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Assessment of Construction Errors in Reinforced Concrete Beams

Afunanya, J.E.
Department of Building, University of Jos Nigeria
jonasafunanya@yahoo.com
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ABSTRACT

This research investigated using the finite element method the effects of construction errors in reinforced concrete beams. This was borne out of the need to investigate one of the many factors responsible for the failure of structures. The variable is the arrangement of tensile steel reinforcements and transverse shear links. Beam models in each series with the original design intent served as the control and are compared with two others each with a typical construction error in either tensile steel reinforcement or transverse shear links. Beam models are designed in accordance to Eurocode2 to fail either in shear or flexure. Analysis and discussion of results based on the ultimate loads sustained and load-displacement relationship are made. Results from simulated beam models indicate that a reduction in the effective depth of a beam model from 112mm to 105mm in the flexural series led to a 13.8% decrease in strength. In the shear series, the incorrect spacing of the entire transverse shear link at 150mmc/c against 130mmc/c used in the control model led to a 23% increase in strength. It is suggested that a high level of quality assurance be maintained during the construction process to limit the occurrence and effects of construction errors.

Keyword: Beam models, construction errors, failure, finite element analysis, human error, tensile steel reinforcement, transverse shear links.

1. Introduction

Human errors are predominant and are one of the many factors responsible for the failure of structures (Kaminetzky, 1991; Stewart, 1993; Rosowsky and Stewart, 1996; Atkinson, 1997; Toutanji et al., 2009; Love et al., 2013; Hong and He, 2015). Human errors arise in the form of lapses and shortcomings that remain unnoticed during the design and construction of structures. Several authors (Love et al., 2012; Skalle et al., 2013) have classified human error as active i.e. caused by frontline operatives that provoke immediate events and latent or hidden, caused by managers and designers that often trigger the active errors. Love et al. (2012) notes further that organisational and project issues have remained a key component of latent error in the project environment. Construction errors sterns from such active and latent errors that remained unnoticed during the construction process and can be made by anyone irrespective of education, competency, skills and experience and recorded failure cases has shown its universality. Even the most competent and qualified professionals are guilty of committing errors during the design and construction of structures (Love et al., 2011).

Kaminetzky (1991) and Busby (2001) have provided definitions for an error whilst Rosowsky and Stewart (1996) defined construction error. However, for the purpose of this study; construction error is viewed as any deviation in the implementation of the original design intent due to human error, on the assumption that the design is in full compliance to acceptable code provisions and standards. Construction error though similar is distinct from design error in the sense that they are human errors consciously or unconsciously made

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during the construction process. Design errors are peculiar to the designer and design documents, and are deviation(s) from acceptable code provisions and standards due to human errors during the design phase whilst construction errors are most often related to the constructor or site operatives during physical construction.

(Carper and Feld, 1997; Love et al., 2011; Hong and He, 2015) have postulated varying rate of failure of structures due to construction errors. Hansson (2011) noted that 25% of failures are due to construction errors whilst Wardhana and Hadipriono (2003) and Kletz (1991) in Saha et al. (1999) reported that 14% and 90% of total failures respectively are as a result of construction errors. In addition, Wardhana and Hadipriono (2003) reported an increasing trend of number of recorded failures and Hong and He (2015) noted that failure of reinforced concrete constructions are on the increase. This, however, can be attributed to the increasing use of the material in construction. Similarly, Stewart (1993) noted that the likelihood of human errors in the construction of in-situ reinforced concrete members is high and Hansson (2011) reported that a greater proportion of reinforced concrete beams in particular fail compared to other structural members. This justifies why this study centred on reinforced concrete beams.

Though several researches (Allen, 1983; Ortega, 2000; Busby, 2001; Love et al., 2011,2012; Hansson, 2011; Skalle et al., 2014; Hong and He, 2015) has been conducted on failures due to construction errors, its causes, effects and mitigation strategies, such events still continue to occur. This can primarily be attributed to the fact that the magnitude, complexity and inconsistent interface typical of most structures make the detection of errors extremely complex and difficult. Busby (2001) notes that in the construction phase, tasks are structured and prescribed in such a manner that site operatives depend entirely on organisational mechanisms for information and if at any point this information mechanism fails then errors are bound to arise. Similarly, Hong and He (2015) noted that construction errors are difficult and complex to quantify as failures resulting from it are not explicitly considered in design codes calibration. These amongst other reasons have meant that construction errors are still prevalent and according to (Allen, 1983; Yates and Lockley, 2002) construction errors are a reality of life and completely inevitable because of man’s innate nature, however, by systematic measures of checking, inspection and effective communication during the construction process such can certainly be minimised.

