Analytical Study on the Beyond Design Seismic Capacity of Reinforced Concrete Shear Walls

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1. Introduction

Protection of NPP structures from earthquakes has become a critical issue after the Fukushima accident in 2011. One of the issues that resurfaced after the accident was to assess the design margin of NPP structures against beyond design earthquakes. The OECD-NEA has organized an international benchmarking program to better understand this critical issue. The benchmark program provides test specimen geometry, test setup, material properties, loading conditions, recorded measures, and observations of the test specimens[1]. The main objective of this research is to assess the beyond design seismic capacity of the reinforced concrete shear walls tested at the European Laboratory for Structural Assessment between 1997 and 1998[2] through participation in the OECD-NEA benchmark program. In this study, assessing the beyond design seismic capacity of reinforced concrete shear walls is performed analytically by comparing numerical results with experimental results.

2. Experimental Program

2.1. Test Specimen

The specimen core is a shear wall of length L = 3 m and height H = 1.2 m. The wall thickness is t = 0.2 m, as represented in Figure 1. At both ends the specimen includes short perpendicular walls (flanges) that reproduce the effect of the corresponding perpendicular walls in actual structures and contribute to the vertical stability. The bottom and the top of the specimen are made of a very rigid beam, with negligible flexibility in the vertical plan.

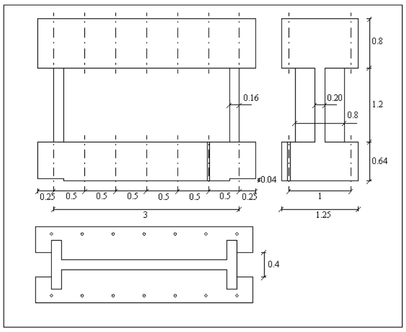


Fig. 1. Specimen geometry (unit : m)

Seismic tests were carried out on the main reaction wall using a pseudo-dynamic test program. For each specimen, the input signal is calibrated to reach the design level of the structure. Additional runs are then applied with higher intensities until the ultimate capacity is reached. For the purpose of this analytical study, only "T6" specimen out of total 12 test specimens is being used.

Actual concrete capacity of the specimen, f_{c28}^{a}, was tested on 15 cm^3 samples and is presented in Table 1. Prior to running the seismic tests, the actual eigen frequency, f^a, of the specimen was measured with low level vibrations and the corresponding elastic stiffness, K^a, was derived. Values of f^a and K^a are presented in the Table 1. The actual damping ratio, ξ^a, was concurrently measured; values are also reported in Table 1. In practice, the masses of the upper beam and loading devices and additional vertical load led to the aimed σ_n^a = 1 MPa.

f_{c28}^{a}	f^a	K^a	ξ^a	σ_n^a
MPa	Hz	MN/m	%	MPa
39.9	10.4	5348	3.7	1.01

Table 1. Observed main features of specimens

The implemented reinforcements and corresponding densities are indicated in Table 2. Anchoring lengths were equal to 50 times the bar diameter. The layout of the reinforcement of walls, including flanges, is represented in Figure 2.

	web			flange
	Φ (mm)	@ (mm)	ρ_v^a, ρ_h^a (%)	Nb x Φ (mm)
vertical	8	125	0.402	20 x 12
horizontal	10	125	0.628	/

Table 2. Detailed implemented reinforcements

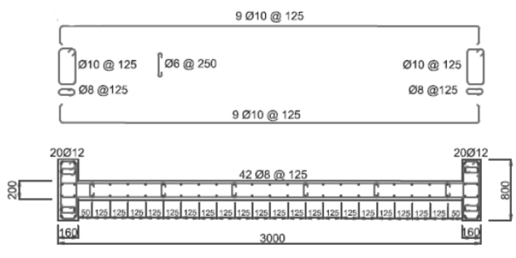


Fig. 2. Reinforcement layout of T6 specimen (unit : mm)

Average of actual yield threshold, f_e^a , obtained from rebar samples and minimum ultimate capacity, f_u^a , obtained from rebar samples are presented in Table 3.

Φ	f_e^a	f_u^a	ε_u^a
mm	MPa	MPa	%
8	594.4	672	23
10	572.8	651	25

Table 3. Mechanical features of reinforcement

2.2. Seismic Input Motions

The test consists of 4 runs of increasing level of amplification factor (α). The value of α in sequences are 1, 1.3, 1.5, 1.8. Horizontal shear forces applied to the specimen is presented in Figure 3.

