COMPUTATIONAL MODELING OF DAMAGES IN BRIDGE PIERS

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Abstract- Bridges are one of the most vulnerable structures to an earthquake damage. Due to an obsolete code for bridge design and poor construction practices in Pakistan, most of the bridges are seismically deficient. Experimental tests are helpful in assessment of bridge piers but requires considerable resources. In that account, numerical tools are also used for the assessment of bridge piers and various numerical techniques are available which can be utilized in this regard. This work focuses on non-linear modeling of bridge piers and validation of proposed computational scheme with experimental data using a Finite element based software–Abaqus. A single circular bridge pier subjected to a monotonic lateral load is modeled in the finite element software. For this purpose, a plasticity based damage model Concrete Damage Plasticity (CDP) is used for modeling damages in Abaqus. CDP considers concrete crushing and tensile cracking as the main failure mechanism. A constitutive model for concrete compression–Modified Kent and Park model–and tensile cracking–exponential relation for the tension stiffening– to obtain the CDP parameters are used. Mesh sensitivity analysis is performed to select a suitable mesh size as well as configuration for the numerical modeling. Additionally, the effect of step size on percentage of kinetic energy produced during the analysis is studied. Computational analysis demonstrates that the proposed scheme predicts damages in accord with the experimental results.

Keywords- Abaqus, Bridge Piers, Concrete Damage Plasticity (CDP), Explicit analysis, Finite Element Method.

1 Introduction

Bridge is an integral component of the transportation network of any country. Complete structural failure or malfunctioning due to non-suitable structural condition can have serious economic and life consequences. Among all the natural disasters earthquakes are most severe due to their unpredictability in magnitude and time. More than 40 deaths and $1.8 billion loss occurred due to bridge damages in the Loma Prieta earthquake [1]. Pakistan lies in one of the most seismically active regions of the world. It is located at the junction of Indian, Eurasian and Arabian tectonic plate. The Himalayan region is alone capable of generating a magnitude 8.0 or above earthquake [2]. But, unfortunately due to an obsolete code for design of bridges and poor construction practices, the existing bridge stock of Pakistan is seismically deficient [3]. In the 2005 Kashmir earthquake many bridges experienced damages beyond repair due to improper seismic design and low quality concrete [3] [4].

Among the different structural elements of a bridge, bridge piers are most vulnerable to earthquake damages. They form the main lateral load resisting system of the bridge. Single column bridges are most vulnerable to earthquake because of a single load path and lack of redundancy in the system. Bridge piers vary in shapes and dimensions. Mostly, bridges having solid circular type of piers ranging in diameter from 3 feet to 5 feet are common in the northern areas of Pakistan – which is the most seismically active region [5]. The average loads on these piers were 760 kips [5].

Bridge piers behavior under the lateral demands can be studied by performing experimental studies. Multiple studies have been conducted in this regard [5]–[8]. However, experimental tests are mostly performed on scaled down models due to lack of space and resources. Furthermore, it is also not feasible to perform experimental tests frequently to study bridge piers under lateral demand for case study research, design and retrofit purposes. Finite element method (FEM) based approaches are better alternative to experimental studies. Developing a finite element model that can predict the behavior of the bridges during an earthquake requires a proper definition of finite element model and material parameters. Different researchers utilize different FEM based tools and material models for both the concrete and steel for computational modelling [8], [9]. One of the models that can be easily employed in this regard is the Concrete Damage Plasticity (CDP) in Abaqus. This model is frequently used to study the nonlinear behavior of concrete structures in static, quasi-static and reverse cyclic condition. CDP is a continuum plasticity based model. It was presented by Lubliner [10] and modified by Lee and Fenves [11]. Quasi-brittle material like concrete, rocks, ceramics and mortar can be modeled with CDP. Compressive and tensile damages are considered by the CDP along with the stiffness recovery under cyclic loading in compression [12]. In the elastic range, the model uses elastic relations for the mechanical properties of the concrete. In the plastic region, the model utilizes the degraded elastic stiffness.

