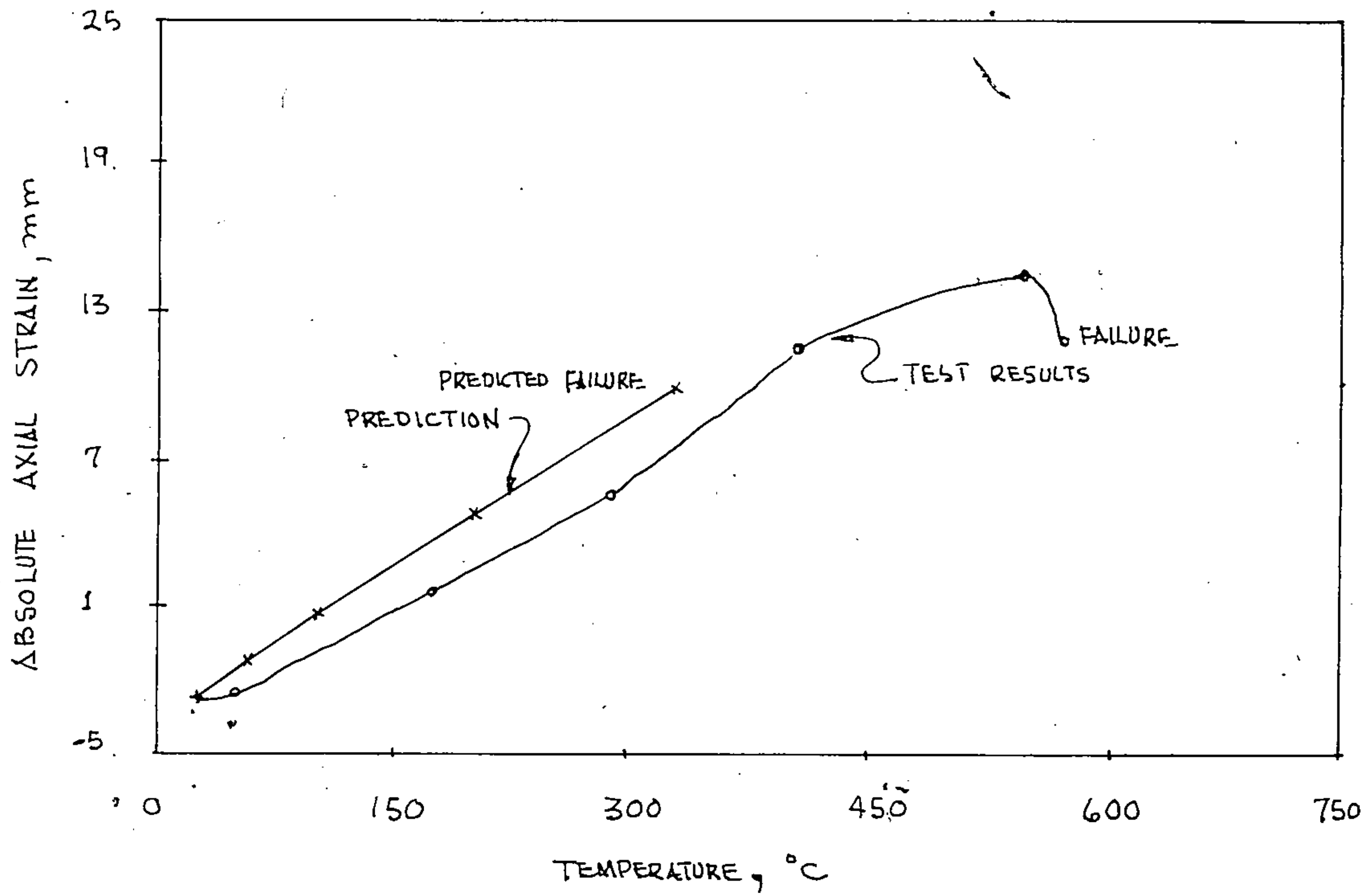


Figure 22

Comparison of Predicted and Test Axial Strains

```
TSS PTF 47+           THE MODULE NAME AND VERSION ARE WIDEFP   ,02/25
OPTIONS-PUBLIC CSLCT(N),BCD MODE(N),ISD(Y),  LISTS-SOURCE(Y),OBJECT

C           FIRE RESISTANCE OF WIDE FLANGE STEEL COLUMNS
           IMPLICIT   REAL*8(A-H,O-Z)
           DIMENSION TS(50),EPSR(10,10),EPSL(10,10),FSR(10,10)
          1EPSLMM(10,10),FSRMM(10,10),FSLMM(10,10),RESID(10,10
           READ(1,1)NTIME,MI,NI,DX,ECC,XKL,FY
       1   FORMAT(3I5,3F10.5,F20.5)
           WRITE(3,1)NTIME,MI,NI,DX,ECC,XKL,FY
C            INITIAL CONDITIONS
           NI1=NI+1
           DO 555 I=1,MI
           DO 555 J=1;NI1
     555   AS(I,J)=0.D0
           EPY=4.D-12*FY
C            CALCULATION OF CO-ORDINATES OF STEEL ELEMENTS
           DO 500 I=1,MI
           Z(I,1)=DX
          RESID(I,1)=55.D6
     500 AS(I,1)=12.0*DX**2
           DO 501 J=2,NI1
           Z(1,J)=DX*(6.0*J-7.0)
          RESID(1,J)=(55.D6-145.D6*Z(1,J)/((NI1*6-7.0)*DX))
     501 AS(1,J)=36.0*DX**2
C              INPUT OF STEEL TEMPERATURES AND CALCULATION OF ST
           DO 502 IT=1,NTIME
          Y=0.0001
           READ(1,3)TS(IT),TINEMI
       3   FORMAT(2F6.1)
           ALPHAS=0.004D-06*TS(IT)+12.D-6
          P3=0.0
           EPTS=ALPHAS*(TS(IT)-20.D0)
           F001=6.9D6*(50.D0-0.04D0*TS(IT))*(1.D0-DEXP((-30.D0
C              MOFF INITIALIZTION
    1500   RHO=1.D0/Y*XKL**2/12.D0
           NUM=0
           IND=0
           INDIC=0
