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Component Method Validation Tests in Precast Concrete Semi-Rigid Connections

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Summary

Full scale testing of a symmetrical precast concrete beam-column connection has been carried out to generate semi-rigid moment-rotation (M-ϕ) data which may be compared with that obtained using the *component method*. In this way the deformabilities of isolated, small scale compression and tension joints, representing the bottom and top fibres of a full scale test, are summed to generate M-ϕ curves. The points where the stiffness of the full scale connection changes, and the magnitude of the connection stiffness are faithfully reproduced by the component method, but the ultimate test moment and rotation capacity are not. This is because little redistribution of flexural stress is possible in isolated tests, and cracking is affected by the presence of floor slabs in the full scale tests. Within these limitations the component method provides a reasonable tool to generate M-ϕ data.

1. Background to Semi-rigid Connection Testing

Precast concrete skeletal frames are designed as braced, unbraced or partially braced structures, in which the columns are usually continuous at the floor level connections, as shown in Fig. 1, where a solid steel billet is awaiting a third beam at an internal connection. The majority of connections however are either single sided (at the edges of buildings) or double sided (at interior columns), and these have formed the basis for all the experimental tests carried out to date. Precast connections are also distinguished between those which include floor slabs (usually hollow cored units) spanning perpendicular to the plane of the beam, and those which do not. In the former, the tie steel at the ends of the floor slabs are an integral part of the stability ties required by most Codes of Practice, and form a vital component of the connection.

Mahdi (1,2) established that the most common types of connection exhibit some degree of in-plane flexural semi-rigidity. Values of strength, stiffness and (M-ϕ) data have been given previously (1). Of course, it rests with the design engineer to decide whether this information justifies a semi-rigid frame design. However, the need to provide further M-ϕ data, without incurring the additional expense of testing, has led to the development of the so called *component method* (3,4). Here M-ϕ data are generated by superposition of individual (and combined) actions within the connection. The component method is accepted in semi-rigid steel connection analysis, and previous work by the authors (4) suggested that it might also be feasible in precast concrete connections.

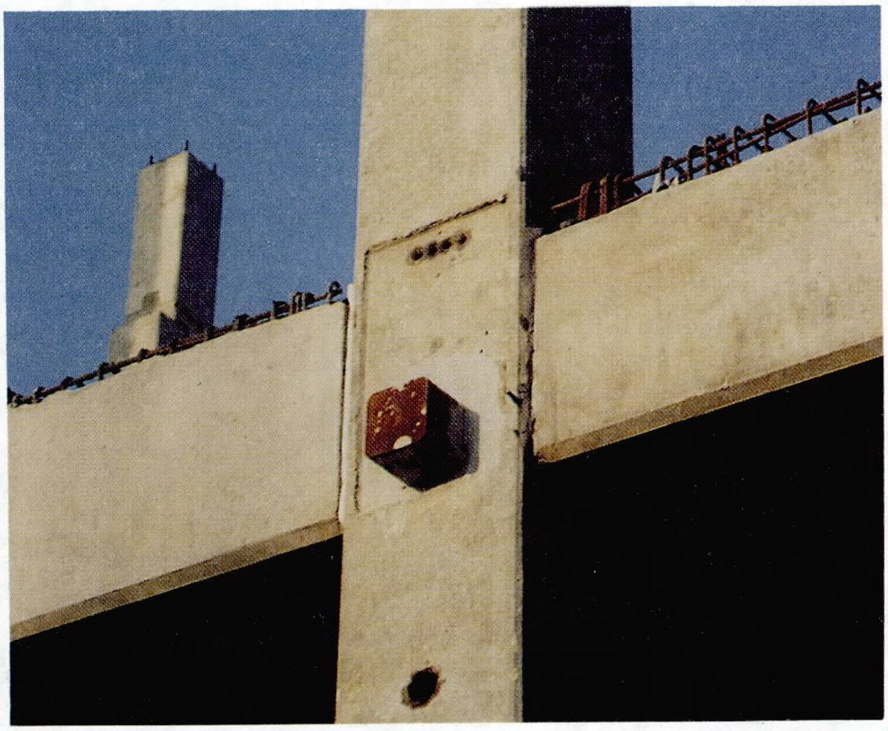

Fig. 1. Three-way precast concrete billet type connector. Note the projecting interface stirrups in the beams and the sleeves in the column to receive continuity tie steel.

This present work takes the above a further step forward by determining the M-ϕ curves for a double sided connection, subjected to equal hogging moments and shear forces, by three methods:

1. direct measurement by full scale testing (called 'Method 1');
2. joint deformations measured and computed in a full scale test (called 'Method 2');
3. joint deformations measured in isolated tests and computed according to the 'Component Method'.

