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Fem Modelling and Analysis of Reinforced Concrete Section with Light Weight Blocks Infill

T. Subramani[1], D. Sakthi Kumar[2], S. Badrinarayanan[3]

[1]Professor & Dean, Department of Civil Engineering, VMKV Engineering College, Vinayaka Missions University, Salem, India.

[2] PG Student of Structural Engineering, Department of Civil Engineering, VMKV Engineering College, Vinayaka Missions University, Salem.

[3]PG Student of Irrigation Water resources and management Engineering, Department of Civil Engineering, VMKV Engineering College, Vinayaka Missions University, Salem,

ABSTRACT

In an attempt to reduce the self weight of reinforced concrete structures, a new development of lightweight sandwich reinforced concrete (LSRC) section has been proposed as an alternative option to solid section. LSRC section is a reinforced concrete section which contains lightweight blocks as infill material. An experimental investigation into the strength of LSRC beams has shown promising results under flexural tests. To ensure the serviceability of LSRC members under service load, it is necessary to accurately predict the cracking and deflection of this section. This paper will focus on analysing the behaviour of the tested beam specimens after cracking occurs. ANSYS 12.1 was employed to study the crack propagation of LSRC beams under bending. The numerical model shows the crack in the area of AAC blocks which associates with the brittle failure of LSRC beams. The crack propagation of the beams analysed by ANSYS agrees well with the results from the experimental investigation.

In structural design, an ideal situation in material saving is to reduce the weight of the structure without having to compromise on its strength and serviceability. A new lightweight sandwich reinforced concrete section has been developed with a novel use of lightweight concrete as infill material. The section, namely LSRC section, is suitable for use as beam or slab members. Experimental investigations into the strength of beams with LSRC section shows promising results under both flexural and shear tests. Based on the test results, the flexural capacity of LSRC beams was found to be almost identical to the capacity of the equivalent solid beam. The shear capacity of the LSRC beams was expectedly reduced due to the low compressive strength of the lightweight concrete infill material. ANSYS 12.1 was employed to develop three dimensional nonlinear finite element models of LSRC beams and was verified against the experimental results.

KEYWORDS: Fem Modelling, Analysis, Reinforced Concrete Section, Light Weight Blocks Infill

I. INTRODUCTION

A newly developed lightweight reinforced concrete (LSRC) section has been experimentally investigated (Vimonsatit et al. 2010). The section is made up of a reinforced concrete with lightweight block infill. LSRC section can be used either as beams or slabs. Figure 1 shows the construction of LSRC beams. The developed LSRC members are suitable for large span construction due to the weight saving benefits and ease of construction.

This paper focuses on analysing the behaviour of the tested beam specimens after cracking occurs. Finite element method (FEM) is a powerful tool commonly used for analysing a broad range of engineering problems in different environments. FEM is employed extensively in the analysis of solids and structures and of heat transfer and fluids.

A nonlinear FEM computer program ANSYS has been widely used for academic research aswell for solving practical problems. Buyukkaragoz (2010) usedANSYS to study on the subject of strengthening the weaker part of the beam by bonding it with prefabricated reinforced concrete plate. Single load was applied in the middle of the beam. solid65 and link8 were employed to model the reinforced concrete with discrete reinforcement, while solid46was used for modeling the epoxywhich is used to bond the prefabricated plate to the beam. The result from experiment in the laboratory is quite similar to the finite element finding. Barbosa and Riberio (1998) used ANSYS to compare the nonlinear modeling of reinforced concrete members with discrete and smeared reinforcement.

Two different modeling were made for the same beam. Concrete was defined with solid65. In the first model, link8 bar was used as discrete reinforcement element. In the second model, steel reinforcement was modeled as smeared concrete element, defined according to the volumetric proportions of steel and concrete. Each model was analyzed four times according to four different material models. Based on their analysis, the results of the load-displacement curves were very similar for both discrete and smeared reinforcement.

