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Comparison between Measured and Computed Structural Response of some Reinforced Concrete Columns in Fire

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1. INTRODUCTION

This paper is the first result of recently commenced cooperation in structural fire engineering research between the Lund Institute of Technology, and the Fire Research Department of the Technical University of Brunswick. The goal of the project is to develop improved calculation methods based on more reliable behaviour models of concrete and steel. For this reason tests on columns carried out at the Swedish Board for Testing in Borås, Sweden, as well as at Brunswick have been used for verification purposes.

The paper deals with a qualitative verification of the structural behaviour of some reinforced columns exposed to fire on three sides. The columns investigated were tested in Sweden. A computer program developed at Lund [1] was used to evaluate the thermal response. The structural behaviour was predicted by another program developed at Brunswick [2]. The concrete and steel analytical model used in the structural program was originated by Schneider and Haksever [3].

The calculations made herein illustrate the need for improved behaviour models for concrete and steel. Therefore, the next step will be to introduce new material models into the structural program in order to achieve more reliable analytical predictions.

2. TESTING PROCEDURE

The principal testing arrangement is illustrated in Fig. 1 which shows the concrete column placed at the vertical furnace opening ($b \times l \times h = 3 \times 1.8 \times 3$) m^3. The hydraulic loading is applied either centrically or eccentrically, as illustrated in Fig. 1. The column is heated on three sides and lightweight concrete walls close the furnace (see Fig. 2).

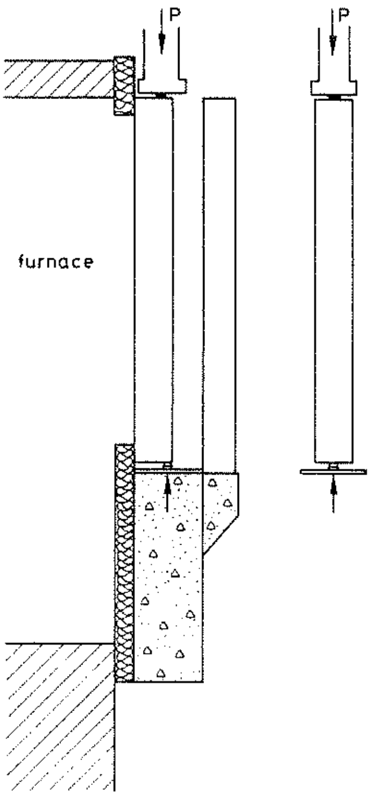

Fig. 1. Testing facilities.

The fire exposure is followed, according to ISO 834, by 9 thermocouples placed around the column as shown in Fig. 3. The horizontal and axial deformations were measured by 10

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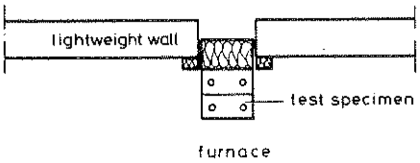

Fig. 2. Section of furnace from above.

