PSEUDO-DYNAMIC TEST AND NON-LINEAR ANALYSIS OF A 1:10 SCALE PRE-STRESSED CONCRETE CONTAIN VESSEL MODEL FOR CNP1000 NUCLEAR POWER PLANT

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ABSTRACT :

As the final barrier to prevent leakage, the pre-stressed concrete containment vessel (PCCV) is an important component of nuclear power plant. In order to verify the seismic safety of a new type PCCV structure for the latest developed nuclear power plant CNP1000 in China, SDOF system pseudo-dynamic test and non-linear analysis of a 1:10 scale PCCV structural model were carried out. To perform the pseudo-dynamic test an artificial earthquake wave was used as the ground motion. According to the scale factor of the test model, the peak acceleration was adjusted to 1g, 2g and 3g respectively, which corresponding to 0.1g, 0.2g and 0.3g for the practical PCCV structure and the corresponding damping ratios of the structure were assumed to be 0.02, 0.05 and 0.05, respectively. The test results show that under earthquake excitation with peak acceleration 2g (the design earthquake level, corresponding to the safety shutdown earthquake level of the US criteria), tensile strains of few concrete monitoring points near the fixed bottom of the model reached the cracking strain, and the equivalent lateral stiffness of the model decreased 5% of its elastic value. When the peak acceleration was 3g, the equivalent lateral stiffness of the model decreased 14% of its elastic value. The static non-linear analysis gives the ultimate lateral load. The dynamic non-linear analysis reveals the strain distribution and the crack pattern on the cylinder of the test model. The analytical results have a good agreement with the test results. It is concluded that under the excitation of the shut down earthquake the PCCV for CNP1000 as a whole is elastic and has sufficient seismic resisting safety.

KEYWORDS: Pre-stressed concrete containment vessel (PCCV), Structural model, Pseudo-dynamic test, Non-linear analysis, Design earthquake level

1. INTRODUCTION

The new generation pre-stressed concrete containment vessel (PCCV) consisting of a cylindrical shell and a semi-spherical dome is the symbolic structure for the latest developed nuclear power plant CNP1000 in China. Since the year 2000 a cooperative research program of the new generation PCCV has been carrying out. A static internal pressure test and non-linear analysis (Chen and Qian, 2002) of a 1/10 scale pre-stressed concrete model were accomplished in 2000. Since the PCCV is required to resist earthquakes, a pseudo-dynamic test of a 1/10 scale new generation PCCV model subjected to an artificial earthquake wave was conducted in 2006. To predict the ultimate lateral load capacity and to evaluate the seismic safety margin of the PCCV, non-linear analyses were carried out with MARC finite element program (MSC, 2005).

2. DESCRIPTION OF THE TEST MODEL

The philosophy of designing the test model was to scale the prototype structure of the new generation PCCV to an acceptable size with as many representative features of the prototype as possible. The geometry scale of the test model was 1/10 of the prototype structure. Material properties, such as concrete, reinforcement and pre-stressed tendon, of the test model were the same as the material properties used for the prototype structure.

According to the elastic similitude law (Wang, 2000), the scale factors of the test model are listed in Table 1.

Table 1 Model scale factors

Item	Length	Time	Frequency	Acceleration	Displacement	Mass	Strain	Stress	Elastic modulus	Force
Scale factor	1/10	1/10	10	10	1/10	1	1	1	1	1/100

The test model consisted of a cylinder, a hemisphere dome, two vertical buttresses, a ring beam and a base mat. The cylinder of the test model was 4800mm high with an inner radius of 2000mm and wall thickness of 110mm. The hemispherical dome had an inner radius of 2000mm and wall thickness of 100mm. The equipment hatch was modeled as a 700mm diameter penetration through the cylinder wall. The plan dimension of the reinforced concrete base mat was about 5m×5m and its thickness was 650mm. To prevent local damage of the lateral loading point of the cylinder, a ring beam with a height of 500mm was cast at the cylinder-dome junction. The geometry of the test model, the global coordinate system, the cardinal azimuths and elevations used to describe the model are shown in Fig.1. The cardinal elevations are numbered 1 through 5. Given this coordinate system the buttresses are located at 0° and 180° respectively while the penetration is located at 90°.

