Fire Resistance of Load-Bearing Reinforced Concrete Walls

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ABSTRACT

This paper describes a numerical method of calculating the fire resistance of load-bearing reinforced concrete walls. Each wall is divided into a number of segments of height and elements of thickness. The calculated temperature profile through the wall gives the temperature in each element which is used with the constitutive relationships and mechanical properties to predict structural behaviour. The model can consider a range of boundary conditions including pinned, fixed and partially fixed ends. It is shown that walls with a small amount of end fixity top and bottom have very good structural behaviour under fire exposure.

KEYWORDS

Computer, concrete, design, fire, load-bearing, model, reinforced, resistance, walls

INTRODUCTION

Concrete structures have a good reputation for fire resistance. There are not many reported cases of serious structural failure resulting from collapse of individual walls or beams or columns. However this may change as buildings are designed with lower safety margins using higher strength concretes.

When a fire occurs in a building, walls are required to limit the spread of fire and maintain their structural adequacy. To be fire resistant, walls should have adequate loadbearing capacity, insulation (limiting the temperature rise of unexposed face of wall) and integrity (no significant cracks and fissures), as shown in figure 1. To satisfy the loadbearing function, a wall is required to carry its imposed loads for the duration of the fire. This paper summarizes a recent study to investigate the loadbearing capacity of concrete walls exposed to fire [1].

LITERATURE REVIEW

New Zealand codes are typical of many countries. The New Zealand fire code [2] specifies the levels of fire resistance required in buildings. Exterior walls and interior

walls between fire compartments are required to have a fire resistance rating of up to 4 hours (1.0 or 1.5 hours in most cases), depending on occupancy and distance to the boundary. Approved fire resistance ratings for elements of construction are described by MP9 [3]. The required minimum thicknesses vary from 75 mm for one hour to 175 mm for four hours. These thicknesses are based on the insulation criterion, and apply to both loadbearing and non-loadbearing walls, with a suggestion that highly loaded or slender walls receive special study. These thicknesses are generally the same as those required for concrete floor slabs.
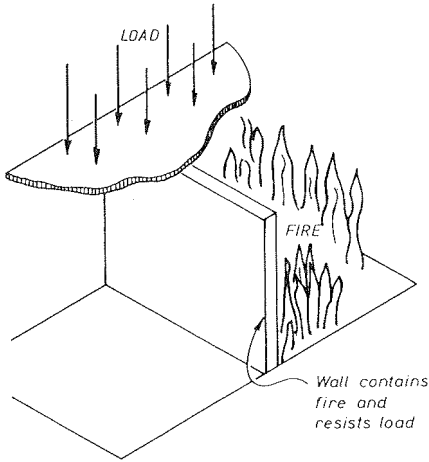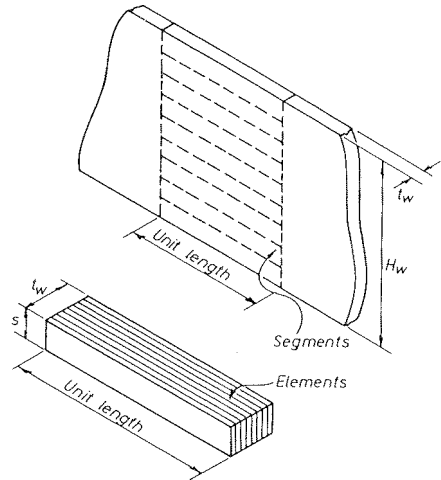

Fig.1 - Functions of wall in a fire

Fig.2 - Wall divided into elements and segments

Elsewhere, the concrete design code [4] specifies that loadbearing and non-loadbearing walls shall have minimum thicknesses of 150 mm and 100 mm respectively. No guidance is given for calculating fire resistance.

Internationally, design guides are becoming available for flexural members such as slabs and beams, but there are apparently no codes or design guides specifying design methods for reinforced concrete walls or columns, other than specifying minimum dimensions and cover.

There have not been many studies of concrete compression members under fire conditions. Becker et al [5] invesigated reinforced concrete frames, and Purkiss and Weeks [6] studied columns. Anderberg and Forsen [7] describe analytical and experimental studies on concrete columns. Cooke [8] has looked at tall walls, but the only significant computer study of walls has been that of O'Meagher and Bennetts [9] which forms the basis for this paper. Lie et al [10] have made an extensive experimental study of concrete columns. No experimental studies of reinforced concrete walls under significant axial loads are known of.

