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Application of Simple Plane Cap Model to Simulate Compression Failure of RC Beam under Impact Loads

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Abstract: The aim of this paper is to present the non-linear analysis for impact response of reinforced concrete (RC) beam with prominence on tension and compression area. In order to envisage the RC behavior, pressure dependant yield criteria Drucker-Prager Plane-Cap (DPPC) type is assumed for the concrete, meanwhile, shear strain energy criterion Von-Mises (VM) is applied for steel reinforcement; to define the accurate strength of material during the short period (dynamic). These material models were incorporated with Adaptive Smoothed Particle Hydrodynamics (ASPH) method. Dynamic Increase Factor (DIF) has been employed for the effect of strain rate (SR) on the compression and tensile strength of the concrete; the orthotropic constitutive equation due to the damage effect is considered during the softening phase on tensile region while constitutive equation of cap model is employed on compression area. A series of experimental studies were also presented in this paper. Several beam elements were tested under low velocity impact loads. Failure mechanism such as shear cracking, bending cracking, compressive behavior of the beam were evaluated by using displacement-time histories as well as overall failure mode. Based on these studies, the investigations enabled a better understanding of the behavior of reinforced concrete beam elements under low velocity impact loads, as well as, it is confirmed that the proposed models give good agreement with experimental results.

Keywords: Plane cap model, reinforced concrete beam, impact loading

1. Introduction

Reinforced concrete (RC) element under impact loading produce dissimilar structural response compared to static loading. In fact, many existing RC elements that are exposed to impact load are not designed to resist that load (Md Noh et al. 2018). Generally, as the impact loading increases, RC beam element will be failed in the form of bending-shear cracks and crushing in the tensile and compression zone, respectively. Further increases of impact loads will result in severe crushing in the top of the beam and expose of bottom and top rebar. Since the advantage of concrete is utilized mostly in compression, the stress condition in compression is of primary attention. Many researchers such as Chen & May, (2009); Tamai et al. (2018); Kishi et al. (2002); Sangi et al. (2010) have been studied experimentally the behavior of RC beam under impact loads. However, its experimental investigation required higher cost in term of preparing the dynamic devices, specimens and also time.

According to the above matters, the numerical simulation could be the one of quick methods to study the behavior of the concrete structure under impact loads. Many literatures can be found such as Sangi et al. (2010); Cotsovos et al.

(2012); Kantar et al. (2011); Pham et al. (2016) have developed the Finite Element (FE) analysis to simulate the impact response of RC beams.

However, some of the FE method faced the numerical problems when dealing with impact loads. The use of mesh are method of generating an eminence mesh and for certain case, it is hard to measure the consistency calculation of crushing failure in compression area. Smoothed Particle Hydrodynamics (SPH) method is one of the meshless method that can be used effectively to simulate the compression behaviour of RC beams under impact conditions. The previous works by Mokhatar et al. (2018) and Fukazawa et al. (2011) have proven the efficiency of SPH method to predict the impact response of RC beam.

Numerical simulation of RC beam for tensile area using SPH have also been extensively discussed by Fukazawa et al. (2011), however, studies focused on compressive area of RC beam under impact loads utilizing SPH method still has not much yet done. Thus, current works aimed to explore the ability of SPH method by employing an adaptive SPH (or ASPH) computation that can model the impact response of RC beam.

The constitutive model, an availability of material law, the improvement of stable and accuracy of non-linear numerical procedure to model RC behaviour is still debated. Moreover, compressive failure model of RC beams under impact phenomena have not as yet been identified entirely. Therefore, an attempt has been made in the current work by developing a simple and reliable damage model, with regards to assessing the response of RC elements (beam) due to impact forces in term of shear cracking, bending failure as well as crushing in the compressive region. The numerical capabilities were extensively verified against existing experimental result by Chen & May, (2009) and Kishi et al. (2002).

2. Numerical Procedure

This part presents a simple and reliable non-linear numerical method and constitutive model of concrete. Four basic schemes to present the localized failure of RC beam subjected to low velocity impact load are: (i) the accurate strength of concrete in tension and compression is represented separately by a parameter, namely DIF (ii) linear pressure-sensitive yield surface Drucker-Prager (DP) with volume dependent Plane-Cap (PC) hardening function when hydrostatic compression occurs between 60 Mpa ~ 150 Mpa are utilized (iii) strain softening in tension is implemented during post-peak regime by adopting φ to degrade the material's stiffness (iv) two kinds of constitutive equations is developed to simulate the crushing, shear cracking as well as bending cracking. All of these features incorporated under ASPH method.

