Effect of Imperfection Factors on Simulation of the Slender RC Columns Strengthened by SFR Using ABAQUS

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Abstract. Due to geometric effects rather than material failure, buckling is the most typical failure mode of slender RC columns. Real columns are never perfect, and their flaws have a big effect on their stability, therefore, to use FEA software to study slender RC columns, requires a number for this imperfection, known as the imperfection factor. By checking experimental fields, codes provide some imperfection factor values to utilize in design and analysis these factors cover all kinds of imperfection that means they will be a flexible value. On the other hand, numerical fields like FEA software manuals dose note give a clear way to specify the imperfection factor according to experimental references especially material imperfection dependency. The results of the analytical model of the slender RC columns are presented in this work. The investigation includes the creation of an analytical model based on the finite element approach using ABAQUS/Standard 2020 software to see how material variation affects the value of the imperfection factor. The analytical model was compared to the experimental work utilizing similar properties to validate the results. All columns were axially loaded. The experimental and finite element results were in good agreement. From the relationship between SFR variation and the imperfection factor, a second-order equation could be suggested.

Keywords: Buckling, Slender RC column, FE analysis, ABAQUS, Imperfection.

1 Introduction

Buckling is a physical phenomenon in which a relatively straight, slender member (or body) bends laterally (typically suddenly) from its longitudinal position due to compression [1]. Buckling, rather than material failure, is a loss of stability caused by geometric effects. However, if the resulting deformations are not controlled, the material can fail and collapse [2]. Real columns are never perfect, and defects in them have a significant impact on their stability. The real columns buckle before the buckling force because of these defects from the perfect shape or material [3]. The material yielding or the column buckling can both cause a column to fail. It is of interest to the engineer to determine when this changeover takes place. In slender columns, buckling is a more common mode of failure. Imperfection refers to the trait or state of being imperfect. These imperfections are divided into geometric imperfection, thickness imperfection, material imperfection, and boundary imperfections [4]. Local and overall (bow, global, or out-of-straightness) geometric imperfections are the two basic categories of initial geometric imperfections. Initial local geometric imperfections can be discovered on the outer or

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inner surfaces of metal structural members in perpendicular directions to the member surfaces. Initial overall geometric imperfections, on the other hand, are global profiles for the entire structural member throughout its length in any direction. An eccentricity of

$$e_j = l_0 /400 \quad (1)$$

where l_0 is the effective length

maybe used as a simplified alternative for walls and isolated columns in braced systems to cover imperfections associated with normal execution variations [5]. Or the eccentricity equal of:

$$e_0 = \alpha m \, L / 500 \quad (2)$$

where L is the span of the bracing system

and $\alpha_m = \sqrt{0,5(1+\frac{1}{m})}$ (3)

in which m is the number of members to be restrained [6]. Equivalent geometric imperfections, with values that reflect the impacts of both global and local imperfections, should be utilized. From the previous equations (1,2) Eurocodes offered a similar range for the imperfection factor in concrete and steel structures. In this study, because of the columns subjected to the concentric load, therefore the eccentricity (e) is equal to the imperfection factor (IMF). Perturbations in geometry are the most common source of imperfections. Abaqus allows you to define an imperfection in three ways: as a linear superposition of buckling eigenmodes, via static analysis displacements, or by directly providing the node number and imperfection values. The perturbations utilized are usually a few percent of a relative structural dimension like a beam cross-section or shell thickness [7]. Nessa Yosef Nezhad Arya in 2015 studied the second-order FE analysis of axial loaded concrete members according to Eurocode 2, the nonlinear analysis focused on concrete material modelling and its nonlinear behaviour, by comparing the FEA results of a benchmark experiment, the finite element model was confirmed [8]. Alaa Hussein Al-Zuhairi & Safaa Qays Abdulrahman in 2020 studied the performance of slender reinforced concrete columns with different cross-sectional shapes, the study aimed to use nonlinear finite element analysis to explain the structural performance of slender SSRC columns both experimentally and numerically [9]. Four samples were selected “first one normal concrete and the other three was strengthened with SFR” from an experimental study where don by Al-Helfi & Allami (the strengthening effect of SFR and CFRP on a part or whole of the of slender RC columns) one was a reference and others were compared [10] to investigate the material variation effect on the imperfection factor using ABAQUS/Standard 2020.