Previous research on human errors in the failure of structures have basically relied on interviews (Love et al., 2013), questionnaires (Yates and Lockley, 2002), experimental tests (Saha et al., 1999; Khalilimard, 2014), human reliability analysis (Rosowsky and Stewart, 1996), finite element analysis (Hong and He, 2015). A lot of which have centred on actual failure cases e.g. (Moncarz and Taylor, 2000; Carson and Holmes, 2003). However, in this study it is thought to model and simulate typical construction errors in every day construction works. This research thus investigates using the finite element method, the effect(s) and extent to which construction errors in reinforced concrete beams affect its strength. The variable investigated is the arrangement of tensile steel reinforcements and transverse shear links. This is achieved by 3D modelling and simulation of beam models using the finite element analysis software ELFEN. The finite element method has proven to be reliable in investigating problems in structural members and (May et al., 2005; Almajed, 2008; Zhao et al., 2012) have successfully used it to simulate the response of reinforced concrete beams.

2. Finite Element Implementation

This section describes the methodology adopted for this work. The modelling stage involves the numerical representation of the physical problem by the input of problem data and control.

2.1 Specimen Details

In order to investigate the effects of construction errors in flexural and shear reinforcement of reinforced concrete beams, beam models measuring (1000×120×150mm) are developed and simulated. Beam models are designed to fail either in shear or flexure in accordance to Eurocode2. Table 1 describes briefly the properties of beam models such as effective depth (d), shear links spacing and the two construction errors investigated with their respective notations, for easy of identification. FCM1 which is the flexural control implies flexural control model one whilst FEM1 whose tensile steel reinforcements are placed at the wrong effective depth (i.e. at 105mm c/c as against 112mm c/c used in the control) implies flexural error model one. SCM1 is the shear control and implies shear control model one whilst SEM1 implies shear error model one and has its entire shear links incorrectly spaced at 150mm c/c as against 130mm c/c used in the control.

Table 1 Model properties and description

Specimen series	Specimen notation	Construction error description	d (mm)	Spacing (mm)
Flexural series	FCM1	Flexural control	112	80
	FEM1	Tensile steel reinforcement placed at the wrong depth	105	80
Shear series	SCM1	Shear control	109	130
	SEM1	Incorrectly spaced shear links	109	150

2.2 Material Properties

Though the choice of material model is limited due to its availability in ELFEN, the selection of appropriate material models for steel reinforcement and concrete were dependent on several factors like the yield criterion, element type and desired material properties. Elastic and plastic material properties are obtained from experimentally determined properties of steel reinforcement and concrete.

2.2.1 Steel Reinforcement

Steel reinforcement is modelled using the von Mises nonlinear isotropic hardening material model. This yield criterion is ideal for isotropic, dense and ductile materials whose failure does not depend on hydrostatic pressure. It requires the specification of uniaxial yield stress as plastic property and input of hardening data in the form shown in Figure 1. This model uses a set of points i.e. the effective plastic strain and uniaxial yield stress in the definition of nonlinear hardening curve and ELFEN utilizes piecewise linear interpolation to obtain the intermediate values (ELFEN Explicit User Manual, 2009). The hardening rule defines the motion of the yield surface during plastic loading. Figure 2 shows the yield surface of the von Mises material model.

Transverse shear links and compressive (top) reinforcements are assigned as single_plate because they were of the same diameter i.e. $\phi 8$ whilst tensile steel reinforcements were assigned as steel_plate with the von Mises nonlinear yield criterion with hardening. It should be noted that the direct tensile test conforming to BS EN ISO 6892-1:2009 were carried out on representative reinforcement samples (3 for each steel reinforcement diameter) to determine elastic and plastic properties as shown in Table 2. Table 3 shows the geometric properties of steel reinforcements used to define the steel reinforcements during modelling.

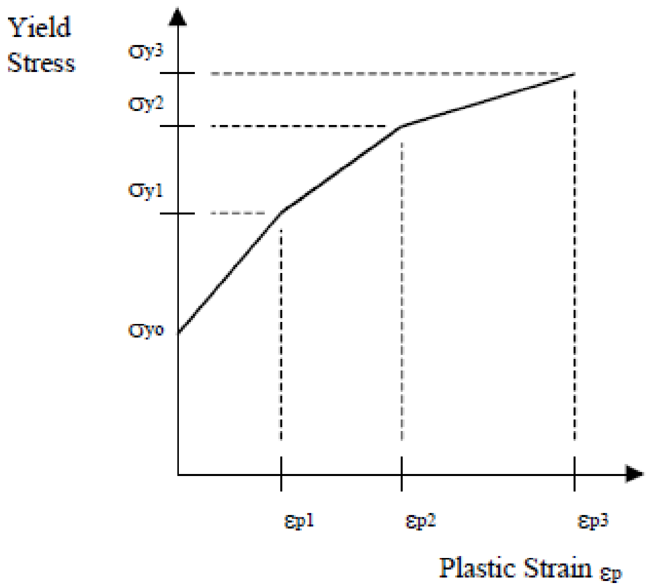

Figure 1: Von Mises nonlinear hardening curve (ELFEN Explicit User Manual, 2009)