2.1 Adaptive Smoothed Particle Hydrodynamics Method

This technique was utilized to investigate the behavior of the RC beams. Generally, the procedure of ASPH method is similar to the conventional SPH as explained in Fukazawa et al. (2011). Some modification has been made to the kernel function by implementing three independent smoothing kernel functions on the orthogonal three coordinates, in which considering anisotropic kernel functions instead isotropic. First derivative of anisotropic kernel function is shown in equation (1), while, the B-spline function that corresponding to the derivation is expressed in (2).

$$\nabla W = \frac{\partial W_x}{\partial x} W_y W_z , W_x \frac{\partial W_y}{\partial y} W_z , W_x W_y \frac{\partial W_z}{\partial z} \qquad (1)$$

$$W(q,h) = factor \times \begin{cases} \frac{2}{3} - q^2 + \frac{1}{2} q^3 & 0 \le q < 1 \\ \frac{1}{6}(2-R)^3 & 1 \le q < 1 \\ 0 & q \ge 2 \end{cases} \qquad (2)$$

The factor of one-dimensional space $1/h$ was utilized in the calculation due to the requirement of the independent three coordinates. Further details for fundamental ideas and crucial formulations of SPH and ASPH have been described clearly by Fukazawa et al. (2011).

The performance of ASPH calculation in this study is deriving from the conventional SPH formulation for hydrodynamics with material strength. The calculations scheme is described as follows:

i. Interactive particles in the influence area are defined prior updating the time increment. Where, only a fixed number of particles are within the support domain used in the particle approximations.

ii. Calculate the kernel function as shown in eq. (1) and (2) or integral approximation.

iii. The performance of ASPH calculation in this study is deriving from the conventional SPH formulation for hydrodynamics with material strength. The calculations scheme is described as follows:

$$\frac{dv_i^A}{dt} = \sum_{B=1}^{N} m^B \left(\frac{\sigma_{ij}^A}{\left(\rho^A\right)^2} + \frac{\sigma_{ij}^B}{\left(\rho^B\right)^2} + \Pi^{AB}\,\delta_{ij} \right) \frac{\partial W}{\partial x_i^A} \quad (3)$$

In this equation, the artificial viscosity of Monaghan Π^{AB} is used in order to prevent the unnecessary penetration for particle during impact. This viscosity formulation and details are also given by Liu et al. (2003).

iv. Compute the strain rate tensor and rotation rate tensor as given in Liu et al. (2003).

$$\varepsilon_{ij}^A = \frac{1}{2} \sum_{B=1}^{N} \frac{m^B}{\rho^B} \left(v_i^{BA} \frac{\partial W}{\partial x_j^A} + v_j^{BA} \frac{\partial W}{\partial x_i^A} \right) \quad (4)$$

and

$$R_{ij}^A = \frac{1}{2} \sum_{B=1}^{N} \frac{m^B}{\rho^B} \left(v_i^{BA} \frac{\partial W}{\partial x_j^A} - v_j^{BA} \frac{\partial W}{\partial x_i^A} \right) \quad (5)$$

where $v_i^{BA} = v_i^B$ - v_i^A and $v_j^{BA} = v_j^B - v_j^A$ is the particle velocity vector.

v. The plasticity theory for yield criterion, the flow rule and hardening rule of both materials concrete and steel is calculated under the stress thread. Explanation of this calculation is described in the following sub-section.

vi. Strain and stress calculation is updated due to the time increment and by applying the constitutive equation, respectively.

2.2 Strain Rate Effects

Effect of strain rate (SR) is tremendously essential for the impact (dynamic) analysis, in order to model the increasing of compressive and tensile strength due to the short period. Thus, SR is considered into numerical model by applying the tensile and compression of DIF recommended by Jaxier et al. (1998) and Zhou et al. (2008). Ratio of dynamic strength to tensile strength in opposition to SR log scale is defined as DIF. Based on these literatures, the magnitude of SR on compression can range from 10^{-6} to 1000 s^{-1} under impact loading and the constitutive equation of DIF is shown below

$$CDIF = \left(\frac{\dot{\varepsilon}_{cd}}{\dot{\varepsilon}_{cs}} \right)^{1.026\eta} \quad \text{for } \dot{\varepsilon}_{cd} \le 30s^{-1} \quad (6a)$$

$$CDIF = \gamma\left(\dot{\varepsilon}_{cd}\right) \quad \text{for } \dot{\varepsilon}_{cd} > 30s^{-1} \quad (6b)$$