C50 concrete was used in the construction of the test model and the measured cubic compressive strength was 65.2MPa on the day of test. Four layers of steel bars were embedded in the cylinder. In total 62 vertical pre-stressed tendons and 44 hoop pre-stressed tendons were used. The vertical tendons were anchored at the base mat and the ring beam, while the hoop tendons were anchored at the buttresses. Each of the tendons was pre-stressed to a level of 1395MPa.

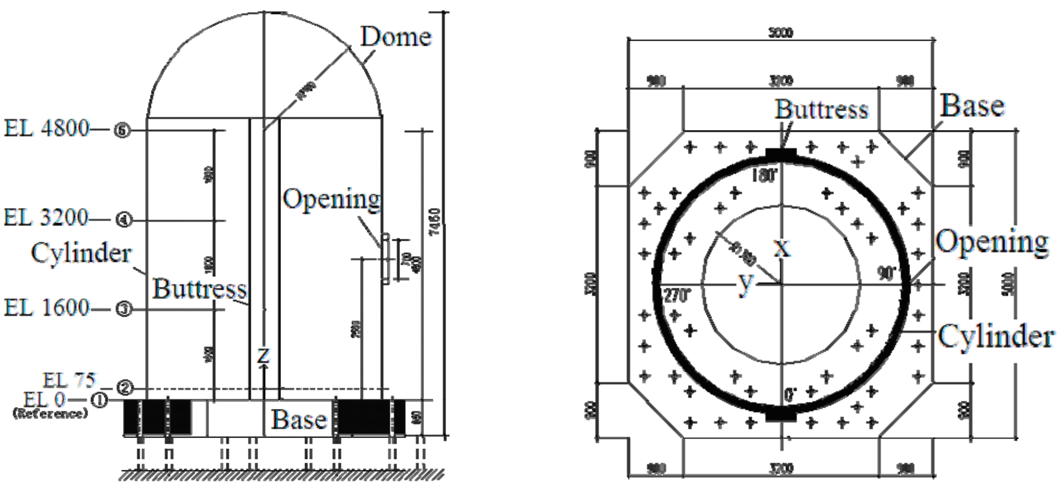


Fig.1 Geometry and coordinate system of the test model

3. OUTLINE OF THE PSEUDO-DYNAMIC TEST

3.1. Test cases and sequences

To perform pseudo-dynamic test the model was regarded as a SDOF system with the identical height of the cylinder (i.e0.0189in). The test was performed in two stages, i.e., in the Y-axis direction (along the 270°~90° line) for stage 1 (TS1) and in the X-axis direction (along the 0°~180° line) for stage 2 (TS2). For each test stage, three tests were performed and they were named as S1, S2 and S3, respectively. Fig.2 displays the test cases and the test sequences. After S3 test of TS2, a pushover test was performed and the applied maximum lateral load was 2000kN.

An artificial earthquake wave, which was the input for time history analysis of the prototype structure, was used as the ground motion for the pseudo-dynamic test, as well as for the time history analysis of the test model. The duration of the artificial wave was 25s and it was scaled to 2.5s for the test and analysis. The peak acceleration was 1g, 2g and 3g for tests S1, S2 and S3, respectively, corresponding to half times of the design earthquake SL-2 level, SL-2 level, that is corresponding to safety shutdown earthquake (SSE) level of the US criteria, and 1.5 times SL-2 level. Fig.3 shows the acceleration time history curve for test S1. The damping ratios of the test model were taken as 0.02, 0.05 and 0.05 for S1, S2 and S3, respectively.

