Zeitschrift: IABSE proceedings = Mémoires AIPC = IVBH Abhandlungen

Band: 8 (1984)

Heft: P-73: Long-term deflections of reinforced concrete frames

Artikel: Long-term deflections of reinforced concrete frames

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DOI: https://doi.org/10.5169/seals-38335

Long-Term Deflections of Reinforced Concrete Frames

Déformation à long terme de cadres en béton armé

Langzeitverformungen von Rahmen aus Stahlbeton

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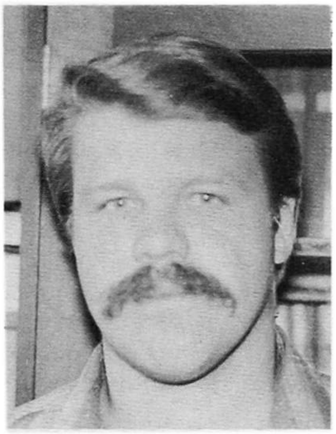

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SUMMARY

Long-term tests on prototype sized, closed rectangular reinforced concrete frames are described and the principal results are presented. A second order nonlinear frame analysis programme is presented. The reliability of the programme is assessed by comparing the values predicted by it with results obtained from the tests on the closed rectangular frames and also from tests conducted by other investigators on fixed base portal frames. The results of ancillary material tests, which define the short-term and long-term material characteristics are given.

RESUME

Des essais de longue durée ont été réalisés sur des cadres rectangulaires fermés, en béton armé. Les résultats principaux sont présentés. Un programme de calcul non linéaire du second ordre est présenté. Sa fiabilité peut être constatée en comparant les valeurs calculées aux résultats d'essais sur les cadres fermés, ainsi qu'aux essais réalisés par d'autres chercheurs sur des cadres portiques. Les résultats d'essais de matériaux, à court et long terme, sont présentés.

ZUSAMMENFASSUNG

Langzeitversuche an geschlossenen, rechteckigen Rahmen aus Stahlbeton werden beschrieben und die wichtigsten Resultate daraus aufgezeigt. Ein nichtlineares Berechnungsprogramm zweiter Ordnung wird vorgestellt. Die Zuverlässigkeit des Programmes kann abgeschätzt werden, indem die berechneten Werte mit den Resultaten aus Versuchen an den rechteckigen Rahmen sowie aus den Versuchen an unten gehaltenen Portal-Rahmen anderer Forscher verglichen werden.

1. INTRODUCTION

The analysis of a reinforced concrete frame under sustained service loads is a difficult and time consuming task. To successfully model frame behaviour, account must be taken of the effects of the various sources of material non-linearity such as cracking, tension stiffening, creep and shrinkage. Additionally in the case of frames with slender compression members, the effects of geometric non-linearity has to be considered. It is only in recent years that this has been achieved using sophisticated computer based models (Lai [1], Schnobrich and Scordelis [2], Aldstedt and Bergan [3]). Such models are extremely valuable research tools from which much can be learned. However they usually place heavy demands on computer resources both in terms of storage requirements and solution time, and are generally unsuitable for routine design. Practising engineers have no simple, reliable method for predicting long-term frame deflections and there is little well documented experimental data from tests on prototype-sized frames from which such procedures could be developed.

The data which is necessary for the calibration of simplified procedures is now potentially available as output from these computer models, provided of course, that the models are accurate and reliable.

Whether a computer model uses sophisticated material modelling laws (Valanis [4], Bažant and Bhat [5,6], Gilbert and Warner [7]) and refined, layered, finite elements (Lai [1], Schnobrich and Scordelis [2], Gilbert and Warner [7]) or a more design-oriented model, which aims for simple input data and more approximate material and numerical modelling (Aas-Jakobsen and Grenacher [8], Corderoy [9], Favre et al [10,11]), its accuracy can only be assessed by comparison of the results with those obtained from physical tests.

In this paper, long-term tests on prototype sized, closed rectangular frames are described and the principal results are presented. A second order non-linear frame analysis programme is presented. The validity of this programme is tested against results from tests, on closed rectangular frames, conducted for this investigation and also against the results of tests, conducted by Drysdale [12], on fixed base portal frames.

2. TEST SPECIMENS, PROCEDURES AND INSTRUMENTATION

2.1 Frame Tests

Long-term tests were carried out on two closed rectangular frames designated 3F1 and 3F2. The cross-section of the frames was 150 mm deep and 100 mm wide with either one or two layers of reinforcment. The reinforcement consisted of structural grade 12 mm deformed bars. The dimensions of the frames and the reinforcement layout are shown in Figure 1.

The closed rectangular shape was chosen because it provided, for each frame, a two-fold replication of beams and columns and a four-fold replication of the beam-column connection. Concentrated loads were applied at the midspan of the beams by a simple reaction yoke which produced a self equilibrating system of applied loads and thereby eliminated the need for column supports with specified boundary conditions. The load was monitored by means of a load ring which was in series with the applied loads. The loading ring was designed to provide a sensitivity of approximately 1% of the level of load used in the tests.

Both frames were cast simultaneously in horizontal impervious plywood moulds and, together with the companion creep and shrinkage specimens, were moist cured for 14 days. The specimens were stripped and tested in a laboratory with controlled temperature (22±2°C) and humidity (the relative humidities are plotted in Figure 7).

