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POSSIBLE ROLE OF INTERNAL SUPPORTS IN CONTINUOUS BEAMS

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SYNOPSIS

At present the design of reinforced concrete (RC) indeterminate structures are primarily based on Code equations which rely heavily on tests performed on determinate members like simply supported beams, slabs, etc. In the present study, efforts have been made to design reinforced concrete continuous members to both ACI 318-89 and a design model which transforms a continuous member into a number of determinate members connected at the points of contraflexure by internal supports. This paper further extends previous findings on statically determinate members to indeterminate ones as regards a more rational approach based on the Compressive-Force Path (CFP) method. Also, in the past, the potentially large discrepancy in load-carrying capacity between the CFP and code methodology was highlighted for members for which the possibility of shear failure was central to the design problem. Implicit in this was the notion that the strength of elements failing in flexure would not significantly be influenced by the design method. The present study shows that, whereas this is indeed likely to be the case as regards the magnitude of the applied load at collapse, the types of failure (ductile/brittle) can vastly differ. The importance of providing "internal supports" at the points of contraflexure has been demonstrated from the presently tested RC continuous beams. The size independence of the role of internal supports has also been recognized.

INTRODUCTION

A study of present-day Codes of practice shows quite conclusively that, despite much research effort, a truely rational and all-encompassing understanding of structural concrete is still lacking. One reason for this is the poor understanding of concrete at the material level. In this respect, the use of current design equations relies on concepts and/or theories that are invariably based on uniaxial (rather than triaxial) stress-strain characteristics. Also, current design Codes [1-4] are based on information predominantly obtained from tests on statically determinate structural concrete members, the most common of these being simply-supported beams; thus, indeterminate reinforced concrete members are virtually designed by applying equations derived specifically for their statically determinate counterparts.

Design rules tend to be based more on design equations than on realistic models. Physical models are, of course, available, but most of these seem to be of doubtful universal validity. For example, present-day design models, being explicitly based on the truss-analogy concept [5,6], are very much dependent upon the residual strength of cracked concrete. The truss-analogy concept also forms the core of new theories (or models), like the modified compression-field theory [7] or the strut-and-tie model [8]. However, recent experimental findings have shown that concrete as a material is brittle in nature [9], and that the conventional strain-softening response is not really a material characteristic, as extensively considered, but merely a "descriptor" of secondary testing effects. Furthermore, aggregate interlock [10-12] which, according to current Codes of practice [1-4], is supposedly a major contributor to the shear-transfer mechanism of a reinforced- or prestressed-concrete (RC, PSC) member, has been found to play a negligible role in such transfer mechanisms [13]. It has also been demonstrated in recent tests [14-19] that RC or PSC members may attain their flexural capacity once their "shear capacity" is exceeded even without behaving like a "truss". It has also been understood that the ultimate limit state behaviour of structural concrete can only be explained in terms of multiaxial effects which are always present in a structure. It was the consideration of these multiaxial effects, the recognition of concrete as a brittle material, and the shortcomings associated with such concepts as the "truss-analogy" and the "aggregate interlock" mechanism, that had led to the introduction of the concept of the compressive-force path (CFP) [20], which has proved capable of providing a realistic explanation of the causes of failure of structural concrete. Design based on this concept has been applied successfully to RC [14, 16, 17, 21] and PSC [18,19,22] statically determinate members made from a wide range of concrete strengths, resulting in economic and, above all, safer design solutions. The method has also been applied to indeterminate structures like continuous beams and portal frames by conducting pilot tests on such structural forms [23]. The detailed description of the CFP concept and CFP design models along with sample calculations are available elsewhere [14, 17, 18, 19, 23].

The present paper aims at further consolidating the applicability of the CFP concept to the design of any form of skeletal structural configuration. First, the possible application of the "tied-frame" model of Kotsovos [20] to cantilever and coupling beams is explored. Then, the results of tests performed on continuous beams are reported. The present series of tests [24] compliments the findings obtained from tests conducted on smaller sized members [23].

