3D Numerical Modeling of RC Deep Beam Behavior by Nonlinear Finite Element Analysis

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Abstract: In order to investigate behavior of reinforced concrete deep beams under static and dynamic loads, conducting a finite element analysis in comparison with an experimental study is very preferable and reliable technique from the viewpoint of difficulty, time saving, human force and budget. The main motivation of the study is to propose a numerical finite element model, including a reliable finite element modeling technique and constitutive material models, to simulate nonlinear behavior of reinforced concrete deep beams. For this purpose in the study, a nonlinear finite element analysis is performed to represent behavior of reinforced concrete deep beams. An existing experimental study is selected from literature and one of the tested specimen of it is verified numerically. Numerical results are compared with experimental ones in terms of load-displacement and damage distribution behaviors. Analysis are performed by using a commercial FE program named ABAQUS. The analysis results are demonstrated that finite element analysis is a highly effective and reliable tool to simulate nonlinear behavior of reinforced concrete deep beams.

***Index Terms*—ABAQUS, Concrete Damage Plasticity, Deep Beam, Finite Element Analysis, Reinforced Concrete.**

I. INTRODUCTION[1]

Earthquakes are one of the most common and severe disasters in which most of the casualties stem from collapse of structures. In order to prevent such a big lost, structures have to be designed in a way that they must have sufficient strength to resist both their own gravity loads and seismic loads occurring during an earthquake. It is known that nonlinear behavior of entire structure should be known well to make an earthquake resistant design. Behavior of a structure depends on behavior of its members such as beams, columns, shear walls etc. Therefore investigation of behavior of reinforced concrete (RC) members under static and dynamic loads is very important to design safe structures [1,2]. Moreover almost all researchers agree that performing a scientific experimental study is the most reliable and accurate technique to investigate nonlinear behavior of RC members.

In terms of inconveniency, time, labor and budget, conducting an experimental study to investigate behavior of a RC member has some disadvantages. Namely experimental studies are difficult to perform, time consuming, and require more budget and human force in comparison with finite element (FE) analysis which is an alternative technique to investigate behavior of RC members. Several studies have demonstrated that FE analysis including an appropriate FE modeling technique and accurate constitutive material models is also a quite reliable and robust tool to simulate nonlinear behavior of RC members. Therefore FE analysis is widely preferred technique by researchers for their scientific studies.

Deep beams are defined both in TS-500 [3] and ACI 318-14 [4] codes as a RC member that is loaded on one face and supported on the opposite face. Moreover it has a clear span not exceeding four times the overall member depth. Bernoulli hypothesis, plane sections remain plane, for slender beams is not valid any more for deep beams due to nonlinear strain distributions on the member causing deep beams act as a tied arch.

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Deep beams are widely used in structures as wall footings, foundation pile caps, floor diaphragms, and shear walls [5]. The practice of them deep beams also have increased substantially in high rise buildings over last decades. A deep beam in a building is shown in Fig. 1 as an example.

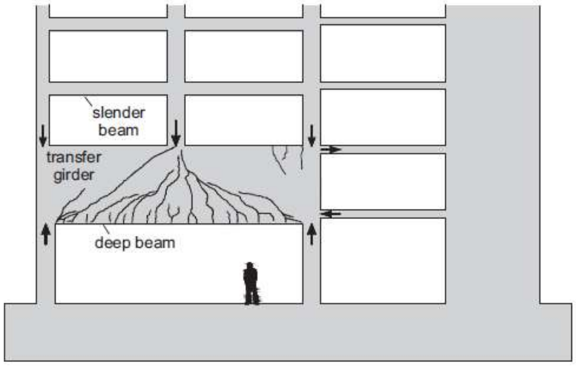


Figure 1. A deep beam in a building [6].

In an experimental study, in order to acquire more realistic results in terms of RC member behavior, dimensions of a deep beam should not be lower than some certain values [7]. It results very big specimen dimensions and that makes the experimental study of deep beams more difficult. Therefore performing such an experimental study of a deep beam requires bigger test setup, more instrumentation, and extra human labor and budged.

Due to difficulties in experimental studies mentioned above, most of the researchers prefer to make a numerical study along with their experimental studies. The number of specimens in an experimental study is determined according to verification process of a numerical model. In other words, first of all necessary number of experimental tests are conducted until the numerical model is verified sufficiently. Later the parametric study is performed computationally by numerical analysis.