2. Full Scale Experimental Tests

2.1 Design of Test Specimens and Test Procedure

Details of the cruciform test assembly are given in Fig. 2. This is essentially a symmetrical beam-column connection, with proprietary slip formed hollow cored floor slabs (supplied by Bison Floors, UK). The length of the beams, and hence the position of the bending load P (Fig. 2[a]), was selected to represent the point of contraflexure in a uniformly distributed loaded beam of about 12 m span. Assuming that the maximum bending moment is recorded at the face of the column, the shear span / beam effective depth ratio for the load is 2365 / 450 = 5.25. The effective depth to the reinforcement is 500 - 50 = 450 mm.

The 200 kN (vertical shear rated) beam - column connector is of the *welded plate* type, comprising a 100 x 100 mm solid steel column billet (Figs. 1 and 3) fillet welded (20 mm leg length x 80 mm long) to a 25 mm thick vertical plate cast into the beam. The 100 mm gap at the end of the beam was filled with 10 mm aggregate insitu concrete (nominal f_{cu} = 40 N/mm^2) up to the top level of the beam.

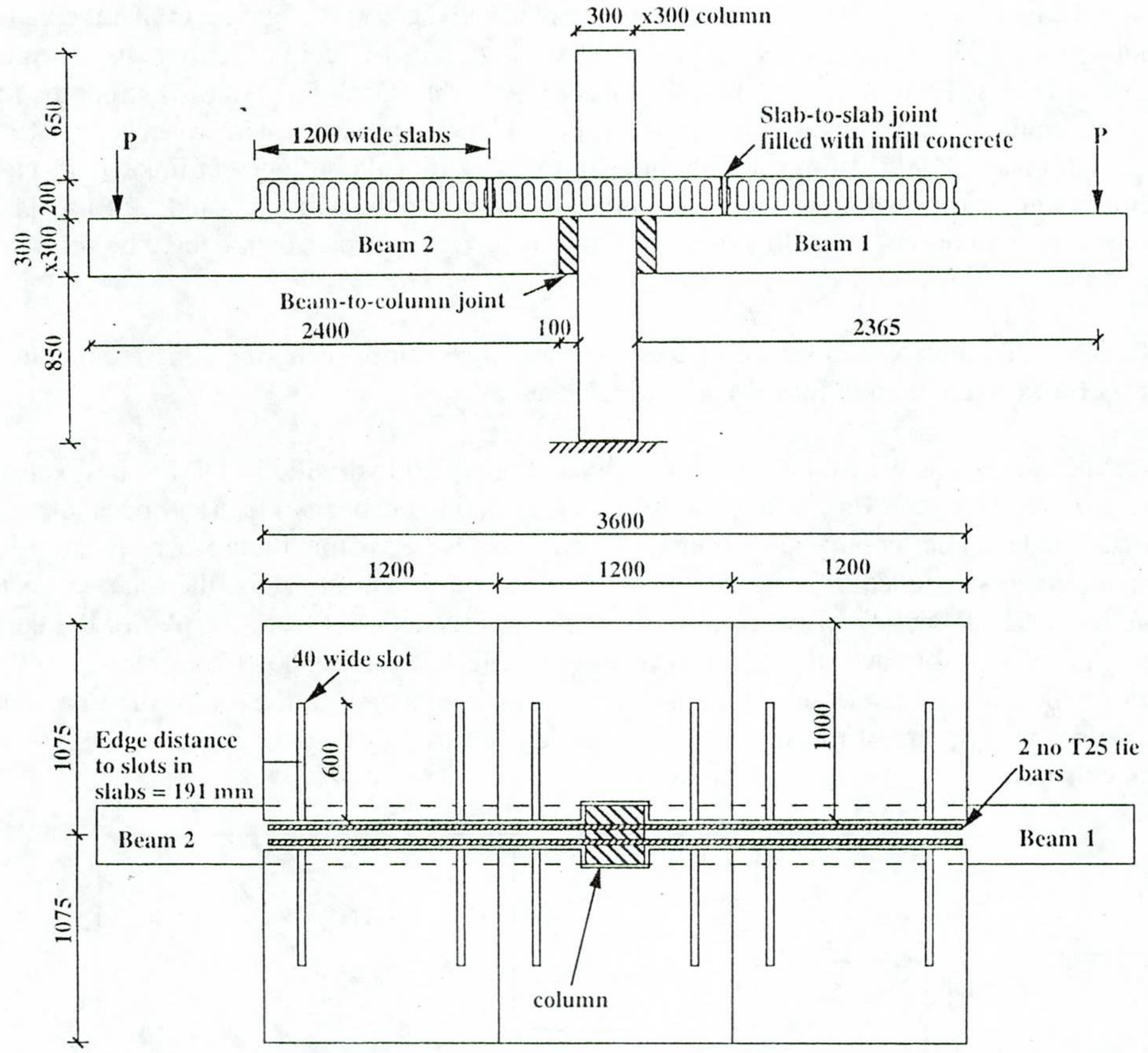

Fig. 2. Arrangement of full scale precast concrete connection test. (a) Elevation (b) Plan.

