Experimental Verification of Automated Design of Reinforced Concrete Deep Beams

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Abstract: Reinforced concrete deep beams (RCDBs) are generally designed using conventional methods such as empirical equations or strut and tie models. These methods however do not take into account the redistribution of forces resulting from non-linear materials' behaviours. Indeed, it is a common belief that deep beams, shear walls and three-dimensional reinforced concrete structures are over-reinforced. The proper determination of the amount of reinforcement is therefore essential for a safe and economical design. To achieve a safe and economical design, non-linear finite element analysis that incorporates non-linear material behavior must be part of the design process itself, and must be applied before and during the design of the reinforcement. In this paper an automated design technique using nonlinear finite element analysis is presented and validated with experiment for RCDBs. The technique was developed using the ABAQUS scripting interface and the Python programming language. This nonlinear finite element automated design technique was used to design the reinforcement for a RCDB. This beam was then manufactured and tested in the laboratory. The experimental results were compared with the automated design results. It was found that the experimental results corroborated those predicted with the automated design method. The failure load was found to be within 1 % of the target load.

Keywords: Deep Beams, Automated Design, Experimental Verifications, Finite Element Analysis and Strain Distributions.

1. Introduction

The application of the reinforced concrete deep beams (RCDBs) within structural engineering practice has risen substantially over the last few decades. More specially, there has been an increased practice of including deep beams in the design of tall buildings, offshore structures and foundations. They differ from shallow beams in that they have a relatively larger depth compared to the span length. As a result the strain distribution across the depth is non-linear and cannot be described in terms of uni-axial stress strain characteristics. Despite a large amount of research carried out on the behaviour of deep beams over the last century (Kong 1990), a rational method for their design is yet to be agreed on; current available methods include empirical methods such as those proposed in the ACI code318-08 (2008) , Canadian Standard (2004), CIRIA Guide 2 (1977) and CEB-FIB Model Code (1993) and the Australian standard AS3600 (2001).

Empirical equations derived from experimental tests have some limitations. They are only suitable for the tests conditions they were derived from, and most importantly, they fail to provide information on serviceability requirements such as structural deformations and cracking. Likewise, the strut and tie model (STM), although based on equilibrium solutions thus providing a safe design, does not take into account the non-linear material behaviour and hence also fails to provide information on serviceability requirements. Cracking of concrete and yielding of steel are essential features of the behaviour of concrete structures and, therefore, they must be taken into account in predicting their ultimate load capacity as well as service behaviour. Failure to do so simply means that the redistribution of stresses in the structure is not taken into account. The obtained designs are believed therefore not to be very economical as they tend to overestimate the amount of reinforcement needed. Thus, the development of an alternative design method is needed to achieve an economical design of RCDBs.

Recent developments in the finite element method and computer aided design have eliminated the need for the long and expensive tests in many areas of engineering. For instance, numerical simulations of metal forming processes have been conducted abundantly and used extensively for the analysis and design of industrial parts (Khelifa, 2007) . The same goes for the automobile industry, which simulates crash tests extensively even though it is possible to develop a product solely through prototyping. On the other hand, this potential has not been fully realized by civil engineering structural designers. Non-linear finite element is still used as a verification tool rather than a design tool. More recently the finite element method has started to be used as a design tool as shown by Khennane (2005) for slabs, and Tabatabai and Mosalam (2001) for deep beams and one way slab. In this paper the technique developed by Khennane (2005) for reinforced concrete slabs has been used to design the RCDBs. This designed beam will be manufactured and tested in the laboratory to validate the developed method.