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HEAT TRANSFER MODEL

Heat transfer through a large wall is a one dimensional heat flow problem which can be solved by conventional means. In this study, a finite difference program has been used to provide a simple description of temperatures in the wall at any time, using thin elements as shown in figure 2. The reinforcing is assumed to be at the same temperature as the surrounding concrete.

Any time-temperature curve could be used as input, but this study uses the standard time temperature curve specified by ISO 834. The fire-exposed face of the wall is assumed to be slightly cooler than the furnace temperature, by a factor which varies from 0.6 at time zero to 0.96 after one hour of exposure, from recent tests [11].

MECHANICAL PROPERTIES OF MATERIALS

This paper uses the total strain model proposed by Anderberg and Thelandersson [12] where the total strain (ε_T) is decomposed into four components; thermal strain (ε_{th}), stress related strain (ε_σ), creep strain (ε_{cr}) and transient strain (ε_{tr}), such that

$$\varepsilon_T = \varepsilon_{th} + \varepsilon_\sigma + \varepsilon_{cr} + \varepsilon_{tr} \tag{1}$$

A series of differing testing regimes is required to evaluate these components of strain. There are no standardized testing methods, so substantial variations will be caused by different rates of straining, rate of temperature rise, specimen size and initial moisture content, and the type of steel. Care is needed in comparing test data from different sources. Unless otherwise referenced, the theory that follows is from references [9,12,13].

Thermal Strain

For concrete this is the unrestrained thermal expansion after allowance has been made for drying shrinkage, It is more convenient to use a composite model which includes drying shrinkage, so that the thermal strain during heating can then be expressed as a simple function of the temperature.

Stress-related Strain

For concrete, the general description of the stress-strain relationship is a parabolic rising branch followed by a linear descending branch. The ultimate stress, the strain at ultimate stress, and the slopes of the ascending and descending branches are all temperature dependant [14,15]. Most tests have been performed on specimens heated without loads, although the most realistic values of ultimate stress are obtained from tests where specimens under constant stress are heated to failure. Ultimate strain is significantly affected by the prehistory of stress. It is possible to express the prehistory of stress at a given time by the accumulated transient strain. The descending branch of the stress-strain curve is less important. The slope is given a constant value independent of temperature and stress history.

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For steel a simple elasto plastic bilinear stress strain relationship is assumed. The modulus of elasticity and yield stress both drop with increasing temperature.

Creep Strain

For concrete, expressions are available which relate the creep under constant stress and temperature to the stress as a proportion of ultimate stress, and to temperature. Such an expression can be modified to give the accumulated creep strain after a given stress history over a period of time. For steel, creep strain varies with temperature, stress and time. The basis of the creep strain model is the creep theory proposed by Dorn and modified by Harmathy [16], who showed that the entire course of the creep curve is uniquely determined by two stress-dependent parameters. In this project, creep is ignored below temperatures of 200°C.

Transient Strain

Until a decade, or so ago, the majority of investigators considered only the need for steady state tests. It was not until Anderberg and Thelandersson [12] performed tests under constant stress and varying temperature, measuring the resultant strain was it realized that transient effects were important. More recently Khoury et al [17] explained this phenomenon very clearly. The transient strain is in part due to aggregate-mortar incompatibilities, problems in the application of instantaneous loading during stress-strain tests and creep tests, and imposition of thermal gradients during heating in all types of tests. Transient strain cannot itself be measured but must be determined from the remaining four components, by re-arranging equation (1). The resulting transient strain component is a very important component in the model for concrete. There is no transient strain for steel.

STRUCTURAL MODEL

There has been an increasing demand for computer programs capable of simulating the response of structures exposed to fire. The aim of this project was to develop a computer model to predict the performance of a concrete wall exposed to fire. The only program readily available [9] was restricted to walls with both ends pinned. In the present project, that program has been modified to model different end conditions.

Scope

The theoretical model is comprehensive and it allows for the variations of the following parameters: wall thickness and height, load magnitude and eccentricity, self-weight of the wall, reinforcement quantity and cover, concrete material properties, reinforcement material properties, and P-delta effects. A range of boundary conditions at the ends of the wall can be considered, including both ends pinned, both ends fixed, one end fixed and the other end propped, one end fixed and the other end free, and both ends with springs of known stiffness.

Axial loads are considered to be constant throughout fire exposure, hence axial restraint forces due to thermal expansion are not included. This is a reasonable assumption for most buildings which rely on structural walls for both gravity load and lateral load

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resistance. The effects of axial restraint should only be considered if the building contains stiff beams which could re-distribute vertical forces to other walls or columns which are not subjected to the same fire exposure.