2 Experimental Programs

All columns were axially loaded, with dimensions of 2000 mm in length and 120 mm x 60 mm in cross-section, column top, and bottom ends were supported by load plate working as a hinge, and the major longitudinal reinforcement was symmetrically 2 x 3Ø6. The stirrups were Ø6 @ 50 mm along with 300 mm from the ends of the column, with 150 mm in the remaining length. The first reference column (SC11) tested was not strengthened, whilst the other columns were strengthened using SFR as detailed in Table 1.

Table 1. Summary of Experimental Results.

Column ID	Strengthened Materials	Length of Strengthened	Fcu (Mpa)	EC (Mpa)	Ultimate Load Test Result (KN)
SC11	Non	Non	35	24870	182
SC21	SFR	L	55	31176	261
SC22	SFR	L/2	55	31176	260
SC23	SFR	L/3	55	31176	244

3 Finite Element Analysis

Because laboratory studies require expensive equipment and tools, as well as a specialized laboratory with qualified workers to complete the experiments, they do not cover a large variety of factors. As a result, the finite element method (FEM) is the most suited tool for expanding the range of parameters to be investigated. A three-dimensional nonlinear numerical analysis has been carried out to simulate the imperfection factor effect on RC columns buckling.

3.1 Analysis Procedure

There are two types of research methods in Abaqus: linear and nonlinear. Because the buckling behaviour cannot be obtained, the nonlinear analysis couldn't depict axially loaded slender column real behaviour in this paper, for both static general and dynamic analysis produced by Abaqus software. This development occurred because of two aspects. First, all FEM equations are based on the equilibrium of stresses and strain compatibility, which means that these equations are impossible to solve due to the discontinuous response at the buckling point. Second, up until final collapse, A perfect (ideal) column is used to illustrate the FE model. Instead of bifurcation, Abaqus handles the discontinues problem by displaying a geometric imperfection mode in the perfect (ideal) geometry of the model. In Abaqus, there are numerous approaches for defining an imperfection. One of these techniques is to use the *IMPERFECTION keyword to directly apply the imperfection in the input file. This necessitates data like eigenvalues and buckling modes, which were provided via linear elastic buckling analysis. In a brief, the FE simulation of the provided specimens necessitated the creation of two models for the same mesh: To determine the likelihood of collapse, an initial model for elastic buckling analysis was developed as shown in Fig 1. This model was analysed with linear elastic buckling to get possible buckling mode and Eigenvalue of this mode which represent the critical load. The plastic model then imports imperfection data (buckling mode, Eigenvalue) from the linear analysis to do the nonlinear analysis of slender RC columns [7]. The input file of both models has been altered as shown in Fig 2.

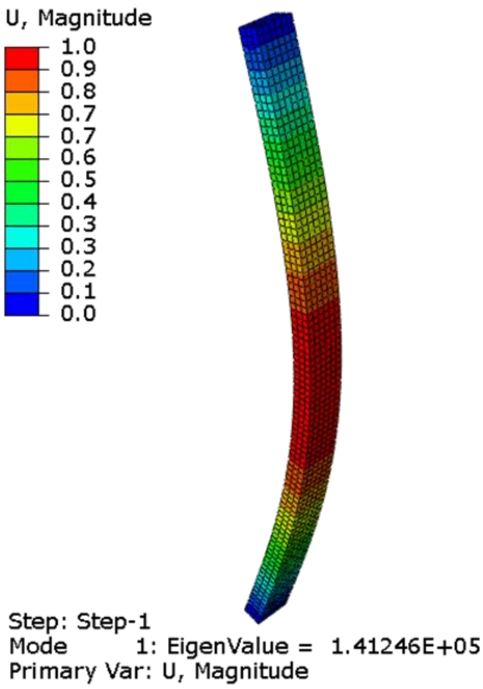


Fig. 1. Linear analysis (buckling mode).

```
**OUTPUT REQUEST
**
*Restart, write, frequency=0
**
**FIELD OUTPUT: F-Output-1
**
*Output, field, variable=PRECELECT
*NODE FILE
U
*End Step
```