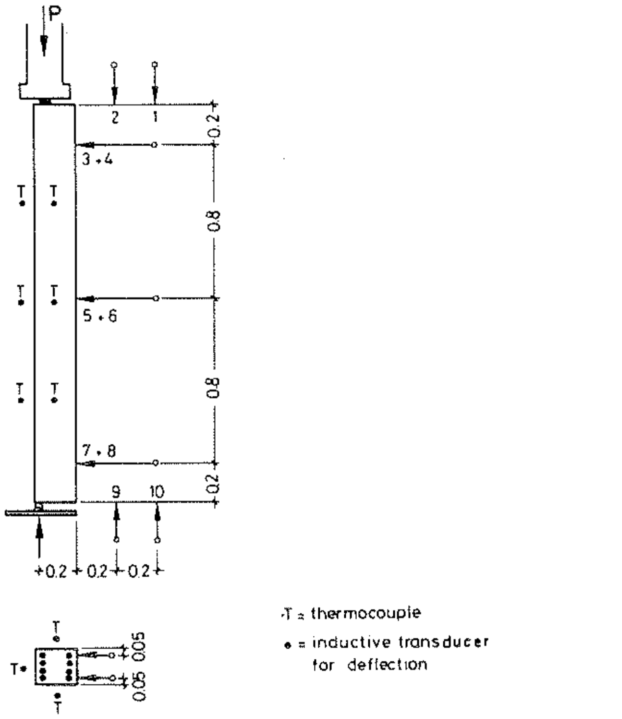

Fig. 3. Situation of thermocouples and inductive transducers for deflection measurements.

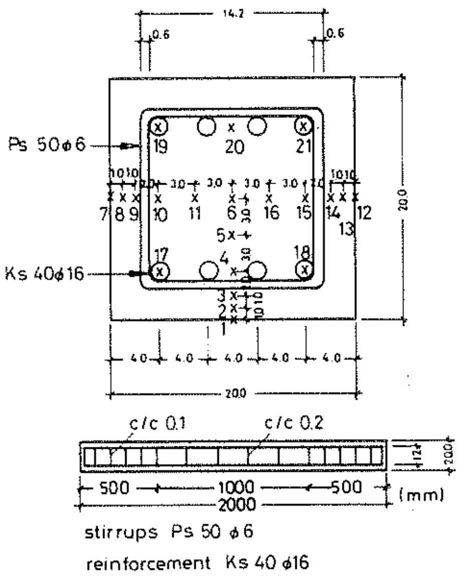

Fig. 4. Section of column. Reinforcement and situation of thermocouples.

inductive transducers placed as illustrated in the Figure.

The reinforcement (Ks 40, ϕ 16, hot-rolled) of the column (length 2 m) and the situation of 21 thermocouples in mid-section is given in Fig. 4.

3. MATERIAL DATA

The concrete used for the column specimens had a cube strength of about 46 MPa referred to the testing age (110 days). The compressive strength used in calculation was $0.8 \times 46 = 34$ MPa. Bending strength was determined on an unreinforced beam, according to the Swedish Standard, to be about 4.4 MPa.

The moisture content of the concrete was measured on cubes $(0.15 \times 0.15 \times 0.15)$ m^3 in connection with the main testing and was about 6%. This value is taken into account in temperature calculations.

The reinforcement Ks 40, ϕ 16 has a proof strength $\sigma_{0.2}$ = 453 MPa and an ultimate strength σ_{ult} = 716 MPa, which was used in the program. The stirrups of the column shown in Fig. 4 are Ps 50, ϕ 6.

4. RESULTS

Three reinforced concrete columns fire-exposed on three sides were studied analytically. Predicted thermal and structural response was compared with measurements for concentrically and eccentrically (± 6 cm) loaded columns.

The loadbearing capacity at room-temperature was determined by testing at Lund for an eccentricity of 0 and 6 cm. The failure load was measured as 1.95 and 0.95 MN, respectively. The data of the three tests carried out at Borås and studied in this paper are given in Table 1.

4.1. Temperature calculations

The thermal properties, thermal conductivity and the enthalpy of concrete used in the calculation were taken from ref. 4. The enthalpy curve has been modified to take account of the high moisture content.

The predicted and measured temperatures are illustrated in Fig. 5. This Figure gives the temperature gradient of six points at the mid-section of the column as a function of time. The very good agreement is significant, but at

TABLE 1

Test and calculation data

Column	Load (MN)	Eccentricity (cm)	Fire resistance (min)	
			Tested	Calculated
SL-1	0.90	0.00	52	65
SL-2	0.60	+ 6.00*	30	55
SL-3	0.30	−6.00**	120	120

*Eccentricity towards the furnace.
**Eccentricity from the furnace.