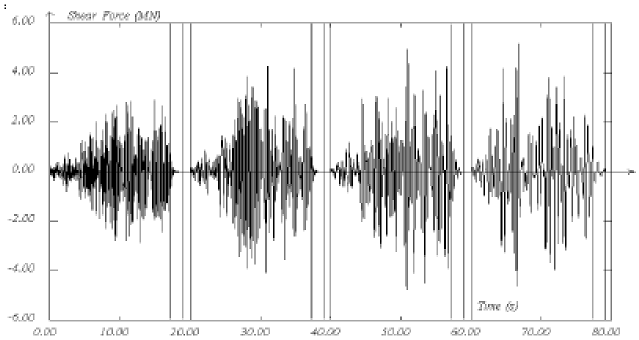


Fig. 3. Horizontal shear force for specimen runs

3. Analytical Study

3.1. Finite Element Modeling of Test Specimen

The T6 specimen was analyzed using the commercial nonlinear finite element program, ABAQUS[3]. Detailed information on the geometry and material of T6 specimen is presented in Section 2.1. The concrete was modeled with solid elements (8-noded fully integrated hexagonal elements, C3D8) and the reinforcement is modeled explicitly with truss elements (2-noded linear truss elements, T3D2). Full bond between concrete and reinforcement was assumed. The Concrete Damaged Plasticity (CDP) model in ABAQUS was used for the material constitutive model of concrete. In this study, the input parameters in the CDP model were 1) uniaxial response in compression and tension, 2) ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress (1.16), 3) ratio of the second deviatoric stress invariant on the tensile meridian to that on the compressive meridian (0.67), and 4) eccentricity (0.1) and dilation angle (55 degrees) for the flow potential, which significantly influenced the predicted response[4]. The poisson's ratio for concrete and steel was set equal to 0.2 and 0.3, respectively. The vertical stress of 1.0 MPa was applied first on the top of the concrete loading block and then horizontal displacement was applied statically up to 0.013 m (1.08% drift) at the top loading concrete block instead of horizontal loading. In this study, only monotonic static analyses (pushover analyses) were condcuted due to the lack of convergence

3.2. Analysis results

The monotonic force-displacement curve for the specimen is presented with experimental results in Figure 4. Overall, the results are comparable, although the strength degredation due to cyclic behavior of the shear wall was not simulated in the analysis. The input parameters in the CDP model influenced signficantly the response. The shear strengh of the shear wall was estimated as 5.5 MN. Figure 5 (a) and (b) show the plastic strain disctribution in web due to cracks and Von-mises stresses of the reinforcements at the displacement of 0.01m of the shear wall, repectively.

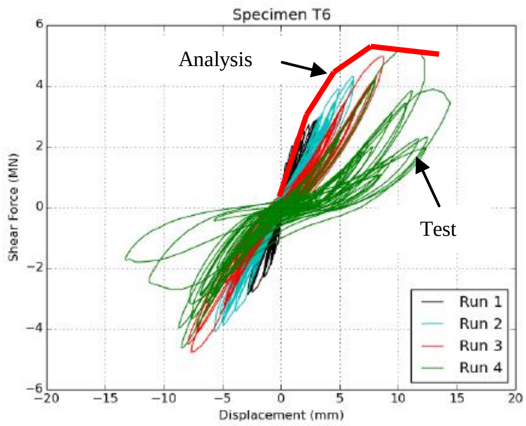
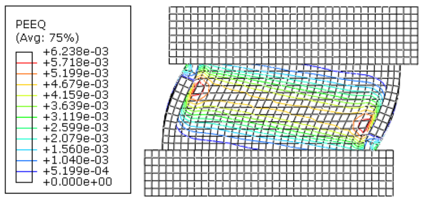
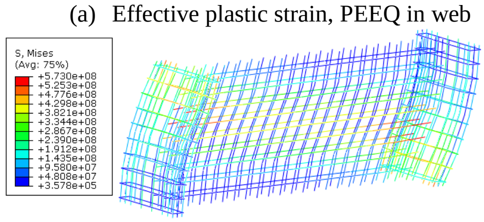


Figure 4. Lateral force vs. displacement



(a) Effective plastic strain, PEEQ in web



(b) Von-mises stresses of reinfocement

Figure 5. Deformed Shape, PEEQ and Von-mises stress

4. Conclusions

The seismic shear capacity of the reinforced concrete shear wall was predicted reasonably well using ABAQUS program. However, the proper calibration of the concrete material model was necessary for better prediction of the behavior of the reinforced concrete shear walls since the response was influenced significantly by the material constitutive model.

REFERENCES

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