(a) Elastic model

```
**---------------------------
--
*IMPERFECTION, FILE=Job-1,
STEP=1
1,4
**
**STEP: Step-1
**
*Step, name= Step-1, nlgeom= YES
*Dynamic, Explicit
,1.
```

(b) Plastic model

Fig. 2. The input file of imperfection factor (highlighted characters).

3.2 Material Properties

Plain concrete's nonlinear behaviour in compression and tension was simulated using the concrete damage plasticity model (CDPM), which took damage characteristics into account. To explain the response of plain concrete with uniaxial and compound stresses, CDPM employs two types of parameters. The reinforcing steel bars were also modelled using Abaqus' plasticity model, which used a bilinear model to characterize the stress–stress relationship of reinforcement. Up to the yielding point, the bilinear model is elastic, and between the yielding point and the start of strain hardening, it is ideal plastic as shown in Fig 3.

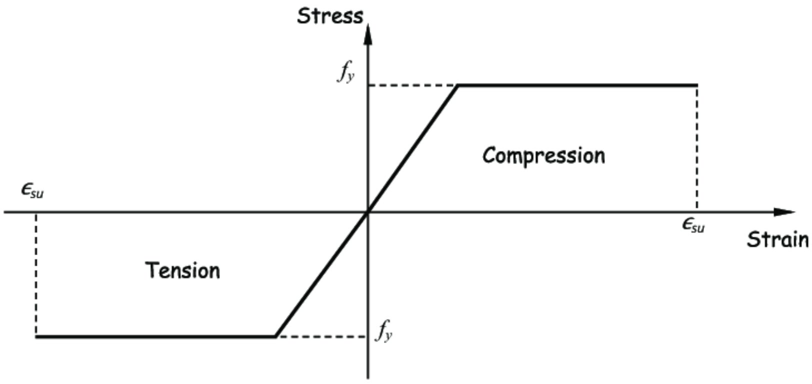
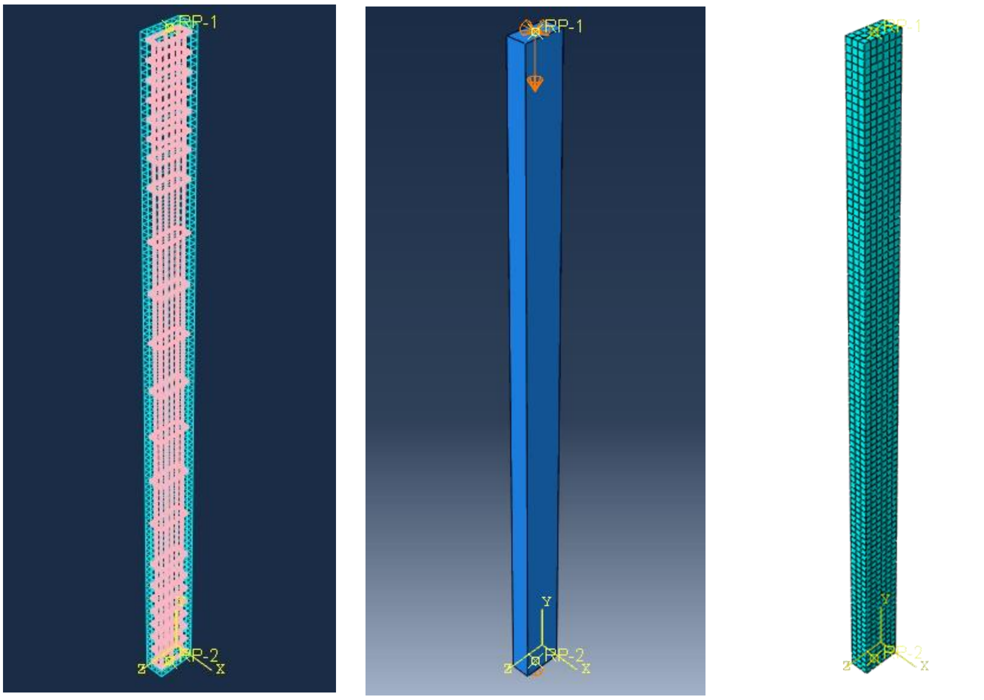