           NUMB=0
           MMM=0
           NNN=0
           EP=0.D0
    1510   P=0.D0
           XM=0.D0
C               CALCULATION OF STRAINS
           DO 503 I=1,MI
           DO 503 J=1,NI1
           IF(AS(I,J)-0.0)503,503,600
     600 RESTRA(I,J)=RESID(1,J)/(F001/0.001D0)
           EPSR(I,J)=-EPTS+EP+Z(I,J)/RHO+RESTRA(I,J)
           EPSL(I,J)=-EPTS+EP-Z(I,J)/RHO+RESTRA(I,J)
C             CALCULATION OF STRESS-STRAIN RELATIONS AND STRESSE
```

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           IF(EPSR(I,J).GE.0.D0)GO TO 4
           C=1.D0
           GO TO 5
       4   C=-1.D00
       5   IF(DABS(EPSR(I,J))-EPY)6,6,7
       6   FSR(I,J)=C*F001/0.001D0*DABS(EPSR(I,J))
           GO TO 8
       7   FSR(I,J)=C*6.9D6*(50.D0-0.04D0*TS(IT))*(1.D0-DEXP((
          1+C*(F001/0.001D0*EPY-F001)
       8   IF(EPSL(I,J).GE.0.D0)GO TO 9
           C=1.D0
           GO TO 10
       9   C=-1.D0
      10   IF(DABS(EPSL(I,J))-EPY)11,11,12
      11   FSL(I,J)=C*F001/0.001D0*DABS(EPSL(I,J))
           GO TO 12
      12   FSL(I,J)=C*6.9D6*(50.D0-0.04D0*TS(IT))*(1.D0-DEXP(
```

,02/25/83 10:45:23 PAGE 001

),OBJECT(N),CROSS REF(Y),SYMBOL TABLE(N),MEMORY MAP(N),LIST DATA SET(Y).

UMNS

R(10,10),FSL(10,10),Z(10,10),AS(10,10),EPSRMM(10,10),-
ID(10,10),RESTRA(10,10)

1 of

NTS

*DX))

ON OF STEEL PROPERTIES

((-30.D0+0.03D0*TS(IT))*DSQRT(0.001D0)))

STRESSES IN STEEL

PAGE 002

0-DEXP((-30.D0+0.03*TS(IT))*DSQRT(DABS(EPSR(I,J))-EPY+0.001D0)))-

0-DEXP((-30.D0+0.03*TS(IT))*DSQRT-

