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ASME BLAST TESTS ON PRE-STRESSED CONCRETE SLABS

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ABSTRACT

The behaviour of reinforced concrete under blast loading is well understood. However, the behaviour of pre-stressed concrete under higher levels of pre-stressing, subjected to blast loading, has not been studied extensively. A blast testing program on pre-stressed, concrete slabs was established to define design acceptance criteria for pre-stressed concrete elements for nuclear structures. The American Society of Mechanical Engineers (ASME) testing program is a part of a code provision update put in place under the Joint Task Group ASME BPVC Section III Division 2 (Nuclear Concrete Containment) and ACI 349 (Nuclear Safety Related Structures). The paper presents six tests on pre-stressed concrete slabs with the same geometry (4.88m x 4.88m x 0.27 m) but with three different designs of the reinforcement and pre-stressing. The reinforcement ratio as well as the pre-stressing levels was chosen to reflect the design of existing nuclear containment buildings as well as the design of new builds currently under construction, or to be constructed in the near future. The pre-stressing for all slabs was introduced using post tensioning with grouted tendons. Two different levels of pre-stressing were introduced in the slabs: 5 MPa and 10 MPa, typical for nuclear containments. The slabs were reinforced with two different amounts of reinforcement: 95 kg/m^3 and 220 kg/m^3. The amount of distant ANFO explosive charge was defined to obtain different target levels of support rotations for each slab design. The main response mode observed in the tests was a flexural mode with brittle failure of the concrete under compression.

INTRODUCTION

The America Society of Mechanical Engineers (ASME) testing program is a part of a code provision update put in place under the Joint Task Group ASME Boiler & Pressure Vessel Code (BPVC) Section III Division 2 Committee for Nuclear Concrete Containment and the American Concrete Institute (ACI) 349 Committee for Nuclear Safety Related Structures and project manged by ASME Standards Technology, LLC. The test program was conducted by Stone Security Engineering, PC to meet project requirements. Structural Technologies VSL performed the assembly and stressing of their post-tensioning system within the slabs. The open-air blast tests using ANFO explosive were performed by Oregon Ballistic Laboratories, LLC at the Stone-OBL, LLC test site near Bend, Oregon, USA.

Considering the properties of pre-stressing steel and the level of compression in concrete, pre-stressed concrete members should present lower ductility than reinforced concrete members under blast loading. However, the available acceptance criteria for pre-stressed concrete in conventional structures, provided in Canadian Standard Association (CSA) Standard S850-12, "Design and Assessment of Buildings Subjected to Blast Loads," and American Society of Civil Engineers (ASCE) Standard 59-11, "Blast Protection of Buildings", have much more stringent acceptance criteria than for reinforced concrete structures. The test data are not available and the technical basis for the difference is unclear. For the cases without transverse reinforcement and relatively low pre-stressing reinforcement index ($\omega_p < 0.3$, Formula (1)), the support rotation of 1 degree represents the upper limit of moderate damage and the rotation limit of 1.5 degrees represents the upper limit for heavy damage; comparing to 2 degrees of moderate damage and 5 degrees of heavy damage for reinforced concrete structures. For higher pre-stressing reinforcement index (ω_p >0.3, Formula (1)), the acceptance criteria are not provided in terms of support rotations but in ductility ratio, smaller than 1, which indicates brittle slab behaviour. This paper deals with the level of pre-stressing typical for nuclear facilities in the range between 5 MPa and 10 MPa. For lower pre-stressing levels the findings from this testing campaign may not be applicable.

Six tests on five slabs are presented in this paper with these two levels of pre-stressing and two reinforcement ratios. The thickness of the slab was defined by the necessary minimum thickness to place the pre-stressing cables in two directions and the reinforcement in two directions on both faces. The size of the slabs was defined to obtain flexural behaviour and flexural failure mode.

SLAB DESIGN

The paper presents tests on pre-stressed concrete slabs with the same geometry (4.88m x 4.88m x 0.27 m) but with three different designs of the reinforcement and pre-stressing. The level of pre-stressing used in nuclear containment buildings is between 5 MPa and 10 MPa and the passive reinforcement ratio varies between minimum reinforcement ratio for existing plants (95 kg/m^3) and higher reinforcement ratio (220 kg/m^3) typical for new builds. The targeted minimum concrete unconfined compressive strength was $f_c^{'}$ = 41.4 MPa. The amount of explosive and the stand-off distance was defined to obtain targeted support rotations of 2 and 4 degrees and to compare the level of damage. Considering the uncertainties, the results of the first tests were necessary to adjust the amount of explosive charge.