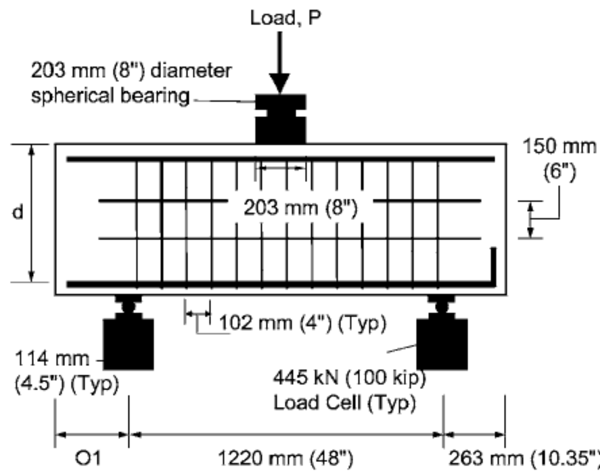
There are some numerical studies to investigate the behavior of RC deep beams. Among some of significant researches are Mohamed, et al. [5], Enem, et al. [8], Islam & Khennane [9], Metwally [10], Najafian & Vollum [11] and Birtel & Mark [12]. In these studies RC beams are modeled generally as 2 dimensional (2D) plane stress FE model and different numerical studies are performed on behavior of RC deep beams in each study. However, in the present study a 3D FEM is used to increase the accuracy of numerical results.

Main motivation of the study is to propose a numerical FE model, including a reliable FE modeling technique and constitutive material models, to represent nonlinear behavior of RC deep beams. Second objective of the study is to demonstrate how robust FE analysis is in terms of simulating actual nonlinear behavior of RC deep beams.

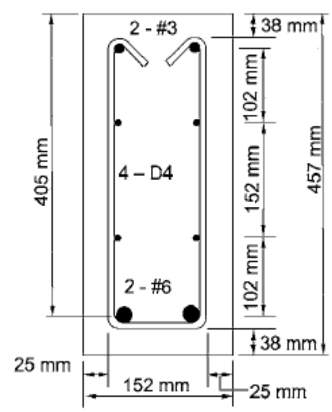
In the study, a numerical verification of an existing experimental study of a deep beam is performed by using a commercial finite element program ABAQUS [13]. Results of the experimental study are compared with the numerical study with regards to load-displacement behavior and damage formation at ultimate failure stage.

II. Experimental Study

To model a RC deep beam behavior numerically, an experimental study conducted by Roy and Brena [14] are selected as a reference study. One of the specimens named as DB1.0-0.75L in their study is selected as reference verification specimen. The ratio of shear zone to effective depth of the section (a/d) is 1.0. The specimen geometry, reinforcement, and experimental test setup are displayed in Fig. 2.



a) Side view



b) Cross Section

Figure 2. Specimen geometry, reinforcement, and test setup [14].

In the reference study, 28-day compressive strength of concrete (f_{ck}) obtained by test of cylinder is given as 29.9 MPa. Yield stresses of No. 5 (19 mm in diameter) reinforcing steel at bottom and No. 3 (10 mm in diameter) at top of the section are 469 MPa and 414 MPa respectively. Moreover diameter of web reinforcement consisting of vertical stirrups and horizontal bars are given as deformed D4 (5.5 mm in diameter) wire. The yield stress of the web reinforcement is 605 MPa.

In the test the specimen was subjected to a single concentrated force at mid-span. In order to distribute stresses over a wider area on the top and bottom surfaces of the member, steel plates were placed at point load and reaction point locations. The beam supports consisted of a pin and a roller in the test.

Load-displacement result of the test is presented in Fig. 3. Since no tabular data for load-deflection graph were given in the reference study, the curve is struck out by a red dashed line. Therefore loads and corresponding displacement values are determined manually.

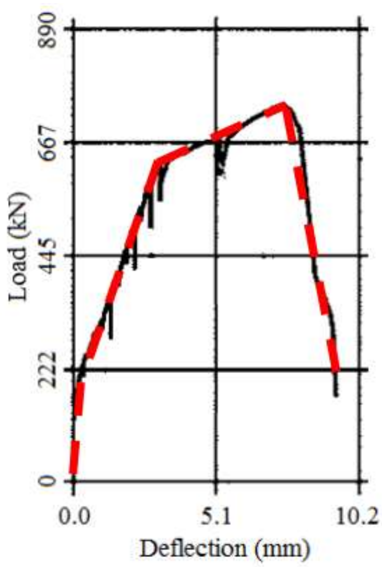


Figure 3. Load-deflection graph of the test

III. NUMERICAL FINITE ELEMENT MODELLING

In the study, numerical simulation is performed by using the FE-code ABAQUS/Standard which is a general-purpose analysis software that can solve a wide range of linear and nonlinear problems.