The frames were tested in the vertical plane with the self weight taken on a

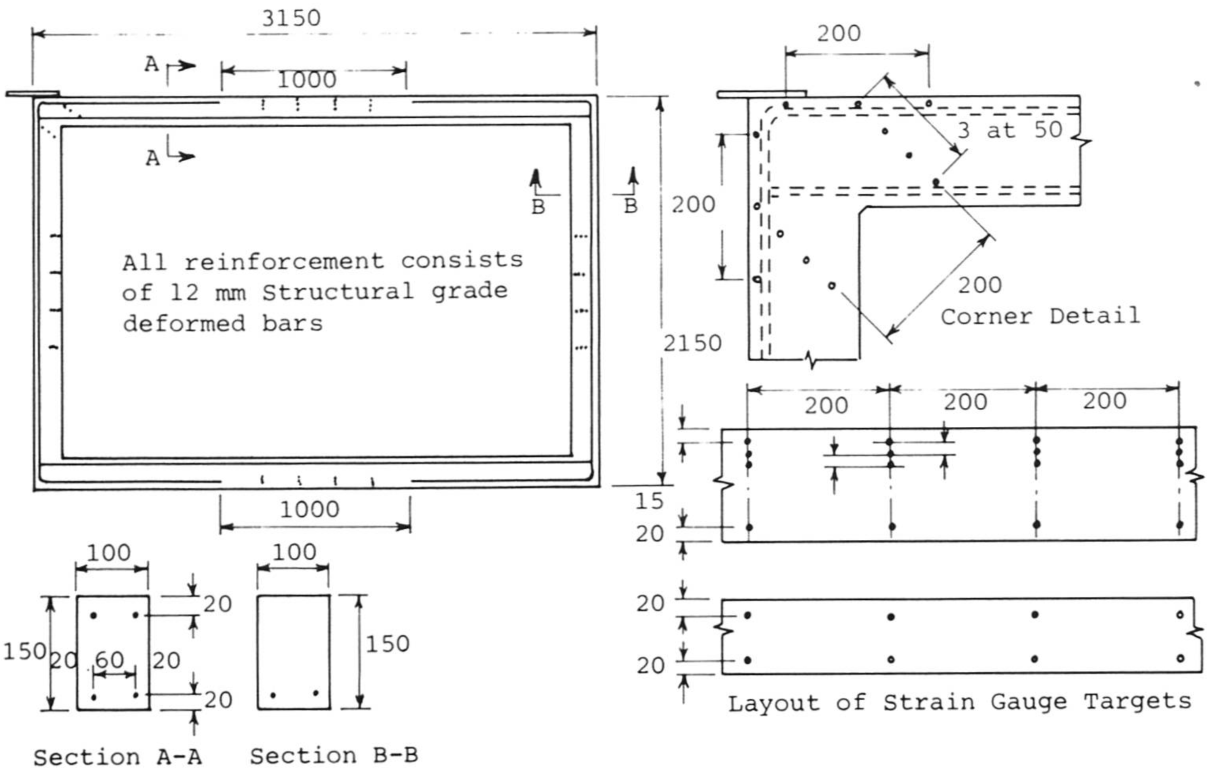


Fig. 1 Details of rectangular closed frames

pinned and a roller support under the columns as shown in Figures 2 and 3. The frames were held in the vertical plane by teflon coated guides attached to a reference frame. The guides were "just" touching the test frames and did not impede their in-plane response.

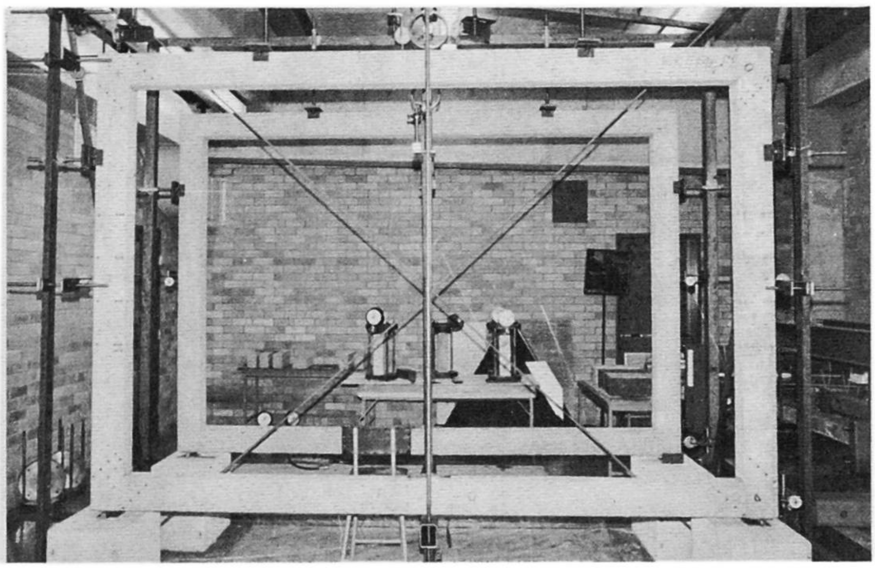


Fig. 2 Frame test set-up

Frame 3F2 carried a sustained load of 7 kN. The same load was briefly applied on frame 3F1 to produce a similar initial crack pattern as that of frame 3F2. Thereafter frame 3F1 carried no applied load.

Deflections were measured with mechanical deflection gauges attached to the reference frame and located as shown in Figure 3. Strain distributions were measured over the middle 600 mm of the beams and columns with a 200 mm gauge length detachable Demec strain gauge. The relative rotations of the beams and columns as well as the time-dependent diagonal compressive strains at the beam-column connections were also measured. The layout of the strain gauge targets are shown in Figure 1.

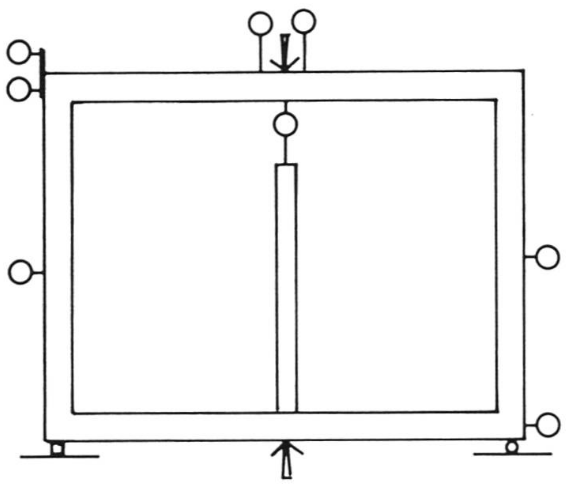


Fig. 3 Layout of deflectometers

2.2 Material Tests

A programme of material testing was carried out to determine the following properties of the concrete:-

(i) compressive strength at various ages,

(ii) modulus of elasticity at various ages,

(iii) flexural tensile strength,

(iv) creep coefficient, and

(v) shrinkage strain.

The compressive strength tests and the flexural strength tests were carried out in accordance with the SAA Code AS 1012 [13]. The moduli of elasticity were determined by tests in accordance with the procedure recommended by the American Society for Testing and Materials, C469-65 [14].

The creep and shrinkage values were obtained from tests on 100 x 150 x 400 mm prism specimens. These specimen sizes were selected because they had the same cross-sections as the test frame. As all the tests were subjected to the same environmental conditions the creep and shrinkage values obtained from the material tests were expected to closely reflect those that occurred within the test frames. The results of the material tests are given in Table 1 and 2. The tests are described in more detail by Bakoss et al [15]. Typical creep and shrinkage curves are given in the Appendix.

The composition of the concrete mix is given in Table 3.