This study focus on non-linear modeling of bridge piers under lateral loading. The main objective of this research is to develop computational scheme that can easily be employed for the prediction of damage in bridge piers without the need for rigorous experimental work. Practically, such a computational scheme can be used for case study research, design or retrofit purposes. An experimental test [5] has been used for the validation of the proposed computational scheme using Abaqus. In the proposed numerical scheme, CDP is used to model damages related to concrete in a bridge pier under monotonic lateral load. Additionally, rebars are modeled using a simple elastoplastic model. Continuum 3-D elements and linear 1-D elements are used to model the concrete and rebars, respectively. Furthermore, fracture-energy based material model is utilized for the tension stiffening of concrete to avoid mesh sensitivity. In addition to this, mesh sensitivity analysis is also performed to select a suitable mesh size for the analysis. The effect of loading time on the kinetic energy of the model is also explored in this study.

2 Methodology

2.1 Numerical model geometry

Experimental test performed by Ali [5] was used for the validation of the proposed numerical technique. The model and test setup is shown below (Figure 1). The model is a scaled down version of a real bridge pier. The model is a single pier

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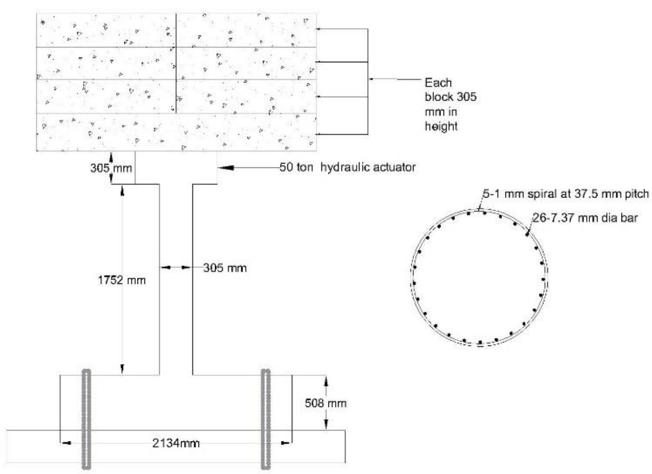

b)


Figure 1: a. Geometry and b. Experimental test setup of the numerical model

having 305 mm diameter and 1750 mm clear height. At the top of the column, a 742 mm square pedestal having depth of 305 mm is provided for the placement of blocks. Furthermore, the column is erected from a base having 2133 mm length, 914 mm width and 505 mm depth–which is rigidly connected to the strong floor. The loads from the superstructure were modeled through concrete blocks, placed above the top of the column. Lateral loads in the form of drift were applied to the model in reverse cyclic manner during the experiment. Numerically lateral loads were applied monotonically in a single direction. Both the experimental and numerical models were subjected to 4% drift. The material properties as determined during experiment are given in the Table 1.

Table 1: Material properties

Material Property	Compressive Strength	Modulus of Rupture	Modulus of Elasticity	Rebar yield Strength
value	2400 psi	674 psi	2798 ksi	60 ksi

2.2 CDP

CDP is a continuum-plasticity based model. The main failures that are considered by the CDP are compressive crushing and tensile cracking. The essential parts of any plasticity based model are yield criterion, flow rule and hardening rule. The yield function adopted by CDP is drucker-pruger hyperbolic function [13] shown in (1) with (2) (3) and (4) defining the dimensionless parameters.

$$F = \frac{1}{1-a}(\bar{q} - 3\alpha\bar{p} + \beta(\varepsilon^{\sim pl})\langle\hat{\bar{\sigma}}_{max}\rangle - \gamma\langle\hat{\bar{\sigma}}_{max}\rangle) - \sigma_c(\varepsilon^{\sim pl}) = 0 \quad (1)$$

$$\alpha = \frac{\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)-1}{2\left(\frac{\sigma_{b0}}{\sigma_{c0}}\right)-1}; 0 \leq \alpha \leq 0.5 \quad (2)$$

$$\beta = \frac{\overline{\sigma_c}\left(\varepsilon_c^{-pl}\right)}{\overline{\sigma_t}\left(\varepsilon_t^{-pl}\right)}(1-a) - (1+a) \quad (3)$$

$$\gamma = \frac{3.(1-K_C)}{2.K_C - 1} \quad (4)$$

The flow rule considered by CDP is non-associated potential plastic flow. The plastic potential flow G is defined as

$$G = \sqrt{(\epsilon.\sigma_{t0}.\tan\psi) + \bar{q}^2} - \bar{p}.\tan\psi, \quad (5)$$