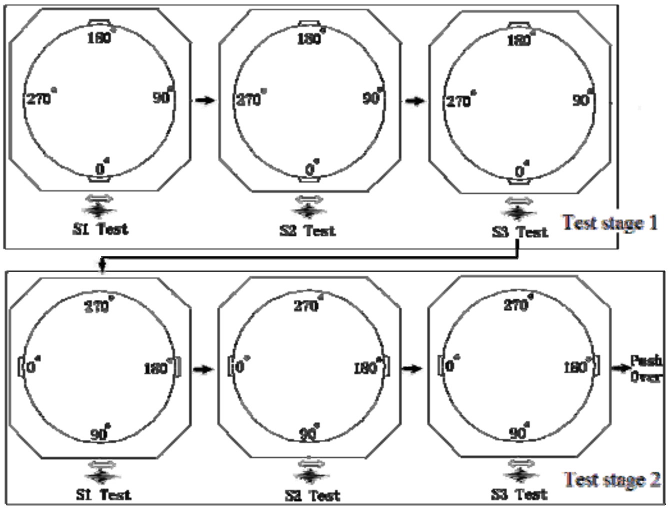


Fig.2 test procedures

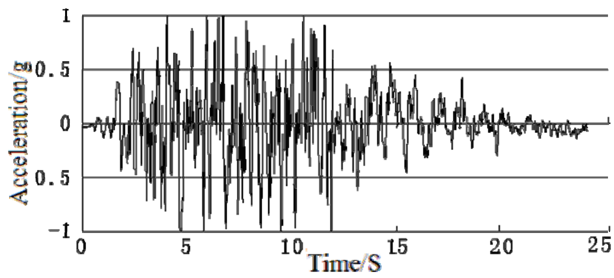


Fig.3 Acceleration time history curve of ground motion

3.2. Measurements

The instrumentation suite was designed to provide information on the overall response of the model as well as local responses which were expected to exhibit damage. 18 displacement transducers and 108 strain gauges were installed on the cylinder. The locations of these instruments were based on preliminary analysis of the test model. The arrangement of the strain gages in the stage TS1 is shown in Fig.4. Fig.5 is a photograph of the test model.

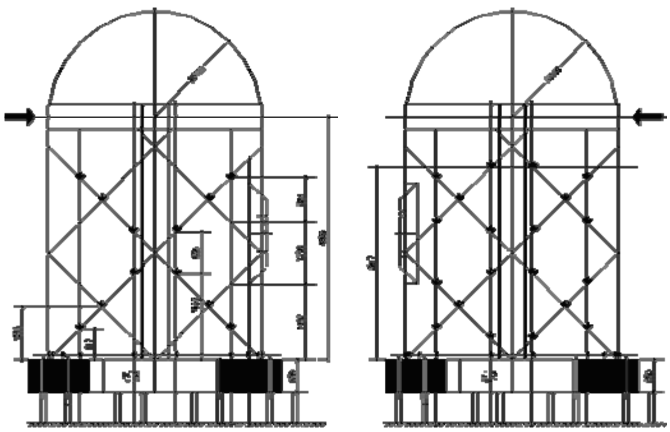


Fig.4 Arrangement of strain gauges in TS1

Fig.5 Photograph of the test model

4. TEST RESULTS

4.1. Initial stiffness and theoretical mass

Before each test stage, the initial stiffness was measured. Table 2 displays the measured stiffness and the theoretical mass concentrated at the top of the cylinder for each test stage.

Table 2 Stiffness and theoretical mass

Test stage	TS1	TS2
Measured stiffness (kN/mm)	1186	1087
Theoretical mass（t）	16.5	14.0

4.2. Equivalent stiffness and maximum responses

The lateral force time history curve under S3 in TS1 is shown in Fig.6. Fig.7 exhibits the hysteretic loops of the lateral force versus displacements in the two test stages. These displacements were obtained from the transducers installed at EL 4800 (Fig.1) and located at 3 & 9 o'clock positions corresponding to the loading direction. Obviously, the model remained approximately elastic throughout the tests.