PHYSICAL MODELS FOR INDETERMINATE RC MEMBERS

The CFP concept stipulates that the strength of an RC member is associated with the strength of concrete in the region of the path along which the compressive forces are transmitted. A cantilever beam subjected to a point loading at its free end can be designed as a simply-supported beam subjected to a point-loading at midspan, since the fixed-end conditions of the cantilever beam are similar to the conditions of the mid-span cross-section of the simply-supported beam. Figure 1a shows the application of the proposed model to such cantilever beams. The same model can be used for the design of a structural concrete wall subjected to horizontal loading. Figure 1b indicates how a beam with fixed-ends, such as a beam coupling two structural concrete walls, can also be designed in compliance with the proposed method, since it can be divided into two portions between the fixed-end and the section through the point of contraflexure (inflection), each of them being essentially a cantilever beam. In this case, however, the design should be complemented so as to allow for the interaction of the two portions of the coupling beam. The latter interaction may be modelled as an "internal support" where the reaction is equal to the shear force that develops in the section through the point of inflection. The provision of web reinforcement in the form of stirrups by an amount sufficient to sustain the action of the shear force should yield a satisfactory design solution. It has been proposed [23] that such reinforcement should extend over a length equal to "2d" symmetrical about the point of inflection. It can be seen from Fig. 1b that the design of the internal support involves the provision of transverse reinforcement to counteract the tensile forces ("reactions" to the "actions" shown in Fig. 1b) acting in that region. Considering the fact that concrete is very weak in tension, it is suggested that only a minor or no contribution from concrete can be considered in designing such internal supports.

The above-mentioned method of design can be extended to any skeletal structural concrete configuration, with the proposed frame model being used to represent elements between consecutive points of inflection and the regions including such points being modelled as internal supports. This design approach has been adopted in the present paper for the modelling of two two-span continuous RC beams. Initially, the continuous beams have been transformed into a number of determinate structural elements. Figure 1c shows how the proposed method can be applied to the modelling of such RC continuous beams. The model of Figure 1c comprises three "tied-frame" models developed by Kotsovos [20] for simply supported RC beams and two internal supports. Failure of the model may take place in the region where the

