Modelling hybrid reactive powder concrete T-beams

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Abstract. The purpose of current study is to evolve a nonlinear finite element model to simulate the influence of the type of concrete on reactive powder concrete (RPC) T-beams. Four simply supported T-beams were modelled and the difference is in the type of concrete (full normal strength concrete (NSC), full RPC, RPC in the web only, RPC in flange only). Commercial ABAQUS software package with the damaged plasticity model (CDP) is proposed for modelling the reinforced concrete (RC) structural members. The compressive stress-strain relationship and uniaxial tensile in addition to the damage parameter of the material were proposed to be effectively utilized in ABAQUS. From the analysis of the finite element model for each beam, the predicted ultimate load capacity against the deformation response was gained and validated with the results of the experimental work.

1. Introduction
One of concrete technology’s newest and most significant evolutions is production of Reactive Powder Concrete (RPC). It has gained tremendous publicity in the last few years for its superior mechanical properties like high strength, high ductility, limited shrinkage, high durability, and high resistance to abrasion and corrosion [1,2]. A report submitted by Richard and Cheryrezy [1] introduced two types of RPC: RPC200 and RPC800 which develop a compressive strength of 200 MPa and 800 MPa respectively. These two types of RPC were manufactured from the same materials that are: cement (800-1000 kg/m³), fine sand with particle size less than 600 µm, silica fume, steel fibres, superplasticizer, and less than 0.2 water to cement ratio with the absence of coarse aggregate. After that, many researchers attempted to study the properties of such concrete and its performance when applied as a structural member. Ridhu et.al. [3] presented an exploration on the shear performance of slender RPC beams with a rectangular cross section with the elimination of web reinforcement. The results of their study referred to that the presence of micro steel fibres with aspect ratio equals to 65 in the RPC did not intensely affect the initial diagonal cracking load whereas it enhanced ductility, absorbed energy, and the ultimate load capacity.

Marzoq and Borhan [4] present an investigational study on the mechanical characteristics of RPC like flexural strength, splitting tensile strength, and compressive strength. The outcomes were linked to the prior experimental work studies and invented equations. Hybrid concrete structure collects all the advantages of precast concrete such as quality, shape, texture, accuracy, colour, and prestressing with entire advantages of in-situ construction method such as mouldability, economy, continuity, flexibility, thermal mass, robustness, and durability. Hybrid concrete construction can meet customer requirements for higher quality and lower costs through simple, buildable, and competitive structures offering consistent performance and quality [5]. Ju and Wang [6] studied the bonding performance between normal strength concrete (NSC) and reactive powder concrete (RPC). The findings referred to that the interfacial roughness was the most significant factor impacting the bonding, splitting tensile strength and shear strength, followed by the steel fibre volume fraction and finally the RPC water to binder ratio.
The most effective method for assessing the accurate performance of the structural elements is to test actual structural members experimentally, however, typical experiments are not always feasible because they are costly and time-consuming. Hence, other methods are needed that take into account concrete’s anisotropic performance including the influence of tensile cracks. Finite Element Modelling (FEM) is one of the important methods to be utilized in structural analysis, which requires less cost and time to implement [7].

One of various marketable (FEM) software which has been utilized throughout the years is ABAQUS software. This software package was utilized in this study to model the hybrid reinforced reactive powder concrete T-beams by and the results were validated against the experimental findings presented in reference [8].

2. Experimental work

Experimentally, four reinforced concrete T-beams were tested to investigate their flexural behaviour. The test was conducted at the laboratory site. The tested parameter was the type of concrete: fully normal concrete, fully reactive powder concrete, or hybrid (normal concrete and reactive powder concrete) as shown in Table 1. The tested beams’ dimensions were: an overall length of 1200 mm and an overall depth of 160 mm. The flange measured 50 mm in thickness and 220 mm in width. The web was made with 100 mm width and 110 mm clear height. The beams were simply supported having a clear span of 1100 mm and the distance between the two loads was kept constant at 367 mm as shown in Figure 1. (Type I) Portland cement was employed to produce RPC and NC beams. Natural sand with a fineness modulus of (2.63) and a specific gravity of 2.64 was utilized for NC mixes. Very fine sand with a maximum size of (600 μm) was used for RPC. Crushed gravel of a maximum particle size of 10 mm was utilized in this study for NC with a 2.74 specific gravity. A grey densified silica fume was used for the RPC mixture following the requirements of (ASTM C1240-03)[9]. Commercially known as Flocrete PC 200 superplasticizer was utilized in the RPC mixes follows the requirements of (ASTM C494-99) type G [10]. More details of the tested beams, loading conditions and the mix proportion of concrete were mentioned in [8].

