PARAMETER ESTIMATION FOR A CONCRETE BEAM FROM EXPERIMENTS USING FIBER-OPTIC STRAIN GAGES

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In this paper, a preliminary laboratory study on a reinforced beam is presented, and the strains measured by optical strain gauges are used to estimate structural parameters of the concrete that can be used to assess the condition of that beam.

Past research has shown that the measurement of relatively noise-free strain response is essential to provide sensitivity in damage detection in structures. In this study, Fiber-Bragg Grating fiber-optic strain gauges, which have been shown to provide strain data that have comparatively lower noise levels and are not affected by electro-magnetic interference, are used to measure the strains. A study on the laboratory beam reported elsewhere has shown that only a specific long gauge length FOS with a bolted connection to the concrete provided accurate estimates of the concrete strains, even when the surface showed extensive cracking.

Damage detection methods need either undamaged baseline data or a robust and accurate numerical model in order that present conditions can be identified. Here, a numerical model of the reinforced concrete beam based on available parametric representation of concrete behaviour is developed. This model is then updated using the experimental strain data in both the reinforcement bar and the concrete surface. The parametric representation of the concrete behaviour that matches the strain data with a minimum error norm is considered to represent the beam. It is shown that this updated numerical model of the beam does indeed represent both the un-cracked and cracked strains over the entire loading ranges considered in this study. The parameters thus identified have potential to detect damage in concrete girders.

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ABSTRACT: A reinforced concrete beam is instrumented using fiber optic strain gauges, standard demec mechanical strain gauges and an instrumented reinforcement rod. The beam is tested using a standard four point loading and the estimates of strains at various locations are used to identify the material parameters of the concrete used. The updated parametric model of the concrete beam is shown to accurately predict the measured strain values in both steel and concrete over a large range of loading from un-cracked to cracked concrete conditions.

1 INTRODUCTION

The authors are collaborating on a project to develop a robust fibre optic based strain measurement system for use in the structural health monitoring (in this instance load testing) of a prestressed concrete post-tensioned box girder railway bridge in Mumbai. The project, a collaboration between Indian Institute of Technology Bombay, Durham University, City University London and Queen's University Belfast, forms part of the UKIERI (UK-India Education and Research Initiative) programme funded jointly by the governments of the UK and India and administered through the British Council.

The work falls into two parts, the first being an evaluation of suitable commercially available fibre optic strain sensors and the second being use of the sensors in the actual load test on the bridge. In the first part, the evaluation of the sensors has been done as has been shown in Banerji, et al (2011). The model updating using the sensor data to form a numerical model which accurately represents the strain measurements forms the subject of this paper. The second part comprising the actual deployment on the bridge has been completed and the results are in the analysis phase.

In the numerical modelling of this test beam, various aspects needed to be considered properly to enable the numerical model to represent the actual behaviour of the test beam under the specific loading that it experienced. These included the material constitutive models for concrete in compression and steel in tension and compression (as there was both top and bottom steel in the test beam), the tensile behaviour of concrete including the concrete tension crack model, and the modelling of the bond between the concrete and the steel reinforcement bars. A review of the current state of art in modelling and incorporating the various behaviours has been shown in Banerji, et al (2011).

A correct characterisation of the concrete properties in tension needs to incorporate concrete tension crack initiation and propagation, since significant cracking was observed during the loading of the beam. As per Rabczuk, et al (2005), the theories of crack modelling by and large fall into two major groups. One group models the crack as a smeared system e.g. Hillerborg, et al (1976) and Jirasek & Zimmermann (1998), where the crack is essentially fictitious, and another group models the crack as a real discontinuity or as a discrete crack, e.g. Xu and Needleman (1996). The modelling of the crack as a real discontinuity imposes constraints on the model by needing pre-knowledge of the crack existence. The model is also mesh dependent since cracking can only occur at the element interfaces. Although a mesh independent approach has been proposed by Belytschko and Black (1999) the cost of computation of these models is comparatively higher than the smeared crack models.

Since the final implementation was designed to be on a real bridge structure, the smeared crack approach, as given by Rabczuk, et al (2005), has been considered for this work to reduce computational costs.

2 PROCEDURE

2.1 Description of Test and Experimental Setup

The reinforced concrete beam (see Fig 1) was 5200 mm long overall (4870 mm between simple supports), 250 mm deep and 300 mm wide. The main tension (bottom) reinforcement was three 16 mm diameter high yield reinforcing bars (the centre one of which was the internally strain gauged bar) and the top reinforcement comprised two 12 mm diameter high yield bars. Cover to the centre of both the top and bottom reinforcement was 30 mm. Links were provided between the supports and the load points but were omitted between the load points to avoid unwanted crack propagation. As shown in Fig 2, the fibre optic strain sensors, strain gauged reinforcing bar and the Demec points were all positioned at the same level on the beam (i.e. the level of the tension reinforcement) thus enabling easy comparison between the readings from all the strain measurement devices. Beam deflections were measured at mid-span and at the load positions using an lvdt at each location.