Inelastic behavior of concrete is defined to FE model by using concrete damaged plasticity (CDP) model providing a general capability for modelling concrete and other quasi-brittle materials in all type of structures. It considers isotropic damaged elasticity concept with isotropic tensile and compressive plasticity [15]. The CDP parameters considered in the FE model is given in Table 1. More detailed information related with CDP parameters can be found in the ABAQUS user's manual.

Uniaxial compressive and tensile constitutive materiel behaviors of concrete are required to define the CDP model in ABAQUS. The stress-strain relations under uniaxial compression and tension loading are, respectively [15]:

$$\sigma_c=(1\text{-}d_c)E_0(\varepsilon_c\text{-}\varepsilon_c^{pl}) \quad (1)$$

$$\sigma_t=(1\text{-}d_t)E_0(\varepsilon_t\text{-}\varepsilon_t^{pl}) \quad (2)$$

where;

E_0 is the initial elastic stiffness of the material.

ε_c^{pl} *and* ε_t^{pl} are plastic strains for compression and tension respectively.

Both d_c and d_t are damage variables explained in detail following part of the study.

TABLE I
CDP PARAMETERS FOR ABAQUS MATERIAL DEFINITION OF CONCRETE

Parameter	Value	Description [15]
ψ	56	Dilation angle
ϵ	0.1	Eccentricity
fb_0/fc_0	1.16	The ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress.
K	0.667	K_c, the ratio of the second stress invariant on the tensile meridian
μ	0.0001	Viscosity Parameter

The uniaxial compressive behavior of concrete is defined for both pre and post peak stresses. The behavior prior the peak strength is obtained by using unconfined concrete model of CEB-FIB 2010 [16] model code. It is however known that the post-peak behavior of concrete both in compression and tension is mesh sensitive [17,18]. Because of this reason, the post-peak compression models of Vonk [19] and Van Mier [20] which takes into consideration the mesh size effect on the post-peak behavior of concrete is used. Compressive behavior of concrete in CDP model of ABAQUS must be defined as stress vs. inelastic strain relationship. The stress-inelastic strain relation (f_c-ε_{ci}) of uniaxial compressive behavior of concrete used in the FE model is given in Fig. 4.

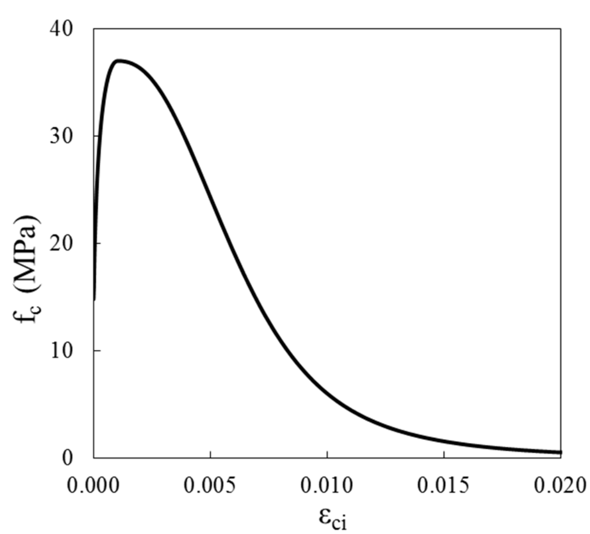


Figure 4. The stress-strain relation of uniaxial compressive behavior of concrete

The tensile behavior of concrete up to peak tensile stress is assumed linear elastic and obtained as peak tensile stress divided by initial elastic modulus. Because of the fact that the peak tensile strength (f_{ctm}) of the tested specimen is not mentioned in the reference study, it is calculated in MPa by using Eq. (3) defined in CEB-FIB 2010 [16] model code.

$$f_{ctm}=0.3(f_{ck})^{2/3} \quad (3)$$

To avoid unreasonable mesh sensitivity due to reasons mentioned above, it is recommended to define the tension softening behavior as a stress-crack opening displacement graph (f_{ct}-w). This brittle fracture concept is defined as the fracture energy (G_f) required to open a unit area of crack (Fig. 5-c) [15]. In CEB-FIB 2010 model code it is stated that in the absence of experimental data, G_f in MPa can calculated for normal weight concrete as follows;

$$G_f=0.073(f_{cm})^{0.18} \quad (4)$$

$$f_{cm}=f_{ck}+\Delta f \quad (5)$$

where:

f_{cm} is the mean compressive strength in MPa and can be calculated by using (5).