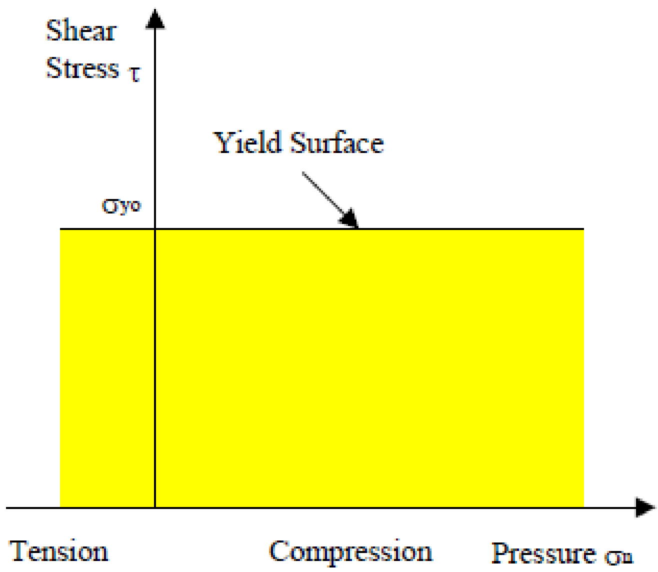

Figure 2: Von Mises yield function (ELFEN Explicit User Manual, 2009)

Table 2: Elastic and plastic properties of steel reinforcement

Type	Diameter (mm)	Elastic modulus (N/m^2)	Poisson's ratio	Density (kg/m^3)	Yield stress (N/m^2)
Single plate	8	2.33E+11	0.29	7800	6.19E+08
Steel plate	10	2.16E+11	0.29	7800	6.04E+08
	16	2.20E+11	0.29	7800	6.27E+08

Table 3: Geometric properties of steel reinforcements

Parameter	ϕ8 (mm)	ϕ10 (mm)	ϕ16 (mm)
Cross-sectional area (m)	5.03E-05	7.85E-05	2.01E-04
First moment of area about YY axis	0	0	0
First moment of area about ZZ axis	0	0	0
Second moment of area about YY axis (m)	2.01E-10	4.91E-10	3.22E-09
Second moment of area about ZZ axis (m)	2.01E-10	4.91E-10	3.22E-09
Product moment of area Ixy	0.00E+00	0.00E+00	0.00E+00
St. Venant torsional constant Kt (m)	4.02E-10	9.82E-10	6.43E-09
Effective shear area Ayy (m)	5.03E-05	7.85E-05	2.01E-04
Effective shear area Azz (m)	5.03E-05	7.85E-15	2.01E-04

2.2.2 Concrete

The standard Mohr Coulomb with nonlinear isotropic hardening i.e. concrete_beam in ELFEN is utilised to model concrete. The Mohr Coulomb material model is a two parameter model and has an added advantage of been able to predict both compressive and tensile failure of brittle materials. It is defined based on the combination of Mohr's failure theory and the Coulomb friction law. The Mohr Coulomb criterion is based on the phenomenon that macroscopic yielding is as a result of frictional sliding between material particles. This yield criterion is implemented with a tension cut-off and indirect softening is as a result of the degradation of cohesion given by Eq.1, which ensures that the normal compressive stress is always on the failure shear plane (ELFEN Explicit User Manual, 2009).

$$\sigma_t \leq c\,(1 - sin\emptyset)/cos\emptyset \qquad \text{Equation 1}$$

The Mohr Coulomb yield surface (Figure 3) has a six-sided conical shape in the principal stress space, and the conical nature of the yield surface reflects the influence of pressure on the yield surface. It requires the specification of cohesion, friction angle, dilatancy angle and the tensile strength of concrete as plastic properties. Hardening is specified by two points using the effective plastic strain, cohesion, friction angle and dilatancy angle.

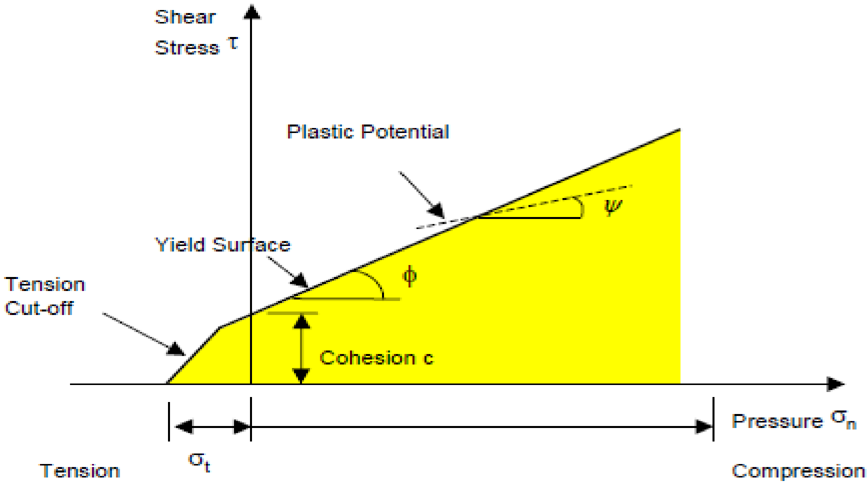

Figure 3: Yield surface of the Mohr Coulomb yield criterion (ELFEN Explicit User Manual, 2009)