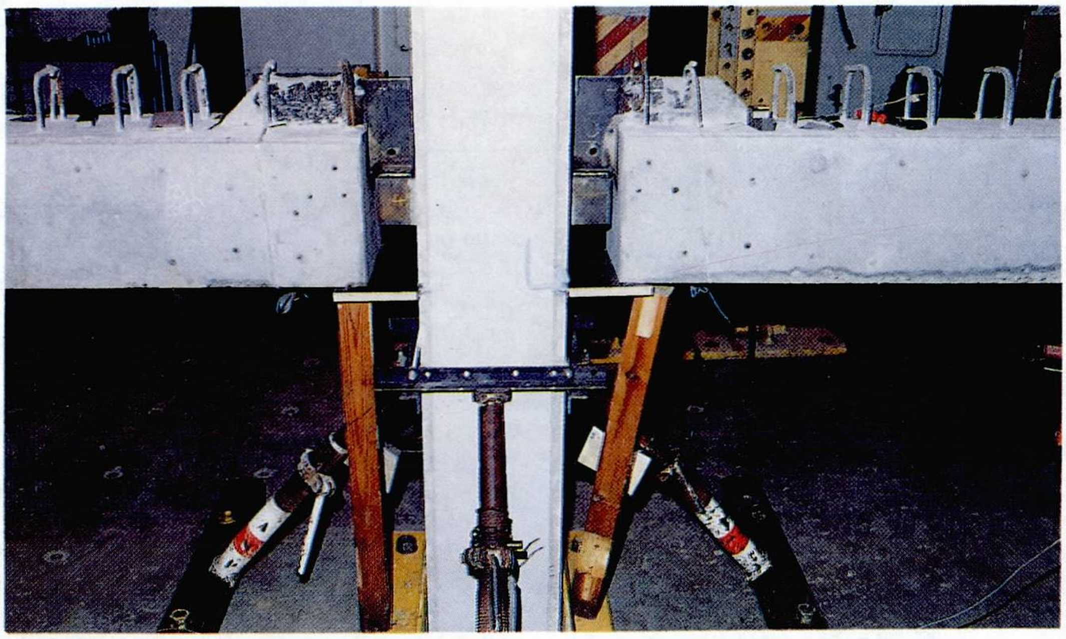

Fig. 3. Construction of connector in laboratory.

The precast floor slabs were concreted into position using 10 mm aggregate insitu concrete (nominal f_{cu} = 30 N/mm^2) after 12 mm diameter high tensile steel (T12, f_y = 460 N/mm^2) tie bars were placed into the 40 mm wide milled slots in the slabs. The nominal strengths for the precast column and beams was f_{cu} = 40 N/mm^2. Column reinforcement consisted of 4 no. T25 main bars and T 12 links at 185 mm centres. Additional confinement links at 75 mm centres were positioned adjacent to the billet. Beam reinforcement consisted of 4 no. T20 bars in the top and bottom, with T10 links at 100 mm centres. Further details may be found in reference 5.

The design moment of resistance of 235 kNm was determined using the concrete rectangular stress block method and unfactored material stresses.

Loading was applied incrementally through hand operated hydraulic jacks (the intervals of loading are evident in Fig. 6) and measured using 200 kN capacity electrical resistance load cells. The beam deflections shown in Fig. 4 were measured using a number of linear potentiometers attached to a rod which was bolted to the column. Thus, the relative rotation **and** the shear deflection of the beam to the column was deduced from the plot of the deflection shown in Fig. 4 for each increment of load (only selected values shown). Compressive deformations δ_B in the bottom of the beam, and crack widths δ_T in the top of the slab were measured using pairs of linear potentiometers, one either side of the beam to check for out-of-plane movements.