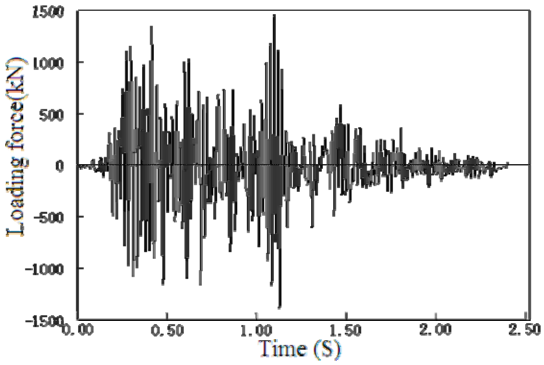
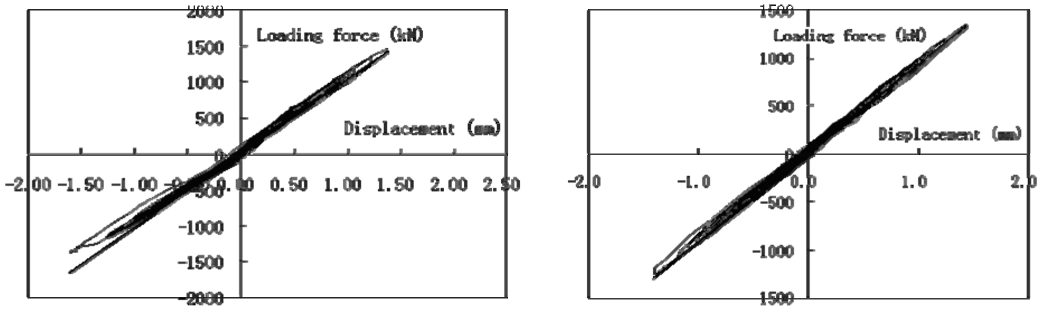


Fig.6 Lateral force time history curve under S3 in TS1



(a) TS1 (b) TS2
Fig.7 Lateral force versus displacement under S3 in two test stages

Table 3 lists the equivalent lateral stiffness of the model in each test case. In S1 of each test stage, no crack was observed. The decrease of the stiffness was in the range of (1-2)%. In S2 of each test stage, a few horizontal cracks appeared at the bottom of the cylinder. The stiffness fell down to about 95%. During S3, the stiffness in TS1 and TS2 dropped to 88% and 84%, respectively. This should due to the crack progressing in the shell. After applying the lateral force of 2000kN on the model, the secant stiffness fell down to 70%. At that time the structure yielded and residual deformation occurred.
The maximum responses are listed in Table 4. The displacement is the mean value of the displacements measured by the four transducers installed at EL 4800(Fig.1).

Table 3 Equivalent stiffness of the test model

Test stage	Equivalent stiffness (kN/mm）				
	Initial stiffness	S1	S2	S3	Pushover
TS1	1187	1176	1131	1050	_
TS2	1087	1065	1044	910	765

Table 4 Measured maximum values

Test stage	Peak acceleration	Loading direction	Push		Pull	
			Lateral force (kN)	Displacement (mm)	Lateral force (kN)	Displacement (mm)
TS1	1g	90°~270º	742	0.637	751	0.688
	2g		978	0.861	922	0.895
	3g		1466	1.432	1384	1.522
TS2	1g	0°~180º	630	0.615	655	0.637
	2g		901	0.906	835	0.860
	3g		1341	1.452	1253	1.448
Push over			2000	2.610		

4.3. Strains and cracks

The medicinal strain and the principal strain of the concrete were monitored by the strain gauges installed on the shell. The onset of cracking would be prophesied by the values of the measured strains. The cracking strain ε_t of concrete is set to 250×10^{-6} in this paper. Table 5 lists the angular locations where the peak medicinal tensile strain (PMTS) exceeds 250×10^{-6} in each test case.

Table 5 Angular locations where PMTS exceeds 250×10^{-6}

Test stage	Peak acceleration	Angular locations where PMTS exceeds 250×10^{-6}
TS1	1g	None
	2g	75°、105°
	3g	60°,75°,90°,105°,135°,255°,270°,285°,300°
TS2	1g	None
	2g	None
	3g	30°, 98°

In TS1, all the PMTS are within elastic limit under S1 test. Yet in S2 and S3 tests, there are 2 and 9 points where PMTS exceeds 250×10^{-6} respectively. The strains at these points kept fluctuating around the residual strain due to crack, as shown in Fig.8. Only after S3 test in TS2, two such points appeared. The stiffening contribution of the buttresses would result in this phenomenon.