Fig. 3. Stress-strain curve for steel reinforcement.

3.3 FEA model boundary conditions, interaction, and meshing

Both ends of the samples were treated as pinned ends in the nonlinear analysis, identical to the test condition. Reference points (RP) were used to simulate the pin ends conditions. At both ends, the movement was constrained in all axes, except the axial movement at the top end was allowed. Although all rotations in all axes were allowed. To ensure a complete link between the reinforcing bars and the concrete, the interactions were represented as an embedded region. In the linear and nonlinear analyses, a mesh size of 20 mm was chosen for concrete and reinforcement. Concrete was modelled using C3D8 element, 8-node linear brick. The steel reinforcement was modelled using D3T2, a two-node linear three-dimensional truss as shown in Fig 4.



(a) Boundary conditions (b) Interaction (c) Meshing

Fig. 4. Slender RC column modelling.

4 Analysis Results and Discussion

4.1 Imperfection Factor Check

In this section two verification are presented: first, check to see which imperfection factors provided by Eurocodes give satisfying results for the reference model, and the second check by choosing different imperfection factor values in addition to reference model imperfection factor to investigate SFR strengthened effect.

4.1.1 Reference Model

Using the numerical method highlights the fact that the results may not be precise because of material properties and modelling errors. Referring to Eurocode 2 [8] and Eurocode 3 [9], two values will be tested through the simulation respectively to compare with the experimental reference model (normal concrete) results:

$$L = \text{clear length of column} = 2000\text{mm}$$

From equation (1)

Imperfection factor = L / 400 = 2000 / 400 = 5 mm

From equation (2)