The beam was loaded in four point bending (see Fig 1) which provided a constant moment zone of 2500 mm. Manually pumped hydraulic jacks were used and the loads applied were measured using load cells incorporated in the loading system. A full set of readings was taken from all the sensors at each load stage during the tests.

The test readings correspond to an incremental loading up to a maximum load of 20 kN.

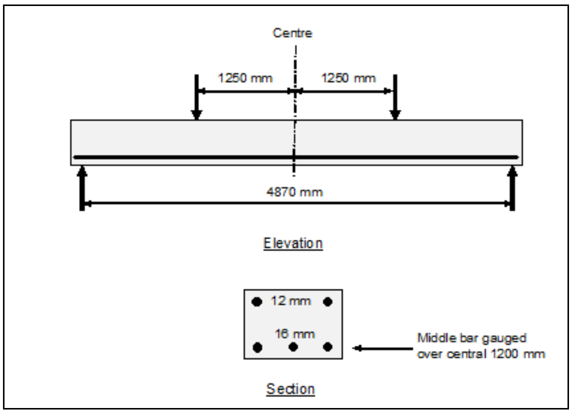


Figure 1. Description and Layout of the test beam

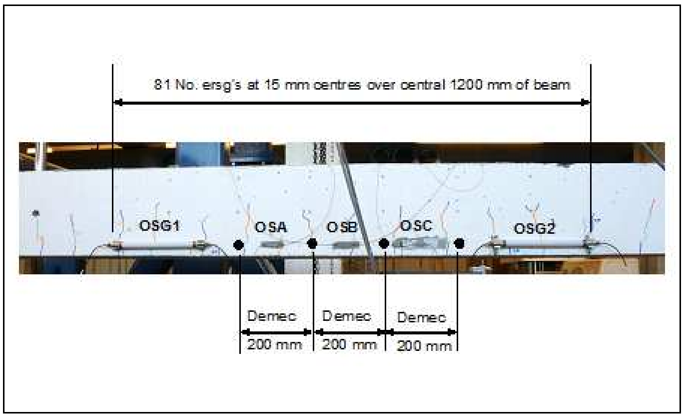


Figure 2. Sensor placements on the test beam

2.2 Formation of Numerical Model

2.2.1 Material constitutive modelling

The steel reinforcement was modelled as an elastic-plastic material in both tension and compression. The Young's modulus was taken as 205 GPa and the yield stress was considered as 463 MPa. The steel was expected to behave linearly elastic up to the yield point and then be perfectly plastic after the yield point.

The stress-strain curve for the concrete material in compression was considered as per Fig 3, where a bilinear model was used to model the curve up to a strain of 0.002, and beyond that a perfectly plastic model was used. In reality, it has been observed that there is a strain softening effect in concrete after achieving full plastic stress, this was not considered here as the strain in the concrete in the range of loads considered in this study did not exceed 0.002. In the numerical model, the stress-strain relation in compression was considered as being linearly elastic up to a stress level of 30 MPa with a Young's modulus of 36.764 GPa which corresponds to CEB – FIP Code (1990) as referenced by Mehta and Monteiro (2006). As the stress level never exceeded this value in the entire load test, the shape of the curve beyond this value was not of any significance. The constitutive relation given in Fig 3 corresponds to unconfined concrete. Since the test beam did not have any links in the region where strains were measured, this seemed a reasonable choice for modelling the beam's behaviour.

The tensile behaviour of concrete is modelled as per Akkerman (2000) who proposed a two parameter concrete constitutive model as is schematically shown in Fig 4. The model is divided into four regions and each region has a different tangent stiffness $E_{c,ti}$ and secant stiffness $E_{c,si}$. In each region, the secant stiffness modulus of elasticity defines the apparent strain with the apparent stress. This model is defined using two dimensionless constants α and β and hence is a two parameter concrete constitutive model.

The concrete-reinforcement bond behaviour is modelled by incorporating a set of non-linear springs using an element scale model as shown by Britto (1998). The stress displacement relation is given in Fig. 5.

