A Numerical Investigation of Punching Shear Behavior in Meso-Scale Arch Panel as Wall Member in Nuclear Reactor Containment Building

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1. Introduction

Nuclear reactor containment buildings are subjected to various load conditions, including internal pressurization, external missile impact, heavy load devices or structural members such as slabs, walls, and columns. Especially, the generation of punching shear and in-plane tensile forces in the tangential direction of the wall due to internal pressure a significant challenge, as concrete is a quasi-brittle material that is highly susceptible to such forces as shown in Fig. 1. These loads can result in punching shear failures, which are a critical concern for the structural integrity of containment buildings.

The containment building is a cylindrical structure with a curved concrete wall, resulting in an arch-shaped structure when modeled in part. While considerable research has been conducted on the punching shear behavior of flat structures, relatively few studies have focused on arch structures. Therefore, this paper presents an analytical investigation of the punching shear behavior of reinforced concrete(RC) arch panels as wall members, aiming to offer valuable insights into the behavior of concrete under these complex loading conditions.

2. Numerical Modeling and Results

2.1 Numerical Modeling

The behavior of RC arch panel subjected to punching shear loads is investigated in this study using MIDAS FEA, a commercial finite element analysis program co-developed by MIDAS IT and TNO DIANA [1,2], purposely designed for advanced nonlinear detailed simulations of concrete structures. The quasi-static simulation using MIDAS FEA was performed for all models. A total strain crack (TSC) model, which is based on the smeared crack approach, is used for the concrete constitutive model. The TSC model is developed along the lines of the Modified Compression Field Theory, originally proposed by Vecchio and Collins (1986) [3], and the three-dimensional extension to this theory is proposed.

The rotating crack model was implemented in MIDAS FEA for finite element simulation in this study. A concise description of the constitutive modeling of concrete is presented in this model. The model incorporates two primary failure mechanisms of concrete, namely tensile cracking and compressive crushing.

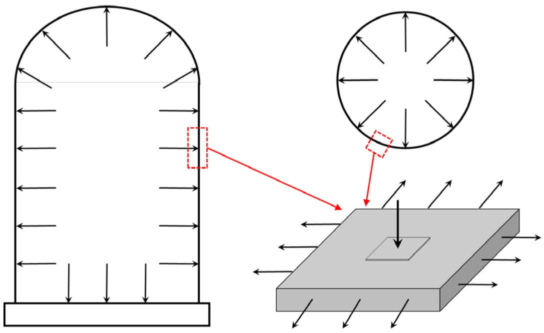
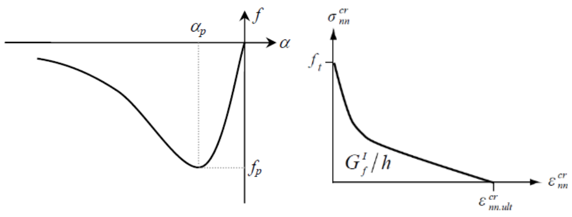


Fig. 1. Punching shear and in-plane tensile forces in containment building wall



(a) Thorenfeldt compression curve (b) Nonlinear tension softening curve

Fig. 2. Concrete constitutive model

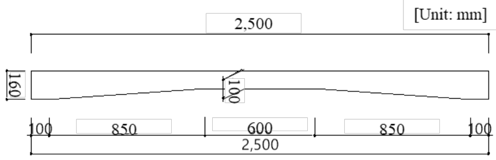
Thorenfeldt parabola, illustrated in Fig. 2(a), was utilized to describe the compressive behavior of concrete. The Thorenfeldt function is expressed using equation as follows:

$$f = -f_p \frac{\alpha_i}{\alpha_p} \left(\frac{n}{n - 1 + \left(\frac{\alpha_i}{\alpha_p} \right)^{nk}} \right)$$