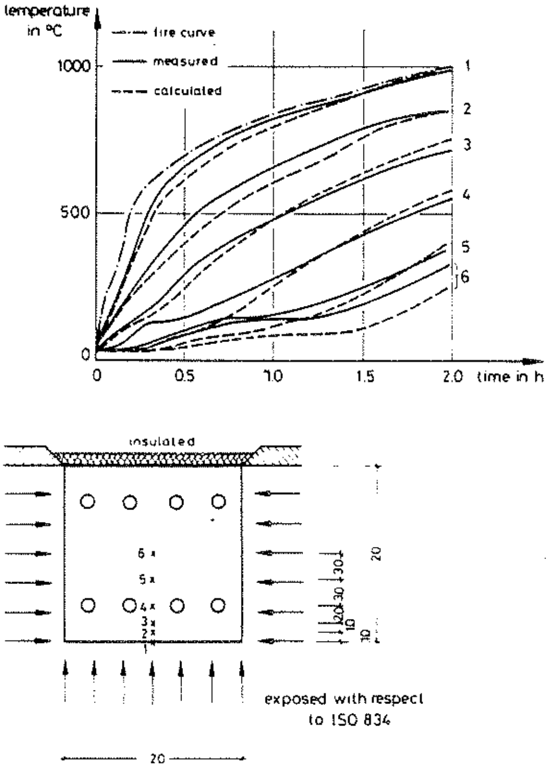

Fig. 5. Measured and calculated temperatures in the midsection of the column.

temperatures below 150 °C the simultaneous moisture vapourization and mass transport cause a slight discrepancy. This is inevitable, as in the Lund TASEF-2 temperature program it is assumed that the moisture content is successively vapourized on site as the steam temperature is reached. This means that the capillary moisture transport is neglected in the program. Predicted temperatures are then inserted into the structural program.

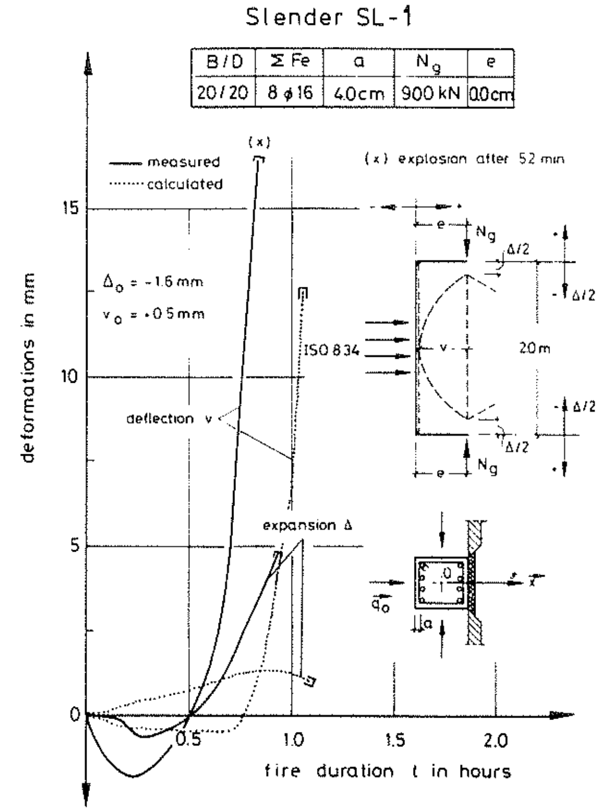

Fig. 6. Measured and calculated behaviour of a reinforced concrete column in a fire, axially loaded to 0.9 MN.

4.2. Structural response

In the original study column SL-1 was centrically loaded to 0.9 MN (= 0.46 P_{ult}). The midpoint deflection and axial deformation are illustrated in Fig. 6. In the test the column exploded after 52 min due to the high moisture content (see Section 3) and high load level, so that the fire resistance time can only be estimated. Failure was, however, imminent, as may be seen by the high deflection rate at the time of the explosion. The estimated failure time is about 60 - 65 min.

The predicted and estimated fire resistance times are relatively close to each other.