Age at Test (days)	Elastic Modulus ($\times 10^3$ MPa)	Compressive Strength (MPa)	Flexural Strength (MPa)
22	27.3	33.1	-
28	31.2	39.0	3.9
42	31.9	41.3	4.0
276	35.7	55.2	-

Table 1 Elastic Moduli, Compressive and Flexural Strengths

Age (days)	58	130	230	330	430	546
Time Under Load (days)	35	107	207	307	407	523
Period of Shrinkage (days)	44	116	216	316	416	532
Creep Coefficient	1.0	1.5	1.9	2.1	2.3	2.5
Shrinkage Strain (microstrain)	210	390	430	540	630	660

Table 2 Creep Coefficients and Shrinkage Strains

Constituents	
Goliath Portland cement Type A (kg/m^3)	350
Munmorah classified fly ash (kg/m^3)	30
10mm crushed Emu river gravel (kg/m^3)	1050
Nepean river sand (8% moist) (kg/m^3)	450
Cronulla dune sand (5% moist) (kg/m^3)	270
Air-entraining agent (ml/m^3)	50
Water, total (l/m^3)	187
Slump (mm)	80
Water/cement ratio	0.53

Table 3 Details of Concrete Mix Used

NOTE : The coarse aggregate, which contained about 60% granites and 35% quartzites, was batched in the saturated, surface-dry condition.

3. COMPUTER ANALYSES

The long-term behaviour of the test frames was predicted using a second order non-linear analysis computer programme. The programme was a modification of an original programme called SONLA which was developed by Corderoy [9] for the analysis and design of slender reinforced concrete frames. The main features of the programme are that it takes into account (a) the geometric non-linearities resulting from sway (P - Δ effect) and from the interaction of the axial forces with flexure in slender members, (b) the material non-linearities caused by cracking, tension stiffening, shrinkage and creep. The main modification of the programme for this project concerned the treatment of the short-term and long-term material non-linearities. The programme also has a design option which can be used to determine the flexural reinforcement that is required in accordance with AS 1480-1982 [16] for given overall member sizes and concrete strengths.

3.1 Short-Term Effects

An iterative piecewise linear procedure is used in the programme which consists of the following three main stages:-

(a) First-Order Analysis

A first-order linear stiffness analysis is carried out using first order member stiffness matrices based on the given gross cross-sectional properties and the short-term material properties corresponding to the age at loading. This analysis provides an estimate of the axial forces that are required for the second-order analysis.

(b) Second-Order Analysis

In this stage of the analysis the element stiffness matrices are modified to include the effect of axial forces on the internal actions of the elements. The second order element stiffness matrices are represented as

$$[K] = [K_1] + [K_2](N) \qquad (3.1)$$

where $[K_1]$ = the first order stiffness matrix,

$[K_2]$ = the non-linear geometric stiffness matrix,

and

N = axial force in the element.

Matrices $[K_1]$ and $[K_2]$ are given explicitly in Figure 4, their derivation is well documented in standard texts.

The geometric stiffness matrix for each element is calculated based on the axial forces previously obtained in the first order linear analysis. In the calculation of the modified element stiffness matrices the same section and material properties are used as in the first order, analysis. The structure is re-analysed using the modified structure stiffness matrix and the element forces obtained are compared with those obtained from the first-order linear analysis. The stiffness matrices are modified and the structure re-analysed iteratively until the calculated element forces converge to within specified limits.

(i) Element co-ordinates

$\frac{EA}{L}$	0	0	$\frac{-EA}{L}$	0	0
	$\frac{12EI}{L^3}$	$\frac{6EI}{L^2}$	0	$\frac{-12EI}{L^3}$	$\frac{6EI}{L^2}$
		$\frac{4EI}{L}$	0	$\frac{-6EI}{L^2}$	$\frac{2EI}{L}$
			$\frac{EA}{L}$	0	0
Symmetric				$\frac{12EI}{L^3}$	$\frac{-6EI}{L^2}$
					$\frac{4EI}{L}$

(ii) First order element stiffness matrix K_1

(N)

0	0	0	0	0	0
	$\frac{6}{5L}$	$\frac{1}{10}$	0	$\frac{-6}{5L}$	$\frac{1}{10}$
		$\frac{2L}{15}$	0	$\frac{-1}{10}$	$\frac{-L}{30}$
			0	0	0
Symmetric				$\frac{6}{5L}$	$\frac{-1}{10}$
					$\frac{2L}{15}$

(iii) Second-order, Non-linear, Geometric element stiffness matrix K_2

Fig. 4 First-order and second-order element stiffness matrices

(c) Section Analysis

An iterative technique is used to calculate for each element a strain distribution which agrees, to within specified accuracy, with the axial force and bending moment obtained from the second order analysis.

The procedure for calculating the bending moment and axial force that act on a section for a given strain distribution follow the general methods described by Aas-Jakobsen and Grenacher [8], Schnobrich and Scordelis [2], Warner and Lambert [17], and others.

The search for the strain distribution that satisfies equilibrium is commenced by assuming an initial value for the extreme fibre compressive strain ε_c, as shown in Figure 5(i) and which is given by:-

$$\varepsilon_c = \frac{|M|(D)}{2EI} - \frac{|N|}{EA} \tag{3.2}$$

where M = moment obtained from second-order analysis,

N = axial force obtained from second-order analysis,

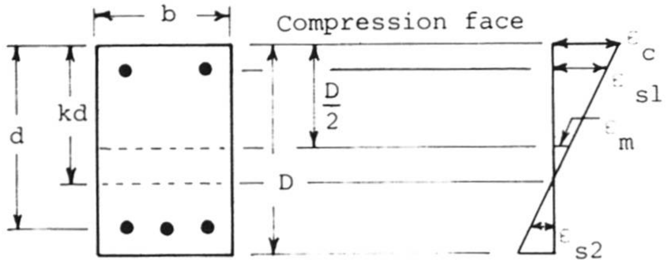
EI = gross flexural rigidity,

EA = gross axial rigidity,

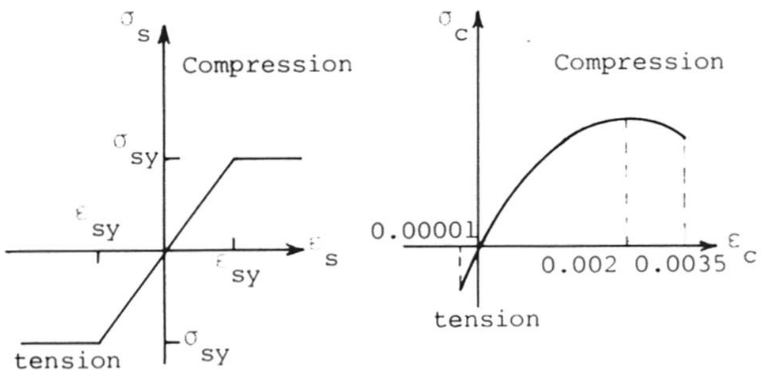
and D = overall depth of section.