In this calculation, the value of $\dot{\varepsilon}_{cs}$ is 30.0e^{-6}. Meanwhile, log $\gamma = 6.156\eta - 0.49$, and η is equal to $[5.0 + 3.0(f_{cu}/4.0)]^{-1}$. For tensile strength, the empirical formula for DIF, defined as:

$$TDIF = \left(\frac{\dot{\varepsilon}_{td}}{\dot{\varepsilon}_{ts}} \right)^{1.016\theta} \quad \text{for } \dot{\varepsilon}_{cd} \le 30s^{-1} \quad (7a)$$

$$TDIF = \beta\left(\frac{\dot{\varepsilon}_{td}}{\dot{\varepsilon}_{ts}} \right)^{\frac{1}{3}} \quad \text{for } \dot{\varepsilon}_{cd} \le 30s^{-1} \quad (7b)$$

The value $\dot{\varepsilon}_{ts}$ on tension area is 3.0e^{-6} and log $\beta = 7.11\theta - 2.33$, where θ is derived by $1/(10.0 + 6.0f_{cu}/f')$. In which the value of 10 N/mm^2 is taken for f'.

2.3 Yield Criteria

Determination of plastic yielding for the concrete material and to control the plastic volumetric change of concrete are using linear DP criterion with PC surface under hydrostatic compression. Yield surface/failure line of this pressure-dependent (DP) model is depending to the value of ϕ and c, where these values could produce α and k as in Fig. 1. The yield strength of compression is higher than tension, where, this phenomenon is employed in this study. PC surface is utilized to control the volumetric expansion as well as to bound the model in the hydrostatic compression axis, Resende (1987).

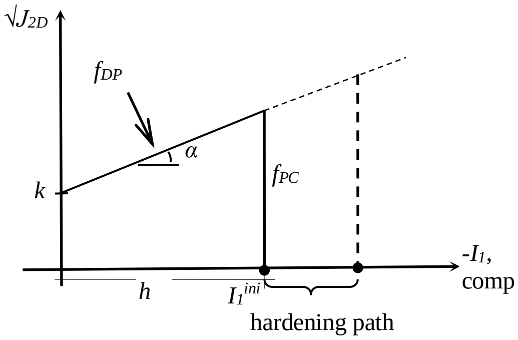


Fig.1. DP yield criterion with PC surface

The generalized equation for the DP criterion can be written as

$$f_{DP} = \sqrt{J_{2D}} - \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)} I_1 - \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)} \quad (8)$$

In order to obtain these two parameters that associated with DP failure criterion, the conventional tri-axial compression (CTC) tests is required. Based on the existing CTC experimental data and derivation of parameters in Desai & Siriwardane (1984) as well as the consistency value used by Lubliner et al. (1989), ϕ and c are assumed to be in range 2.80 – 3.40 N/mm^2 and 32^0, respectively.

Study in Poinard et al. (2010) has been identified by hydrostatic test, when the hydrostatic compression arises at 60 MPa, the cement matrix starts to damage. Thus, initial compression cut-off is adopted at 60 MPa, where CP surface take place at this point to control the plastic volumetric response

$$f_{PC} = I_1 - h\left(\varepsilon_v^p\right) = 0 \quad (9)$$

where, h is location of the cap related to the $\varepsilon_v{}^p$ as defined

$$h = \frac{1}{D}\ln\left(\omega - \frac{\varepsilon_v^p}{w}\right) \therefore \omega = 1 - \varepsilon_{vo}^p / w \quad (10)$$

For steel reinforcement, Von Mises' criterion is adopted. In this paper, the failure mode is only considered on the concrete materials. Thus, this conventional shear strain energy criterion is applied for both compression and tension part with 1/100 of initial stiffness for strain hardening.

2.4 Failure Model of Concrete

For describing the failure model of concrete, an illustration of non-linear stress-strain curve is used. In tension, the tensile and flexural cracking is identified by strain-softening as shown in Fig. 2. The softening effect is directly defined after exceeding the yield stress, σ_t.

Method to identifying softening is extracted from previous study in Fukazawa et al. (2011) and the further details about this technique could be obtained from the paper. In order to defy the cracking on concrete, w_c is calculated by G_f and the depend on particle size. The value of 100N/m is used for fracture energy. In tension, the value of w_c is 13000 (με) and it is assumed that the material strain-softens until reach the value of w_c.