In the test, the deflection was towards the furnace and changed sign at 0.5 h. This behaviour was not obtained in calculation. The reason for this is connected with the shortcomings in material modelling. The axial deformation during the first 0.5 h of the test was almost zero, but subsequently increased. At commencement the predicted deformations were greater, but after half the fire resistance time they were smaller than the measured deformations. The discrepancy is apparent,

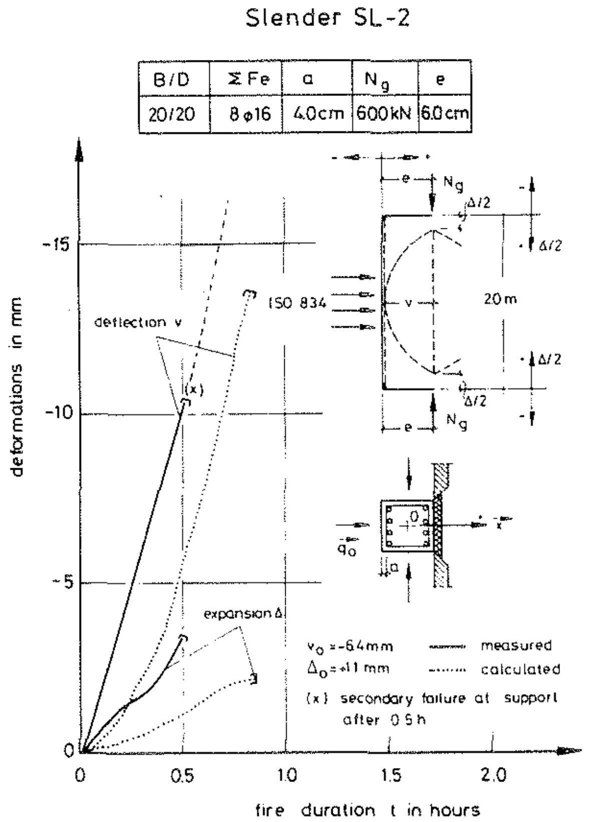

Fig. 7. Measured and calculated behaviour of a reinforced concrete column in a fire, eccentrically loaded to 0.6 MN.

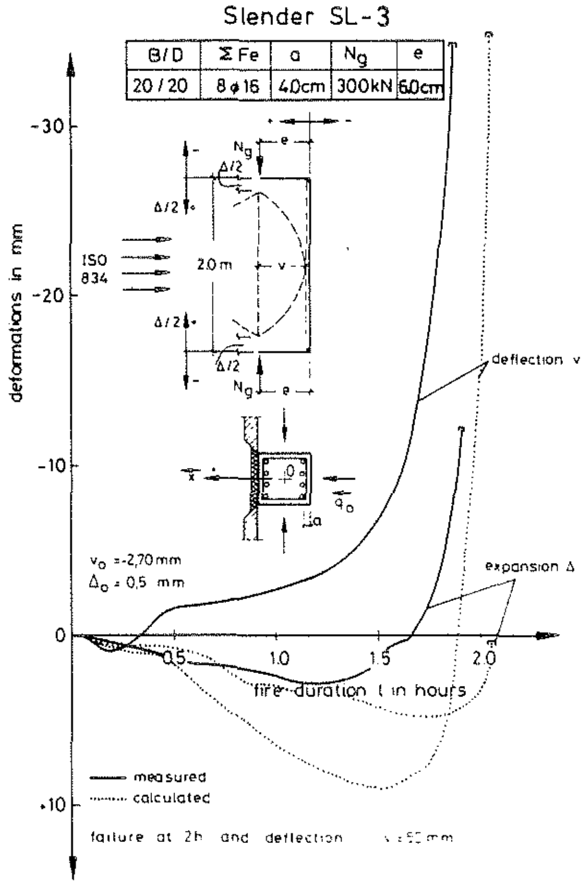

Fig. 8. Measured and calculated behaviour of a reinforced concrete column in a fire, eccentrically loaded to 0.3 MN.

but the mode of failure was predicted correctly, *i.e.*, the results are qualitatively quite acceptable.