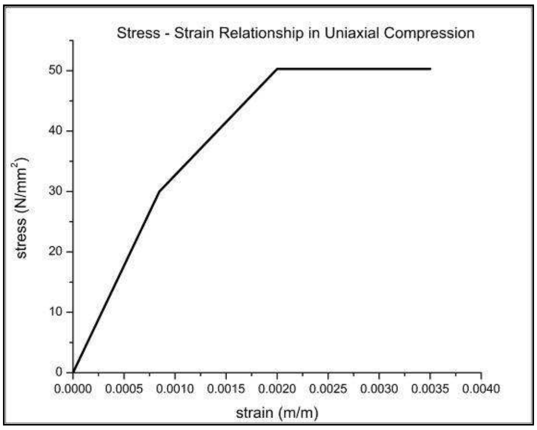


Figure 3. Compressive stress-strain relation in concrete

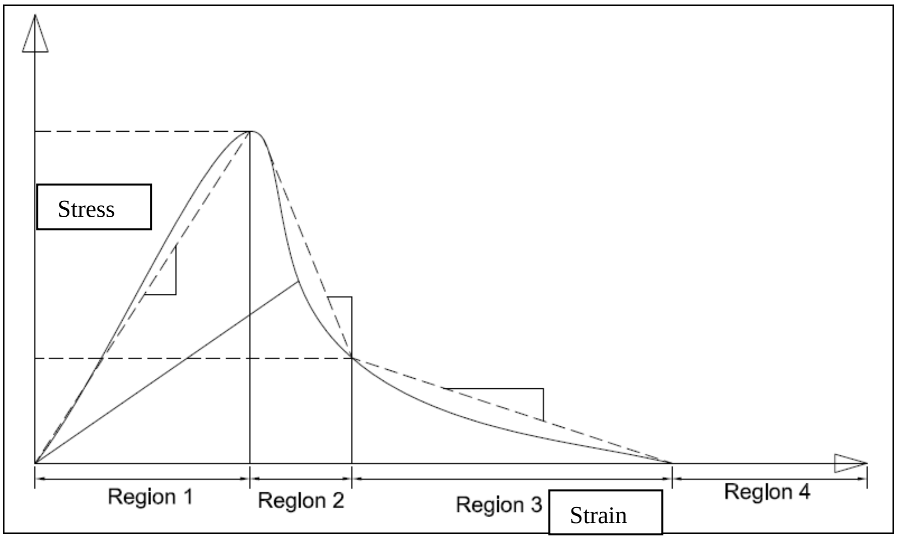


Figure 4. Tensile stress-strain relation in concrete

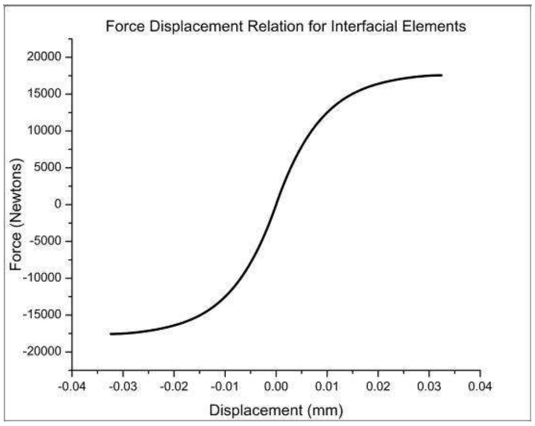


Figure 5. Bond behaviour between concrete and reinforcement

2.3 Analysis Procedure

The reinforced concrete beam was analysed using standard non-linear finite element software that incorporated all the constitutive models discussed above. The concrete was modelled using eight-node isoparametric brick elements, and the reinforcement using two-node truss elements.

Since the loading was symmetrically applied on the beam, only half the beam was modelled and symmetry conditions were applied on the mid face. The support boundary conditions were modelled as a line support condition.

The load was applied on the model in two steps. In the first step, an inertial load was applied which was then propagated on to the next step, as required by the software. In the second step the appropriate vertical load was applied on the corresponding application points.

3 RESULTS AND DISCUSSIONS

3.1 Extraction of results

The numerical model was monotonically loaded and the responses were recovered at each sub-step of completed iteration. The load was increased gradually up to a total of 20 kN as done in the test. The responses were then plotted along with the experimental results. Since there were multiple strain gauges placed upon the steel reinforcing rod, all of which were in the region of nominally constant moment, these are being represented in the graphs in the form of a mean value and a band of +/- one standard deviation.

The numerical model was first analysed using the constitutive model parameters given in the section above. The compressive strength was taken as 50.3 MPa as measured in the laboratory for the concrete used in the test. The characteristic strength in tension for concrete was taken from Mehta and Monteiro (2006), who state that the value should be between the bounds of 2.79 MPa and 5.43 MPa, with a possible mean value of 4.11 MPa for the above compressive strength. The mean value of tensile strength was first used in the numerical analysis.