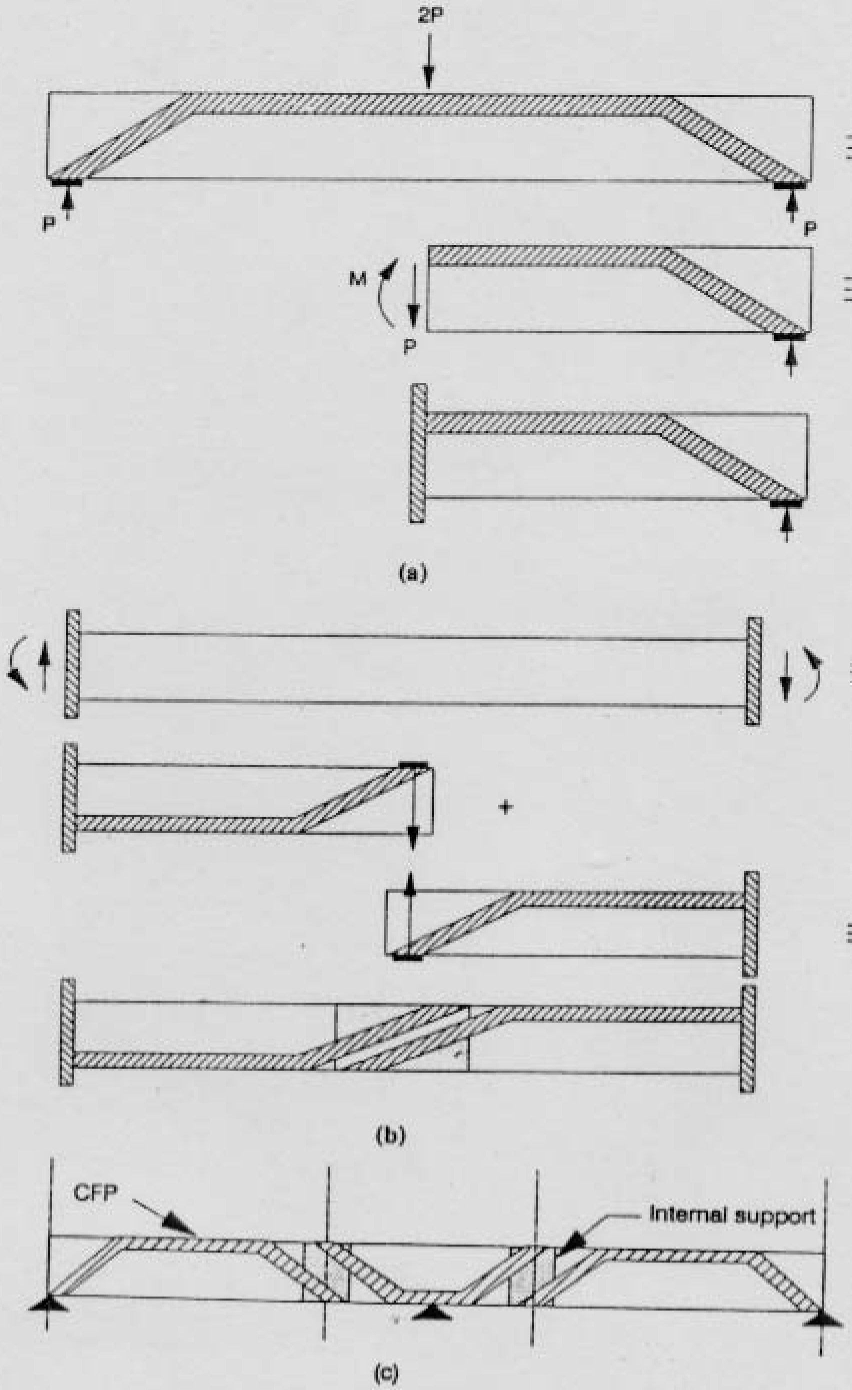

Figure 1: Proposed Model for (a) Cantilever Beam, (b) Coupling Beam, and (c) Two-Span Continuous Beam

CFP changes direction, or within the horizontal portion of the path, or at the internal supports.

It is important to mention here that the design of individual "statically determinate" structural elements, separated by the internal supports, follows the design method described elsewhere [14, 17, 18, 19, 23].

EXPERIMENTAL STUDY OF RC CONTINUOUS BEAMS

Beam details A total of five RC continuous beams of two types have been tested [24] and will be reported in this paper. The beams had two equal spans of 1125 mm each with an overhang of 50 mm beyond the supports. The beams were loaded equally at the middle of each span. During the "discretization" of the continuous beams, designed in accordance with the CFP concept (denoted as CB2C and CB3C), as portrayed in Fig. 1c, the indeterminate members have, essentially, been transformed into three determinate members following the model adopted earlier by Seraj, et al. [23] each of these being modelled using the tied-frame model of Kotsovos [14]. The domains between these discretized members are modelled as internal supports. Here, the points of inflection (centre of internal supports) have been located on the basis of elastic bending moment diagram.

The dimensional, cross-sectional and general design details of the RC continuous beams of series 2 (CB2A, CB2N and CB2C) and series 3 (CB3A and CB3C) are presented in Fig. 2. The transverse reinforcement of CB2A, comprising 40 closed vertical stirrups of 6 mm diameter, was designed according to ACI 318-89. Whereas beams of both series had 4-12 mm ϕ bars as longitudinal reinforcement at the bottom, series 2 and 3 beams had, respectively, 5-12 mm ϕ bars in two layers and 4-12 mm ϕ bars in one layer as longitudinal (top) reinforcement. The stirrup spacing in the outer shear spans were 78 mm centre-to-centre and the stirrup spacing in the inner shear spans was 45 mm centre-to-centre. The beam CB2N was only provided with nominal transverse reinforcement comprising 28 stirrups. The beam CB2C was designed in accordance with the proposed method with 9-6 mm ϕ stirrups placed over a distance of 320 mm (= 2d) around the points of contraflexure (internal supports). The rest of the beam required only nominal transverse reinforcement. Similarly the series 3 beams CB3A and CB3C were designed following ACI 318-89 and the proposed method, respectively.

The beams were designed using concrete having a cylinder strength of 27.58 MPa (4000 psi). At the time of testing, concrete cylinder strength for series 2 and 3 were found to be 34 MPa and 39 MPa, respectively. These strengths were used in calculating the predicted strength of the members. The strength characteristics of the reinforcing bars used in manufacturing the continuous beams reported in this paper were determined. Whereas the yield and ultimate strength of 12 mm deformed bar was 507 MPa and 682 MPa, respectively, such strengths for 6 mm diameter plain bar was 361 MPa and 461 MPa, respectively.