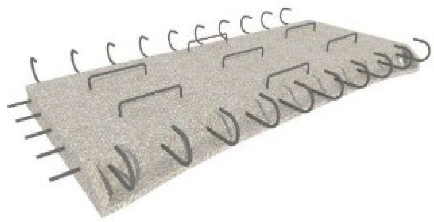
The tension limit in concrete is defined by a stress-fracture energy approach proposed by Hordijk (1991), as shown in Fig. 2(b). This approach assumes that softening phenomena occur when the tensile strength is surpassed, and the slope of softening is governed by the fracture energy and mesh size parameters. The fracture energy was obtained from the CEB–FIP code, with the value varying for each member based on the compressive strength of concrete and the coarse aggregate size. For all concrete, the maximum aggregate size was set to 25 mm, and the mesh size was selected as 50 mm for all specimens. The model is calculated as follows:

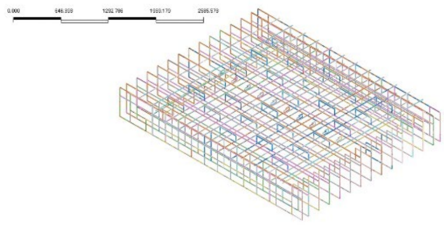
Table 1. Material models and input parameters in FE model

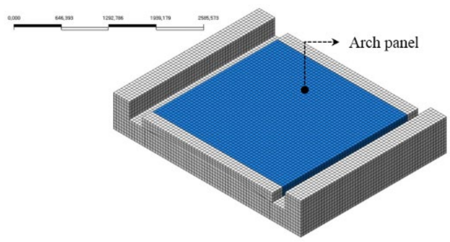
Material	Models / criteria	Young's modulus (MPa)	Weight density (kN/m3)	Poisson's ratio	Strength (MPa)
Concrete	Hordijk (Tension) Thorenfeldt (Compression)	30,000	24.5	0.167	AD: 49.5 CIP: 38.5
Rebar	von Mises (Yield criterion)	200,000	78.6	0.3	400



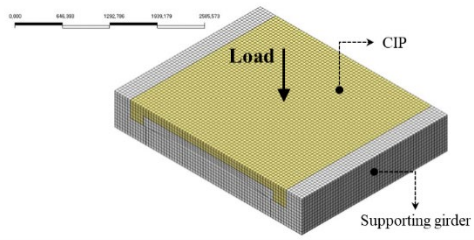
(a) Front view of arch panel


(b) Arch panel schematic


(c) Rebar modeling



(d) Concrete modeling w/o CIP



(e) Concrete modeling with CIP

Fig. 2. FE modeling

$$\frac{\sigma_{nn}^{cr}(\varepsilon_{nn}^{cr})}{f_t} = \begin{cases} \left(1 + \left(c_1 \frac{\varepsilon_{nn}^{cr}}{\varepsilon_{nn,ult}^{cr}}\right)^3\right) exp\left(-c_2 \frac{\varepsilon_{nn}^{cr}}{\varepsilon_{nn,ult}^{cr}}\right) \cdots \\ -\frac{\varepsilon_{nn}^{cr}}{\varepsilon_{nn,ult}^{cr}}(1 + c_1^3) exp(-c_2), \quad if\ 0 < \varepsilon_{nn}^{cr} < \varepsilon_{nn,ult}^{cr} \\ 0, \qquad if\ \ \varepsilon_{nn,ult}^{cr} < \varepsilon_{nn}^{cr} < 0 \end{cases}$$

Here, c1 = 3 and c2 = 6.93.

The yielding of embedded rebars for nonlinear behavior was modeled using the von Mises plasticity function. The material properties of concrete and rebar for arch panel are tabulated in Table 1.

A three-dimensional simulation was conducted on the arch panel composite specimens using a 20-node isoparametric element and an embedded bar element for concrete and rebar, respectively. The simulation of model, which are comprised of concrete and rebar, as depicted in Fig. 2. The boundary conditions of all specimens were vertically and laterally restrained at contact surface nodes between supporting girders. MIDAS FEA supports the construction stage analysis function, and this study has leveraged this feature to implement lateral restraints of arch panel and casting of CIP overlay concrete in each stage. The FE model was precisely calibrated and validated in the previous experimental study results [4].