Imperfection factor = L / 500 = 2000 / 500 = 4 mm

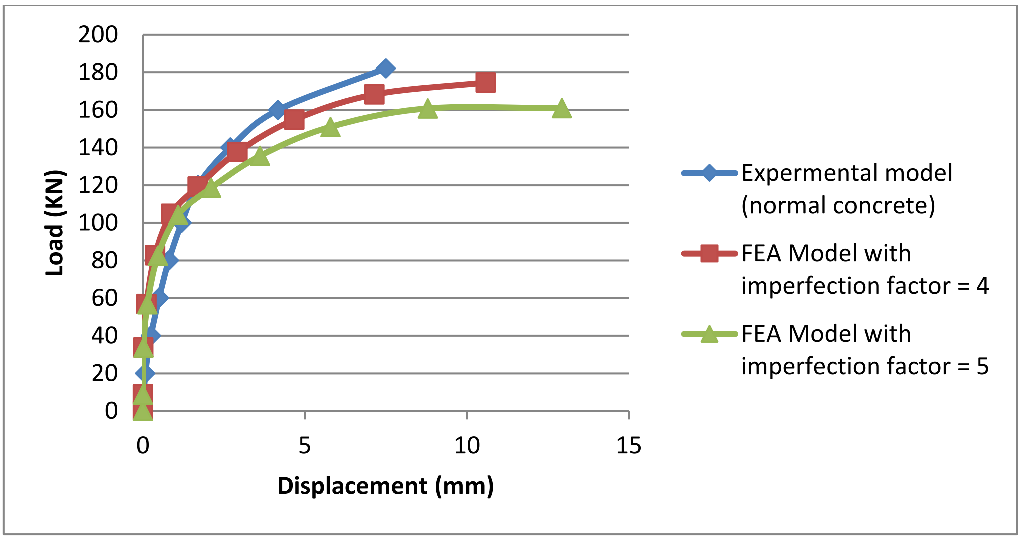
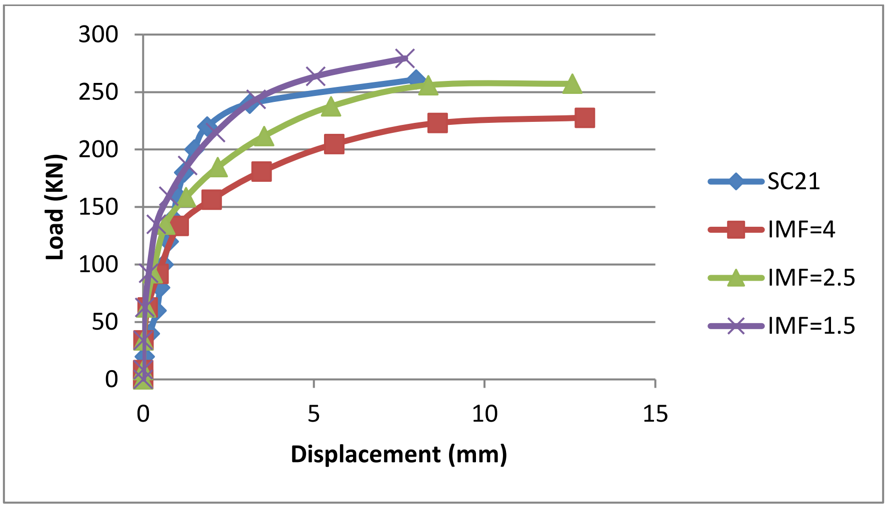


Fig. 5. Comparison between two different imperfection factor values of the load-deflection curve for FEA model and experimental model.

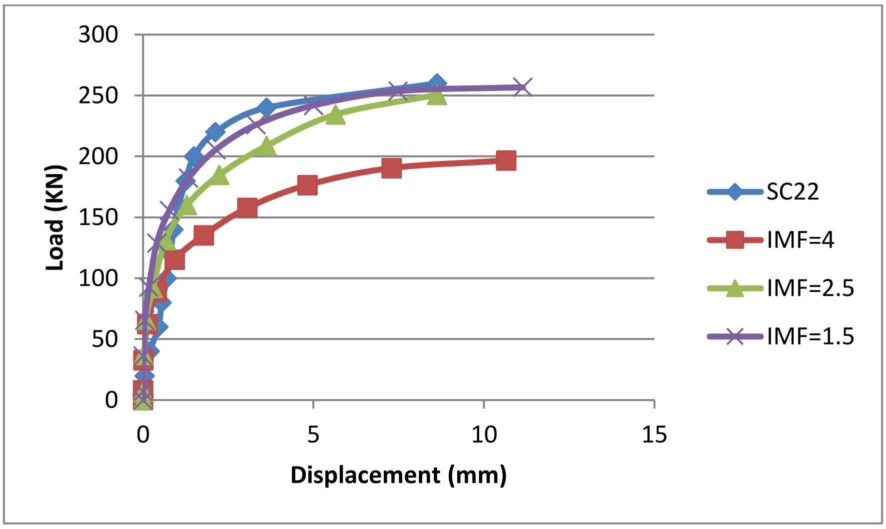
As illustrated in Fig 5. Which shows that the imperfection factor (4) is considered satisfactory.