2. Automated Design Technique

The algorithm shown in **Error! Reference source not found.**, schematically shows the different steps of the design methodology. Initially the beam, without any reinforcement, is analysed as a linear elastic medium to identify the areas of potential cracking. A minimum reinforcement is then incorporated in these areas (hereafter named reinforcing fields) and the target design load is applied in increments. The analysis is carried out in a non-linear fashion, and the amount of reinforcement is updated as required. At the end of a load increment, and before the solution proceeds to the next step, the reinforcement is checked whether it has yielded or not. If no yielding has occurred, the analysis progresses to the next load increment. Otherwise, the reinforcement is updated as to avert yielding. The design process is carried out iteratively until the target load has been achieved and no yielding is detected.The smart fictious material model for steel (Hoogenboom 1998) is used to update the reinforcement in a yielded reinforcement field. The steps in the algorithm are implemented using the Abaqus Version 6.10 (2010) scripting interface, which is an extension of the object oriented programming language Python 2.5 (2008) . The advantage of using the Abaqus platform is the availability of robust concrete models; one of them is the CDPM (Concrete Damage Plasticity Model) based on the works of Lubliner et al.(1989) and Lee and Fenves (1998). A detailed description of the model and its implementation can be found in the Abaqus 6.10 documentation (2010).

BEGIN

Step 1: **Load the Abaqus Solver to read the input file and carry out a linear analysis to identify the regions of potential cracking. It is important to make sure that the job is run interactively.**

Step 2: **Group all the elements belonging to regions of potential cracking into element sets, called herein reinforcing fields.**

Step 3: **Provide these reinforcing fields with minimum reinforcement ratios.**

Step 4: **Set the target load for which the reinforcement is to be optimised, and divide it into load increments.**

Step 5: **While the applied load is less that the target load**

- **Carry out a nonlinear analysis of the current model**
- **Access the Abaqus database file (extension .odb)**
- **Loop through the reinforcing fields (elements sets) and retrieve the maximum and minimum strains at the reinforcement level, and check whether the reinforcement has yielded or not.**

  IF no yielding of reinforcement
    - **Load = load + load_increment**

  ELSE
    - **Update any reinforcement that has yielded.**
    - **Keep load constant.**

  END IF

END

Figure 1. Automated design process.

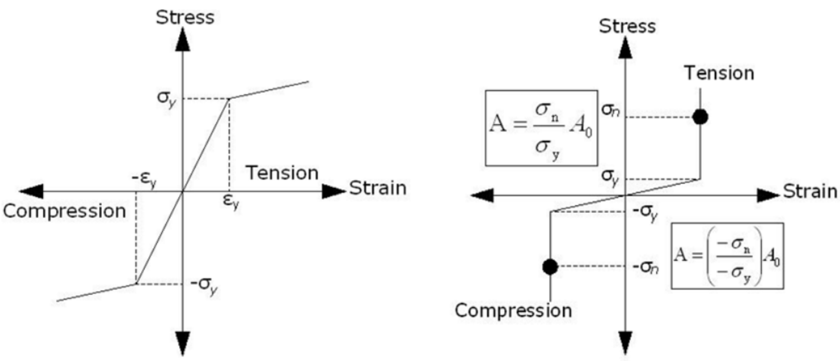

Figure 2. Smart fictious model.

The criterion is that the steel should be strained as close as possible to its yield strain. The smart fictious material model for steel (Hoogenboom 1998) is used for this purpose. The steel modelmodel is reversed to decide the amount of reinforcement as shown on Figure 2.

The calculated stress, σ_n , either in tension or compression, is compared to the yield stress, σ_y . If the calculated stress is less than the yield stress, no action is taken. Otherwise, the new area of steel required to inhibit yielding is obtained as:

$$A = A_o \frac{\sigma_n}{\sigma_y}$$

where, A is the updated steel area and A_0 is the initial steel area. This process is equivalent to a plasticity algorithm where the state of stress is scaled back to the yield surface. However, instead of redistributing the excess stress as a pseudo-load vector as done in a plastic analysis, it is the area of steel that is increased to keep the strain just at yielding. A detailed description of this process termed strengthening behaviour as opposed to plastic behaviour is explained in Hoogenboom (1998).

3. Finite element model and its Validation

Before applying the automated design method, it is necessary to develop a finite element model for RCDBs. A two dimensional finite element model was developed for the beam. Four nodded continuum plain stress (CPS4) element was used for concrete parts of the beam. The same element was used for the modelling of loading and support plates. Two nodded truss element was used for reinforcing bars model that embedded within the concrete. Perfect bond was assumed between the reinforcement and the surrounding concrete. This type of representation allows the reinforcement to be treated as an integral part of the basic element and its stiffness contribution can be evaluated by the principle of superposition. 50mm × 50mm mesh size was used for modelling of concrete and reinforcing bars.