Testing The load from a hydraulic jack was evenly divided by a spreader beam and roller-plate arrangement, and applied at the middle of both spans of the continuous beams. The testing rig is shown in Fig. 3. During the loading of the beams to failure, LVDTs were engaged, for monitoring midspan and out-of-plane displacements. At each load increment, the load was maintained constant for about 4 minutes in order to monitor the load and deformation response of the beam, mark the cracks (if any), and take photographs of the member's crack patterns. Details of the testing procedure, loading sequence, and validation of the testing set-ups are available elsewhere [24].

DISCUSSION OF RC CONTINUOUS BEAM TEST RESULTS

Load-carrying capacity The transverse reinforcement provided for the beams CB2A, CB2C, CB3A and CB3C were deemed sufficient for an RC beam to attain its flexural capacity. The actual and predicted load carrying capacity of the beams are given in Table 1. The ACI 318-89 code predicted a very low failure load as current codes overestimate the significance of stirrup within the shear span. It is evident from the table that present code provisions are rather inefficient (i.e. conservative). The predictions of the failure load by the proposed method, although slightly better than its code counterpart, is not commendable either. Whereas the proposed method could not predict the failure load of CB2A, CB2N and CB3A, since they did not have internal supports, it predicted deformation behaviour of all the beams tested satisfactorily.

Table 1. Predicted and Measured Load-Carrying Capacity (Behaviour)

Beam	Total sustained load, kN (Behaviour)			Predicted/Measured load	
	Prediction		Measured		
	ACI 318	Proposed		ACI 318	Proposed
CB2A	293.8 (Ductile)	- (Brittle)	436.0 (Brittle)	0.67	-
CB2C	200.7 (Brittle)	293.8 (Ductile)	426.0 (Ductile)	0.47	0.69
CB2N	189.0 (Brittle)	- (Brittle)	382.0 (Brittle)	0.49	-
CB3A	270.5 (Ductile)	- (Brittle)	382.0 (Brittle)	0.71	-
CB3C	195.0 (Brittle)	270.5 (Ductile)	462.6 (Ductile)	0.42	0.58

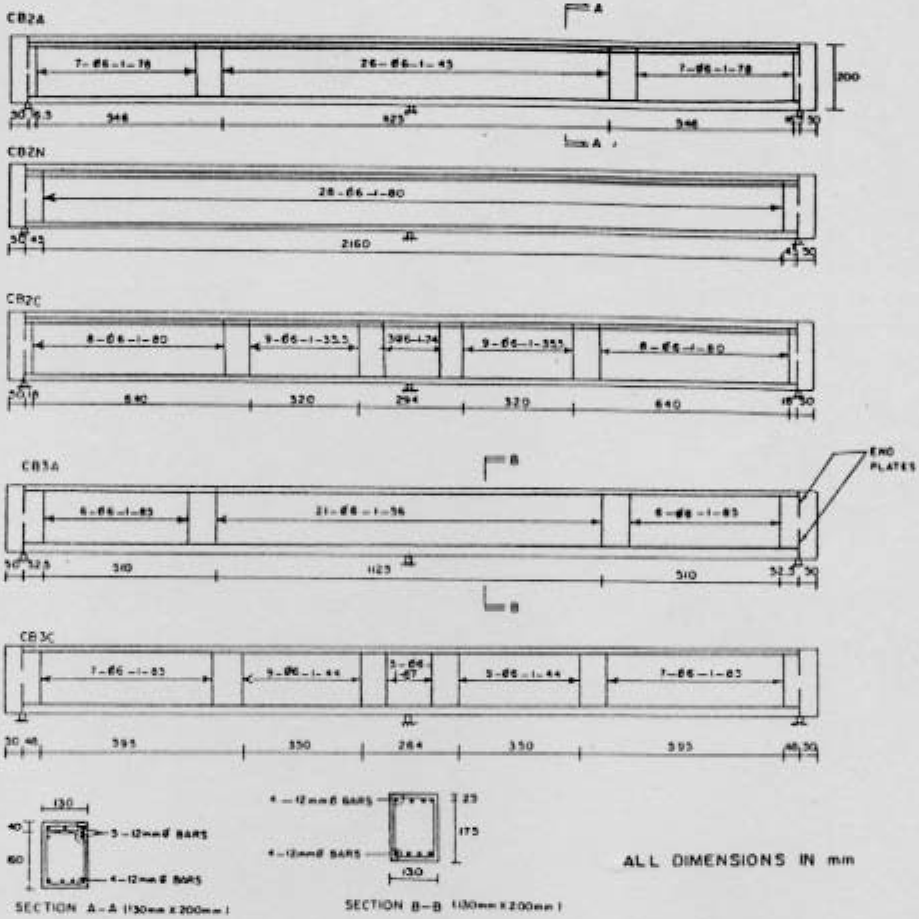

Figure 2: Dimensional, Cross-Sectional and Design Details of RC Continuous Beams Tested