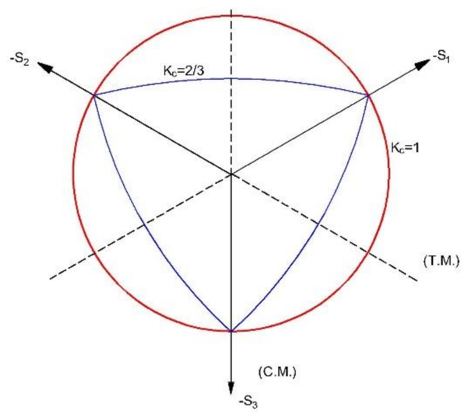

Figure 2: : Yield Surface in deviatoric plane

The definition of CDP requires certain parameters. These parameters include: dilation angle ψ , eccentricity ϵ, the ratio of biaxial to uniaxial yield compressive strength ($\frac{\sigma_b}{\sigma_c}$), the ratio of the second stress invariant on the tensile meridian to the compressive meridian K_C ,and viscosity μ. The yield surface in deviatoric plane with the effect of K_C on the shape of yield surface is shown (Figure 2). There has been no consensus in literature regarding the selection of a specific value [13], however, the values of these parameters used in this study are given in the Table 2.

Table 2: CDP model parameters

CDP Parameters	$\psi(°)$	ϵ	$(\frac{\sigma_b}{\sigma_c})$	K_C	μ
value	40	0.1	1.16	0.667	0

2.3 Concrete material models

CDP data inputs requires the Inelastic strain ε_c^{in} where subscript c and t are for tension and compression, and stiffness degradation variable d_c. The calculation of plastic strain and degradation variable are shown in (6) and (7).

$$\varepsilon_c^{pl} = \varepsilon_c^{in} - \frac{d_c}{(1-d_c)}\frac{\sigma_c}{E_0} \text{ (compression) }, \varepsilon_t^{pl} = \varepsilon_t^{ck} - \frac{d_t}{(1-d_t)}\frac{\sigma_t}{E_0} \text{ (tension)} \tag{6}$$

$$d_c = 1 - \frac{\sigma_i}{\sigma_{cu}} \text{ (compression) }, \quad d_t = 1 - \frac{\sigma_i}{\sigma_{tu}} \text{ (tension)} \tag{7}$$

Where ε_c^{in} and ε_t^{ck} show strain hardening and tension stiffening in compression and tension, respectively.

Complete compressive and tensile stress-strain data is required to calculate the values of these parameters. The modified Kent and Park model [14] is used in this study to define the concrete compressive behavior (Figure 3). Stress-strain material models for tensile behavior of concrete encounters mesh-sensitivity issues, therefore, it is recommended [12] to use models based on crack opening displacement and stress for the definition of strain-softening in tension . These are fracture energy based material models and which relates the crack opening displacement with the stress. Linear, bilinear and exponential relations are available in literature (Figure 4) . Any of these relations can be used unless the areas under the curve are equal.

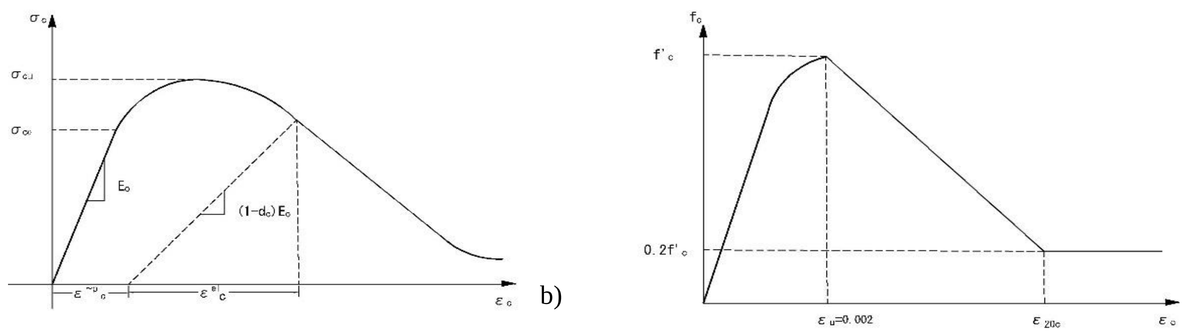

Figure 3: a. Typical compressive uniaxial stress-strain curve, and b. Modified Kent and Park model