The neutral axis depth kd is then incremented from zero until the resultant of the compressive forces in the concrete and compressive steel and the tension forces in the tension steel and tension stiffening zone are in equilibrium with the axial force N.

The internal stresses that correspond to the assumed linear strain distributions are calculated by using the stress v strain relationships for concrete and steel which are given in Figure 5(ii). The tension stiffening contribution of the tensile concrete located between cracks, for a member subjected to flexure, is expressed in accordance with the results of Clark and Speirs [18]. The tension stiffening force TF is given by equation 3.3.



(i) Strain Distribution



Steel σ-ε curve

Concrete σ-ε curve
$\sigma_c = 850\,F'_c\,\varepsilon_c(1-250\varepsilon_c)$ KPa

(ii) Idealised Stress v Strain Curves

Fig. 5 Strain distribution and stress-strain curves used in SONLA

$$(TF) = 0.3\ F'_t\ b\ (D-kd)(\varepsilon_{sp}/\varepsilon_s) \qquad (3.3)$$

where F'_t = ultimate tensile strength of concrete,

D = overall depth of section,

kd = neutral axis depth,

ε_{sp} = steel strain calculated, ignoring the tensile concrete, at first cracking,

and ε_s = steel strain with tensile concrete neglected, at the bending moment under consideration.

The value of ε_{sp} is evaluated in this programme from the expression, developed by Clark and Speirs [8]

$$\varepsilon_{sp} = 200\ (I_u/I_{cr})\ 10^{-6} \qquad (3.4)$$

where I_u = moment of inertia of uncracked section,

and I_{cr} = moment of inertia of cracked section.

The moment M_i, which results from this strain distribution, is compared with the moment M given by the second-order analysis. If they agree to within a specified accuracy then the correct short-term strain distribution has been found. If, however, M_i is larger or smaller than M then the extreme fibre compressive strain is decreased or increased by an increment and the neutral axis depth is again incremented until the sum of the forces on the cross-section equal the axial force N. This procedure is iterated until the extreme fibre compression strain and the neutral axis depth are such that the internal stress resultants are in equilibrium with both the axial force N and bending moment M.

3.2 The Effects of Creep and Shrinkage

The influence of creep on the iterative section analysis is taken into account by increasing the instantaneous strains due to sustained loads by a factor of $k_r\phi$, so that the concrete strains at time T after loading are given by

$$\varepsilon = \varepsilon_{ci}\ (1 + k_r\phi(T)/\gamma) \qquad (3.5)$$

where ε_{ci} = instantaneous concrete strain,

k_r = a reduction factor to account for the restraint offered by the reinforcement,

γ = ratio of initial load to sustained load,

$\phi(T)$ = creep coefficient at time T after loading.

The factor k_r is evaluated from the expression 3.6 given by Branson [19]

$$k_r = \alpha/(1 + p'/2) \qquad (3.6)$$

where $\alpha = 0.5$

and p' = ratio of the area of compression steel to the area of concrete expressed as a percentage.

The creep coefficient at any time T is given by the hyperbolic expression

$$\phi(T) = \phi(\infty)\{T/(T + H)\} \qquad (3.7)$$

where $\phi(\infty)$ = limiting value of creep coefficient at $T = \infty$,

T = time under load,

and H = time required for half of the limiting creep to take place.

Shrinkage effects are taken into account by calculating the ratio of the shrinkage curvature to load induced curvature, as determined in the section analysis, for every element. A weighted average value of this ratio is used as an amplification factor by which the computed deflections are multiplied. This modulates the effects of shrinkage so that shrinkage deflections are included even in exceptional circumstances where the load induced deflections are low.

The shrinkage curvatures of the elements are evaluated from the expression recommended by ACI Committee 435 (Branson [19])

$$\psi_s = (0.7)(\varepsilon_{cs})(\sqrt[3]{(p-p')})\{\sqrt{(p-p')/p}\}/D \qquad (3.8)$$

for $(p-p') \leq 3.0\%$

or

$$\psi_s = \varepsilon_{cs}/D \quad \text{for } (p-p') > 3.0\% \qquad (3.9)$$

where ε_{cs} = unconstrained shrinkage strain,

and p = percentage of tensile reinforcement.

Intermediate values of the unconstrained shrinkage strains are obtained from the ultimate shrinkage strain $\varepsilon_{cs}(\infty)$ by the expression

$$\varepsilon_{cs} = \varepsilon_{cs}(\infty)\ \phi(T)/\phi\ (\infty) \qquad (3.10)$$

The total load induced curvature ψ at time T is then given by

$$\psi = \psi_i\ (1 + k_r\phi(T)/\gamma) \qquad (3.11)$$

where ψ_i = Instantaneous curvature.

Finally to complete the first cycle of non-linear analysis the modified element stiffnesses are calculated as

$$(EI)_m = M_i/\psi \qquad (3.12)$$

and

$$(EA)_m = N/\varepsilon_m \qquad (3.13)$$

In which ε_m is the midplane strain as defined in Figure 5(i).

The modified stiffnesses are then used to re-analyse the structure and re-iterate the steps of the section analysis until the change in the calculated curvatures that occur between successive cycles of calculation does not exceed specified values.

The calculations of one cycle of the section analysis are schematically summarised in Figure 6.

4. MEASURED AND PREDICTED RESULTS

The long-term deflections measured at the midspans of the beams of frames 3F1 and 3F2 are given in Figure 7. In Figure 8 the total long-term beam deflections of frame 3F2 are compared to the corresponding values predicted by the computer programme.