In test SL-2 the column was loaded to 0.6 MN with an eccentricity of 6 cm from the furnace. The load corresponded to 63% of the ultimate load under normal atmospheric conditions. In the test the measurements were stopped after 0.5 h due to a support failure, which was a mishap, but a comparison is still of interest. The predicted and test estimated fire resistance times were about 1.0 and 0.8 h, respectively, and deformations here were closer to measurements. The predicted curves are almost parallel to the measured curves and give smaller deformations (Fig. 7.)

In the final comparison between measured and predicted behaviour column SL-3 was loaded to 0.3 MN with an eccentricity of 6 cm towards the furnace. This load corresponded to 31% of the ultimate load under normal atmospheric conditions. The fire resistance time predicted is in good agreement with the measured value of 1.9 h, but the deflection was only qualitatively in agreement.

The predicted deflection was directed towards the furnace for most of the time, but after 1.5 h it diminished, changed sign, and followed the measured curve until failure occurred. In both calculation and measurement the column expanded axially during the first 1.5 h, but then transferred into compression. The reason for the sudden change into the failure state is due to the fact that the compression zone of the cross-midsection moves inwards and the tensile stress in the reinforcement is increased to such a high value that yielding occurs. This behaviour results in a rapid deflection of the column and instant failure. This mode of behaviour can be seen in both test and calculation (see Fig. 8).

In the comparisons made here it must be emphasized that the measured deflections were taken from only one test specimen and the accuracy of measurement cannot be stated. This means that these latter values are somewhat uncertain and must be used with caution.

5. CONCLUSIONS

The Lund TASEF-2 temperature program predicts the temperature range, for use in the Brunswick structural HP 010 program with reasonable accuracy, as illustrated in Fig. 5. Both programs are efficient and of great importance in solving the thermal and structural response to fire of any concrete structure.

The predicted results from the current version of the HP 010 program are extremely dependent on the developed rules used for concrete and steel at elevated temperatures. This means that the material models must be improved. Predicted and measured behaviour are, as illustrated in Figs. 6 - 8, in satisfactory agreement. The mode of failure and fire resistance time are quite acceptable, but the discrepancy in the deflection process is unsatisfactory.

The problem in predicting detailed structural behaviour is due mainly to the fact that the analytical models of concrete and steel need to be improved and further developed. For instance, the influence of the loading history and the heating rate are not taken into account in the concrete model and only to some extent in the steel model. The steel model is only based on transient tests carried out at a constant load and constant heating rate [5, 6]. It has been shown in ref. 4 that in such tests the heating rate in a particular temperature and loading range has a great effect.

Neglect of the loading history influence often results in a higher calculated compression stiffness of the column and lower axial deformation. In a column where the internal stress distribution changes continuously, it may be of great importance to consider the influence of the loading history in conjunction with the heating rate. This hypothesis will be checked both experimentally and analytically.

6. FUTURE PLANS

The next step in the cooperative project will be to introduce into the structural program concrete and steel material behaviour models [7] developed at Lund in an attempt to determine whether the discrepancy in the detailed structural response can be reduced.

Every material model is a simplified "real life" version and can evaluate real behaviour only to a certain degree; the goal is to develop an improved system, which is suitable for practical purposes. Models developed in this way will be more effective in such a structural study if the loading history and heating rate are taken into consideration.

Earlier predicted behaviour of axially restrained, reinforced concrete columns [8] tested at Brunswick have hitherto not been successful unless the heating rate was considered empirically. These tests will also be studied analytically.

The aim of the cooperation is to develop improved computational methods so that realistic predictions can be made of the response of any concrete structure to fire. This will reduce the need for expensive and comprehensive fire testing and will simultaneously create a basis for the development of fire design methods for more complicated structural systems.

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