Δf is a constant value that can be taken as 8 MPa.

Because there was no experimental data for G_f in the reference research, Eq. (4) is used to calculate fracture energy in the study.

In literature tree different model types for behavior of tension softening part after cracking have been proposed as linear, bilinear and exponential in terms of stress-crack opening displacement curve, Fig. 5.

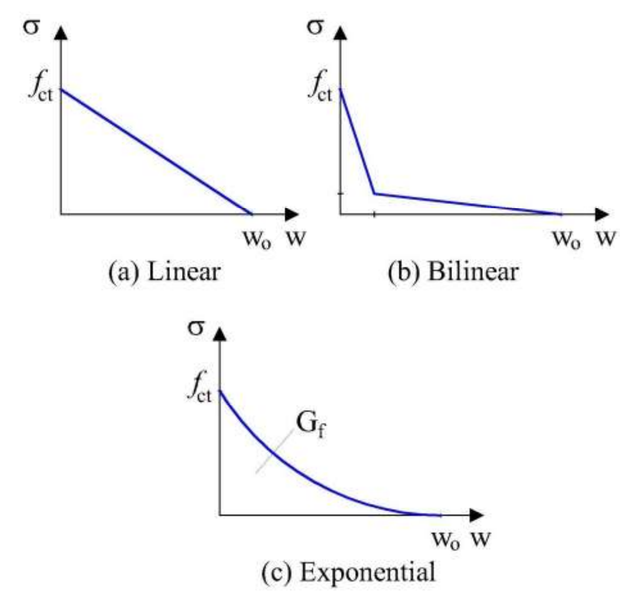


Figure 5. Post-cracking tensile behavior of concrete

In this study the behavior of tension softening part after cracking is defined by using exponential post-peak tension softening model of Hordijk [21] as stress-crack opening displacement behavior which is shown in Fig. 6. ABAQUS enforces a lower limit on the post-cracking stress equal to one hundred of the initial cracking stress to avoid potential numerical problems, σ ≥ fct/100 [15]. Therefore the post-cracking stress values are defined to the FE model by considering this limitation.

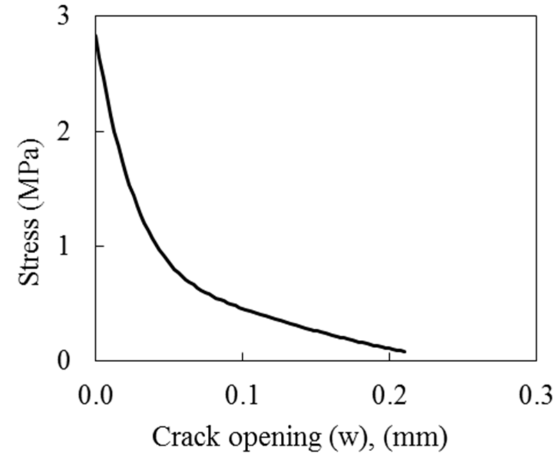
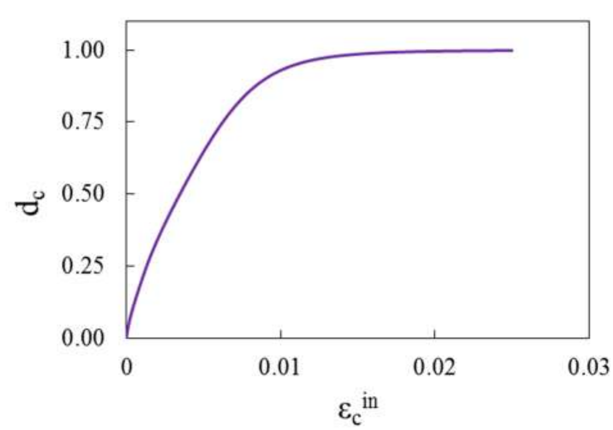
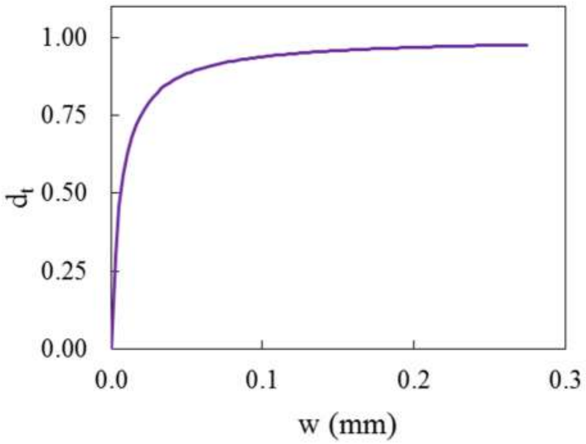