Subsequently, a brute-force model updation procedure was done to ensure that the numerical results for both steel and concrete matched the experimental results in a least-square error sense. The variables considered in the updation process were the tensile strength of concrete and the post-cracking strain softening behaviour, as the numerical results were most sensitive to these parameters. The final value of the characteristic tensile strength was estimated as being 1.76 MPa, and the post-cracking strain softening given in Fig. 6 showed a merging of Region 2 and 3 without having a distinct kink as suggested by Akkerman (2000) and shown in Fig. 4. It is interesting to note that the final tensile strength of 1.76 MPa is below even the lower bound value of 2.79 MPa as suggested by Mehta and Monteiro (2006). This may be ascribed to the compressive strength of concrete in the cracked region being less than the characteristic value because of the inherent heterogeneity and consequent changes in the local strength of the concrete in the tested beam.

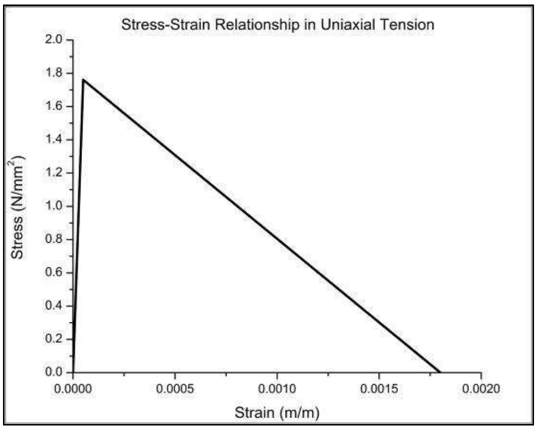
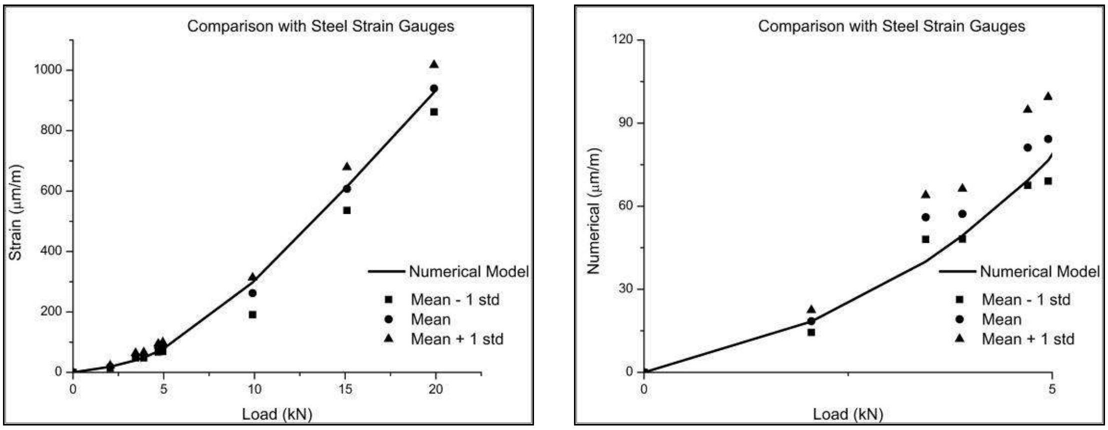


Figure 6. Concrete Tensile behaviour as determined by best fit of results to experiments

The comparison of the estimates of the strains in both steel and concrete obtained from the final updated numerical model with the experimental values for different loads up to an upper load limit of 20 kN is shown in Fig. 7 and Fig. 8. The experimental values for steel are taken as the mean and mean ± 1 standard deviation of the 81 ersg strain values in the steel reinforcement. The experimental values of the concrete considered are firstly the average of the three demec gauge readings at the level of the tension reinforcement, and secondly the readings from the optical fiber strain gauge OSG 2 as is shown in Fig. 2. This is because from an initial processing this is the strain gauge which has performed best throughout the test. It can be seen the numerical results for steel strains match the mean strain values in steel for the entire loading domain from 0 to 20 kN, and the numerical results for concrete strain match the average of the 3 strain values obtained from the Demec gauges and also the strain estimated by the optical sensor OSG 2, which is the optical strain sensor that provided the most accurate estimates of strain in concrete.