Figure 3: Testing Set-Up

Deformational response The load-deflection relationship of the beams tested are shown in Fig. 4. Although two LVDTs were employed for recording the midspan deflections of each of the two spans, the LVDT readings of the span demonstrating slightly higher deflection are plotted in this figure. It is clear that whereas the beams CB2C and CB3C underwent ductile failure, their code counterparts CB2A and CB3A failed in a brittle manner. The beam CB2N underwent a brittle load-deformation behaviour, as expected.

Cracking process The final cracking pattern of the various beams tested are shown in Fig. 5. For the beam CB2A, first crack was formed above the central support at a total load of 111 kN. With the increase in load, nearly vertical cracks were developed simultaneously below the loading points and above the central support. As the load increased, these cracks grew in length and also new inclined cracks, oriented towards the loading points and the central support, were formed. Failure occurred near the left point load at 436 kN. Similarly to CB2A, the cracking process of CB2C started at 111 kN with the formation of nearly vertical cracks which developed simultaneously below the loading points and above the central support. Cracking was evenly distributed over all the three critical points and the extent of cracking was very much enhanced near failure indicating its greater potential for stress redistribution and superior ductility. At 258 kN load, spalling of concrete beyond the right support (as well as the end plate) took place. This affected the deformational behaviour slightly. The failure crack extended from the loading point of left span to the central support at a total load of 427 kN. Whereas the initial stages of the cracking process of the beam CB2N was akin to the other beams of this series, number of cracks over the central support was more than the other beams. At 382 kN load, failure cracks extending from near the right loading point to the central support became prominent and eventually failure took place.

For the beam CB3A, first crack was visible below the right loading point at 102 kN. More flexural cracks were formed in the subsequent load steps. First web crack was formed at 175 kN below the right loading point. In the following load steps, additional shear cracks were formed. At 290 kN spalling of concrete beyond the left support (i.e. the end plate) took place. Failure occurred at a load of 383 kN when two shear cracks, one in each of the spans became prominent. For the beam CB3C, first flexural crack was formed above the central support. Although the cracking process of this beam was similar to that of CB2C, the formation of first web crack and its penetration deep in to the compression zone was slightly delayed in comparison to CB2C. At failure, failure crack started from the point load of the right span and extended to the central support.