The differences exhibited at the load greater than the service load when the effects of material modeling led to the difference in the nonlinear behavior and ultimate load capacity. Ibrahim and Mubarak (2009) used ANSYS to predict the ultimate load and maximum deflection at mid-span of continuous concrete beams, which were pre-stressed using external tendons.

This model accounts for the influence of the second-order effects in externally pre-stressed members. The results predicted by the model were in good agreement with experimental data. Padmarajaiah and Ramaswamy (2001) investigated the prestressed concrete with fiber reinforcement.

In the present study, ANSYS version 12.1 is employed for the numerically modeling of the LSRC beam because of its proven useful 3-D reinforced concrete element provided in the element library. In the following sections, beam details used in the experiment will be briefly described, followed by the description of the developed finite element modeling of concrete and steel reinforcement. The crack development of beams will be presented to compare with the experimental results.

II. FINITE ELEMENT MODELLING

The concrete was modeled with solid65, which has eight nodes with three degrees of freedom at each node, i.e., translation in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. A link8 element was used to model the steel reinforcement. This element is also capable of plastic deformation.

Two nodes are required for this element which has three degree of freedom, as in the case of the concrete element. Discrete method was applied in the modelling of the reinforcement and stirrups used in the tested specimen. The two elements were connecting at the adjacent nodes of the concrete solid element, such that the two materials shared the same nodes. By taking advantage of the symmetry of the beam layout, only half of the beam in longitudinal direction has been modeled in the finite element analysis.

2.1 Concrete

For concrete, ANSYS requires an input data for material properties, which are Elastic modulus (*Ec*), ultimate uniaxial compressive strength (*f _ c*), ultimate uniaxial tensile strength (modulus of rupture, *fr*), Poisson's ratio (*v*), shear transfer coefficient (*βt*). The modulus of elasticity of concrete was 32000 MPa which was determined in accordance withAS 1012.17 (1997). Poisson's ratio for concrete was assumed to be 0.2 for all the beams.

The shear transfer coefficient, *βt* , represents the conditions of the crack face. The value of *βt* , ranges from 0 to 1 with 0 representing a smooth crack (complete loss of shear transfer) and 1 representing a rough crack (i.e., no loss of shear transfer) as described in ANSYS. The value of *βt* specified in this study is 0.4. The numerical expressions by Desayi and Krisnan (1964), Eqs. (1) and (2), were used along with Eq. (3) (Gere and Timoshenko 1997) to construct the multilinear isotropic stress-strain curve for concrete in this study.

$$f = \frac{E_c \varepsilon}{1 + (\frac{\varepsilon}{\varepsilon_0})^2} \quad (1)$$

$$\varepsilon_0 = \frac{2 f_c'}{E_c} \quad (2)$$

$$E = \frac{f}{\varepsilon} \quad (3)$$

f =stress at any strain *ε*
ε=strain at stress f
εo =strain at the ultimate compressive strength *f' c*

The concrete used was grade 40, having the compressive strength of 43.3MPa at 28 days. The strength value of AAC blocks used in the model was 3.5MPa. The compressive stress at 0.3 of the compressive strength was used as the first point of the multi-linear stress-strain curve. The crushing capability of the concrete was turned off to avoid any premature failure (Barbosa and Riberio 1998).

2.2 Steel reinforcement

All beams were provided with top and bottom longitudinal bars, N20 bars were used as the bottom steel in all beams with tensile strength at yield was 560MPa while the yield strength of R-bars which was used as the top bar and the stirrup was 300MPa. The steel for the finite element modelswas assumed to be an elastic-perfectly plastic material and identical in tension and compression. Poisson ratio of 0.3 was used for the steel. Elastic modulus, Es =200,000MPa

III. LOAD DEFLECTION RELATION OF BEAMS FAIL IN FLEXURE

The load deflection characteristics from the Finite Element Analysis (SB1F, LB1F, LB2F) are plotted to compare with the flexural test results in Figure 2. All results show similar trend of the linear and nonlinear behaviour of the beam. In the linear range, the load-deflection relation from the finite element analysis agrees well with the experimental results.