The concrete damage plasticity model (CDPM) was used to present the behaviour of concrete in RCDBs. This model uses damage plasticity formulation in compression and cracking combined with damage elasticity in tension. The input parameters required for defining this material model included the uniaxial compression stress-strain curve, uniaxial tension softening curve, concrete dilatation angle, biaxial compression strength ratio and ratio of tensile to compression meridian. In this model, the compression harding and damage data are provided as a table under heading compressive stress-inelastic strain and compression damage parameter-inelastic strain. The stress-strain curve can be defined beyond the ultimate stress into the strain-softening regime. The uniaxial compression stress-strain curve reported by Wee et al. (1996) was adopted to produce these data. The post-failure tension stiffening and tension damage are also required in the concrete damage plasticity model. The tension stiffening and damage data are also respectively supplied as tables under the headings of stress-cracking displacement and damage parameter-cracking displacement. In this study, the concrete behaviour in tension is defined using the fracture energy criterion developed by Hordijik (1991). The dilatation angle was assumed to be 50^o. A detailed description of the CDPM model and its implementation can be found in the Abaqus 6.10 documentation (2010). A linear elastic perfectly plastic model was used for reinforcement bars to analyze the beams. Only half beam was modelled for analysis due to the symmetry of loading and geometry to save processor time and memory allocations since ABAQUS generates a large number of files.

Boundary conditions that represent the structural support specify values of displacement and rotations at appropriate nodes. XSYMM ($u_x = 0$, $\phi_y = 0$ and $\phi_z = 0$) was applied to the symmetry plan of the beam and y-displacement was applied at the left support of the beam for the analysis. Four point bending load was applied to the beam for analysis. Concentrated loads were employed to the specific nodes as increments.

A beam tested by Tanimura et al. (2005) was used to validate the developed finite element model, whose details are shown on Figure 3, and for which an experimental load versus mid-span deflection is available, is analysed as is; that is with its original reinforcements. The yield strengths of longitudinal and transverse bars were 458 and 388 MPa respectively.

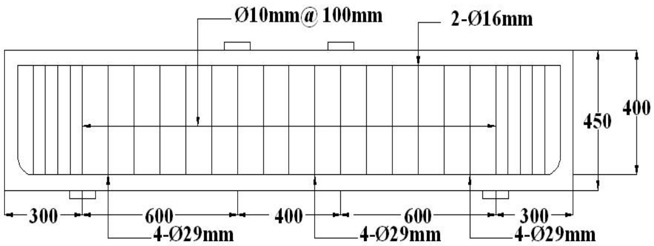

Figure 3. Dimensions in mm and reinforcements details.

Figure 4 shows that the finite element analysis using the damaged plasticity model for concrete reproduces the experimental behaviour of the tested beam very well. This places enough confidence in its performance to be used in the presently developed optimisation process.

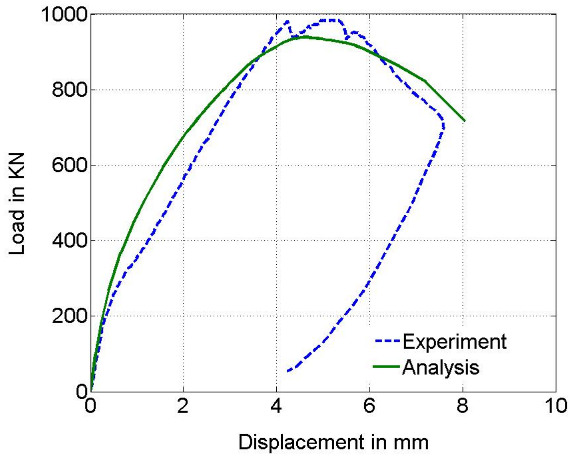

Figure 4. Model validation.