Structural Analysis

Structural analysis of members with combined bending and axial load is not simple, and is complicated by geometrical non-linearities (P-delta effect) and material non-linearities. No closed form analytical solution is possible even under cold conditions, so numerical methods must be used. A sophisticated calculation scheme has been developed by Nathan [18] for reinforced or prestressed concrete beam-columns, producing column-deflection curves for members of various length or end condition. The scheme has also been used for timber members [19]. In a fire study such as this one, the geometry and material properties are altered due to thermal exposure at each step, so a complete structural solution must be found at each time step and Nathan's scheme is not easily applicable. This leads to the use of considerable computer time, but produces the required results.

Cross Section Behaviour

The wall of unit length is divided into a number of segments top to bottom as shown in Figure 2. Each segment is identically divided into a number of transverse elements across the thickness. Within any segment it is assumed that plane sections remain plane, hence the curvature is considered to be constant.

At each cross section at each step, a trial strain diagram (or cross section curvature) is proposed for the segment, from which the strain in the concrete and steel in each element of thickness can be obtained. The stress related strains are then obtained from the total strain using the constitutive law.

The concrete and steel stresses are obtained using the stress-strain laws for the materials, modified for increased temperature in each element. These stresses are multiplied by the element area to give element forces which are summed over the cross section to give axial load and bending moment at the cross section. These actions are compared with the external axial load and moment to see if equilibrium has been achieved. If not, a new set of strains is proposed and the process is repeated until successful.

Once a plastic hinge occurs, there is no longer a unique set of strains (or section curvature) for a given combination of axial load and moment. When no set of strains can be found to achieve equilibrium, the wall is regarded as having failed.

Deflections of Wall

To allow for wall deflections and "P-delta" effects it is necessary to calculate the lateral displacement of the wall at each segment boundary. From the curvatures of the wall at each segment boundary, the deflected shape of the wall can be determined using incremented calculations from a point of known deflections or slope. If the change in the displaced shape between successive calculations is significant then the revised displaced shape is used to estimate a new "P-delta" moment for each segment boundary

and the analysis is repeated for each segment boundary up the wall. When a very small change between successive displaced shapes is obtained it is considered that a solution for the current time step has been found.
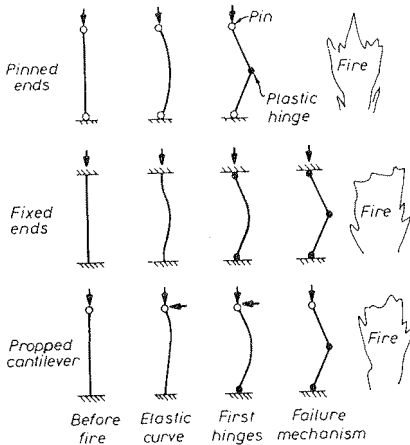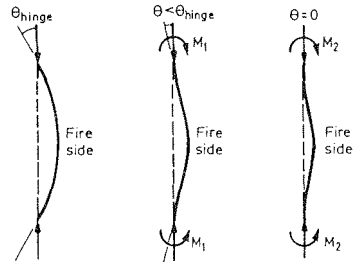

Fig.3 - Failure mechanisms   Fig.4 - Solution for fixed end wall

Boundary Conditions

The calculation procedure depends on the boundary conditions. For statically determinate systems such as walls pinned at both ends and cantilever walls, the first cross section failure implies failure of the whole system (figure 3, top).

For statically indeterminate systems, two or more plastic hinges must occur before the system fails (figure 3). To obtain a solution at each time step, a trial and error process is required, varying the initial actions until a selected rotation or displacement has the required value. This increases computational effort by an order of magnitude with yet another layer of trial and error loops. In this study the selected variables were the end rotation for fixed end walls (figure 4), and the prop deflection for propped cantilevers (figure 5).