2.3 Development of ELFEN Finite Element Model

Upon initiation of modelling mode, tokens are created and nodes inserted in the global coordinates according to the beam dimensions. A complete geometry (Figure 4) is attained by connecting nodes to produce lines, lines to form surfaces and surfaces to form a volume. Each beam model is divided into vertical strips/layers along the beam span taking into consideration the position of steel reinforcements and shear links. The shear links spacing determined the number of layers, as adjacent longitudinal steel reinforcements were connected to each other at adjoining nodes of transverse shear links. The division into strips ensured adequate connection at nodes between the longitudinal steel reinforcement and shear links and also ensured adequate bond with the surrounding concrete elements. Constraints are defined globally and applied to line elements. Structural constraints are defined by restricting vertical and horizontal displacement and vertical displacement at adjacent ends (from left to right) of the beam models respectively as shown in Figure 5. Loading is by applied displacement and was applied symmetrically to the nodes of two strips of surfaces each measuring (0.025 × 0.12mm at 205mm c/c) created on top of the beam models as shown in Figure 6. Ramp load is specified as the load type using a load curve.

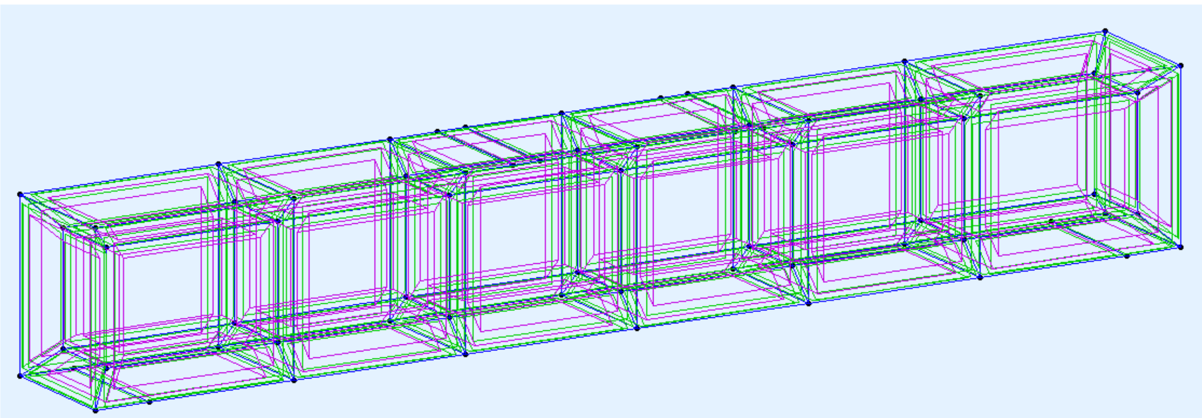

Figure 4: Model geometry

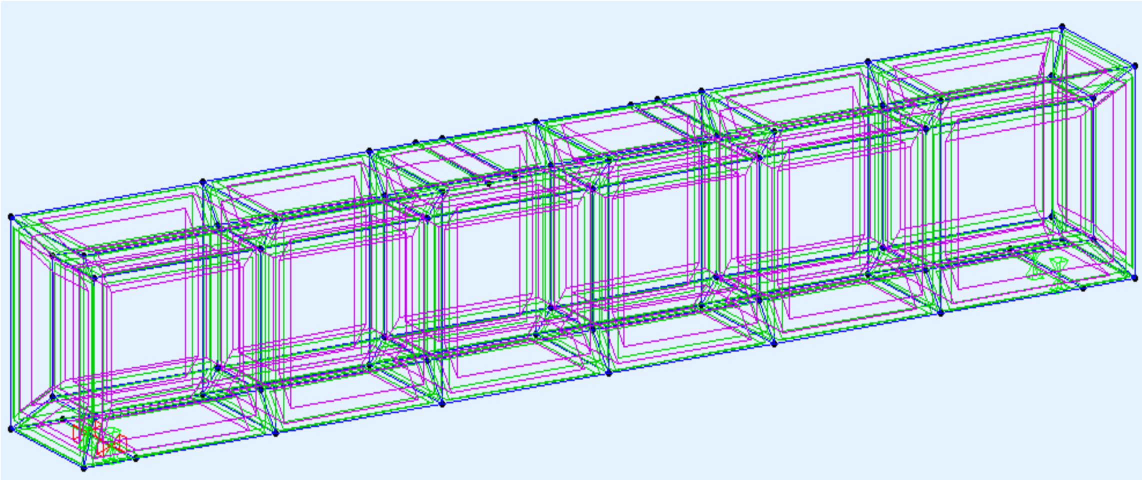

Figure 5: Structural fixities of model

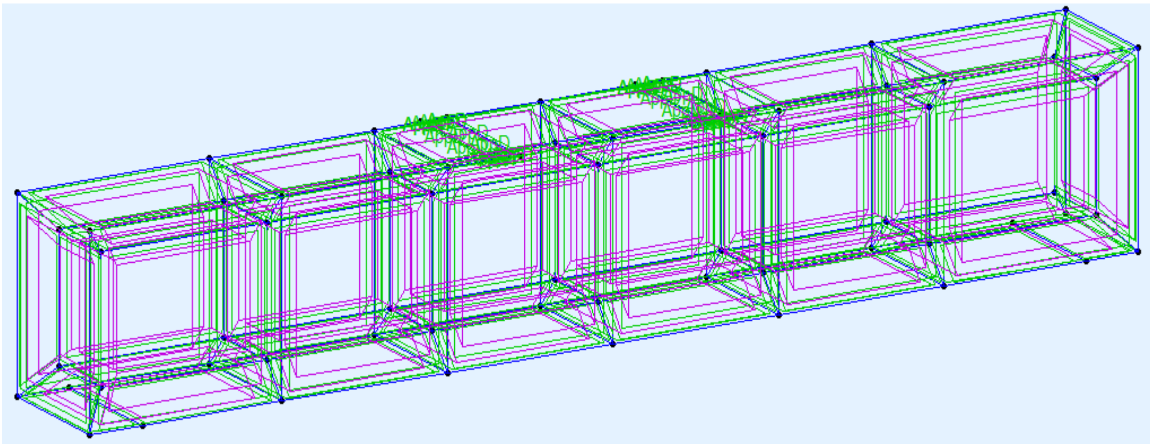

Figure 6: Applied displacement load on beam model

2.4 Finite Element Mesh