4. Automated design procedure for the RCDBs

The automated design technique was used to design the reinforcement for the beams shown in Figure 5 so that they could support the target load of 450 kN.

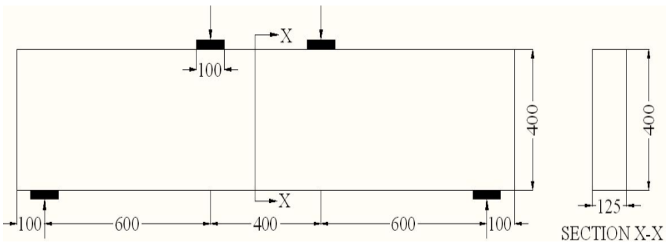

Figure 5. Dimensions in mm and loading.

4.1 Identification of reinforcing fields

To identify the areas of potential cracking, the beam was initially analyzed as linear media without any reinforcement. Figure 6 shows the contours of the principal strains. It can be clearly seen that there are regions of high tensile and compressive strains respectively at the bottom and the top parts of the beam in the flexural span. In addition, it can be observed that the direction of the principal strains changes to 45^{o} in the shear regions.

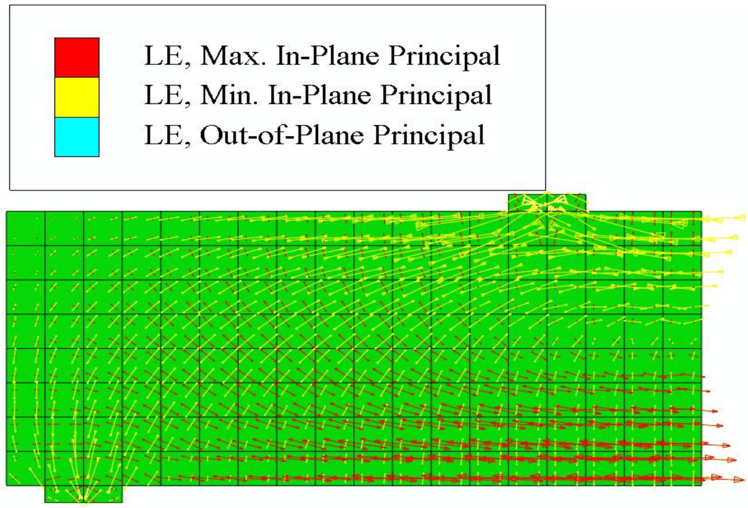

Figure 6. Linear elastic analysis of RCDB for the identification of regions of potential cracking

Based on these regions of high strain intensities, seven reinforcing fields are identified for the solid beam as shown on Figure 7. They are named according to their positions, and then translated into element sets in Abaqus, and each assigned with an initial φ10 mm bar.

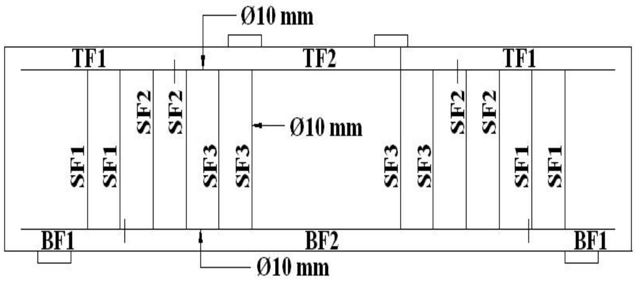

Figure 7. Reinforcing element sets for the RCDB.

4.2 Design process

As mentioned previously, the RCDB will be design for a load of 450 kN . Applying the algorithm described in Figure 1 results in the evolution with load of the steel areas in the different reinforcing fields as shown in Figure 8 .