No principal strain measured by the gauges installed at the middle part of the cylinder exceeded 250×10^{-6}. The upper part of the cylinder remained elastic and no crack was observed.

After each test stage, horizontal cracks appeared at the bottom of the shell flange (i.e. on the 90° and 270° sides for TS1, while 0° and 180° sides for TS2). All the cracks were within the elevation of 100mm from the top plane of the base mat.

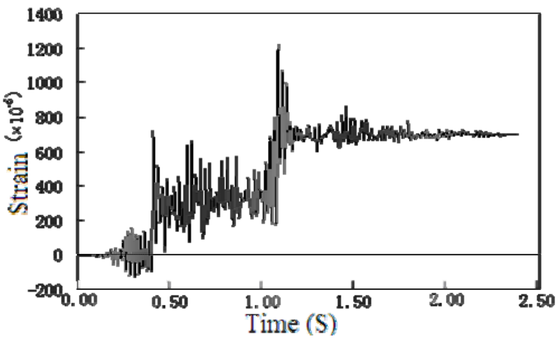


Fig. 8 Strain time history curve under S3 in TS1 (location of 270°)

5. NON-LINEAR ANALYSIS

In order to evaluate the seismic safety margin of the PCCV, non-linear analysis of the 1/10 scale PCCV model was carried out with MARC (MSC, 2002) finite element program.

5.1. Modeling

The cylinder and the dome were modeled with four-node shell element. The ring beam was modeled with 8-node solid element, and the buttresses with beam element. The reinforcing bars were considered as membrane layer of equivalent smeared thickness within the thickness of the shell element, assuming a perfect bond between the reinforcement and the surrounding concrete. The tendons were modeled with truss element, which were tied to concrete (Wang et at, 2006).

Concrete was modeled as a non-linear material throughout the deformation range, with full representation of pressure-dependent compressive plasticity and tensile cracking utilizing the shear-retaining smeared-crack modeling approach. The material properties used in the analysis are given in Table 6.

Table 6 Material properties

Concrete			
Compressive strength f_c	Tensile strength f_t	Young's modulus E	Ultimate compressive strain ε_{cu}
50MPa	4MPa	35GPa	3300×10^{-6}
Tendon		Reinforcing bar	
Yield strength f_y	Young's modulus E	Yield strength f_y	Young's modulus E
1860MPa	195GPa	500MPa	200GPa

5.2. Nonlinear time-history analysis

The artificial earthquake ground motion time history curve, as shown in Fig.3, was used as the input for the time-history analysis in Y-axis direction. The duration was scaled to 2.5s and the peak acceleration was 3g. A time step of Δt=0.0005s with a total of 5000 steps was used for the integration.

The mean-displacement of the two nodes (located at 0° and 180° respectively, at EL 4800) was considered as the horizontal displacement of the top of the cylinder. As shown in Fig.9, the displacement time history of the cylinder showed fair agreement with the measurements of the pseudo-dynamic test, indicating that the modeling assumptions and parameters were reasonable.