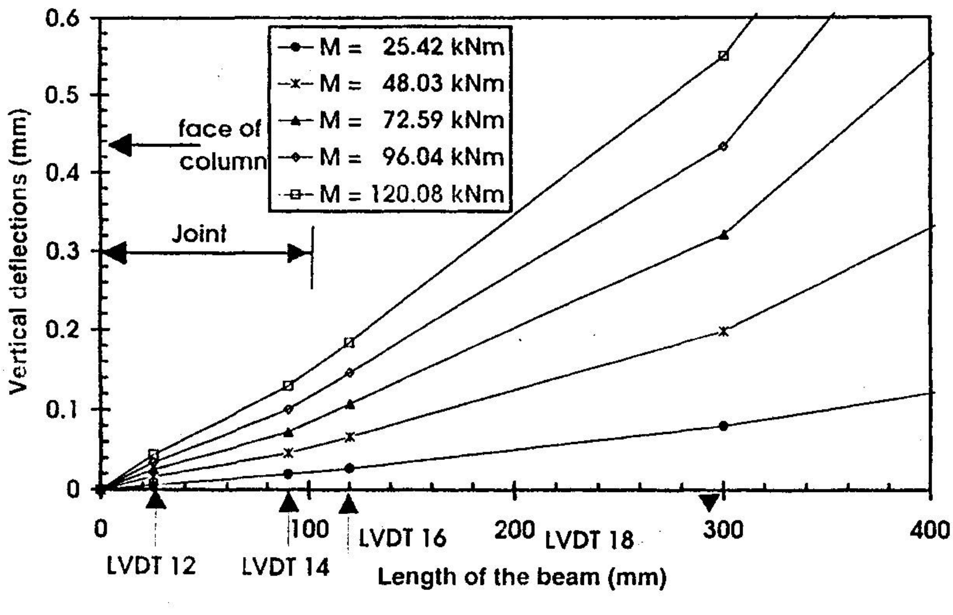

Fig. 4. Profiles of vertical beam deflections at selected values of applied bending moment.

2.2 Test Results

The M-ϕ data presented in Fig. 5 are average values for the mean results obtained for both Beams 1 and 2 (Fig. 2[a]). M_{con} refers to the applied hogging bending moment at the face of the column = 2.365 P (kNm units). The relative rotation ϕ refers to the **total** rotation of the beam relative to the column, and was determined using two methods:-

Method 1. By measurement of the gradient of the vertical deflection vs distance curves in Fig. 4 over a distance of 300 mm from the face of the column. Shear deflections are thus eliminated. Fig. 4 also shows that the rotation in the joint region (up to 100 mm from the face of the column) is approximately two-thirds of the total beam-column rotation.

Method 2. By the addition of horizontal joint deformations in the bottom (δ_B) and top (δ_T) fibres of the connection divided by the total depth of the connection, i.e. (δ_B + δ_T) / 500 (mm units). This method assumes full shear interaction between the floor slab and the beam. The data for δ_B and δ_T are presented in Fig. 6, which shows that crack widths at the top of the slab are some five times greater than the compressive deformations in the beam.

Fig. 5 shows the two methods are in exact agreement for M < 75 kNm, and within 10 per cent of one another thereafter. This shows that, within the normal scatter in experimental work of this type, either method may be used to generate M-ϕ data, and is the first step towards the validation of the component method.

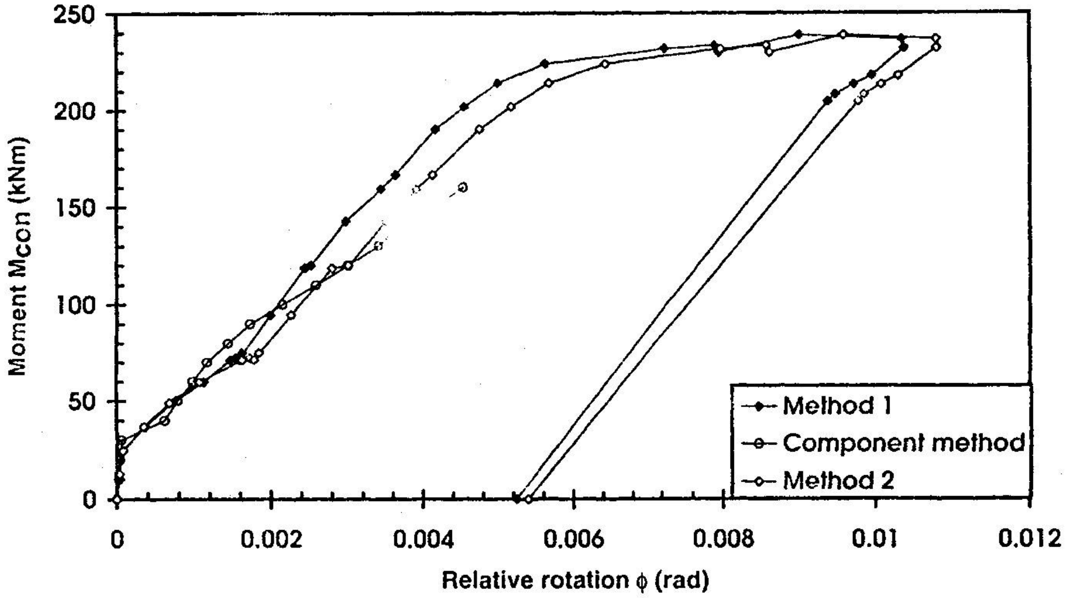

Fig. 5. Moment-rotation data obtained using three different methods.