3. Analytical investigation

With the ABAQUS software, the FE simulations were done. In this section, a comparison is provided between the theoretical and the experimental work to ensure adequate modelling for beams involving:
material properties, elements type, real constant, and convergence study. The nonlinear finite element method was performed to analyse the T-beams tested in the current research as shown in Table 1. The following references were utilized to model the material properties in ABAQUS software and will be explained in detail in subsections.

3.1. Modelling of material properties
Modelling the elastic performance of concrete in ABAQUS composed of two parameters: The Poisson’s ratio and the modulus of elasticity ($E_c$) as follows: For NSC, the elastic modulus can be determined according to the ACI-318 [11] formula as shown in equation (1). The modulus of elasticity of RPC was calculated by Graybeal [12] as shown in equation (2):

\[
E_c = 4700 \sqrt{f_c''} \\
E_c = 3840 \cdot \sqrt{f_c''}
\]

The commonly accepted value of Poisson’s ratio for NSC ranged from 0.15 to 0.25, while it is widely taken as 0.2 in the analysis [13]. For Ultra-High-Performance Concrete (UHPC), Poisson's ratio does not vary too much compared to other types of concrete, but it may possess a smaller range. Poisson's ratio for all UHPC investigations is about 0.2 within the elastic phase [14]. The ratio of 0.2 is adopted in the current finite element analysis. The concrete damaged plasticity (CDP) model treats both the compressive crushing and the tensile cracking of concrete as probable failure modes and was adopted in this research. The CDP utilizing several parameters like dilation angle ($\Psi$): The effective range of the angle of dilation in values between $30^\circ \leq \Psi \leq 40^\circ$ according to Malm [15]. $30^\circ$ for NSC and $40^\circ$ for RPC were adopted in this study according to the results of a sensitivity study conducted by the authors. The default value ($0.1$) of the flow potential eccentricity ($\varepsilon$) and the default value of the ratio of biaxial compressive yield stress/ uniaxial compressive yield stress ($fb0/ fc0$), which equals $1.16$, were adopted. The default value of Kc is $2/3$ which is the ratio of the second stress invariant in the tensile meridian to the compressive meridian ($0.5 \leq Kc \leq 1$).

Figure 2 displays the typical stress-strain curves of the uniaxial compressive and tensile behaviour defined by the CDP. In tension and compression, the uniaxial nonlinear performance of concrete can be defined as a tabular input in the shape of stress-inelastic strain. The inelastic or cracking strains can be determined as follows:

\[
\varepsilon_c^{in} = \varepsilon_c - \frac{\sigma_c}{E_c} \\
\varepsilon_t^{in} = \varepsilon_t - \frac{\sigma_t}{E_c}
\]

In which the symbols c and t refer to compression and tension, respectively, $\varepsilon_c^{in}$ and $\varepsilon_t^{in}$ are the inelastic strains, $\varepsilon_c$ and $\varepsilon_t$ are the total strains, $E_c$ is the initial elastic modulus, $\sigma_t$ and $\sigma_c$ are the stresses.
Uniaxial compressive stress-strain behaviour of NSC is modelled utilizing Hoggestad parabola [17]. The linear-elastic branch of the Hoggestad model continues until $\sigma_c$ that is taken as $0.4 \sigma_{cu} (0.4 f'_c$).

The relationships of the Hongestad model are presented as follows:

\[ \sigma_c = f'_c \left( 2 \frac{\varepsilon}{\varepsilon_0} - \left( \frac{\varepsilon}{\varepsilon_0} \right) \right) \]  
\[ \varepsilon_0 = \frac{2 f'_c}{E_c} \]  
\[ \sigma_c = f'_c \left[ \frac{a x (\varepsilon_c)^b}{c (\varepsilon_c)^c} \right] \]  
\[ \varepsilon_0 = 1.17 \times 10^{-5} \times f'_c + 4.59 \times 10^{-4} \times v_f + 1.92 \times 10^{-3} \]

where: $\sigma_c$: compressive stress of concrete (MPa), $\varepsilon_c$: compressive strain of concrete $a = 3.805$, $b = 0.919$, $c = 2.831$ and $d = 3.970$, $f'_c$ and $\varepsilon_0$ are compressive strength and its corresponding strain respectively. $\varepsilon_c$: compressive strain corresponding to stress $f'_c$. $f'_c$: cylinder compressive strength (MPa) and $v_f$: steel fibre volumetric ratio.