The central process in finite element modelling is the transformation of a geometric model into a finite element mesh. 3D unstructured mesh is utilised and for 3D unstructured meshing in ELFEN, only triangular elements are available which gives 4 noded solid tetrahedral element. This element is used to mesh the concrete (Figure 7) with a mesh size of 0.025m. It has three degrees of freedom U, V and W in the global coordinate and six stress/strain components. The 4 noded tetrahedral element is based on the standard isoparametric approach where the same shape functions are used in the interpolation of displacements and geometry. The strain-displacement continuum is determined by utilizing the velocity strain measure from which incremental strains are computed. The Jaumann rate algorithm is used to eliminate the effects of rigid body rotations during stress update. This element uses one point quadrature to integrate element force contributions whilst stabilization terms restrict hourglass deformation (ELFEN Explicit User Manual, 2009).

Steel reinforcement is modelled using the 2 noded 3D Simo beam element (Figure 8) which corresponds to element number 25 in the ELFEN element library. They are 1D pin jointed bars that are assumed to deform axially by stretching and allow only translational displacement U, V, W at the nodes. Axial strain is defined by logarithmic strain measure and rigid body rotations are eliminated during stress update by using a co-rotational algorithm (ELFEN Explicit User Manual, 2009). The mesh size for longitudinal steel reinforcement is 0.025m and was kept constant for all beam models.