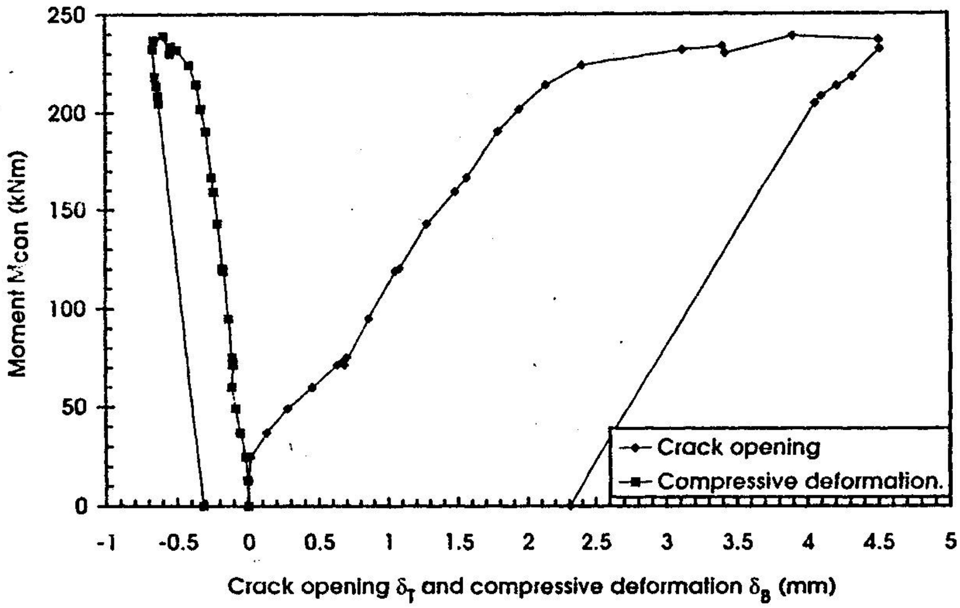

Fig. 6. Horizontal deformations vs applied moment.

Fig. 7 shows the damaged area of the joint. (The notation refers to applied load P in kN.) A circle has been drawn around the bottom right hand corner of the joint to indicate the extent of the concrete compression zone and the final position of the neutral axis, i.e. about 100 mm from the bottom of the beam. Horizontal bursting cracks are a clear indication of unconfined concrete compressive failure in the insitu concrete infill. A second horizontal crack occurs at the level of the top surface of the solid steel billet, and is possibly indicative of local stress concentrations there. The largest cracks are, as expected, at the column to joint interface. These initiated at M = 30 kNm, which coincides with the large reduction in stiffness seen in Fig. 5, and may be interpreted as the point at which the section is cracked flexurally.