In this study, the stress-displacement relationship (bilinear) was offered to model the tensile behaviour of NSC and the stress-displacement (trilinear) was offered to model the tensile behaviour of RPC as shown in Figure 3. For NSC, the tensile strength ($f_t$) is calculated by the equation proposed by Wong et. al. [19] as shown in equation (9), and the fracture energy ($G_f$) is according to CEB-FIP Model [14]. The value of $G_f$ can be obtained as a function of the maximum size of aggregate ($D_{max}$) and the concrete’s strength needed as shown in Table 2.

\[ f_t = 0.33 \sqrt{f'_c} \]  

| $D_{max}$ (mm) | C12 | C20 | C30 | C40 | C50 | C60 | C70 | C80 |
|---------------|-----|-----|-----|-----|-----|-----|-----|-----|
| 8             | 40  | 50  | 65  | 70  | 85  | 95  | 105 | 115 |
| 16            | 50  | 60  | 75  | 90  | 105 | 115 | 125 | 135 |
| 32            | 60  | 80  | 95  | 115 | 130 | 145 | 160 | 175 |
For RPC, the tensile strength ($f_t$) can be calculated according to the equations proposed by Danha [18], the ($f_t$) is calculated from the splitting tensile strength ($f_{sp}$) or the flexural tensile strength ($f_r$) as follows:

$$f_t = 0.53 \times f_{sp}$$ (10)
$$f_t = 0.37 \times f_r$$ (11)

![Stress-displacement relation for NSC and RPC](image)

**Figure 3.** Stress-displacement relations for NSC and RPC [21,22].

To model the initiation and propagation of a crack in the concrete, the tension damage parameter ($d_t$) is utilized. This variable represents the damage to the elastic stiffness of concrete. The $d_t$ can be taken with a range from zero to one corresponding to the undamaged and fully damaged materials respectively [23]. The tension and compression damage are determined as follow [24]:

$$d_t = 1 - \frac{\sigma}{f}$$ (12)

Where $\sigma$ is the tension stress of concrete at the phase of analysis where the damage parameter is to be determined and $f$ is the ultimate tensile stress of concrete.

The plasticity model which focuses on elastoplastic hardening material is utilized to model the steel reinforcement and the elastic model only is adopted for the steel of the plates and the supports. The values of the stress-strain are achieved from the tensile test of the steel conducted by the authors. The stress and strain utilized the true strain and the true stress as illustrated in Figure 4 and the equations 13 and 14 [25]:

$$\sigma_t = \sigma (1 + \varepsilon)$$ (13)
$$\varepsilon_t = \ln (1 + \varepsilon)$$ (14)

Where $\sigma_t$ is the true stress, $\varepsilon_t$ is the true strain, $\sigma$ is the engineering stress and $\varepsilon$ is the engineering strain. The input data is characterized as tabular true stress-plastic strain data. The plastic strain is equivalent to the overall strain excluding the elastic strain which corresponds to the yield stress.
The elements used in the representation of the adopted specimens and geometrical properties in this model is C3D8R which is an 8-node brick element, reduced integration. This element was used to represent the beams, the support, and the load plates. T3D2 which is 2 nodes truss element was used to represent the steel bars.

A significant step in the modelling of finite elements is to choose the mesh size. In the current model, different element sizes of 10, 20, 30, 40, and 50 mm were examined. It can observe from Figure 5 that the variation of the load-deflection curve can be ignored in the case of the size enlarged from (10) mm to (20) mm. In addition, the curve of load-displacement is more precise with experimental results. Hence, a 20 mm element size was chosen for the mesh size.