The experiment and FEA results are presented in Fig. 3. The load–displacement curve obtained from the FEA at mid-span exhibited bi-linear behavior, which was similar to the experiment results. The initial stiffness corresponded very closely to the measured data, but after approximately 370 kN. This is similar to the experimental results, where the stiffness declined due to cracking at approximately 370 kN. The ultimate loads of arch panel were 820 kN, respectively, which were approximately 1.7 times higher than the design punching shear load of 483.3 kN as calculated from the ACI 318 code [5]. The ultimate load of the test was lower than the FEA results, because the applied loading was stopped in the test at approximately 800 kN to prevent catastrophic failure of the specimens for safety reasons. Therefore, it is expected that the arch panel specimens could resist higher loads, because there was no significant decrease in stiffness.

Figs. 4a–d illustrates the failure mode at the bottom surface from the initial cracking load to the ultimate load as observed from the experiment and the FE simulation. The cracking propagation process in the FE simulation results followed the four loading steps. The TSC model based on fracture energy is implemented in the simulation, as smeared crack models. As shown in Figs. 4a–d, all crack patterns acquired from the FE simulation were similar to the cracking patterns observed from the experiments.

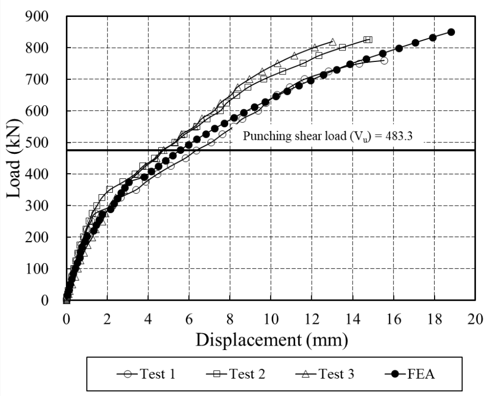
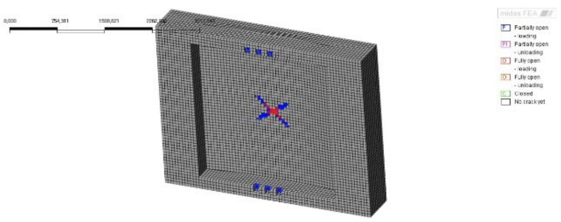
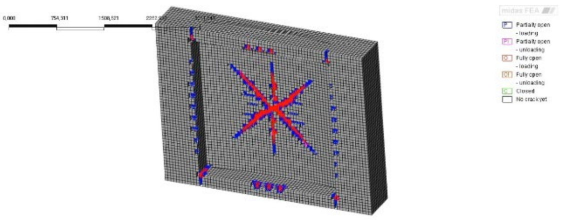


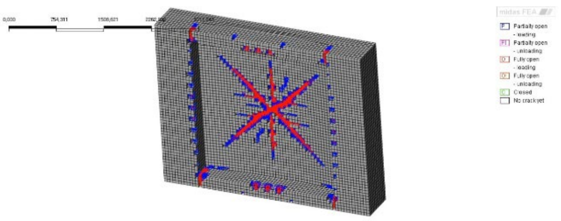
Fig. 3. FE modeling



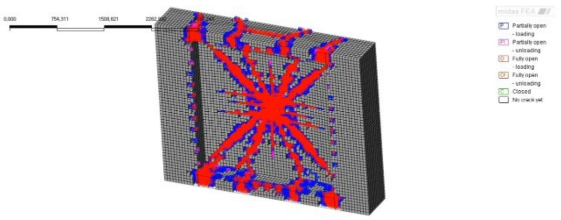
(a) 374 kN



(b) 500 kN



(c) 650 kN



(d) P_u

Fig. 4. Failure modes

3. Conclusions

This paper presents the study of finite element analysis verification for RC arch panel structural behavior under punching shear load. FE simulations were performed to validate the experimental results for punching shear strength of the specimen. The ultimate loads of arch panel were 820 kN, respectively, which were approximately 1.7 times higher than the design punching shear load of 483.3 kN as calculated from the ACI 318 code.

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