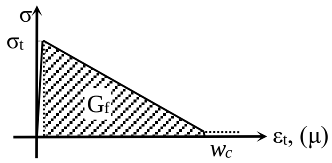


Fig.2. Strain-softening in tension

Since investigation on tensile failure has been studied by Fukazawa et al. (2011), this paper deals more on the compression area. The simple implementation model without required various parameters by experimental tests is considered. The non-linear behavior in compression is illustrated in Fig. 3.

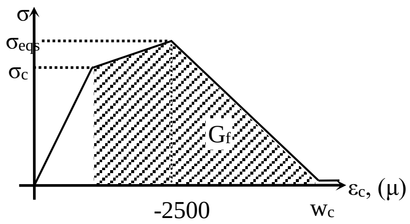


Fig.3. Hardening and strain-softening in compression

Compression failure is due to the great localized failure zone, however, still bounded under damaged zone Resende (1987). In the present model, the hardening is defined after reach the compressive yield strength σ_c followed by compression softening. The incremental plastic strain vector (hardening parameter) as expressed in (11) is used to control the size of the yield surface.

$$d\vec{\varepsilon}_{ij}^{p} = \sqrt{\frac{2}{3}(d\varepsilon_{ij}^{p}.d\varepsilon_{ij}^{p})} \qquad (11)$$

While, the volumetric expansion in compression through loading process is derived by

$$d\varepsilon_{ij}^{p} = \lambda \frac{\partial f}{\partial \sigma_{ij}} \qquad (12)$$

using flow rule associated with the DP yield criterion. The gradient of yield surface represented by associated flow rule in (12) is given as

$$\frac{\partial f}{\partial \sigma_{ij}} = \frac{\sigma'_{ij}}{2\sqrt{(1/2)\sigma'_{ij}\,\sigma'_{ij}}} - \alpha\delta_{ij} \qquad (13)$$

Thus, the incremental plastic strain is evaluated using

$$d\varepsilon_{ij}^{p} = \lambda\left(\frac{\sigma'_{ij}}{2\sqrt{\frac{1}{2}\sigma'_{ij}\,\sigma'_{ij}}}\right) - \alpha\delta_{ij} \qquad (14)$$

Generally, concrete material fails in crushing, if the materials has crushed in compression, thus, it is assumed that there is also strain softening occurred. Compressive Damage Zone (CDZ) model has been introduced by Resende (1987). In that model, the tensile fracture energy in the splitting cracks is introduced as a parameter of CDZ model. In addition, the compressive crushing is related to the specimen length and specimen slenderness. However, rationality of the parameter needs to be verified suitably with further experimental test.

In present study, degradation of material stiffness is employed to establish the failure process during compressive crushing. The idea of integrity tensor φ_i (i = x,y,z) is taken from Fukazawa (2011), where damage variable is the negative values of three principle strain. Similarly in tension area, damage variable used in compression area can be written as follows:

$$\varphi_x^2 = (1-d), \therefore \left(d = \frac{\varepsilon_x}{w_c} \right) \qquad (15a)$$

$$\varphi_y^2 = (1-d), \therefore \left(d = \frac{\varepsilon_y}{w_c} \right) \qquad (15b)$$

$$\varphi_z^2 = (1-d), \therefore \left(d = \frac{\varepsilon_z}{w_c} \right) \qquad (15c)$$

where the value of w_c is 1300 (με) that calculated by 100N/m fracture energy and 0.005 m particle diameter. Damage formulations from (16) are multiplied to the initial fourth-order isotropic elastic matrix. Thus, the orthotropic constitutive equation is given by the following expression:

$$\sigma_{ij} = \varphi_x \varphi_y \varphi_z \left[D^{el} \right] \varepsilon_{ij} \qquad (16)$$

3. Experimental Work

Practical test were carried out by Chen & May (2009) and Sangi et al. (2010) investigating high mass – low velocities (7.3 m/s and 5.3 m/s) impact behavior of reinforced concrete beam and the resulting dynamic response of the total structure. Tests were carried out on several beams under drop-weight loads. Beam size that used in the experimental works is 200 mm x 100 mm in depth and width, while, 2700 mm span beams in length. Beam is designed with 6 mm diameter of top steel reinforcement and 12 mm diameter for bottom reinforcement. Concrete cover between the main reinforcement bars and the top and bottom edges of the beam is 25 mm. Stirrups of 6 mm are spaced at 200 mm intervals. In addition, a flat impact mass is used. Diameter of it is 100 mm, and the weight of it is 98.7 kg. The details of reinforcement arrangement and dimension of projectile are shown in Fig. 4.