The assembly of the parts, that were utilized in modelling these beams and the details about the steel reinforcement are shown in Figure 6.
3.4. Loading and boundary conditions
The model of the finite element was loaded with a similar position in the experimental test of all beams as shown in Figure 7. The load is presented as a uniform pressure, where the total applied load is divided on the area of the bearing plate. The boundary conditions of the displacement were defined by a simply supported restrain. The simply supported restraints of this model consist of pin support to resist the translation of vertical direction (Y) and longitudinal direction (Z), (Uy=Uz=0 where U2=U3=0). While roller support translating only in a vertical direction (Y) (Uy=0 where U2=0).

3.5. Interaction
After the assembly phase, parts should be interacted together to make a composite system. To contact all parts, the steel reinforcement parts were treated as one part by selecting all of them and were renamed as one part and then re-meshed. To model the contact between concrete and the reinforced steel bar in ABAQUS, interaction with embedded reign was utilized. To model the interaction between the RC beams, the support, and the bearing plates, the surface-to-surface kind tie interaction was utilized.
3.6. Analysis of the model

This section displays the results obtained from the finite element analysis (FEA) and compare them with the experimental test results. The structural analysis was carried out utilizing one step only (static general analysis step). Table 3 provides a comparison of the ultimate loads between the experimental results [8] and the finite element analysis of the investigated beams. The results possess a good convergence between the numerical and the experimental results ranged from 89% to 98%. The load-deflection curve for each beam gained from the experimental work [8] together with the FEA curves is displayed and compared in Figure 8.

As shown in Figure 8, fewer deflections (stiffer behaviour) were noticed in the FEA models than those of experimentally investigated T-beams. This can be attributed to many factors as follows:

a- The concrete was estimated as a homogenous material in the FEA model, but it was a heterogeneous material.
b- In concrete, the micro-cracks created by handling and drying shrinkage, decrease the stiffness in the actual concrete beams, but these influences were not taken into account in the FEA model.
c- In FEA, it is presumed that an ideal bond exists between concrete and steel bar of reinforcement. However, this assumption cannot be reached in the actual specimen.

Table 3. Numerical and experimental results of the tested beams

| Beam | EXP.[8] | Ultimate load [kN] | F.E.M | FEM/EXP |
|------|--------|---------------------|-------|---------|
| B1   | 105.04 | 99.48               | 0.95  |
| B2   | 147.50 | 145.77              | 0.98  |
| B3   | 138.50 | 123.92              | 0.89  |
| B4   | 123.72 | 114.60              | 0.93  |
Figure 8. The numerical and experimental load-deflection curves for the T-beams tested

3.7. Crack patterns
The failure shape is displayed in Figure 9. It can be observed that the T-beams are bent in their plane because of the applied bending moment (My). The crack patterns were noticed in the theoretical FEA model as shown in Figure 8 where the flexural and the shear cracks were observed at the flexural test of (B1, B2, B3, B4).
Conclusions

1. Results of the FEA exhibited a reasonable agreement with the experimental load-deflection behaviour.
2. Numerically, the ultimate load increased by 46.53%, 24.57%, and 15.2% when using full RPC, RPC in the web only, RPC in flange only respectively compared to fully NSC.
3. RPC T-beams showed a stiffer performance than NSC T-beam due to the higher RPC’s compressive strength and the existence of steel fibre compared to NSC result in decreasing the deflection of the beam.

Figure 9. Numerical and experimental crack patterns of the T-beams
4. Utilizing NSC in the web and RPC in the flange develops the flexural behaviour of the beam since the gain of the compressive strength and the aid of the steel fibre of RPC offer more ductility to the beams. The neutral axis is moved up to equalise the constant tensile reinforcement force and this can lead to enhancing the moment capacity resistance of the member; thus, the ultimate load amplified.

5. Also, utilizing RPC on the web improved the behaviour of T-beams effectively more than the situation when RPC is in the flange. This can be attributed to RPC's capability to absorb energy and the impact of steel fibres in stopping the cracks from the propagation as well as the growth in the section's corresponding moment of inertia.

6. The cracks in the investigated T-beams were flexural appeared at the tension zone in the middle third of the T-beam and flexural-shear cracks formed at the tension zone.

7. The cracks of the NSC beam become less in number and wider compared with the cracks of RPC. The failure mode of RPC showed ductile behaviour due to the existence of steel fibre. The steel fibres reduce the propagation of the crack and enhance the resistance to deformation.

5. References

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