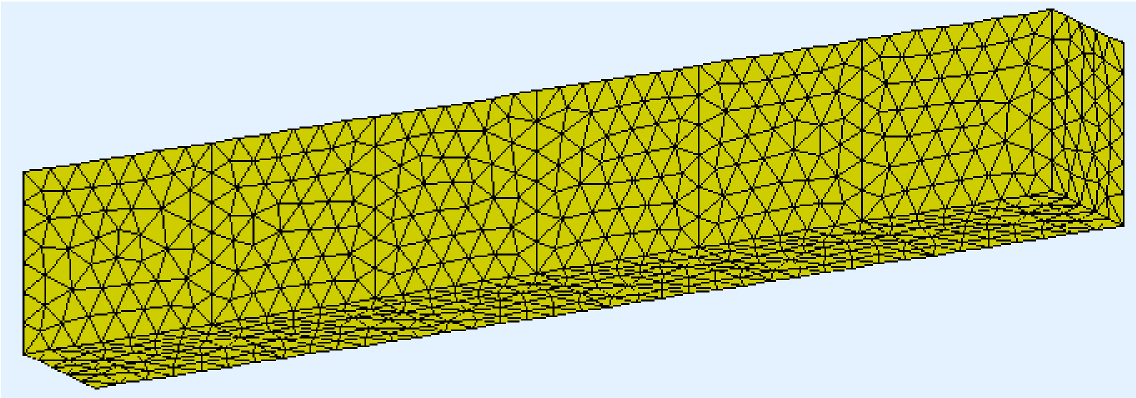

Figure 7: Concrete mesh

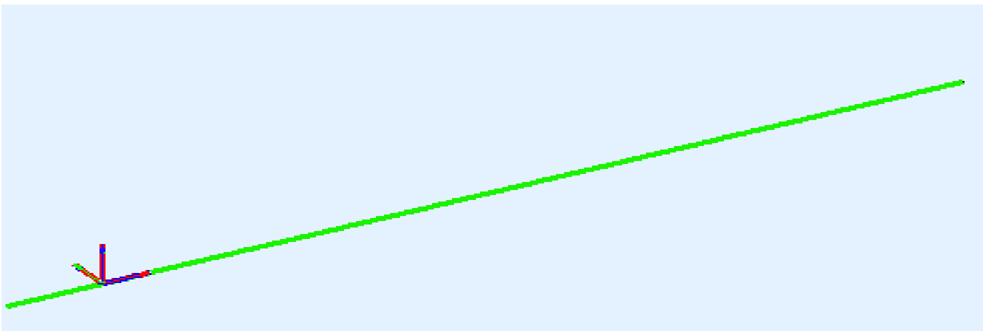

Figure 8: Steel reinforcements mesh

2.5 Analysis and Control

The initial and maximum time step is set as one, the initial time is usually set large as ELFEN will use the automatically computed value in the analysis. The maximum time step change was set at 101, it specifies the largest increment between consecutive time-steps, and can lead to instability in the analysis if it is set large. The factor critical time step is set as 0.9. The analysis is terminated when either the termination time of 1.5s is attained or 1E+08 time steps have been performed. The termination time of 1.5s though conservative is utilized as simulations attain steady state at approximately 1.2s. Appropriate checks on the termination time were conducted by plotting the Y reaction vs. pseudo time.

3. Results and Discussions

Results obtained from the finite element simulation of beam models are summarised in Table 4. Simulated beam models are analysed and discussed based on ultimate load sustained and load displacement relationships. The load-displacement plot is obtained by taking reactions in the Y direction and multiplying the respective applied displacement loading with the pseudo times. Recalling that notations were made earlier to ensure easy identification of beam models e.g. FCM1 implies flexural control model one and FEM1 means flexural error model one. SCM1 implies shear control model one whilst SEM1 implies shear error model one. In the flexural series, simulations with parameters other than those determined by experimental procedures were carried out to validate the developed models and to investigate the effects of such parameters on the beam models i.e. FCM2 and FEM2 whilst all other beam models are simulated with experimentally determined properties.

Table 4: Summary of results

Specimens series	Model notation	Ultimate load, kN	% difference, kN	Applied displacement, mm
Flexural failure	FCM1	80.71	-	8.00
	FCM2	79.18	1.9	15.00
	FEM1	69.60	-	7.00
	FEM2	69.10	0.72	7.00
Shear failure	SCM1	64.56	-	5.00
	SEM1	79.61	-23.3	5.00

3.1 Flexural Series Models

This series consist of two model types; FCM which served as the control and FEM with its tensile steel reinforcement placed at the wrong effective depth i.e. effective depth reduced from 112mm to 105mm.

3.1.1 Flexural Control Models (FCMs)

FCM1 and FCM2 sustained an ultimate load of 80.7kN and 79.2kN respectively, indicating a 1.9% decrease in strength by FCM2. Both models differed only in the magnitude of applied displacement loading of 15mm in FCM1 and 8mm in FCM2. The varying magnitude of applied displacement loading used in this simulation is to determine if the magnitude of applied displacement loading affected significantly the ultimate load sustained by the beam models. The result suggests not. However, it affected significantly the shape of the load – displacement curve. Figure 9 shows the load displacement plot of control models in the flexural series. Both beam models exhibited similar load displacement behaviour with initial response been linear elastic until the yield stress is attained when beam models exhibited nonlinear behaviour until ultimate conditions is attained.

3.1.2 Flexural Error Model One (FEM1 and FEM2)

Of the two beam models simulated; the only difference is in the specification of the tensile strength of concrete i.e. FEM2 has a higher tensile strength of concrete than FEM1. FEM1 sustained an ultimate load of 69.6kN whilst FEM2 sustained an ultimate load of 69.1kN, indicating a 0.72% decrease in strength by FEM2. This implies that the concrete tensile strength did not affect significantly the ultimate load sustained by the beam models. Figure 10 shows the load displacement plot of both beam models. The shape of the load displacement curve in the elastic region is the same for both beam models. However, the effect of concrete tensile strength can be observed from the difference on both curves at the onset of nonlinear behaviour on attainment of the yield stress. This is because the Mohr Coulomb yield surface is defined based on the tensile strength and in uniaxial tension, the tensile strength is defined by the elastic limit.