Figure 6. Stress-crack opening displacement behavior for tension

Additionally, in order to take into account the degraded response of concrete, two independent uniaxial compressive and tensile damage variables, d_c and d_t are defined in CDP model. These variables represent the damage on elastic stiffness that it occurs when a concrete specimen is unloaded from any point on the strain weakening branch of the stress-strain graphs. That elastic stiffness degradation is significantly different for both compressive and tensile behaviors. The uniaxial degradation variables are functions of plastic strains, temperature, and field variables. They can take values ranging from zero to one corresponding to undamaged and fully damaged materials respectively [15]. Compressive and tensile damage behaviors of the concrete defined in FE material model are demonstrated in Fig. 7.



a) Compressive damage



b) Tensile damage

Figure 7. Damage behaviors of the concrete

Material behavior of reinforcing steel is defined as elastic and inelastic stress-strain relationships with strain hardening effect. ABAQUS enforces to define behavior of reinforcement as a stress vs. plastic strain (σ_s-ε_s^{pl}) curve. Moreover true stress is considered to take into account instantaneous cross-sectional area of reinforcing bars during loading. A sample true stress-plastic strain curve of No. 5 steel bar used in the specimen is presented in Fig. 8.

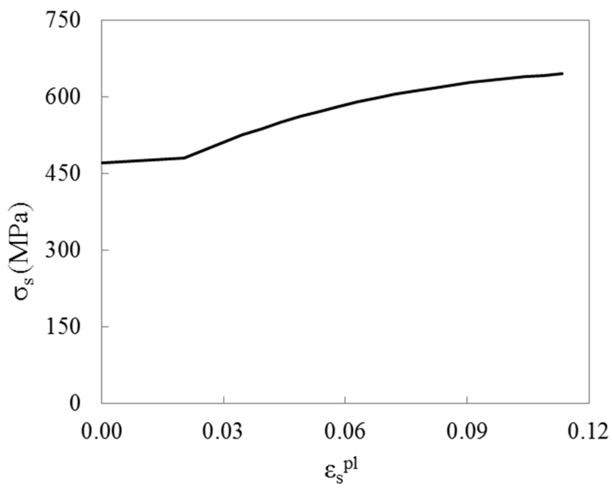


Figure 8. True stress-plastic strain curve of No. 5 steel bar.

In order to avoid stress concentration at support and loading points, steel plates with given dimensions in the test are added the FE model. The load is applied as a linear vertical displacement on loading plate. Supports are considered as a pin and a roller as in the test.

Instead of a 2D plane stress simulation, a 3D FE Model is used to increase the accuracy of numerical results in the study. The element types used in FE model for either of concrete and steel plates are 8-node linear bricks (C3D8R). Moreover reinforcement steel is a 2-node linear 3-D truss (T3D2). The reinforcement is assumed fully embedded into concrete to take into account the interaction between reinforcement steel and concrete. The general view of ABAQUS model of reinforcement is displayed in Fig. 9.

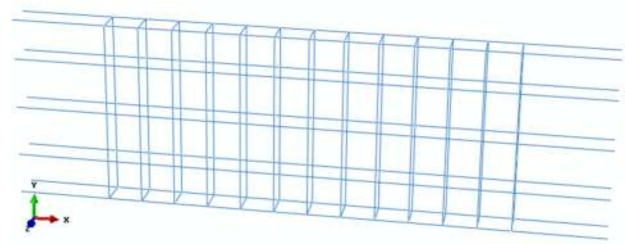


Figure 9. ABAQUS reinforcement model.

In order to determine more accurate mesh size for FE model, a parametric study is performed. As a result an optimum mesh size is determined as 50 mm with an aspect ratio of 1. The meshed model is demonstrated in Fig. 10.