```
      IF(EPSR(I,J).GE.0.D0)GO TO 4
      C=1.D0
      GO TO 5
    4 C=-1.0D0
    5 IF(DABS(EPSR(I,J))-EPY)6,6,7
    6 FSR(I,J)=C*F001/0.001D0*DABS(EPSR(I,J))
      GO TO 8
    7 FSR(I,J)=C*6.9D6*(50.D0-0.04D0*TS(IT))*(1.D0-DEXP((-30.D0+0.0
     1+C*(F001/0.001D0*EPY-F001)
    8 IF(EPSL(I,J).GE.0.D0)GO TO 9
      C=1.D0
      GO TO 10
    9 C=-1.D0
   10 IF(DABS(EPSL(I,J))-EPY)11,11,12
   11 FSL(I,J)=C*F001/0.001D0*DABS(EPSL(I,J))
      GO TO 13
   12 FSL(I,J)=C*6.9D6*(50.D0-0.04D0*TS(IT))*(1.D0-DEXP((-30.D0+0.(
     1(DABS(EPSL(I,J))-EPY+0.001D0)))+C*(F001/0.001D0*EPY-F001)
C       CALCULATION OF LOAD
   13 PS=(FSR(I,J)+FSL(I,J))*AS(I,J)*2.D0
      P=P+PS
C      CALCULATION OF BENDING MOMENT
      XMS=(-FSR(I,J)+FSL(I,J))*AS(I,J)*Z(I,J)*2.D0
      XM=XM+XMS
  503 CONTINUE
C      CALCULATION OF MOMENT GENERATED BY AXIAL LOAD
      PECCY=P*(ECC+Y)
      IF(TINFMI-0.D0)420,2002,2003
 2002 FACT=2.D0
      IF(ECC.LT.0.004D0)FACT=20.D0
      GO TO 2004
 2003 FACT=20.D0
 2004 IF(DABS(XM-PECCY)-0.02D0*XM)2303,2303,2005
 2005 IF(NUMB-1)2006,2110,2110
 2006 IF(NUM-1)2010,2008,2008
 2008 IF(DABS(XM-PECCY)-0.02D0*XM)2303,2303,2110
 2010 IF(P-0.D0)2015,2015,2020
 2015 EP=EP-0.001/FACT
      GO TO 1510
 2020 IF(XM-0.D0)2300,2030,2030
 2030 IF(DABS(XM-PECCY)-0.02D0*XM)2303,2303,2040
 2040 IF(XM-PECCY)2050,420,2080
 2050 IF(INDIC-0)420,2060,2070
 2060 EP=EP+0.001/FACT
      IND=IND+1
      GO TO 1510
 2070 EP=EP+0.0005/FACT
      NUM=NUM+1
      GO TO 1510
 2080 IF(IND-0)420,2090,2100
 2090 EP=EP-0.001/FACT
      INDIC=INDIC+1
      GO TO 1510
```



```
 2100 EP=EP-0.0005/FACT
      NUM=NUM+1
      GO TO 1510
 2110 IF(XM-PECCY)2120,2303,2130
 2120 IF(NNN.EQ.1)GO TO 2300
      EP=EP+0.0001/FACT
      NUMB=NUMB+1
      MMM=1
      GO TO 1510
 2130 IF(MMM.EQ.1)GO TO 2300
      EP=EP-0.0001/FACT
      NUMB=NUMB+1
      NNN=1
      GO TO 1510
 2300 WRITE(3,2302)NUMB
 2302 FORMAT(////'0',I20,'MOMENTS NOT BALANCED WITHIN 2 PERCENT; N
 2303 IF(Y.EQ.0.001)GO TO 2304
```

9