Experimental work has shown that the impact results in the beam realizing a peak loading followed by flexural failure with crushing beneath the projectile and shear cracking in the impact zone. Shear crack pattern and bending failure of the beam are most important for further analysis and comparison to the results of numerical simulation. Of particular interest are the compressive area and tensile region of RC beam.

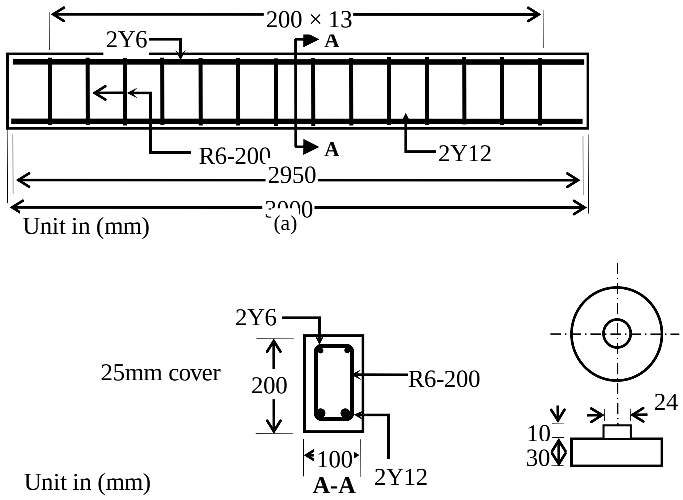


Fig.4. Reinforcement and projectile details

4. Analysis

This section describes the material properties used in the non-linear simulation as well as the comparisons of displacement-time histories and failure mode between analysis and experimental results. Table 1 shows the linear material properties of concrete and steel that used in the simulation.

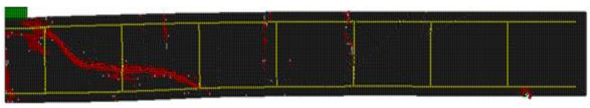
Table 1 Concrete and steel linear properties

Material	**Young Modulus (N/m^2)**	**Poisson's ratio**
Concrete	2.06e10	0.22
Steel	2.06e11	0.3
Density (kg/m^3)	**Yield Stress, σ_c (N/m^2)**	**Yield stress, σ_t (N/m^2)**
7800	300e6	300e6
2400	30e6	3e6

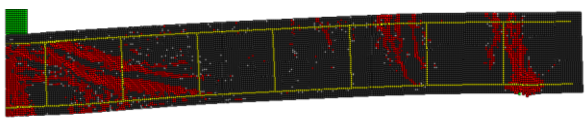
Numerical simulation of the beam exploits the quarter model to simplify the analysis. Where, 3000 mm length of beams (2700 mm span) with simply and pin-end support are examined. All beams were supported 150 mm from the ends.

4.1 Simply Supported Beam

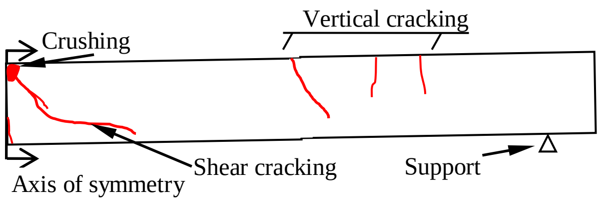
In the study of Chen & May (2009), the result shows the flexural failure and shear cracking as well as some crushing in the impact region (see Fig. 5c and 5d). In numerical analysis, it can be seen in Fig. 5a that the proposed model gives similar crack pattern and failure modes with the practical results. Additionally, vertical cracks are identified closely with experimental at the upper side of beam section away from impact region. This paper also compares the proposed analysis with the numerical idea of Fukazawa et al. (2011). Based on Fig. 5b, the impact response of beam indicates the major diagonal crack pattern and damage. Failure of the beam is also identified significantly. It can be noted that by applying the damage variables to the compression area, the crushing can be appeared beneath the impact area.