RESULTS

Precast Walls

Pinned connections. Figure 6 shows typical output for a wall with pins at both top and bottom. This output is for a wall 3000mm high, 150mm thick, with $275mm^2/m$ reinforcing in the centre and a vertical axial load of 50kN. It can be seen that the model predicts a mid-height deflection of about 100mm after two hours of fire exposure. When P-delta effects are considered the deflections increase by only about 10%, but this reduces the fire resistance from four hours to two-and-a-half hours.
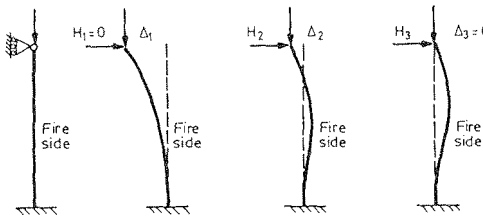
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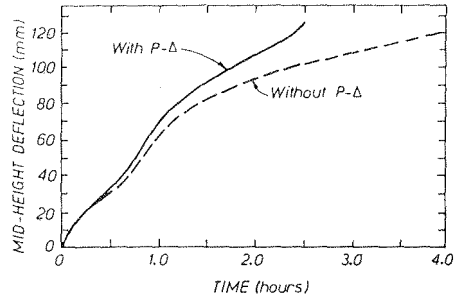
Fig.5 - Solution for propped cantilever   Fig.6 - Influence of P-delta effects

End eccentricity. The assumption of pins at both ends is a poor one. For a wall with square ends and no continuity, the eccentricity of the applied load will change as the wall deforms, as shown in figure 7. The current program cannot allow for incremental change as the wall deforms, but if the analysis is carried out with an end eccentricity of 75mm, the deflection of the wall is reduced by approximately 20% and the fire resistance is increased significantly.

Effect of load and slenderness. The effects of axial load and slenderness on the same pinned-pinned wall analysed above are shown in figure 8. It can be seen that for loads above 50 kN, the fire resistance drops steadily with increasing slenderness, roughly proportional to the level of axial load.
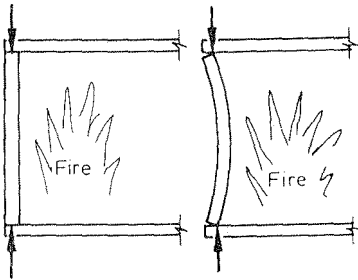

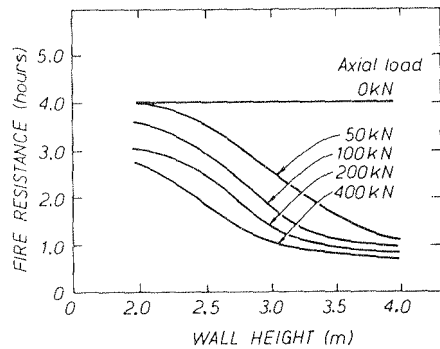Fig.7 - Eccentricity due to wall deformation   Fig.8 - Effect of load and height on FRR

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Continuous Walls

Most walls in real buildings have some continuity at top and bottom, especially in seismic regions such as New Zealand. A typical continuous wall in a multi-storey building is shown in figure 9. Such a wall can be modelled as a single height wall with pins at each end, and springs of rotational stiffness matching the walls in the storeys above and below. Typical output is shown in figure 10. The line for 100% stiffness is for the same wall extending over several equal height storeys with a pin connection to the slab edges. Deflections for this case are very small, never exceeding 10mm. Even with 25% of this stiffness the deflections are very much less than for the pinned condition (zero stiffness), and the fire resistance increases accordingly.
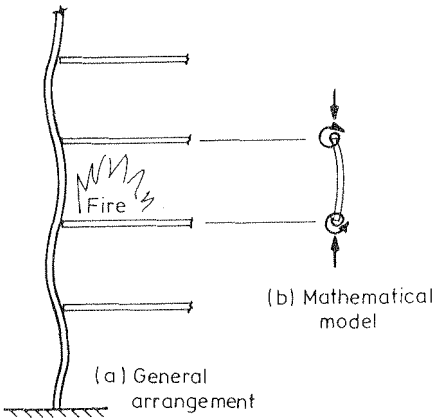

(b) Mathematical model

(a) General arrangement

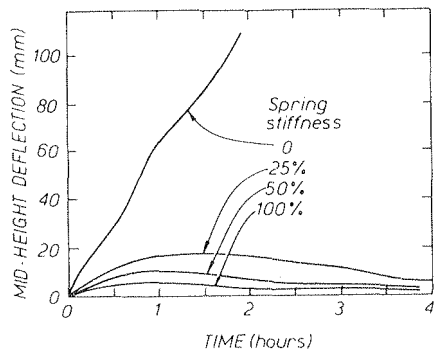Fig.9 - Wall in multi-storey building

Fig.10 - Effect of spring stiffness on deflection

Cantilever Walls

Many industrial buildings have fire resistant walls supporting non-fire-resistant roof structures. When a fire occurs in such a building, the roof collapses leaving a cantilever wall exposed to fire from one side. There are no axial loads other than self-weight, and lateral loads are ignored. The calculated deflections for such a wall become very large and failure occurs as a result of P-delta moments under self weight, assuming no lateral loads.