IV. CRACK PROPAGATION OF SOLID AND LSRC BEAMS

During the experiment, The specimens was carefully observed for crack and its propagation. Figures show the crack pattern obtained at failure for beams SB1F, LB1F and LB2F. The experimental results are compared with the crack pattern obtained fromANSYS. In this figure, small dash lines indicates the crack location at the certain load level

4.1 Control beam (SB1F)

In the control beam which failed in flexure, the crack started to occur underneath the loading point at 32.9 kN load level. This flexural crack expanded as the load level increased. Figure 3 shows the crack propagation until load level 89.9 kN. However, the crushing capability of ANSYS was turned off, so the crushing related crack at the top of the beam could not be observed.

4.2 Beam with maximum amount of AAC blocks (LB1F)

The crack pattern of the beam contains maximum amount of AAC blocks is illustrated in Figure 4. Beam LB1F has eight AAC blocks placed within the beam which was the maximum possible amount of blocks based on the gap size between each blocks to ensure smooth concrete flow without any restriction during pouring.

The flexural cracks started to occur at 32.2 kN. Figure 2 shows the crack pattern up to 76.8 kN load level. It is clear that the ANSYS model for LB1F shows more cracks compared to the SB1S. The crack of AAC blocks is noticeable in this model which related to the brittle failure in the actual beam.

4.3 Beam with half amount of AAC blocks (LB2F)

This beam contains half amount of AAC blocks compared to LBF1. In this case, the flexural crack started to appear at the load level of 32.9 kN. The increasing load caused the crack propagation in the beam. Figure 5 shows the crack pattern of this beam up to 78.6 kN.

The only different is, the LSRC beams have more cracks compared to the equivalent solid beam due to the crack which also appear in the AAC blocks. The noticeable cracks of the AAC blocks in ANSYS model correlated to the brittle failure in the LSRC beams. The crushing related crack at the top of the beam could not be observed because the crushing capability of ANSYS was turned off.

Research significance

The paper presents a novel use of lightweight concrete as infill of a reinforced concrete section. This new developed section can be used as beam or slab, which has advantage due to its lighter weight. The weight reduction leads to several benefits in terms of cost and construction time.

Based on the presented experimental and numerical works, the new proposed lightweight section shows great potentials for industrial use. The weights saving benefits also contribute towards sustainability and buildability design objectives of concrete structure.

V. EXPERIMENTAL INVESTIGATION

LSRC section

In reinforced concrete, the structural properties of the component materials are put to efficient use. The concrete carries compression and the steel reinforcement carries tension. The relationship between stress and strain in a normal concrete cross-section is almost linear at small values of stress. However, at stresses higher than about 40% of the compressive concrete strength, the stress-strain relation becomes increasingly affected by the formation and development of microcracks at the interfaces between the mortar and coarse aggregate (Warner et al., 1998). In determining the flexural capacity under the bending theory, a typical strain, stress and force diagram of a reinforced concrete section is as seen in Figure.

Concrete has low tensile strength, therefore when a concrete member is subjected to flexure, the concrete area under the neutral axis of the cross-section is considered ineffective when it is in tension at ultimate limit states. In creating an LSRC section, prefabricated lightweight (in this case AAC) blocks are used to replace the concrete within this ineffective region. The developed LSRC section can be used for beams or slabs. Typical LSRC beam and slab sections are as shown in Figures, respectively.

5.1 Construction of LSRC members

As per any reinforced concrete members, the construction of LSRC members can be either fully precast, semi-precast, or cast *in-situ*. Lightweight blocks can be technically placed between the lower and upper reinforcements of the section. In a beam member, the encasing shear stirrups can be installed before or after the placement of the blocks.

The lower part of concrete section can be cast with the lower reinforcing steels in which the shear stirrups and lightweight blocks are already put in place. The semi-precast LSRC members can be depicted in Figure. Alternatively, the precast can be done with the portion below the underside of the blocks, which means that the concrete can be cast prior to the placement of the blocks.