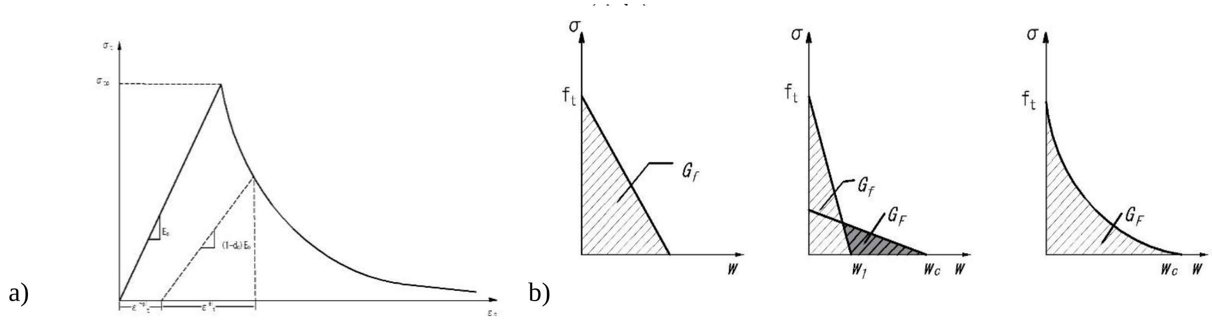

Figure 4:a. Typical uniaxial tensile stress-strain curve and, b. linear, bi-linear and exponential relations for tension stiffening.

2.4 Rebar

A simple bi-linear (Figure 5) relation is used to model the rebars. The rebars are modelled using 1-D beam element.

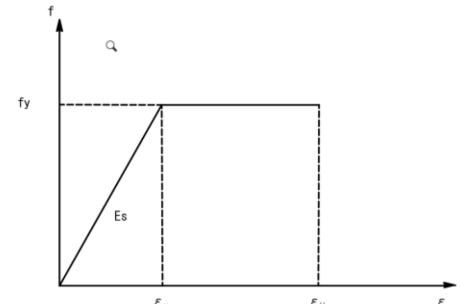

Figure 5: Bi-linear relation for rebar modeling

3 Results

As material degradation in the form of damages is modeled using CDP, therefore, an explicit dynamic analysis is performed to avoid the convergence error–encountered in the implicit analysis. The main failure that the CDP depicts is the concrete crushing in compression and cracking in tension. Since CDP is a smeared crack model, therefore, the damages are not depicted as a separate entity. Mesh sensitivity studies were carried out to check the suitable mesh size and arrangement, since modeling the whole model with same mesh element size will increase the computational time. Three types of mesh arrangements were used: a complete instance mesh with the same mesh size, a 1.5 mesh ratio between column and the foundation, and a localized mesh in the vicinity of the column. The complete instance mesh shows damages in the column as well as in the footing due to the connectivity of the meshed elements, which are in contrast to the real behavior observed during the experimental test. In the 1.5 mesh ratio, due to contact issues, an uplift was observed at the base of the column. The localized mesh comes out to be the most suitable arrangement. The damage patterns observed during the localized mesh are in good agreement with the experimental results (Figure 8) as shown in (Figure 7).