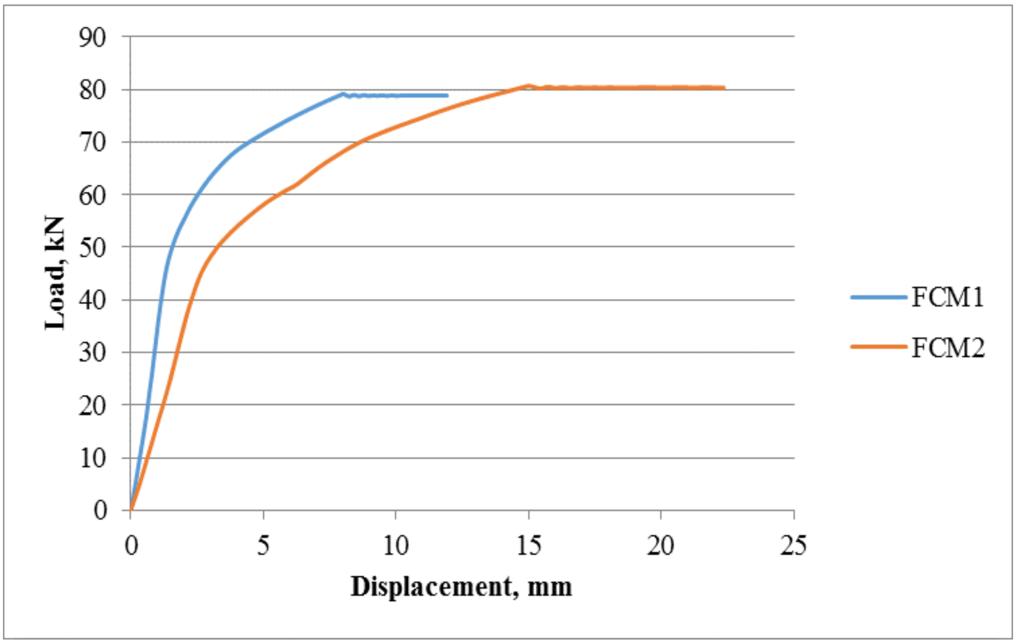

Figure 9: Load displacement plot of the flexural control beam models (FCM1 and FCM2)

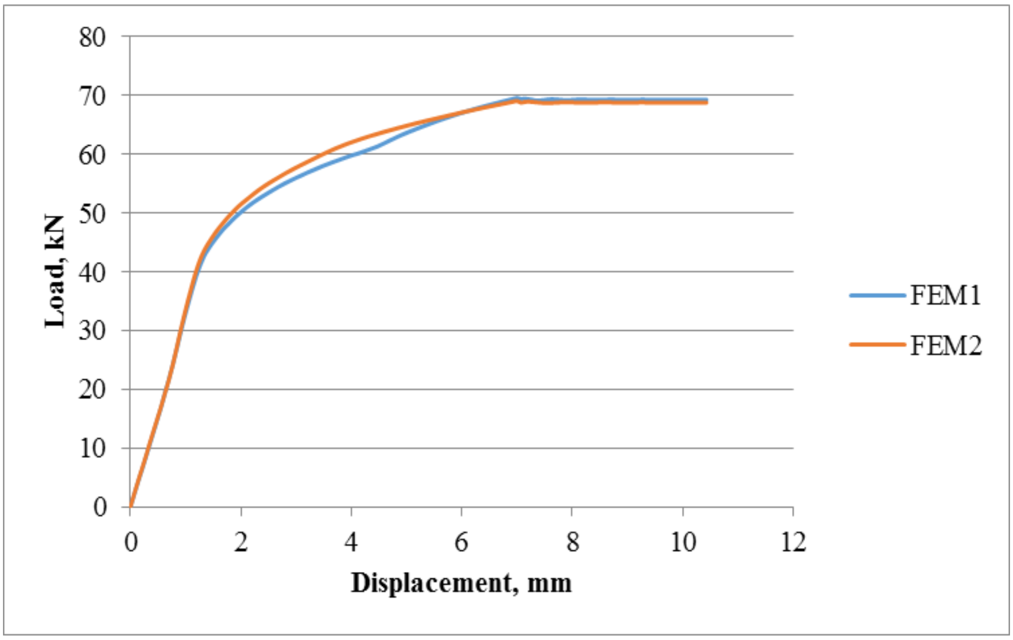

Figure 10: Load displacement plot of the flexural error beam models (FEM1 and FEM2)

3.1.3 Comparison between Flexural Beam Models (FCM1 and FEM1)

For the purpose of this comparison, only FCM1 and FEM1 will be discussed as they both contain the true properties intended for this research. FCM1 and FEM1 failed at an ultimate load of 80.7kN and 69.6kN respectively. This indicates a 13.8% decrease in strength by FEM1. However, both beam models exhibited similar load displacement behaviour as shown in Figure 11. The initial response is linear elastic until the yield stress is attained where nonlinear behaviour initiates. The identical shape of the load displacement curve particularly in the elastic region gives credibility and confidence to the model developed as been able to significantly predict the response of beam models.