4.1.2 SFR Strengthened Models

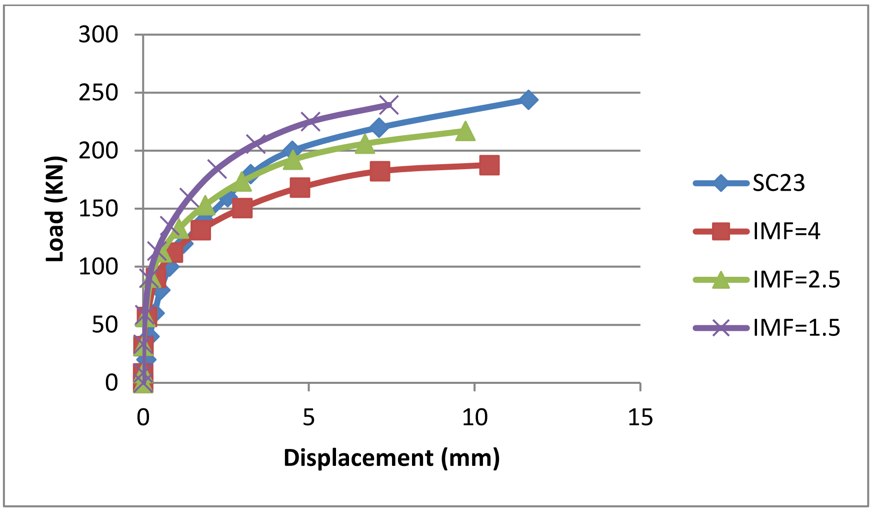
As mentioned previously, there are no imperfection factor values within the specifications, considering the effect of SFR, two values will be tested in addition to reference column imperfection factor value (1.5, 2.5, 4) to see how well the results match the experimental work as shown in Fig 6.



(a) SC21 (All lengths of the column strengthened with SFR).



(b) SC22 (L/2 of the column strengthened with SFR).



(c) SC23 (L/3 of the column strengthened with SFR).

Fig. 6. SFR effect on the imperfection factor for each strengthened column

Fig 6-a shows the comparison between load-deflection curve for each experimental (all length of the column strengthened with SFR) and FE model, FE model tested with three different values of imperfection factors to see which is the nearest to experimental results, also Figure 6-b and 6-c show the comparison between experimental and FE model for each L/2 SFR strengthened column and L/3 SFR strengthened column respectively. Table 2. showing the best values for the imperfection factor that apply to the behaviour of the real columns tested in the laboratory.

Table 2. Columns IMF values.

Column	IMF
SC11	4
SC21	1.5
SC22	1.5
SC23	2.5

4.1.3 IMF-SFR relationship

Fig 7. illustrates the relationship between the imperfection factor and SFR distribution length by a curve showing different values for this factor which gives a clear idea about the material deviation imperfection factor.

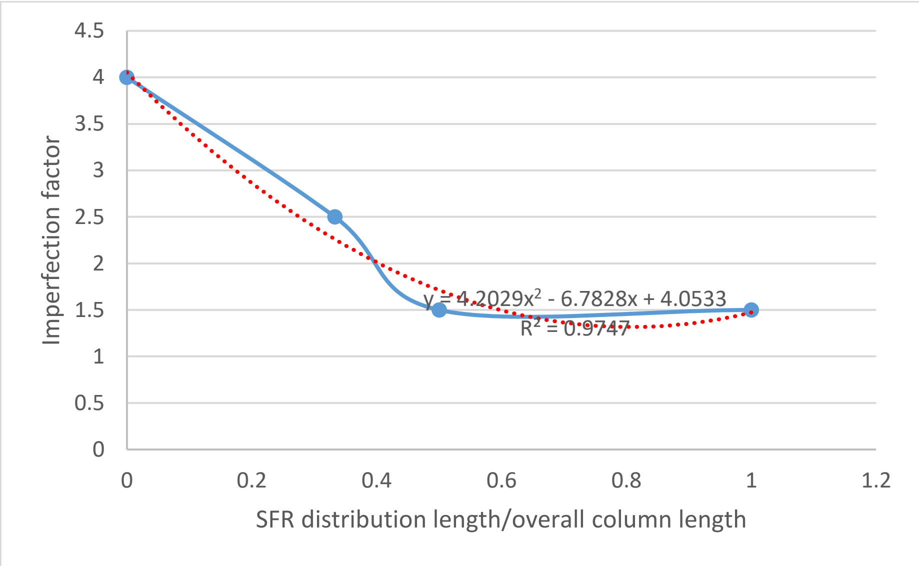


Fig. 7. IMF-SFR relationship.