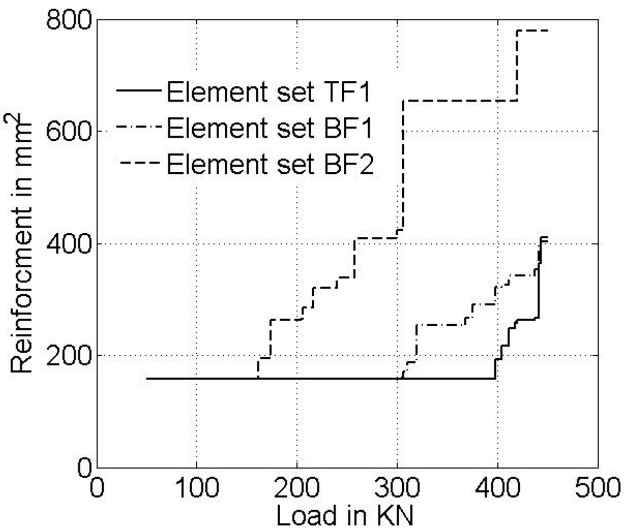

Figure 8. Increase in steel areas for the RCDB.

As a result of stress redistribution, the steel areas do not increase uniformly in the reinforcing fields. The first yielding occurred in the flexural reinforcing field BF2 at an applied load of 162 kN. The next reinforcing field to yield is the flexural reinforcement in the shear spans BF1. This takes place at a load of 300 kN. The reinforcement in the compressive region TF2 starts to yield at a load of 398 kN. It can be also noticed that the target load of 450 kN is reached without any shear reinforcements (SF1, SF2 and SF3) and compressive reinforcement (TF1) yielding, thus keeping the original φ10 bars provided. When it reaches the target load of 450 kN, the reinforcement in the tensile region BF1 and BF2 have increased from 157 mm^2 (equivalent to 2 φ10) to 402.32 mm^2 (equivalent to 2 φ16) and 778.2 mm^2 (equivalent to 4 φ 16) respectively. The reinforcement in the compressive region TF2 on the other hand has increased from 157 mm^2 (equivalent to 2 φ10) to 410.43 mm^2 (equivalent to 3 φ10 and 1 φ16).

The optimized reinforcement details hence obtained are represented in Figure 9. It is clear that the same amount of tensile and compressive bars is not needed throughout the beam span. It is also observed that no shear reinforcement is required at the mid-span of the beam.

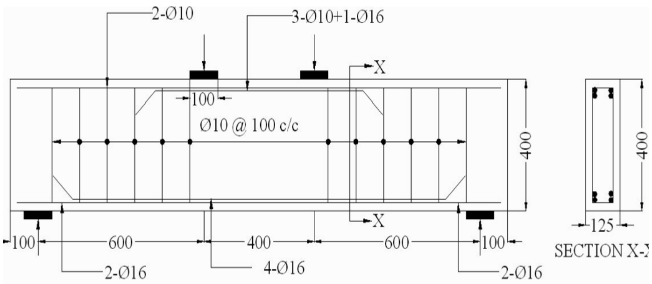

Figure 9. Optimized reinforcements for the RCDB.

5. Experimental Program

The objective of the experimental program is to validate the automated design results presented in the previous section. A RCDB will be manufactured according to the design presented on Figure 9 and tested until failure.

5.1 Materials and Fabrications

The cement used for the concrete mix was Type GP Portland cement. Crushed stone and river sand used in concrete mix. The maximum size of coarse aggregate was 10 mm. The concrete mix proportion by weight per m^3 is given in Table 1. The beam specimens and the cylinders were cured in the same condition for 28 days. The cylinders were tested on the same day as the beam test, and the 28 days strength was found to vary between 53 MPa to 55 MPa.

Table 1 Concrete mix proportion

Mix content	Unit weight, Kg/m^3
Water	207
Cement	414
Coarse aggregate	816
Fine aggregate	938

Deformed steel bars were used for longitudinal and transverse reinforcements. Their yield strength was 500 MPa. The reinforcement cages were fabricated in the workshop as per Figure 9. Longitudinal and web reinforcements were cut into the required lengths. The web bars were then tied to the longitudinal bars by point welds. The tensile bars were prepared for the application of strain gages. This consisted of grinding the bars for approximately 80 mm in length at the gage application point. All beams were cast in oiled wooden forms.