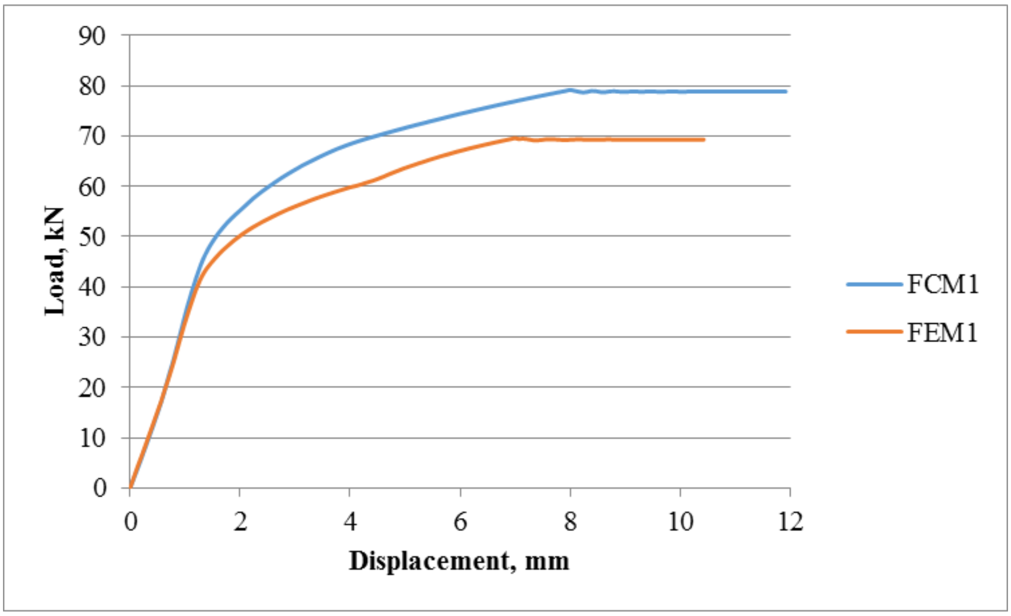

Figure 11: Load displacement plot of FCM1 and FEM1

3.2 Shear Series Beam Models (SCM1 and SEM1)

This series consist of two model types; SCM1 serves as the control with transverse shear links spaced at 130mm c/c whilst SEM1 has its entire shear links spaced incorrectly at 150mmc/c. This implies that one shear link is missing in SEM1. It should be noted that unlike in the flexural series, the effect of other parameters are not investigated here i.e. only actual parameters are used. From Table 4, it can be observed that SCM1 and SEM1 sustained an ultimate load of 64.56kN and 79.61kN respectively at 5mm applied displacement loading. This indicates a 23.3% increase in strength by SEM1. Though this is inconsistent with the pattern of results obtained in the flexural series and seems rather surprising and illogical as both beam models have approximately same properties besides the number of elements i.e. 8863 and 8398 for SCM1 and SEM1 respectively. However, no logical reason(s) can be attributed to this other than the Mohr Coulomb material model has not captured accurately material nonlinearity and as (Chen, 1982; Malm, 2009) postulates, the Mohr Coulomb failure criterion with tension cut- off (type used in this research) cannot predict accurately the results of concrete, though it can be used as a fair approximation. Figure 12 shows the load displacement plots of SCM1 and SEM1. The shape of both curves particularly in the elastic region is same but for a marked difference on the initiation of nonlinear behaviour. The exhibition of identical elastic behaviour by both models gives confidence that the models predict reasonably the response of the beam models.

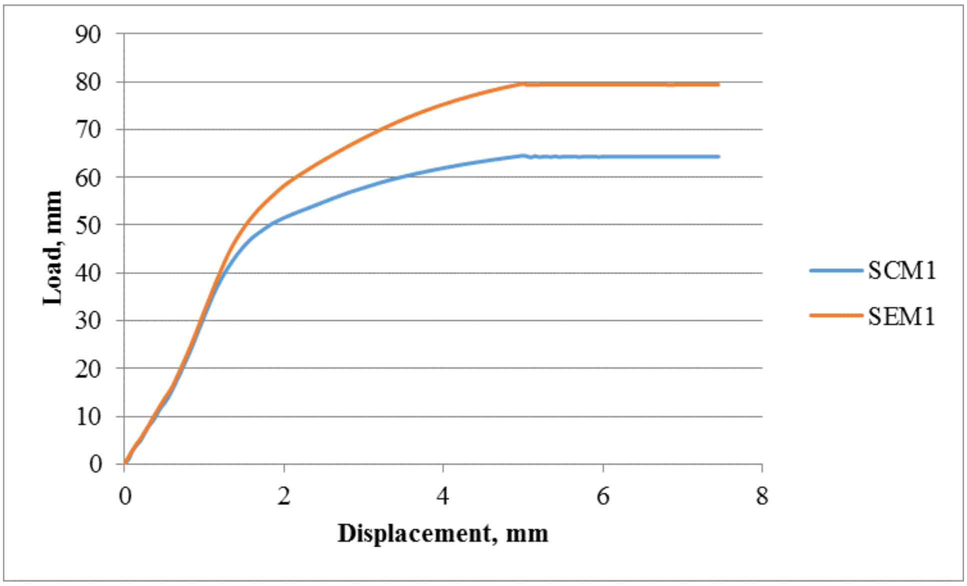

Figure 12: Load displacement plot of the shear series beam model SCM1 and SEM1

4. Conclusion

This research investigated using the finite element method the effects of construction errors in reinforced concrete beams. Results of this research work suggest that construction errors during the construction of reinforced concrete beams, affect significantly the strength of beam models. In addition, the Mohr Coulomb material model with tension cut-off has reasonably predicted the response of simulated beam models particularly in the elastic region. Since human error cannot be explicitly removed from the construction process due to human cognitive shortcomings, it is therefore imperative to ensure a high level of quality assurance during the construction process to limit the occurrence and effects of construction errors. In addition, since construction errors are made by site operatives, it is important that they are made to understand the consequences of their actions or inaction particularly on shortcomings thought to be minor. At the organisational and project levels; processes, practices and actions employed by management should be such that reduces the propensity to provoke construction errors.

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