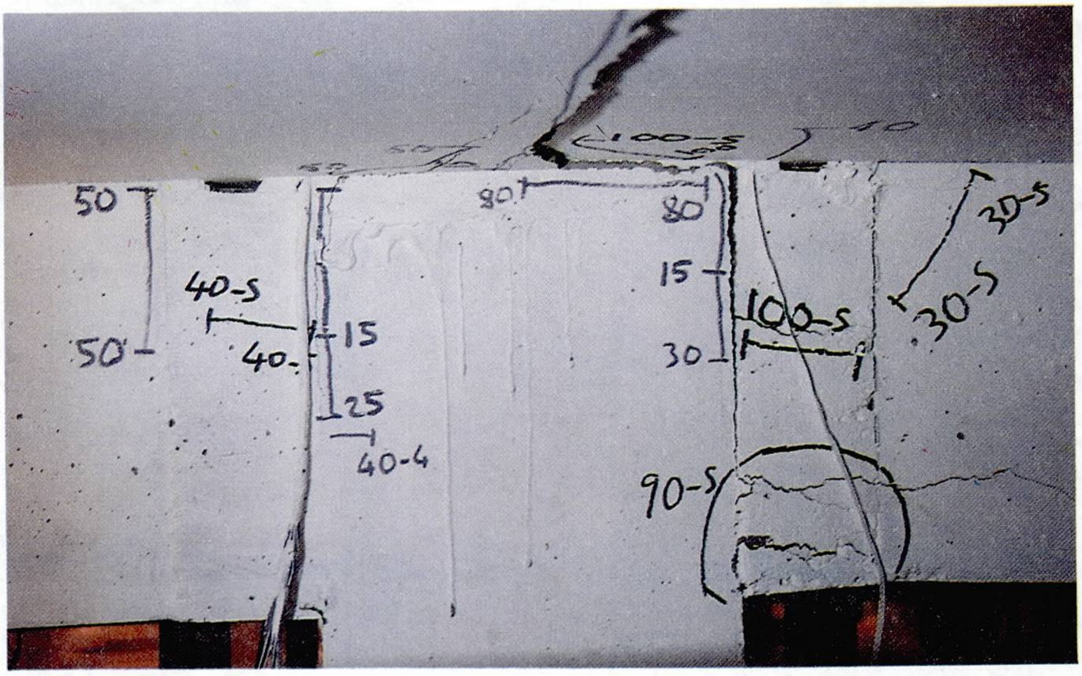

Fig. 7. Crack patterns in the vicinity of the connections in the full scale test.

3. Isolated Joint Tests

The compression zone in the bottom of the beam can be simplified as a pair of precast concrete blocks of say 100 x 100 mm cross section, one representing the beam and the other the column, joined together using a narrow strip of infill concrete of the same cross section and thickness 't' to represent the insitu infill. The shapes of the specimens are shown in Fig. 8, with the infill concrete shaded. The top of the floor slab is represented by a single reinforced concrete joint, containing 2 no. T25 bars subjected to flexural tension. The design and testing methods are described in references 4 and 5. Data for δ_T are being presented in reference 5 to enable a direct measurement of tie force vs crack width to be made.

δ_B is a function of three parameters; i) the applied stress σ; ii) the 'effective' Young's modulus E_{ce} of the concrete; and iii) the interface deformability λ of the precast-insitu joint interface. The expression in Fig. 8 gives E_{ce} in terms of the Young's modulus for the precast and insitu concretes, E_{cp} and E_{ci}, respectively (determined experimentally from the solid specimens) and the number n of interfaces each of deformability λ. The term 'x' is the distance over which E_{ce} was determined (200 mm). Fig. 8 shows that if $E_{cp} = E_{ci}$, i.e. both concretes are the same, then $E_{ce}/E_{cp} = 1$. The fact that it is not so is indicated by the reduction attributed to the effect of t/x

(E_{cp}/E_{ci} - 1). A second reduction in E_{cc} is attributed to the joint(s) interface deformability, i.e. $n\lambda E_{cp}/x\sigma$. The small variation in E_{cc}/E_{cp} with infill thickness t may be conservatively ignored with the result that E_{cc}/E_{cp} = 0.5.

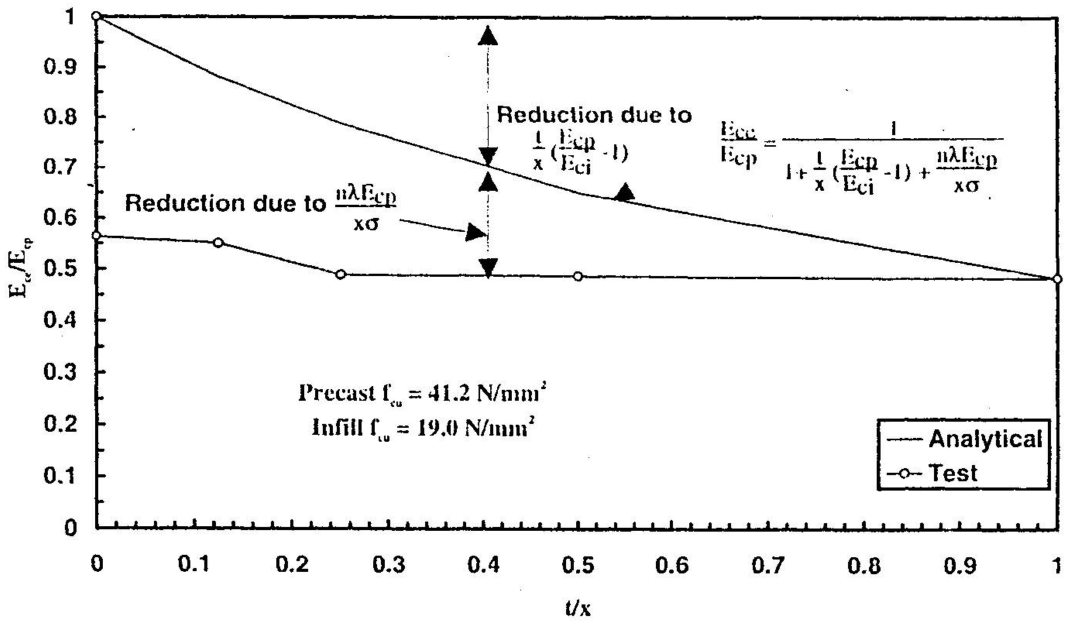

Fig. 8. Effective Young's modulus data in isolated compression tests.