5.2 Instrumentation

Six, Y11-FA-8-120-VM3T type electric strain gauges were used to measure the strain at specific points along the tensile bars. The resistance and the gauge factor of the strain gauges were respectively equal 120.0 ± 0.3% and 2.05 ± 1%. Displacements are measured at mid span using LVDT's for the beams.

5.3 Test Setup and Procedure

5.3.1 Test setup

Testing is done on a Shimadzu Universal Testing Machine of a capacity of 1000 kN. The test setup is shown in Figure 10. All strain gages, LVDT's and the load cell are connected to a SCXI-1000 data logger interfaced with a computer. LabVIEW software is used to record all of the data. Prior to testing, the data acquisition system is electronically zeroed.

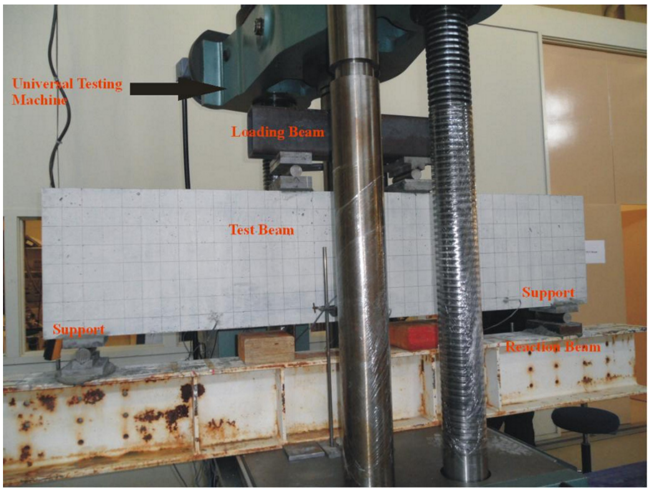

Figure 10.Test setup

5.3.2 Test procedure

The beams were tested under four-point bending. Displacement control mode was adopted at an increment 0.5mm/minute until the beams either completely collapsed or their resistance decreased with increasing deformation.

6. Results and Discussions

6.1 Load-displacement response

Figure 11 shows the experimental load versus deflection curves as well as the optimized load deflection curve for the beam. It can be clearly seen that the automated design method predicted well the load deflection behavior. Most importantly, the design resulted in a very ductile behavior as is proven experimentally.

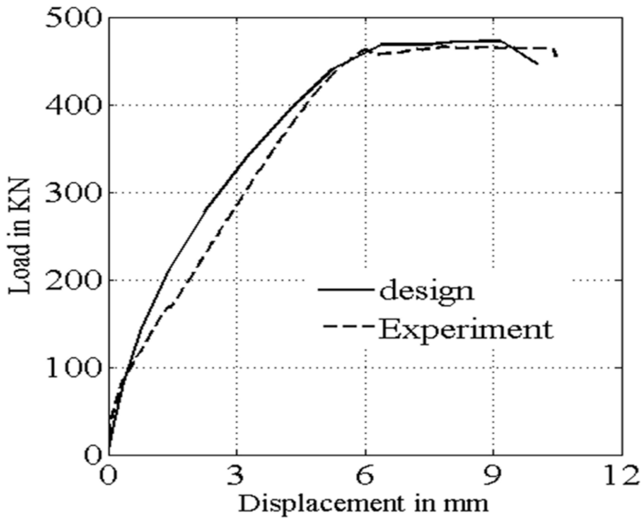

Figure 11. Design and experimental load displacement curves.

6.2 Crack Pattern

The first diagonal shear crack was formed in the shear span at 115 KN. Flexural cracking occurred at a load of 132 KN at near the center of the beam. The beam failed at a load of 466.38 KN by diagonal cracking in the shear span. The final crack pattern is represented in Figure 12. Figure 13 shows the computed crack pattern. It can be clearly seen that it is very similar to those shown on Figure 12.