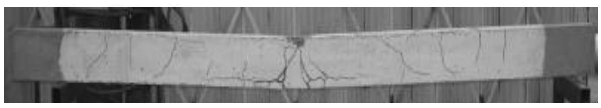
(a) Simulation results (quarter model)



(b) Simulation results (quarter model) by using model of Fukazawa et al. (2011)



(c) Experimental results (symmetry)

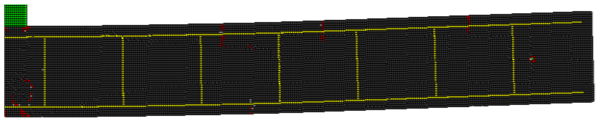


(d) Experimental results (full view)

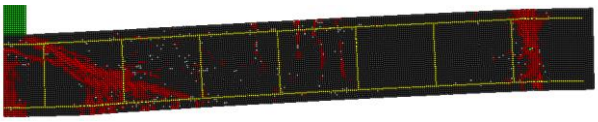
Fig.5. Comparisons of simply supported beam between numerical with experimental results

4.2 Pinned Ended Supported Beam

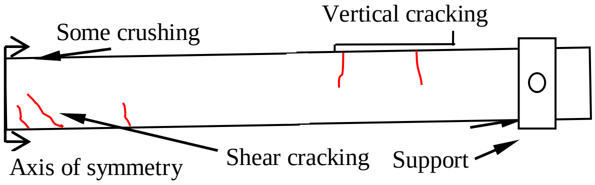
For pin-ended support beam, the velocity of projectile is reduced to 5.3m/s. Decision of boundary conditions and numerical data input correspond to practical tests by Sangi et al. (2010). The reduced velocity resulting shear and flexural cracking approximately about 1 to 3 mm (see Fig. 6c and 6d), it is confirmed by the Fig. 6a, that, proposed model provides comparable phenomena in term of propagations of vertical cracking, inclined cracking tendency as well as crushing on compression region. Moreover, based on Fig. 7, the maximum value and shape of displacement curves from the numerical analysis are in reasonable agreement with the experimental results. However, numerical model by Fukazawa et al. (2011) does not work well to analyze the failure behavior of RC beam subjected to impact loads. As in Fig. 6b, the total damage for shear cracking and a large amount of vertical cracking has been investigated critically. From Fig. 7, it has been also proved that excessive bending cracking is determined. Therefore, it is noteworthy that concrete performs differently in both tension and compression, thus the yield criterion could not be assumed to be similar. Application of pressure dependent criterion in this study can avoid the over prediction of deformation Zhou & Hao (2008).



(a) Simulation results (quarter model)



(b) Simulation results (quarter model) by using model of Fukazawa et al. (2011)



(c) Experimental results (symmetry)

(d) Experimental results (full view)

Fig. 6. Comparisons of pin-ended support beam between numerical with experimental results

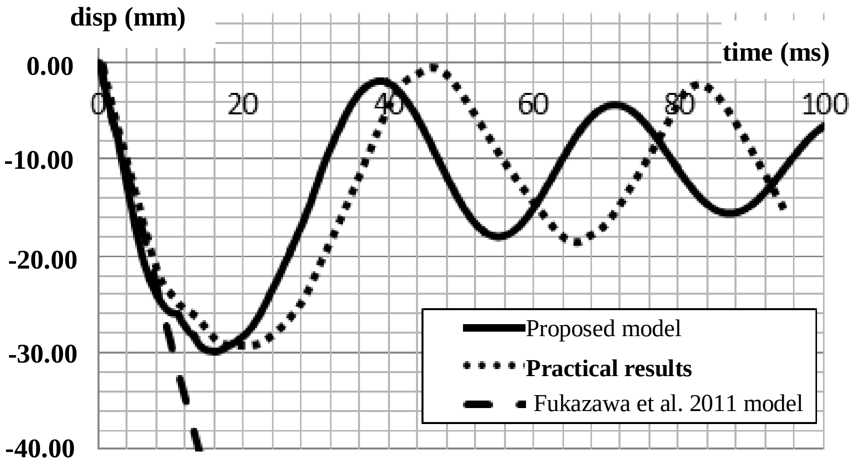


Fig.7. Comparisons displacement-time histories

5. Conclusion

Results have shown that the proposed model is reliable to calculate both shear and bending cracking as well as the compressive damage such as crushing under the impact zone. Moreover, this numerical simulation has produced as closed as experimental results in term of displacement time histories. However, to obtain more reasonable and realistic estimation of damage behavior of RC elements subjected to low velocity impact loads, further study on non-linear simulation scheme should be conducted numerously.

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