If the roof in a single storey building has fire resistance, then the roof may hold the top of the wall in position, hence it may act as a propped cantilever as shown in figure 5. The effect of the top restraint is to greatly reduce deformations, hence increase   fire resistance.

CONCLUSIONS

This paper has shown that a powerful analytical tool can be developed for analysing the behaviour of reinforced concrete walls in a fire environment. To model structural behaviour correctly it is essential to understand the behaviour of the wall correctly, especially the boundary conditions. The case studies described in the paper and the original report [1] lead to the following conclusions about wall behaviour:

(1)   The quantity of reinforcement is not as significant as cover, slenderness ratio or end restraint.

(2)   Increasing axial load decreases the fire resistance of all walls.

(3)   End conditions are very important. Walls with rotational restraint at one or both ends behave very well because deflections are very small.

(4)   Slender walls with high axial loads and pinned supports have poor fire resistance, because deflections soon get out of control.

(5)   Cover to main reinforcement is very important. Hence 200 mm thick walls with two layers of reinforcement have less fire resistance than 175 mm thick walls with one layer of reinforcement, all other parameters being equal.

This model should be refined to consider end conditions more accurately, especially to allow for changes in the end eccentricity as the fire develops. The model should be used to investigate a larger number of realistic case studies to identify which types of walls are at risk, for further study. This should include the use of real fire time-temperature curves.

ACKNOWLEDGEMENTS

REFERENCES

1.   Munukutla, V.R. "Modelling fire performance of concrete walls". Civil Engineering Research Report 89-5. University of Canterbury, New Zealand, 1989.

2.   Fire resisting construction and means of egress. NZS 1900, Chapter 5. Standards Association of New Zealand, 1988.

3.   Fire properties of building materials and elements of structure. Document MP9:1989. Standards Association of New Zealand, 1989.

4.   Design of concrete structures NZS 3101:1982. Standards Association of New Zealand, 1982.

5. Becker, J.M. and Bresler, B. Reinforced concrete frames in fire environments. University of California, Berkeley, Report No. UCB FRG WP 75-6, 1975.

6. Purkiss, J.A. and Weeks, N.J. "A computer study of reinforced concrete columns in a fire". he Structural Engineer 65B(1), pp. 22-28, 1987.

7. Anderberg, Y. and Forsen, N.E. "Fire resistance of concrete structures". Division of Building Fire Safety and Technology, Lund Institute of Technology Report LUTVDG/(TVBB-3009), 1982.

8. Cooke, G.M.F. "Fire engineering of tall fire separating walls". Fire Surveyor, June 1987, August 1987.

9. O'Meagher, A.J. and Bennetts, I.D. "Behaviour of concrete walls in fire". Proc. First National Structural Engineering Conference, Melbourne, pp.438-445, 1987.

10. Lie, T.T., Lin, T.D., Allen, D.E. and Abrams, M.S. "Fire resistance of reinforced concrete columns". Division of Building Research, National Research Council of Canada, DBR paper No.1167, 1984.

11. Woodside, A. Ruiter, J.J. and Wade, C.A., "Fire resistance of New Zealand concretes", SR34 Building Research Association of New Zealand, 1991.

12. Anderberg, Y. and Thelandersson, S. "Stress and deformation characteristics of concrete at high temperatures". Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology, Bulletin 54, 1976.

13. Purkiss, J.A. "High temperature effects". Proc. of the International Conference on Design of Structures Against Fire, Aston University, Birmingham, 1986.

14. Dougill, J.W. "Modes of failure of concrete panels exposed to high temperatures". Magazine of Concrete Research 24, 71-76, 139-148, 1982.

15. Furamura, F. "The stress-strain curve of concrete at high temperatures. Annual Meeting of the Architectural Institute of Japan, No.1, Paper No. 7004, 1976.

16. Harmathy, T.Z. "A comprehensive creep model". Journal of Basic Engineering, Transactions of the ASME, 89-D (3) 496-502, 1967.

17. Khoury, G.A., Grainger, B.N. and Sullivan, P.J.E. "Transient thermal strain of concrete: literature review, conditions within specimen and behaviour of individual constituents". Magazine of Concrete Research, 37(132):131-144.

18. Nathan, N.D. "Slenderness of prestressed concrete columns". Journal of PCI, 28(2):50-77, 1983.

19. Buchanan, A.H. "Combined bending and axial loading in lumber". Journal of Structural Engineering, 112 (12): 2592-2609, 1986.