If this is the case, side formworks will be required when prepare the upper part of the section for concreting. It is necessary to ensure that the section is monolithic by making sure during casting that the concrete can flow in properly through to the sides of the beam and in the gaps between the lightweight blocks.

5.2 Materials

The concrete used was grade 40, having the compressive strength of 43.3 MPa (6280 psi) at 28 days. Superplasticiser was added to the concrete mix to increase the workability of the concrete to ensure the concrete filled all the gaps for beam specimens with AAC blocks in it. The maximum size of aggregate was 10 mm (0.39 in).

The strength value of AAC blocks used was 3.5 MPa (507 psi). All beams were provided with top and bottom longitudinal bars, N20 bars (dia. 0.78 in) were used as the bottom steel in all beams with tensile strength at yield was 560 MPa (81221 psi) while the yield strength of R-bars which was used as the top bar and the stirrup was 300 MPa (43511 psi).

5.3 Beam specimens

The flexural test was to compare the flexural capacity between the solid and LSRC beams. Three beams were prepared, one solid (SB1F) and two with AAC blocks (LB1F and LB2F). LB1F beam had the maximum number of blocks that could be placed in it, while LB2F has half the amount of that contained in LB1F. In the shear test, two beams were prepared, one solid (SB1S) and one with AAC blocks (LB1S).

As a result, when the tied blocks were placed, there were gaps between the blocks and the stirrups, and the blocks and the longitudinal bars. These gaps were useful in enhancing the grip of the reinforcing bars in the concrete section. Figure shows a typical LSRC beam with AAC blocks infill.

5.4 Test set-up

Three beams were designed to fail in flexure, and two beams to fail in shear. The beams were simply supported and were subjected to two point loads. The distance between the two point loads was 800 mm (2.62 ft) and 1680 mm (5.51 ft) in the flexure and shear tests respectively.

The typical test set up is as shown in Figure. The beams were loaded to failure using a 20 tonne (4.4 kips) capacity hydraulic jack to apply each of the two point loads. The jacks were attached to a reaction frame. Two supporting frames with 200 mm (7.87 in) long × 150 mm (5.91 in) diameter steel rollers were used as the end support.

To ensure a uniform dispersion of force during loading and to eliminate any torsion effects on the beam due to slight irregularities in the dimension of the beams, plaster of paris (POP) and 100 mm (3.94 in) wide × 250 mm (9.84 in) long × 20 mm (0.79 in) thick distribution plates were placed on the rollers and also under the jacks.

Instrumentation

The vertical deflections of the test beams were measured using Linear Variable Differential Transformers (LVDTs) which were placed at 200 mm spacing within 2.8 m span. LVDTs were also attached on each loading jack to capture the vertical deflection at the loading point.

The LVDTs were attached to a truss frame as seen in Figure. With this arrangement, the curvature of the beam can be identified in relation to the loading increment. During the initial set up of the LVDTs, the instruments were calibrated before the test commenced. An automated data acquisition system with a Nicolet data logger system was used to record the load-deformation from the jacks and the LVDTs.

VI. INTRODUCTION TO FINITE ELEMENT MODELING

Engineering analysis of mechanical systems have been addressed by deriving differential equations relating the variables of through basic physical principles such as equilibrium, conservation of energy, conservation of mass, the laws of thermodynamics, Maxwell's equations and Newton's laws of motion. However, once formulated, solving the resulting mathematical models is often impossible, especially when the resulting models are nonlinear partial differential equations.

The response of each element is expressed in terms of a finite number of degrees of freedom characterized as the value of an unknown function, or functions, at a set of nodal points. The response of the mathematical model is then considered to be approximated by that of the discrete model obtained by connecting or assembling the collection of all elements.

The disconnection-assembly concept occurs naturally when examining many artificial and natural systems. For example, it is easy to visualize an engine, bridge, building, airplane, or skeleton as fabricated from simpler components. Unlike finite difference models, finite elements do not overlap in space.