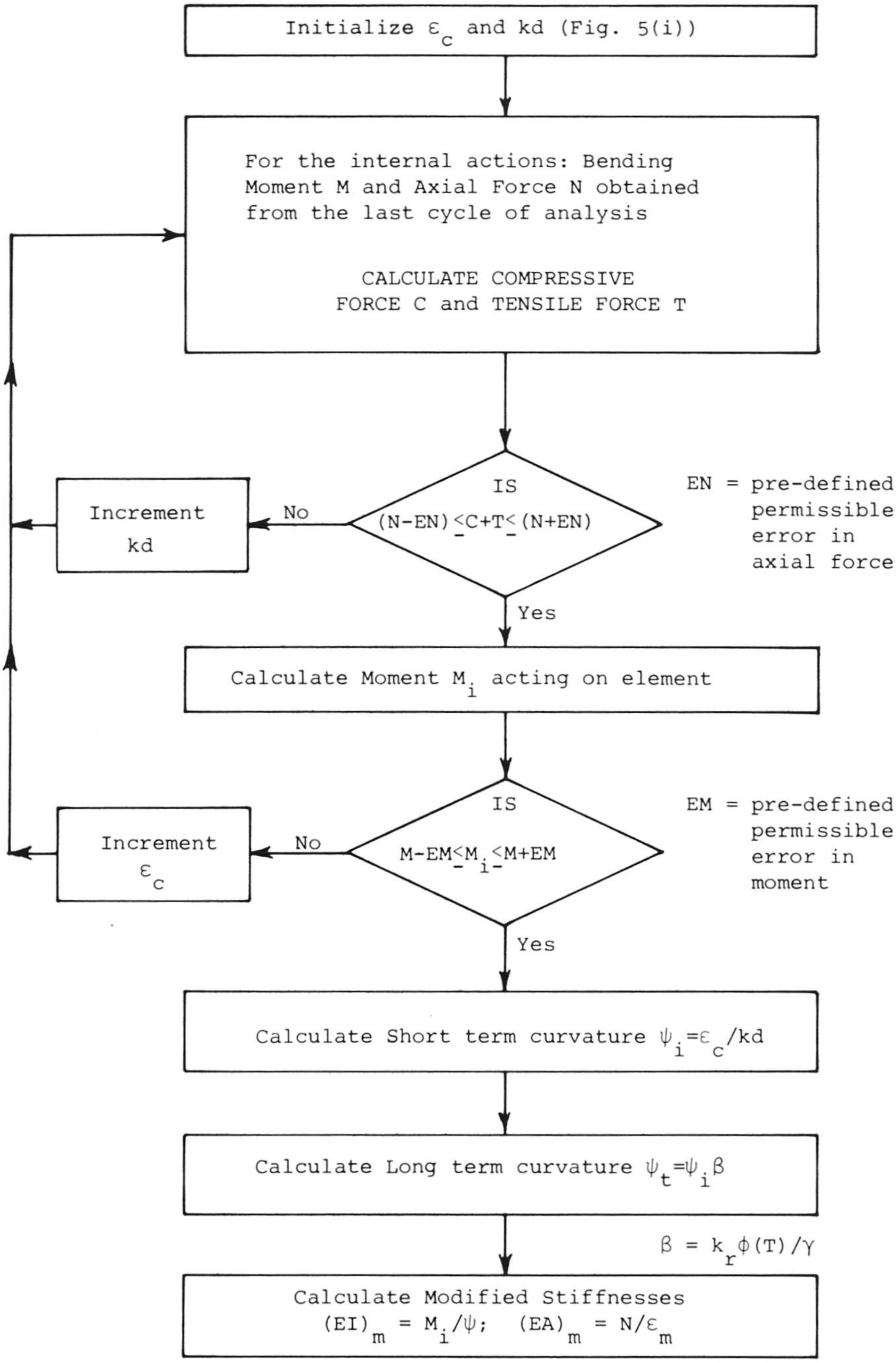


Fig. 6 Iterative procedure for calculating strain distribution-curvature-stiffness for given axial force and bending moment