Fig. 9 shows the variation in λ with applied axial stress σ for the specimens shown in Fig. 8. Although the deformability of the 'dry' joint, i.e. two precast pieces with no intermediate medium, is much greater than the 'cast' joints, it is the latter which is of interest to us. A mean constant value for λ/σ = 0.003 mm per N/mm² may be used without loss in accuracy.

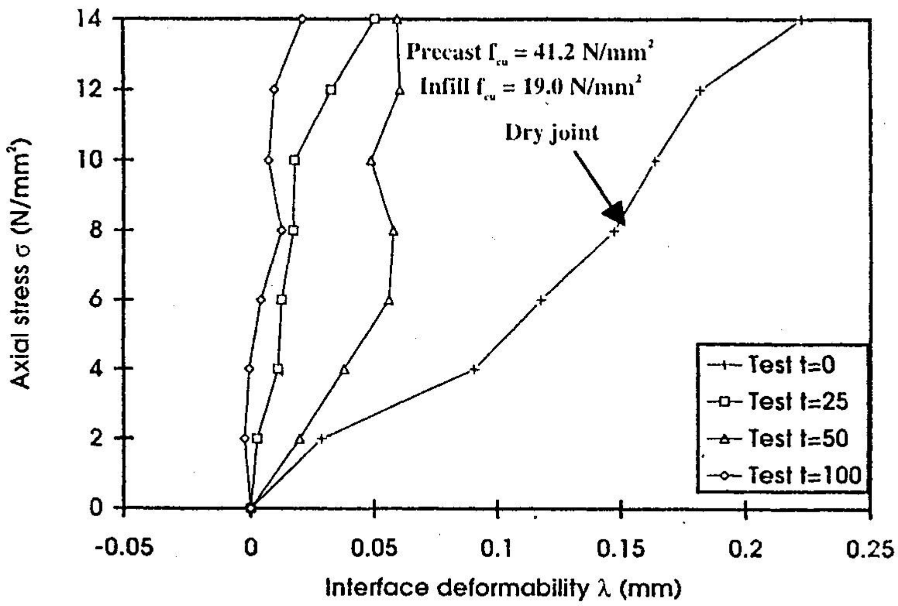

Fig. 9. Variation of joint interface deformability per interface with applied axial stress in isolated compression tests.

4. Comparison of M-ϕ Derived from Full Scale Tests and The Component Method.

M-ϕ data is derived from the component method as follows:

1. Using the flexurally uncracked section properties Z_u of the beam and floor slab (neglecting the welded plate connector), the compressive flexural stress σ in the beam is determined for a given bending moment M, i.e. $\sigma = M/Z_u$.
2. The compressive strain in the beam $\varepsilon = \sigma/E_{cc}$, where E_{cc} is given in Figs. 8 and 9. Compressive deformation δ_B is determined over a gauge length of 180 mm.
3. The tie force in the top steel is equated to the total compression force in the beam. δ_T being determined directly from the aforementioned crack width test data.
4. Rotation $\phi = (\delta_B + \delta_T) / 500$.
5. Repeat steps 1 to 4 using the flexurally cracked section properties where the flexural tensile stress in the concrete exceeded the limiting value. This point coincided with the commencement of the first crack in the full scale test, i.e at M = 30 kNm.

The results of this exercise are also shown in Fig. 5. The agreement with the full scale results varies between +15 and -20 per cent of the moment. However, the maximum moment achieved is only 160 kNm, i.e. two-thirds of the full scale test result, and the maximum rotation is 4.4 mrad, less than half of that achieved in the full scale test. The post-cracking stiffness of the connection in the full scale test is 38.5 kNm/mrad, whereas in the component method it is approximately 33.0 kNm/mrad.

5. Discussion

In making comparisons between the M-ϕ results obtained from the different methods there are a number of important features in the behaviour of the full scale test worthy of further discussion. These points are discussed in the context of gaining confidence in using the component method, and to qualify some of the (inevitable) assumptions (in italics) made.

In the full scale test a transverse flexural crack was first observed at an applied bending moment of 30 kNm (Fig. 7). This crack appeared at the column face and spread to the outer edge of the hollow cored slab. The crack widths at this point were 0.019 and 0.017 mm on either side of the column. Apart from one or two minor deviations in the results (see Fig. 5) the behaviour was generally anticipated with non-linear behaviour commencing at about 80 per cent of the ultimate moment, i.e. 190 kNm. Signs of compressive concrete failure in the bottom of the beam were evident.