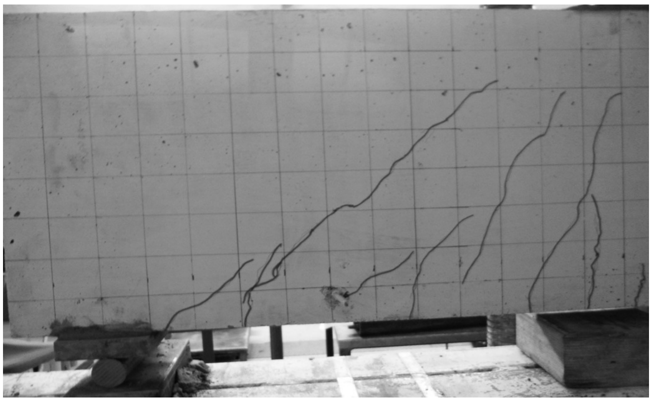

Figure 12. Left side crack pattern for beam

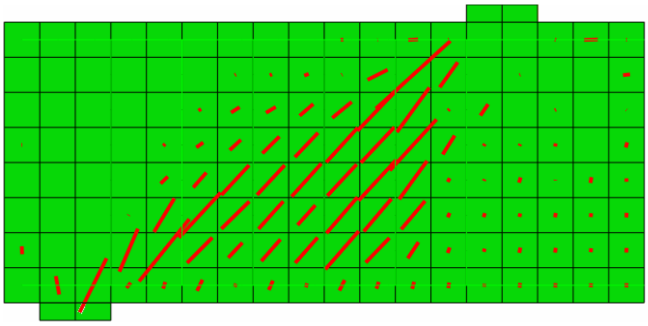

Figure 13. Crack pattern at peak load for RCDB obtained from design

6.3 Steel strains

Figure 14 shows the experimental and anticipated strain distributions in the bottom reinforcement when the applied load reaches the target load. The results show that the longitudinal steel is slightly overstrained in the flexural zone as the recorded strains exceed the yield strain for both beams. Nonetheless, it can be observed that the proposed design technique uses the reinforcing steel more efficiently since most of the recorded strains at failure are within the vicinity of the yield strain of the steel.

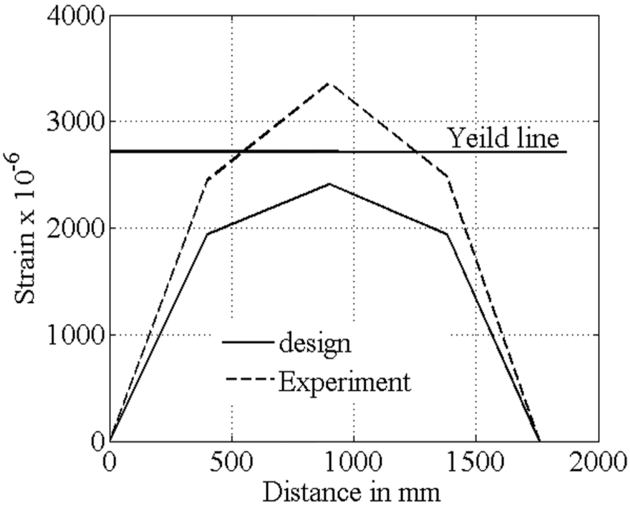

Figure 14. Strain in tensile bars at failure

7. Conclusions

An algorithm making use of professionally developed finite element software is presented for the design of the reinforcement. It was used to design the RCDB. The rationale for the design is that the steel bars carrying the loads once the concrete is cracked should be strained as close as possible to the steel yield strain. The developed approach can assist practising engineers in achieving very economical and safe designs.

To validate the method, a beam specimen with the obtained steel were cast and tested in the laboratory. It was found that the experimental results corroborated those predicted with the automated design method. The failure load was found to be within 1 % of the target load. The measured mid-span displacements at failure were also consistent with those predicted by the method. It was also found that the design resulted in a more ductile behaviour. The measured steel strains were also found to be in the vicinity of the yield strains as predicted by the method. Most importantly, it was experimentally proven that the method uses the reinforcing steel more efficiently.

Yet, one can still argue that the new design method may not be economical after all as it involves a lot of cutting of steel bars to comply with the design. This may be true for a one-off job, but it is definitely not the case for the pre-cast industry that manufactures thousands of panels, where automated cutting and welding are used and the savings on steel could be substantial.

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