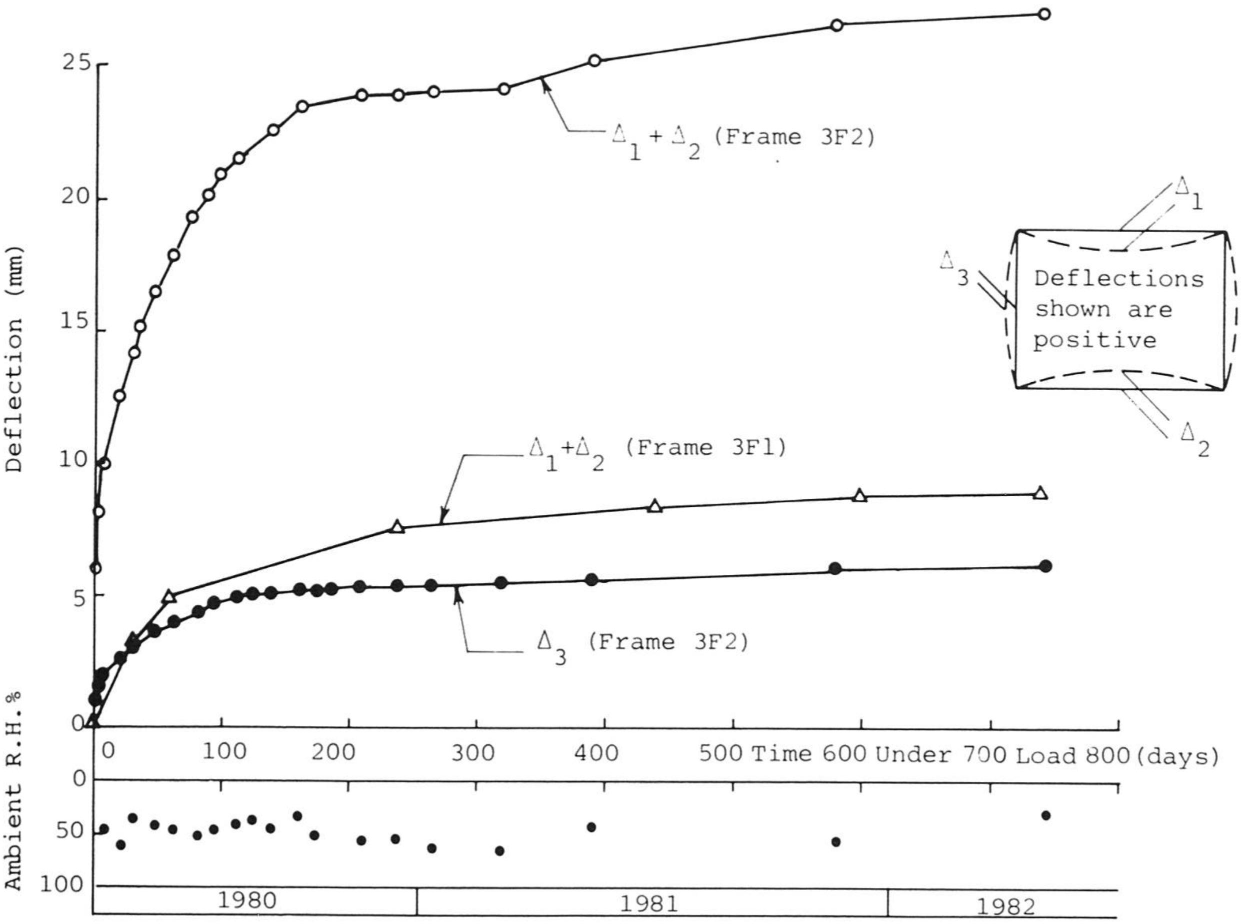


Fig. 7 Midspan deflections of frames 3F1 and 3F2

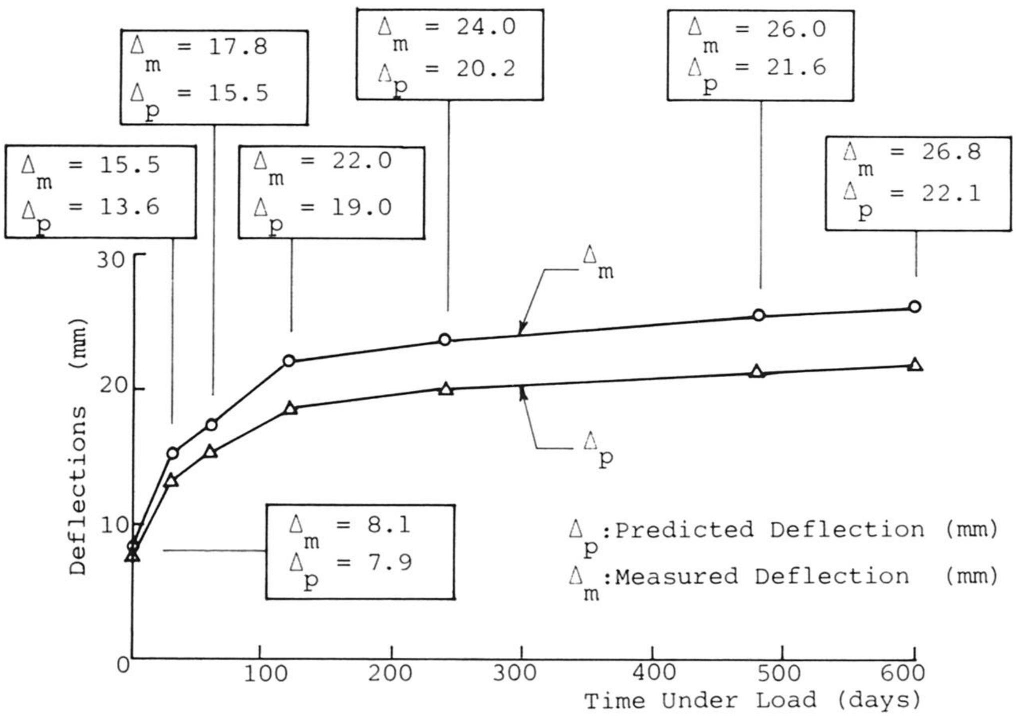


Fig. 8 Predicted and measured total midspan deflections of Frame 3F2

The long-term strain distribution within the corners in the direction of diagonal gauge lines are given in Figure 9. The measured rotations of the beam-column connections and the rotations predicted with the computer are shown in Figure 10.

It should be noted that the values for material properties used as data in the computer analysis were those obtained in the associated materials testing programme.