```
.D0-DEXP((-30.D0+0.03*TS(IT))*DSQRT(DABS(EPSR(I,J))-EPY+0.001D0)))-
```

2 of

```
.D0-DEXP((-30.D0+0.03*TS(IT))*DSQRT-
.001D0*EPY-F001)
```

```
.D0
```

LOAD

5

0

0

9

```
'11-IT 2 PERCENT; NUMBER=',I3)
```

```
2302  FORMAT(////'0',T20,'MOMENTS NOT BALANCED WITHIN 2 PERCENT; NUMBER=
2303  IF(Y.EQ.0.001)GO TO 2304
      IF(PB.GE.P)GO TO 2304
      PB=P
      IF(Y.GE.0.053)GO TO 2304
      Y=Y+0.001
      GO TO 1500
2304  WRITE(3,2305)TINEMI
2305  FORMAT('OTIME=',1X,F7.1,1X,'MIN')
      WRITE(3,14)TS(IT)
  14  FORMAT('  TEMP STEEL (C) ',T26,F6.1)
      DO 520 M=1,MI
      DO 505 N=1,NI1
      EPSRMM(M,N)=EPSR(M,N)*1.D3
      EPSLMM(M,N)=EPSL(M,N)*1.D3
      FSRMM(M,N)=FSR(M,N)*1.D-6
 505  FSLMM(M,N)=FSL(M,N)*1.D-6
      WRITE(3,15)(EPSRMM(M,N),N=1,NI1)
      WRITE(3,16)(EPSLMM(M,N),N=1,NI1)
      WRITE(3,17)(FSRMM(M,N),N=1,NI1)
      WRITE(3,18)(FSLMM(M,N),N=1,NI1)
  15  FORMAT('STRAIN ON RIGHT  MM/M',T26,16F6.1)
  16  FORMAT('  STRAIN ON LEFT  MM/M',T26,16F6.1)
  17  FORMAT('  STRESS ON RIGHT  MPA ',T26,16F6.1)
      WRITE(3,222)(RESTRA(M,N),N=1,NI1)
 222  FORMAT(10F8.6)
  18  FORMAT('  STRESS ON LEFT  MPA',T26,16F6.1)
 520  CONTINUE
      PWR=P*1.D-3
      XMWR=XM*1.D-3
      PECCYW=PECCY*1.D-3
      WRITE(3,19)PWR
      WRITE(3,191)XMWR
      WRITE(3,192)PECCYW
  19  FORMAT(1X,' TOTAL LOAD = ',F10.2,1X,'  KN')
 191  FORMAT(1X,' TOTAL MOMENT (SUM OF MOMENTS OF ELEMENTS) = ',F10.2,1X
 192  FORMAT(' MOMENT (LOAD*(ECCENTRICITY+DEFLECTION)) ='F10.2,1X,'KN-M
      EPTOT=EP*3100.0
```

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```
      YMM=1000.D0*Y
      WRITE(3,20)EP
      WRITE(3,201)EPTOT
      WRITE(3,202)YMM
  20  FORMAT('  RELATIVE AXIAL STRAIN =',F10.7)
 201  FORMAT('  TOTAL AXIAL STRAIN =',F10.4,1X,' MM')
 202  FORMAT('  LATERAL DEFLECTION AT MIDHEIGHT =',F10.2,1X,' MM')
      IF(Y.NE.0.001)GO TO 502
      Y=Y+0.001
      GO TO 1500
 502  CONTINUE
 420  CONTINUE
      STOP
      END
```

2 PERCENT; NUMBER=',I3)

3 of 3

MENTS) = ',F10.2,1X,'KN-M')
)) ='F10.2,1X,'KN-M')

9

0.2,1X,' MM')