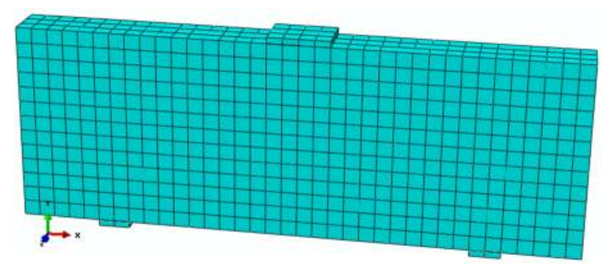


Figure 10. ABAQUS meshed FE model.

IV. Results and discussion

Results of numerical study and test are plotted in Fig. 11 in terms of load vs. mid-displacement curves. It seems clearly on the graph that yield and ultimate load levels and sudden reduction in stiffness due to shear failure beyond ultimate displacement are simulated very successfully. In other word, flexural and shear behaviors of the experimental test are represented very accurately by FE model.

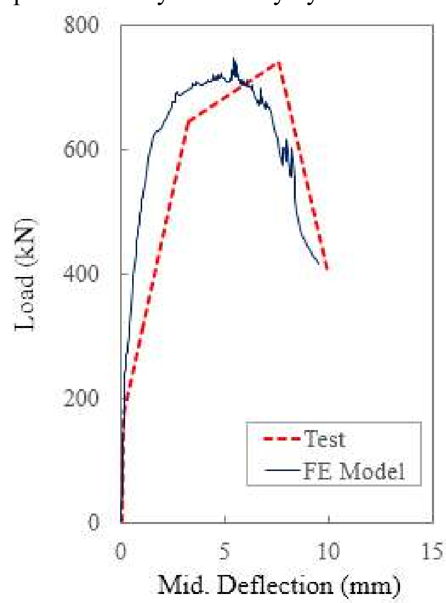


Figure 11. Verification results of FE model with test result.

On the load-displacement graph on the contrary, FE model curve is stiffer than the experimental test result. After first crack occurred on the member, stiffness of the FE model is higher than that of the test. Such kind of higher stiffness in FE models may stem from micro-cracks reducing the stiffness of the RC member. It is know that they are present in real the concrete while FE models do not include them [5]. Moreover some unknown ambient factors may have reduced the stiffness of the tested specimen.

Damage occurrences at the ultimate level for both experimental and numerical studies are displayed in Figs. 12 and 13 in terms of equivalent plastic strain (PEEQT) respectively. It is shown that the ABAQUS FE model demonstrates sufficiently nonlinear damage distribution of the experimental test member.

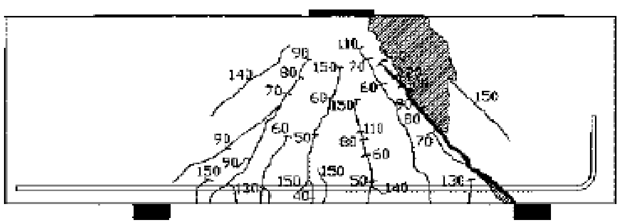


Figure 12. Ultimate damage result of the test [14].

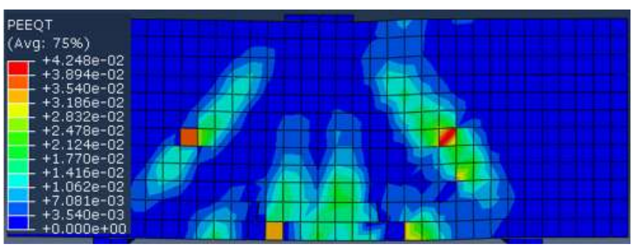


Figure 13. Ultimate damage result of the numerical FE model.

V. Conclusion

In the study, a numerical nonlinear FE study is performed by considering an existing tested RC deep beam. For that aim the tested specimen is verified numerically until sufficient degree of accuracy by realization of an adequate FE modeling technique and quite accurate material modes.

At the end of FE modeling process, the test specimen is verified sufficiently with ABAQUS in terms of load-displacement behavior and damage distribution at ultimate failure stage. The results are deduced that the FE analysis including an appropriate FE modeling technique and accurate constitutive material models is highly successful to represent nonlinear behavior of RC deep beams.

When taking into consideration the difficulties, necessary time, and budget and labor to conduct an experimental test, parameterized nonlinear finite element analysis is very reliable and preferable technique to investigate nonlinear behavior of RC deep beams.

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