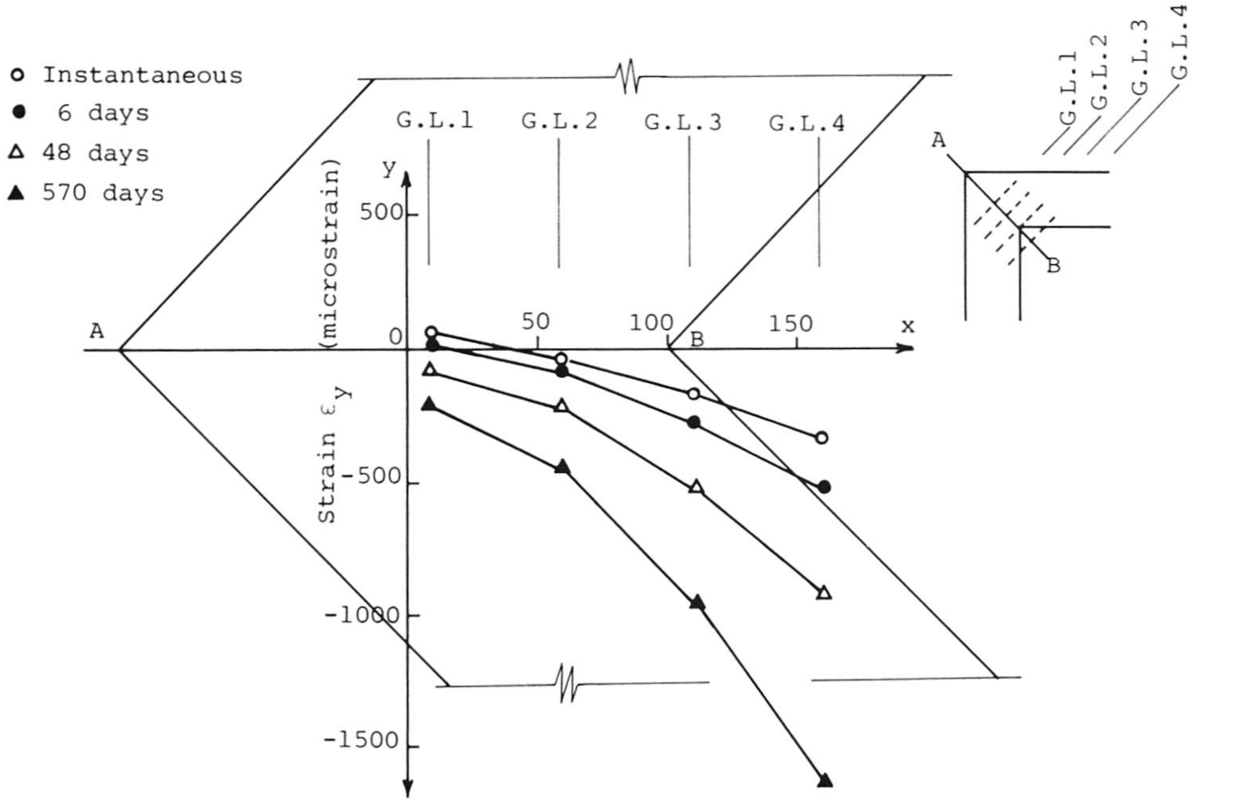


Fig. 9 Strain distribution of beam-column connection

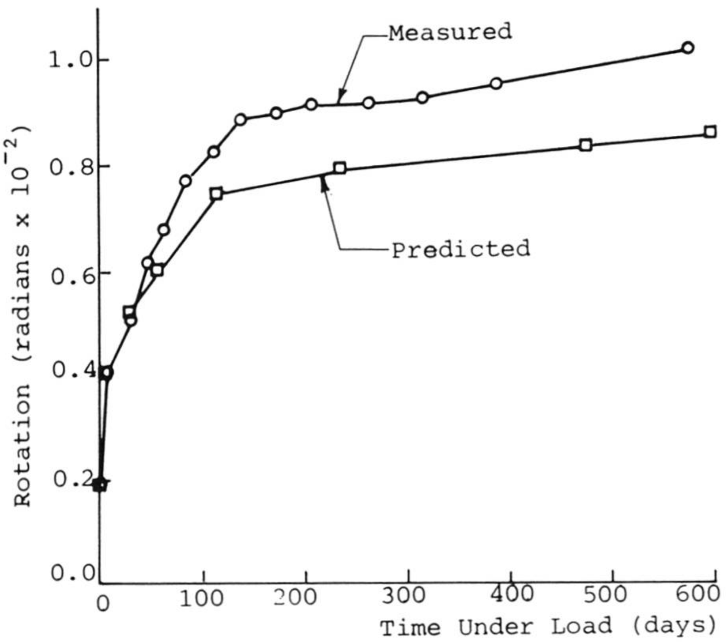


Fig. 10 Rotation of beam-column connection

Long-term deflections observed by a number of researchers have also been compared with corresponding values predicted by the computer programme used for this investigation. A set of these results relating to fixed-base rectangular portals tested by Drysdale [12] is given in Table 4. Particulars of these frames are shown in Figure 11. Full details of the materials and results of these tests are given by Drysdale [12].

Loading				Deflection (mm)			
Test No.	P_H (kN)	P_V	Type	Predicted Δ_H	Measured Δ_H	Predicted Δ_V	Measured Δ_V
1	17.8	35.6	Short term	8.1	9.0	4.0	4.0
4	26.7	53.4	Sustained 53 days	17.8	18.0	8.3	7.0

Table 4 Comparison of deflections of portal frames from tests by Drysdale with values predicted using computer programme

Further examples of comparisons of predicted and measured deformations of reinforced concrete structures are given by Bakoss and Corderoy [20].

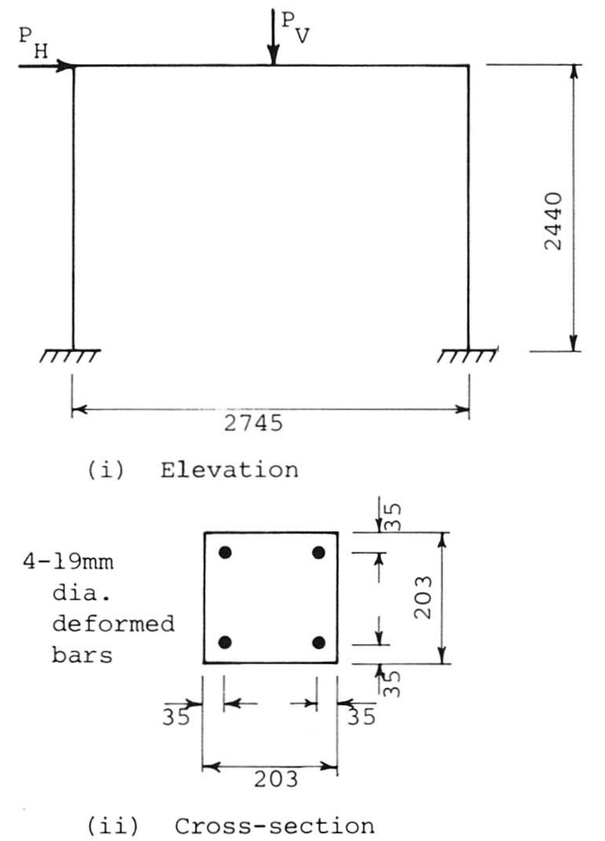


Fig. 11 Fixed-base portal frame tested by Drysdale (10)

5. SUMMARY AND CONCLUSIONS

Results from an experimental test programme on reinforced concrete frames have been presented and compared with theoretical predictions made using a second order non-linear frame analysis programme.

Both the instantaneous and time-dependent deformations caused by sustained loading for a period of over 600 days were measured in the laboratory. Companion specimens were also tested from which the material properties, including the creep and shrinkage characteristics of the concrete, were recorded.

The time-dependent deflection at 600 days under the sustained service load first applied at age of 29 days was approximately three times the initial instantaneous deformation. Shrinkage induced deflections of 1/600 of span were measured with approximately 80% of the deformation at 700 days taking place within the first 200 days after curing.

The theoretical computer predictions of long-term deformations were shown to provide acceptable agreement with the measured values. The maximum difference between the predicted and measured beam deflections and rotations of the beam column connection at 600 days after loading did not exceed 20%.

6. REFERENCES

1. LAI, K.L., Non-Linear Time-Dependent Behaviour of Reinforced Concrete Structures. Ph.D. Thesis Univ. of New South Wales, 1975.
2. SCHNOBRICH, W.C. and SCORDELIS, A.C., Finite Element Analysis of Reinforced Concrete Structures. International Seminar, Part 2, Politecnico di Milano, Milan, 1978, pp. 61-334.
3. ALDSTEDT, E. and BERGAN, P.G., Non-Linear Time-Dependent Concrete Frame Analysis. ASCE Journal of the Structural Division, Vol. 104, No. ST7, July, 1978, pp. 1077-1092.
4. VALANIS, K.C., A Theory of Viscoplasticity Without a Yield Surface. Archivum Mechaniki Stosswanej, Vol. 23, 1971, pp. 517-551.
5. BAŽANT, Z.P. and BHAT, P., Endochronic Theory of Inelasticity and Failure of Concrete. Journal of the Engineering Mechanics Division, ASCE, V. 102, 1976, pp. 701-722.
6. BAŽANT, Z.P. and BHAT, P., Prediction of Hysteresis of Reinforced Concrete Members. ASCE Journal, Vol. 103, Jan., 1977, pp. 153-167.
7. GILBERT, R.I. and WARNER, R.F., Time-Dependent Behaviour of Reinforced Concrete Slabs. IABSE, Proc., p-12/78, 1978, pp. 1-12.
8. AAS-JAKOBSEN, K. and GRENACHRE, M., Analysis of Slender Reinforced Concrete Frames. Publication of IABSE, Vol. 34-1, Zürich, 1974, pp. 1-17.
9. CORDEROY, H.J.B., Analysis of Slender Reinforced Concrete Frames by SONLA. NSW Institute of Technology, Civil Engineering Monograph, No. C.E.78/1, ST.E., Sydney, 1978, pp. 124.
10. FAVRE, R., KOPRNA, M. and PUTALLAZ, J.-C., Deformation of Concrete Structures, Theoretical Basis for the Calculation. IABSE Surveys, S16/81, Feb., 1981, pp. 24.
11. COMITÉ EURO-INTERNATIONAL DU BETON, Manual du CEB, Fissuration et déformations. Ed. R. Favre, Ecole Polytechnique Fédérale de Lausanne, 1983.
12. DRYSDALE, R.G., Prediction of the Behaviour of Concrete Frames, IABSE Symposium on Design of Concrete Structures for Creep, Shrinkage and Temperature Changes, Madrid, May, 1970, pp. 395-404.
13. STANDARDS ASSOCIATION OF AUSTRALIA, Method for the Determination of Compressive Strength of Concrete Specimens AS 1012, Part 9, Method for the Determination of the Flexural Strength of Concrete Flexure Specimen, AS 1012, Parts 10 and 11, 1974.

14. AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Test Method for Static Modulus of Elasticity and Poissons Ratio of Concrete in Compression. ASTM C469-65, 1978.

15. BAKOSS, S.L., FAULKES, K.A. and PULMANO, V.A., Predicted and Measured Concrete Properties for Long-Term Deflections of Reinforced Concrete Beams. Civil Engineering Transactions, Institution of Engineers, Feb., 1983, pp. 21-28.

16. STANDARDS ASSOCIATION OF AUSTRALIA, Concrete Structures Code. AS 1480, 1982, pp. 101.

17. WARNER, R.F. and LAMBERT, J.F., Moment-Curvature-Time Relations for Reinforced Concrete Beams. UNICIV Report, No. R-94, Univ. of New South Wales, Oct., 1972.

18. CLARK, L.A. and SPEIRS, D.M., Tension Stiffening in Reinforced Concrete Beams and Slabs Under Short-Term Load, Technical Report 42-521, C and CA, London, July, 1978, pp. 19.

19. BRANSON, D.E., Deformation of Concrete Structures. McGraw-Hill, New York, 1977, pp. 546.

20. BAKOSS, S.L. and CORDEROY, H.J.B., Analysis of the Non-Linear Behaviour of Reinforced Concrete Frames. Conference on Computers and Engineering, The Institution of Engineers, Australia, Sydney, Aug.-Sept., 1983.

APPENDIX

Creep and shrinkage curves obtained from tests on the concrete used in frames.

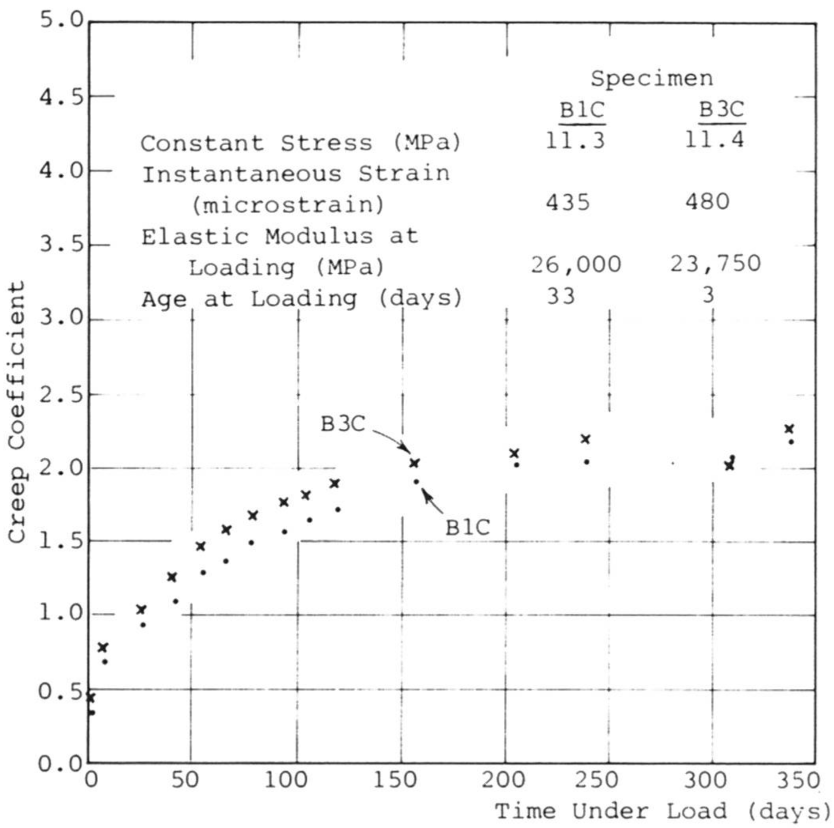


Fig. A1 Typical creep curves for concrete mix used in frames

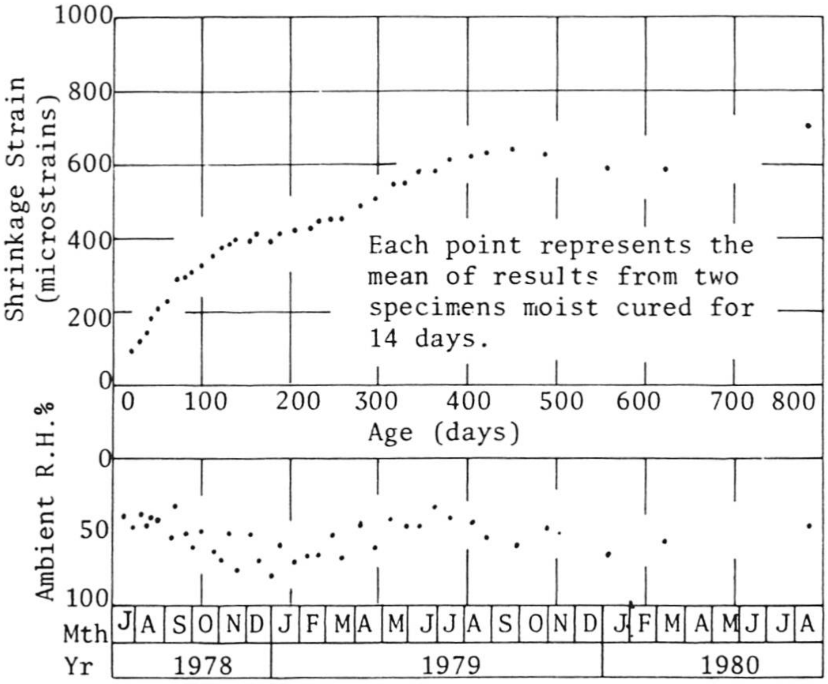


Fig. A2 Typical shrinkage curve for concrete mix used in frame