Although, by getting a graphical relationship it's easy to convert to a mathematical equation that can be used to determine imperfection factor for RC slender column strengthened with SFR of any distribution length.

Since y = IMF (imperfection factor)

x = SFRL (steel fibre reinforcement distribution length)

The suggested equation will be

$$IMF = 4.2029\ SFRL^2 - 6.7828\ SFRL + 4.0533 \quad (4)$$

4.2 Deformation Diagram

Most compressed structural elements are designed using Euler's theory of buckling, or one of Euler's adjustments to account for inelastic behaviour . The practical applicability of these ideas is typically assessed by comparing them to buckling loads acquired using a traditional mechanical or hydraulic testing machine. A half sinewave will buckle a column with hinged ends, the deformation diagrams of slender RC column at failure load are shown in Fig 8.

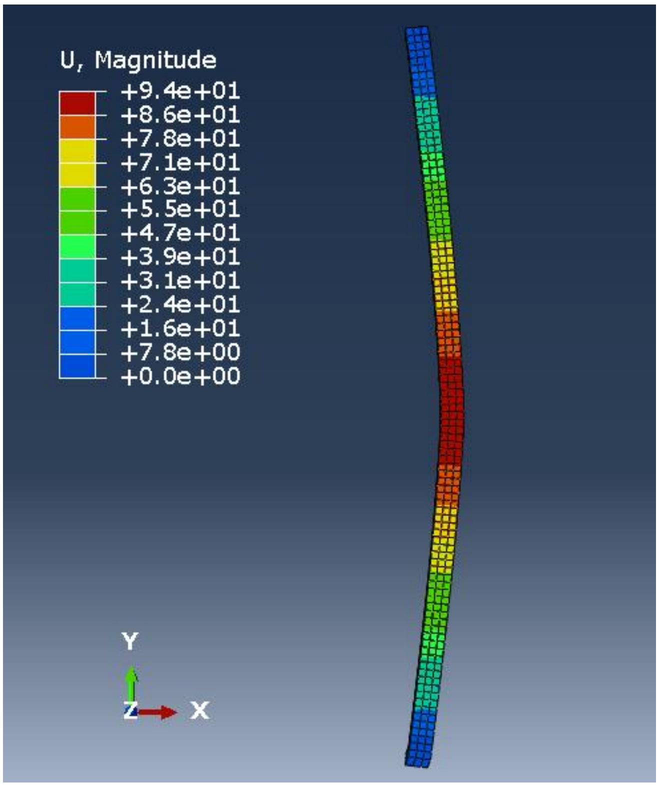
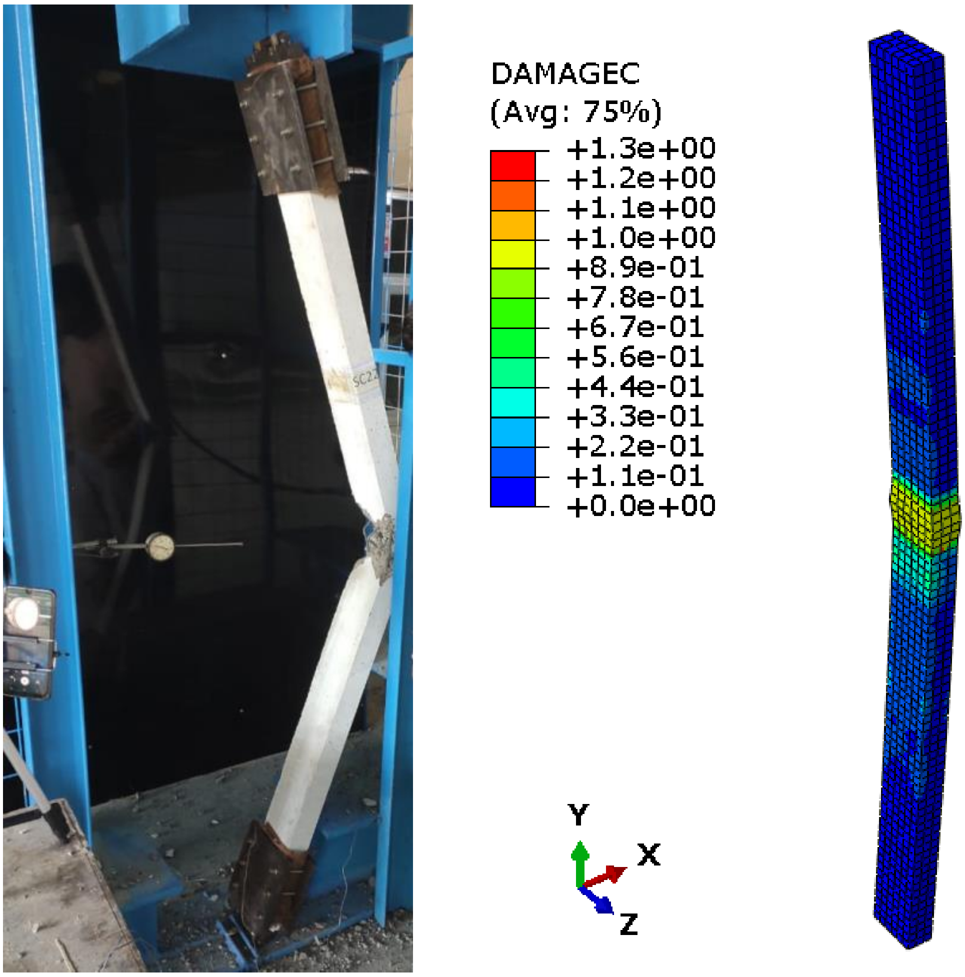