The localized mesh configuration which is the most suitable configuration for the proposed numerical scheme is used for the selection of suitable mesh size. Mesh size of 1-inch, 2-inch, 2.5-inch and 3-inch is used to study the effect of mesh size on damage pattern the results are shown below (Figure 7). Dispersed damage patterns, which are not in agreement with the experimental results, are observed with coarser mesh. Finer mesh shows clustered damages but with higher computational time. Hence, 2-inch mesh is the most optimum mesh which produces good results with a comparatively smaller computational time. The actual test was a quasi-static and the total time of the test was 3600 seconds, however, modeling lateral load with a time step of 3600 seconds requires high computational cost and time. Therefore, the step-time for the lateral load should be such that no dynamic effects are produced in the model with reasonable results. Two step-times are used ,40 seconds and 60 seconds, with both producing high kinetic energy which are not desirable in quasi-static analysis. The high kinetic energy is due to the dynamic effect caused by the high rate of loading in the explicit dynamic analysis. The recommended value of kinetic energy to total energy is 5 to 10 percent. The ratio of kinetic energy to total energy for 40 and 60 time steps was varying between 50% to 0.03%. Therefore, such analysis should be performed either with a higher time step. Contrarily, the analysis should be done through a combination of modified time-step with either mass-scaling or damping or both.

4 Conclusion

The material model for concrete by the name of CDP is used, to model the damages in bridge piers using Abaqus. In this study, a proposed scheme for the damage modeling was used on a single bridge pier to study the effect of mesh size and step-time. The following are the conclusions:

1 Mesh size has a significant influence on the damage pattern and the time required for analysis.
2 Coarser mesh shows a dispersed damage pattern which is not in agreement with the experimental results.
3 Finer mesh shows clustered damages but with higher computational time.
4 Step-time should be selected based on the recommended ratio of kinetic energy to potential energy.

5 In case of smaller step size, mass-scaling should be used to avoid excessive kinetic energy content.

6 Compressive crushing, tensile cracking and rebar yielding (Figure 7)— smeared over the finite elements— were observed during the numerical analysis which are in good agreement with the experimental results (Figure 8).

The above conclusion represents a number of variables that affect the damage in bridge piers when modeled with CDP. Further trials are required to study the effect of model parameter, concrete strength, and column geometry on the damages.

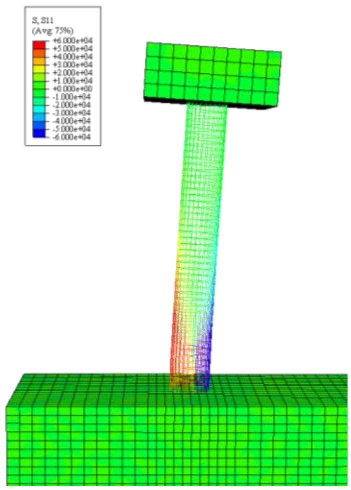

Figure 6: Rebar yielding

Figure 8: Failures observed in the experimental test a) Tensile cracking, b) Compressive crushing of concrete and rebar yielding

a)
b)

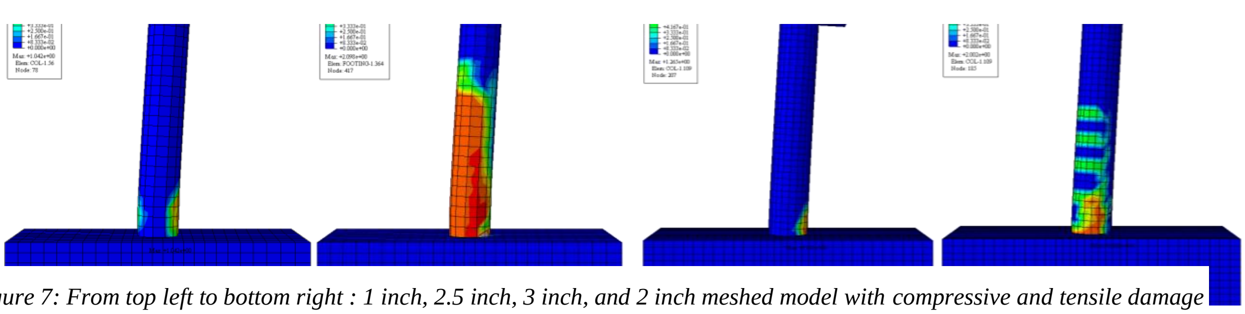

Figure 7: From top left to bottom right : 1 inch, 2.5 inch, 3 inch, and 2 inch meshed model with compressive and tensile damage patterns

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