Objectives of FEM in this Course

- Understand the fundamental ideas of the FEM
- Know the behavior and usage of each type of elements covered in this course
- Be able to prepare a suitable FE model for structural mechanical analysis problems
- Can interpret and evaluate the quality of the results (know the physics of the problems)
- Be aware of the limitations of the FEM (don't misuse the FEM - a numerical tool)

Finite Element Analysis

A typical finite element analysis on a software system requires the following information:

- Nodal point spatial locations (geometry)
- Elements connecting the nodal points
- Mass properties
- Boundary conditions or restraints
- Loading or forcing function details
- Analysis options

Because FEM is a discretization method, the number of degrees of freedom of a FEM model is necessarily finite. They are collected in a column vector called u. This vector is generally called the DOF vector or state vector. The term nodal displacement vector for u is reserved to mechanical applications.

FEM Solution Process
Procedures

- Divide structure into pieces (elements with nodes) (discretization/meshing)
- Connect (assemble) the elements at the nodes to form an approximate system of equations for the whole structure (forming element matrices)
- Solve the system of equations involving unknown quantities at the nodes (e.g., displacements)
- Calculate desired quantities (e.g., strains and stresses) at selected elements

Basic Theory

The way finite element analysis obtains the temperatures, stresses, flows, or other desired unknown parameters in the finite element model are by minimizing an energy functional. An energy functional consists of all the energies associated with the particular finite element model. Based on the law of conservation of energy, the finite element energy functional must equal zero.

The finite element method obtains the correct solution for any finite element model by minimizing the energy functional. The minimum of the functional is found by setting the derivative of the functional with respect to the unknown grid point potential for zero. Thus, the basic equation for finite element

$$\frac{\partial F}{\partial p} = 0$$

analysis is
where F is the energy functional and p is the unknown grid point potential (In mechanics, the potential is displacement.) to be calculated. This is based on the principle of virtual work, which states that if a particle is under equilibrium, under a set of a system of forces, then for any displacement, the virtual work is zero. Each finite element will have its own unique energy functional.

As an example, in stress analysis, the governing equations for a continuous rigid body can be obtained by minimizing the total potential energy of the system. The total potential energy P can be expressed as:

$$\Pi = \frac{1}{2}\int_{\Omega} \sigma^T \varepsilon dV - \int_{\Omega} d^T b dV - \int_{\Gamma} d^T q dS$$

where and are the vectors of the stress and strain components at any point, respectively, **d** is the vector of displacement at any point, **b** is the vector of body force components per unit volume, and **q** is the vector of applied surface traction components at any surface point.

The volume and surface integrals are defined over the entire region of the structure W and that part of its boundary subject to load G. The first term on the right hand side of this equation represents the internal strain energy and the second and third terms are, respectively, the potential energy contributions of the body force loads and distributed surface loads.

In the finite element displacement method, the displacement is assumed to have unknown values only at the nodal points, so that the variation within the element is described in terms of the nodal values by means of interpolation functions. Thus, within any one element, **d** = **N u** where **N** is the matrix of interpolation functions termed shape functions and **u** is the vector of *unknown nodal displacements*. (**u** is equivalent to **p** in the basic equation for finite element analysis.) The strains within the element can be expressed in terms of the element nodal displacements as **e** = **B u** where **B** is the *strain displacement matrix*. Finally, the stresses may be related to the strains by use of an elasticity matrix (e.g., Young's modulus) as **s** = **E**

VII. EXPERIMENTAL RESULTS

The failure loads of the solid and LSRC beams under the flexure test were found to be of insignificantly different. It was found that beam LB1F, which had the maximum number of AAC blocks, failed at an average load of 78.9 kN (17731 lbs), LB2F and SB1F beams failed at 78.6 kN (17664 lbs) and 78.5 kN (17641 lbs), respectively.

These load values were taken from the average of the loads applied from the two hydraulic jacks.When a beam is more critical in shear, rather than in flexure, an LSRC beam is expected to exhibit lower shear resistance than the equivalent solid beam. This is because the inserted AAC blocks in an LSRC beam have lower compressive strength than the normal concrete.