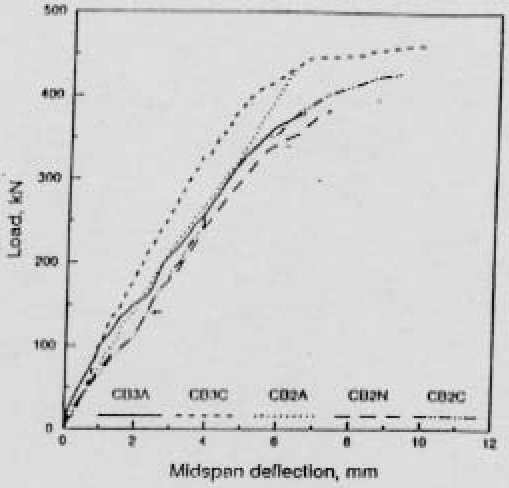

Figure 4: Load-Midspan Deflection Curves of RC Continuous Beams Tested

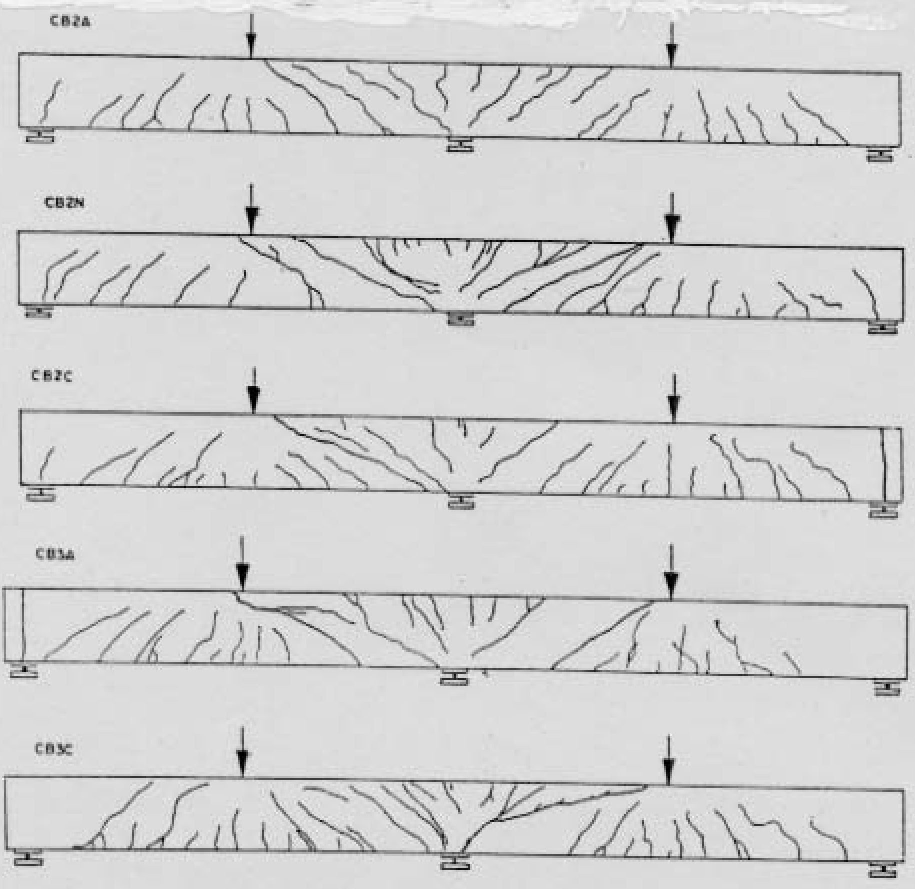

Figure 5: Crack Pattern of RC Continuous Beams Tested

Causes of observed behaviour and failure mechanism It is interesting to note that the failure load predicted by both the American Code and the proposed method was much lower than the actual failure load. This difference in the experimental and the calculated values is largely due to the use of the elastic moment diagram in calculating the design load. Moreover, if, in the calculation of the flexural capacity of the section, the ultimate failure stress of the longitudinal bars was used instead of the yield stress, the flexural capacity of the beam would have been much larger. At loads near failure, triaxial stress conditions exist in the compressive zone of an RC member. Due to the presence of such triaxial stresses, the depth of neutral axis reduces, and consequently the moment capacity of the section increases. Such an increase in the moment capacity of the beam section, at the ultimate limit state, appears to be the only possible explanation that can bridge the gap between the calculated and the experimental findings.

For all the beams tested other than CB2N, the amount of transverse reinforcement furnished was actually designed for sustaining stresses which turned out to be much lower than the stresses which were actually borne at the critical sections at failure. As mentioned earlier, the beams designed to the proposed (CFP) method, was substantially more ductile than their American Code counterpart. It appears that the transverse reinforcement provided in CB2C AND CB3C (in order to make an effective internal support at the point of inflection, as advocated by the proposed method) played an important role in bringing these members to ductile failure. Seemingly, this additional transverse reinforcement helped in the smooth "flow" of load throughout the whole structure by distributing moments from the more stressed to the less stressed regions. It is important to note that the favourable consequences due to the presence of such reinforcement cannot be explained by the traditional truss-analogy concept, as such reinforcement did not cover the full length of the shear critical span.

It is important to note here that the findings of the present series of tests match well with the observations gathered during tests conducted on two-span continuous beams having about one-fifth volume [23] in comparison to the specimens tested here. It proves, quite conclusively, that the significance of modelling internal support is not dependent on the size of the specimen.

CONCLUSIONS

The importance of providing "internal supports" at the points of contraflexure was confirmed from the results of the two RC continuous beams. These internal supports greatly increased the ductility of the continuous beams designed to the proposed method. The other continuous beams, which did not cater for such internal supports, suffered a brittle type of failure. From this investigation it became clear that all the members tested sustained loads much above the design loads showing the inefficiency of flexural design of members based on uniaxial behaviour of concrete at the ultimate limit state. The size independence of the role of internal supports has also been recognized by comparing the findings with the test results of smaller-sized continuous beams, available in the literature.

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