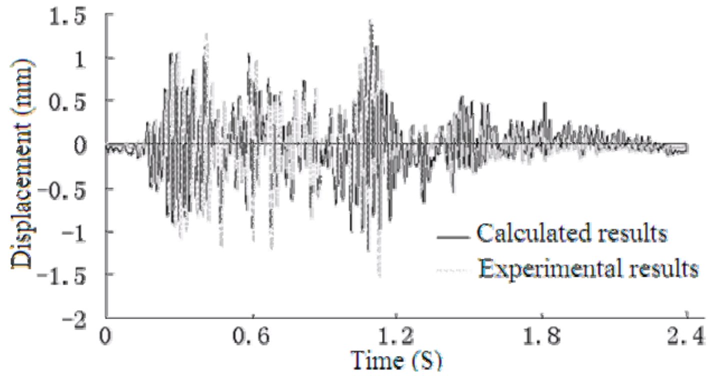


Fig.9 Displacement time history curve

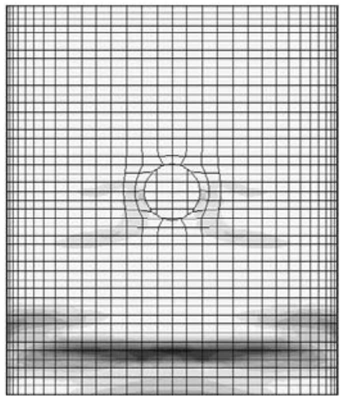


Fig.10 Crack pattern at the end of analysis

5.3. Non-linear static analysis

To perform non-linear static analysis a monotonically increased lateral force was applied at the top of the cylinder of the test model. The loading direction was in Y-axis.

The sequence of crack progressing is as follows. The first surface horizontal crack in concrete appears at the bottom of the shell flange (around 90°side) at the lateral force of 800kN. Cracks on inner surface are observed with a crack depth of 50% wall thickness at 1750kN. At 2210kN, cracks appear around the opening. Further at 4810kN, cracks occur in the area about 300mm~660mm high from the base, followed by crack density increasing. At the lateral force of 7190kN, diagonal shear cracks are observed near the buttresses. Fig.10 shows the final crack pattern at the end of analysis.

The relationship of the lateral force versus displacement is shown in Fig.11. The straight line AB represents that the structure is elastic. At point B, about 2900kN, the stiffness begins to degrade. After 5600kN(point C), vertical reinforcement yielding initiated at base-cylinder junction on 90°side followed by cracked area spreading to a larger region. This results in continuous decrease of the structure stiffness. At 8030kN (point D), the vertical tendons yielded along the azimuth of 90°. Finally, at the loading level 8760kN (point E), crushing strain of concrete was reached at the base of the cylinder within the range of 270°±40°.

There are two failure criteria for PCCV model: one is the tendon yielding criterion (Basha et al, 2003), the other is the concrete crushing criterion (ε_{cu} =3300×10^{-6}). According to the first criterion, the ultimate lateral load capacity is 8030kN, 5.48 times the maximum base shear force (1460kN) developed by earthquake excitation with peak acceleration of 3g.

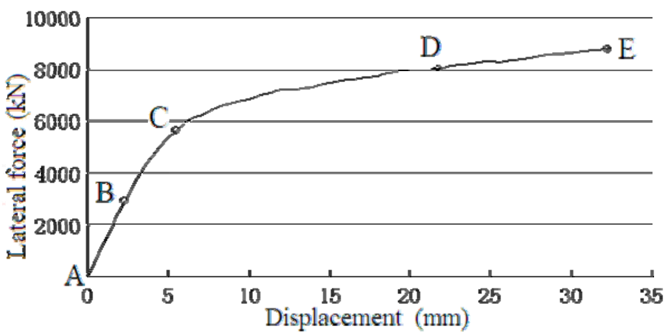


Fig.11 Lateral force-displacement relationship

6. CONCLUSIONS

The test results and the analysis results of the 1:10 pre-stressed concrete containment vessel model led to the following conclusions:

1) The PCCV model under S1 and S2 excitation in each test stage remained elastic. The equivalent stiffness fell down to 98% and 95% in S1 and S2 respectively.

2) A few horizontal cracks at the bottom of the shell flange occurred when the model subjected to S3 motion, while no crack was observed at the upper part of the cylinder. The whole model remained approximately elastic under S3. So far the equivalent stiffness dropped to 86%. After the lateral force of 2000kN (exceeding twice the peak lateral force in S2) was applied, the equivalent stiffness decreased to 70%.

3) From the experimental results of the 1:10 scale model subjected to S2 motion, it is safely concluded that the prototype would remain elastic under the design earthquake SL-2 level.

4) From the non-linear static analysis, the ultimate lateral load capacity is 8030kN, 5.48 times the maximum base shear (1460kN) developed by earthquake ground motions with peak acceleration of 3g.

5) The PCCV for CNP1000 has sufficient seismic safety margin.

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