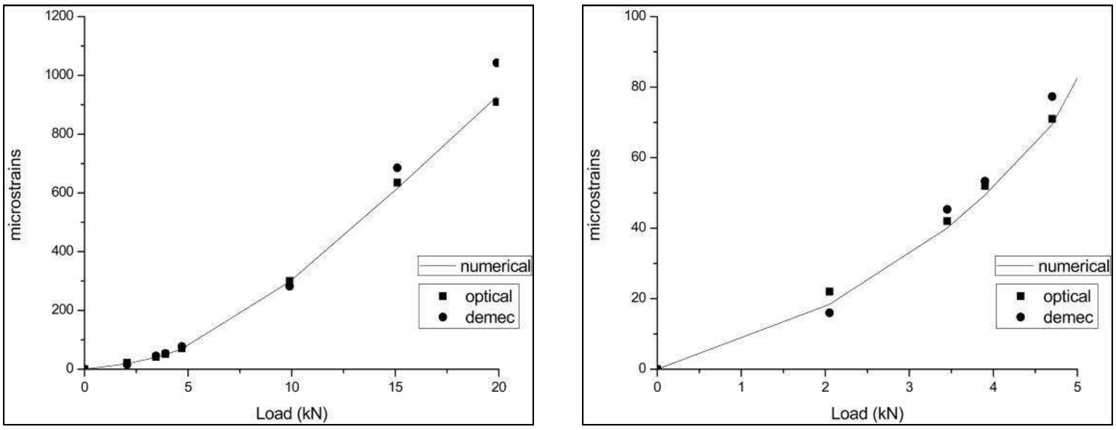
A comparison of the numerical values of the centre span and load position displacements with those obtained from the experiments for the 0 to 20 kN load domain are, respectively, presented in Figs 9 and 10. It can be seen from these figures that the numerical and the experimental values match very well.

Thus it is clear that the numerical model and the model updation procedure used here provide a very good model for analysis of the reinforced concrete beam considered in this study. Finally, this numerical model has good potential of being used for the test bridge to be instrumented and studied later in this project.



(a) (b)

Figure 7. Results for Steel Strain Measurements – a) Full Range and b) For 5 kN



(a) (b)

Figure 8. Results for Concrete Strain Measurements – a) Full Range and b) For 5 kN

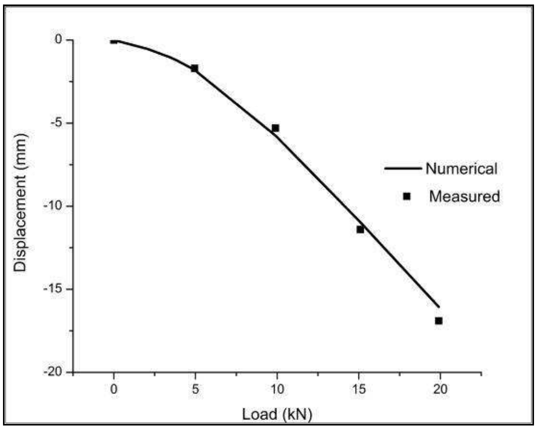


Figure 9. Results for displacement measurements at centre of span

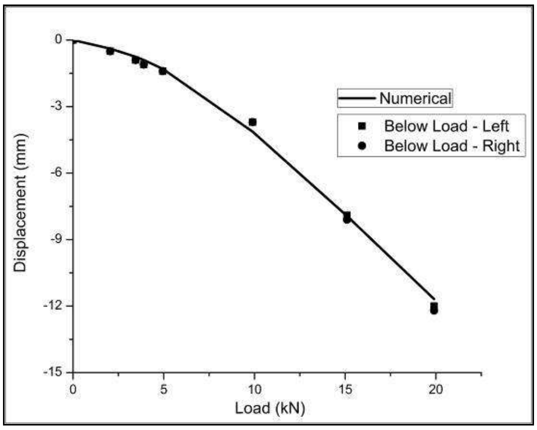


Figure 10. Results for displacement measurements below load application point

3.2 Conclusion

Although the results of the numerical model and the experimental data showed good correspondence, a significantly larger number of tests would need to be performed before parameters drawn from a well corresponding numerical model could be considered representative of the test concrete. The experience of the tests showed that long gauge length optical strain gauges, if properly mounted can give very accurate strain readings and can be used for model updating purposes.

Both optical and demec strain gauges gave very good correspondence in the service range of the loads, where predominantly un-cracked concrete exists. In the higher load ranges, the demec gauges tended to over-estimate the strains whereas the optical strain gauges gave good correspondence with the numerical estimate for the entire range of loads applied.

The evaluation of the parameters on which the concrete behaviour is dependent, and its consistent behaviour over the entire range of the loads applied, gives confidence in using the particular formulation of concrete for the main problem of structural health monitoring of the box girder railway bridge envisaged. This concrete model being numerically tractable allows an accurate depiction of complex problems as exist in the real world.

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