As a result, an LSRC beam has less effective concrete area to resist the shear when compared to the solid beam of identical height. Based on the two beam tests, the failure loads of SB1S and LB1S were 128 kN (28766 lbs) and 102 kN (22923 lbs), respectively. A significant 20% reduction in the shear capacity of LSRC beam compared to the equivalent solid beam.

The load-deformation behaviour of all the tested beams was found to be similar and followed the same trend. The loads versus deflections at the mid-span of all the beams under flexure and shear are plotted in Figure.

Under the flexural test, the main flexure cracks were developed within the two loading points and widen up as load increased. At failure, the concrete in the compression region crushed. It was seen that the exposed reinforcing steel in this region buckled. The typical crack formations at failure under the flexural test of solid and LSRC beams are as shown in Figures, respectively.

For beams tested in shear, the behaviors of the two tested beams were similar. Small flexure cracks occurred first at the midspan region of the beam. Subsequently, the flexure cracks extended as flexure-shear cracks were developed between the support and the loading point. At the load approaching the failure load, critical web shears crack were developed diagonally within the shear span. The cracks continued to widen as the load increased, and failure occurred soon after depicting a typical sudden type of shear failure.

The typical progressions of the cracks and the failure modes of the beam tested in shear are shown in Figure 8. After the test, it was of concern to determine whether the inclination of the critical shear crack was influenced by the position of the AAC blocks within the crack region.

After the beam failed, the beam was cut using concrete saw to examine the actual position of the blocks. It was found that the cracks propagated right through the blocks as if the section was monolithic. This behavior indicates good bonding between the concrete and the blocks.

Correlation of test results with design prediction

The test results on the failure loads of the beams are compared with the predicted values obtained from design equations based on Australian standard for concrete design (AS3600-2009). In the calculation, rectangular stress block concept was adopted in which a uniform stress of magnitude $0.85f'c$ was used to replace the nonlinear stress distribution above the neutral axis.

A single parameter γ was used to define both the magnitude and the location of the compressive force in concrete. Based on AS 3600 (2009), the value γ for normal concrete with $f'c$ up to 50 MPa (7252 psi), is $\gamma = 1.05 - 0.007(f'c)$, $(0.65 \leq \gamma \leq 0.85)$.

The predicted flexural capacity was calculated from the solid beam section, which was equal to 82.7kNm (18585 lbs). Based on the test results of the maximum load at failure, the moment of the tested beams was 78.5 (17641), 78.6 (17664) and 78.9 (17731) kNm (psi) for solid, LB2F and LB1F, respectively. These results show good correlation with the ultimate design moment value, having only 5% difference. Based on these results, the concrete replacement by AAC blocks, as tested on LB1F and LB2F, seems to virtually have no effect on the flexural strength of the section, which is as expected.
The predicted shear capacity obtained from the design calculation based on AS3600 (2009) also shows good correlation with the LSRC beams. The design value of the shear capacity appears to be conservative for the solid beam. The test/predicted shear capacity ratios for the solid and LSRC beams were 1.27 and 1.01, respectively. Therefore, it seems that design adjustment needs to be made should the designer wish to maintain the same level of conservativeness in predicting the shear capacity of an LSRC beam, as that of an equivalent solid beam.

Numerical investigation

ANSYS 12.1 (2010) was employed to simulate the flexural and shear behaviour of the beam by finite element method. The concrete was modelled with solid65, which has eight nodes with three degrees of freedom at each node, that is, translation in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing.

A link8 element was used to model the steel reinforcement. This element is also capable of plastic deformation. Two nodes are required for this element which has three degree of freedom, as in the case of the concrete element. Discrete method was applied in the modelling of the reinforcement and stirrups used in the tested specimen.

The two elements were connecting at the adjacent nodes of the concrete solid element, such that the two materials shared the same nodes. By taking advantage of the symmetry of the beam layout, only half of the beam in longitudinal direction has been modelled in the finite element analysis.