Fig. 8. Slender RC column deformations.

4.3 Failure mode

The inception of cracks is seen at the mid-height of the column as the load increased until failure, Fig 9b shows the failure mode in the slender RC column that has been analysed and which also gave good compatibility with the experimental model shown in Fig 9a.



(a) Experimental failure mode **(b)** Analytical failure mode

Fig. 9. Failure mode.

4.4 Failure Load

The comparison of ultimate load and maximum deflection between finite element and experimental results are shown in Table 3. which shows that the agreement between finite element and experimental results is quite good.

Table 3. Ultimate load and deflection results.

Column ID	Ultimate load (KN)		Maximum deflection (mm)	
	EXP.	FEM	EXP.	FEM
SC11	182	174	7.5	10.5
SC21	261	279	8	7.7

SC22	260	256	8.6	11.1
SC23	244	217	11.6	9.7

The presence of SFR distribution deviation affected the values of the failure load which reflecting the difference of the imperfection factor for every case.

5 Conclusion

Geometrical nonlinearity of slender RC column makes buckling the most common failure mode when it analysed using a suitable FEA software, ABAQUS one of the powerful software for this purpose, Abaqus user manual mentions to the geometric imperfection without considering material perturbation effects on this factor, the nonlinear FEA prosses depends on constant imperfection factor provided to the input data of the FEA models if they have the same geometry and material, but the factor will be variable if the material variation distribution through column length changes from a model to another, this variation could be written as a suggested second-order equation resulted from representing load-deflection curve relationship for all models, the analytical model was compared to the experimental work and the results were in good agreement.

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