Strains were also measured in the T25 tie bars in the full scale test. Because the bars were continuous through the column and the loading was symmetrical their anchorage was assured at the column face. After the concrete in the tension zone failed to take any more tensile force, these were then taken by the tie bars and this gave a rise in the steel strains. At the ultimate moment two of the strain gauges recorded strains of 7000 $\mu\varepsilon$ and 5400 $\mu\varepsilon$, indicating significant yielding of the bars.

In generating the M-ϕ data using the component method it is assumed that the strains are transferred to the steel tie bars in the isolated joint test in the same manner as in the full scale tests, even though the presence of the hollow core slabs will have an influence of this.

Compressive deformations δ_B (Fig. 6) were measured over a distance of 180 mm, i.e. 100 mm joint plus 40 mm precast beam and column. The maximum concrete strain calculated from these values is 0.0037, and being greater than 0.0035 ultimate strain at which concrete is normally assumed to fail explains the onset of failure at M = 240 kNm. It is important to note that the compressive concrete strain obtained from the strain gauges in the beams near to the joint zone at failure was 0.00347. The maximum moment achieved by the component method was 160 kNm, and the limiting rotation was 4.4 mrad. The failure was due to concrete crushing failure in the isolated compression tests (Fig. 8).

In the isolated tests it is impossible for strains to exceed the uniaxial limit and therefore no redistribution of stress is possible in the component method.

A good agreement was obtained between the rotations derived from Methods 1 and 2. At no point do the rotations differ by more than 0.4 mrads. Horizontal deformations at the top of the beam were in linear registration with δ_T and δ_B, showing that the beam and slab were rotating as a rigid block. These data also showed that the neutral axis for the flexurally cracked section was near to the level of the welded plate connector.

In using the component method it may be assumed that plane sections remain plane, and that full horizontal interface shear interaction between the beam and slabs is possible. It is not necessary to include for the effects of the welded joint between the steel billet and narrow plate as this point coincides with the neutral axis.

6. Conclusions

M-ϕ joint data were obtained from full scale precast concrete beam - column connection tests and compared with similar data generated using the component method. A two stage approach was used to validate the component method for this particular type of precast concrete connection:

Stage 1. True M-ϕ data were obtained from vertical beam deflections measured within 300 mm of the face of the column. These values were within 10 per cent of those obtained by summating extreme fibre horizontal deformations.

Stage 2. M-ϕ data were generated by summating horizontal deformations obtained from isolated, small scale compression and tension joint tests.

In comparing the results obtained from the full scale tests and the component method, it is noted that both concrete and tie steel uni-axial yield strains are exceeded in the former, whereas this is not possible in the isolated tests. For this reason the full scale ultimate test moment of 240 kNm and rotation capacity of 11 mrad are not achieved; the values being 160 kNm and 4.4 mrad, respectively. This is because no redistribution of stress is possible in isolated tests, and cracking is affected by the presence of floor slabs in the full scale tests. However, the points where the stiffness of the full scale connection changes, i.e. after the first flexural crack at 30 kNm moment, and the magnitude of the stiffness are both faithfully reproduced in the component method.

In conclusion it is such that, within the limitations described the component method provides a reasonable tool to generate M-ϕ data, and needs to be developed further.

References

1 Elliott, K. S., Davies, G. and Mahdi, A. A. Semi-rigid joint behaviour on columns in precast concrete buildings, COST C1 Proceedings of the 1st Workshop, Semi-rigid Behaviour of Civil Engineering Structural Connections, E.N.S.A.I.S., Strasborg, Oct. 1992, p 282 - 295.

2 Mahdi, A. A. Moment rotation effects on the stability of columns in precast concrete structures, PhD thesis, University of Nottingham, 1992.

3. Briquet, C., Guisse, S., Jaspart, J. P., Lognard, B. and Maquoi, R. Research activities under COST C1 at the Department MSM of the University of Liege, COST C1 Proceedings of the 2nd Workshop, Semi-rigid Behaviour of Civil Engineering Structural Connections, Prague, Oct. 1994, p 75 - 88.

4. Elliott, K. S., Davies, G. and Görgün, H. The determination of moment-rotation in semi-rigid precast concrete connections using the component method, as ref 3, p 31 - 41.

5. Görgün, H. Semi-rigid behaviour of connections in precast concrete structures, Dept. of Civil Engineering Report No. SR 96-001, University of Nottingham, Feb. 1996, 344 pp.

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