Concrete

ANSYS requires an input data for material properties concrete in terms of Elastic modulus (Ec), ultimate uniaxial compressive strength (fc'), ultimate uniaxial tensile strength (modulus of rupture, fr), Poisson's ratio (V), and shear transfer coefficient (βt). The modulus of elasticity of concrete used was 26500 MPa (3843.5 ksi) which was determined in accordance with AS 1012.17 (1997). The initial Poisson's ratio for concrete was assumed to be 0.2 for all the beams.

The shear transfer coefficient, βt, represents the conditions of the crack face. The value of βt, ranges from 0 to 1, with 0 representing a smooth crack (complete loss of shear transfer) and 1 representing a rough crack (that is, no loss of shear transfer) as described in ANSYS. The value of βt specified in this study is 0.2, which is recommended as the lower limit to avoid having convergence problems (Dahmani et al., 2010).

The numerical expressions by Desayi and Krisnan (1964), Equations 1 and 2, were used along with Equation 3 (Gere and Timoshenko, 1997) to construct the multi-linear isotropic stress-strain curve for concrete in this study.

$$f_c = \frac{E_c \varepsilon}{1 + (\frac{\varepsilon}{\varepsilon_0})^2} \quad (1)$$

$$\varepsilon_0 = \frac{2 f_c'}{E_c} \quad (2)$$

$$E_c = \frac{f_c}{\varepsilon} \quad (3)$$

Where *fc* is the concrete stress at any strain ε, and εo is the strain at the ultimate compressive strength *fc'*. The compressive stress at 0.3 of the compressive strength was used as the first point of the multi-linear stress-strain curve. The crushing capability of the concrete was turned off to avoid any premature failure (Barbosa and Riberio, 1998).

Steel reinforcement

The steel for the finite element models was assumed to be an elastic-perfectly plastic material and identical in tension and compression. Poisson ratio of 0.3 was used for the steel. Elastic modulus, *Es* = 200,000 MPa (29008 ksi).

Comparison of numerical and experimental results

The typical finite element model of the beam and the results at failure are illustrated in Figure. The load deflection characteristics from the finite element analysis (SB1F, LB1F and LB2F) are plotted to compare with the flexural test results in Figure. All results show similar trend of the linear and nonlinear behavior of the beam. In the linear range, the load-deflection relation from the finite element analysis agrees well with the experimental results when the applied load is below 40kN (8989 lbs).

After the first cracking, the finite element model shows strength of AAC infill material. The comparison of greater stiffness than the tested beam. The final load for the model is also greater than the ultimate load of the actual beam by 16%. Based on these results, the concrete replacement by AAC blocks, as tested on LB1F and LB2F, has virtually no effect on the flexural strength of the section, which is as expected under the shear (SB1S and LB1S),

There are several factors that may cause the greater stiffness in the finite element models. Microcracks produced by drying shrinkage and handling are present in the concrete to some degree. These would reduce the stiffness of the actual beams; however, the finite element models do not include micro cracks during the analysis.

Perfect bond between the concrete and reinforcing steel elements was assumed in the finite element analysis but the assumption would not be true for the actual beams. As bond slip occurs, the composite action between the concrete and steel reinforcing is lost. Thus, as also pointed out by (Kachlakev et al., 2001), the overall stiffness of the actual beams could be lower than what the finite element models would predict, due to the factors that have not been incorporated into the models.

VIII. CONCLUSION

The experimental results of the flexural and shear tests of solid beams and the developed numerical model of LSRC beams are presented. Crack propagation of the beams are closely monitored and the experimental results are compared with the results from FEM analysis. Based on the results, the crack propagation from ANSYS model compares well with the results from the experiment. ANSYS could predict the similar behaviour of crack propagation in each beam specimen. The crack in AAC block correlated to the brittle failure of the sandwich beams. The benefit of this investigation is that the developed FEM model can be used to analyse similar beam sections with different structural configurations and loading parameters to gain more insights of the behaviour of LSRC members.

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