Table 1: Three different slab designs

Slab Design	Pre-Stressing Level [MPa]	Passive Reinforcement Ratio [kg/m^3]	Targeted support rotations [Degrees]
A	5	220	2 and 4
B	10	95	2 and 4
C	10	220	2 and 4

Passive reinforcement

The tests were performed using three different slab designs: A, B and C, as shown in Table 1. Slab Design B passive reinforcement is constituted of 12.7 mm diameter bars at 152 mm, each way, each face, which represents 0.1% reinforcement ratio per direction, per face. Slab Design A and C passive reinforcement is constituted of 15.9 mm diameter bars at 102 mm, each way, each face, which represents 0.23% reinforcement ratio per direction, per face. The shaped spiral reinforcement of 9.5 mm diameter with 152.4 mm spiral diameter with 5 turns of 50.8 mm pitch, was designated by the supplier as adequate. Spiral bursting stress reinforcement was provided for all the slabs. U-bars with 9.5 mm diameter were added at 152 mm around the perimeter of each slab and were placed as close as possible to the edges. At a minimum, three U-bars were placed between the trumpets for Slab Design A, and two U-bars were placed between the trumpets for Slab Designs B and C.

Pre-stressing cables

Slab Design A was stressed to approximately 5 MPa. This panel has 14 ducts (56 strands) in each direction (Figure 1). Slab Designs B and C were stressed to approximately 10 MPa. These panels had 18 ducts (72 strands) in each direction. Slabs were post-tensioned using the VSL 6-4 strand Post-Tensioning System with 15mm strands, Grade 1860 MPa complying with ASTM A416. Lower-level stress strands (

56 strands) were pulled to approximately 60% of the ultimate stress f_{pu}; higher level stress strands were pulled to 85% of f_{pu}. VSL confirmed that their systems have been validated beyond these target values and have been tested up to 95% of f_{pu}. The length of the post-tensioned strands was 4880 mm. Strands were placed in PVC flat ducts measuring 35 mm high × 86 mm wide. The ducts, which have a capacity of four strands each, were centered in the thickness of the slabs in one direction and staggered in the other direction to minimize the eccentricity of the applied force. Anticipated anchorage seating was 6.4 mm. With a strand seating loss, the anticipated force in the lower-level stress strand was approximately 118 kN; the anticipated force in the higher-level stress strand was approximately 182 kN. The ducts were grouted after the pre-stressing of the cables. The pre-stressing reinforcement index (Formula (1)) was provided form PDC-TR-06-08:

Figure 1. Pre-stressing cables and passive reinforcement in representative panel (Slab Design A).

$$\omega_p = \frac{A_{ps}}{bd_s}\frac{f_{ps}}{f_C^{'}} \tag{1}$$

Where: ω_p is reinforcement index for pre-stressing cables, A_{ps} is area of pre-stressed reinforcement in tension area, b is the member width, d_p is the depth to centre of pre-stressing steel, f_{ps} is the calculated stress in pre-stressing steel at design load, d_p is the depth to centre of pre-stressing steel and $f_C^{'}$ is the concrete compressive strength at 28 days.

TEST SET-UP

The Stone-OBL reaction structure , has maximum dimensions of 7.6 m wide, 5.8 m tall and 4.3 m deep. The walls, floor slab and roof slab are constructed of 0.610 m thick, heavily reinforced concrete. The supporting steel frame was designed to allow free rotation at two-way slab supports.

The explosive used for the testing of the panels was ANFO (i.e., ammonium nitrate/fuel oil), a widely used bulk industrial explosive mixture. ANFO consists of 94% porous prilled ammonium nitrate (AN), which acts as the oxidizing agent and absorbent for the fuel, and 6% number 2 fuel oil (FO). The density of ANFO is approximately 913 kg/m^3. The stand-off was chosen to obtain approximately uniform loading on the slab. The stand-off was the same in all tests and equal to 13.7 m. The centre of mass of the explosive was placed 1.2 m above the slab bottom, 1.7 m above the ground. The weight of ANFO explosive used in the tests varied between 770 kg and 1040 kg.

INSTRUMENTATION

The electronic instrumentation was set to trigger upon explosive detonation and consisted of several different types of sensors to document the pre-stressed, post-tensioned slab response to the blast load and consisted of :

- Strain gauges inside the test panel
- Laser displacement sensors behind the test panel
- A high-speed video camera behind the test panel
- Real-time video cameras behind the test panel and at a far distance in front of the test panel
- Reflected (P3 to P7) and free-field pressure sensors within set locations.

In this paper only displacements of the panels and the reflected pressure are presented, along with the photo of the specimens after the tests. Other data can be found in ASME ST-LLC (2017).

TEST RESULTS

Test 1, Slab Design B

The first test was performed with Slab Design B. The pre-stressing level in concrete was 10 MPa and the amount of passive steel reinforcement was 95 kg/m^3. The concrete compressive strength was $f_c^{'}$ = 52.5 MPa. The panel was subjected to an explosion of 1040 kg of ANFO at a standoff of 13.7 m.

Figure 2. Front and rear face of the slab after Test 1

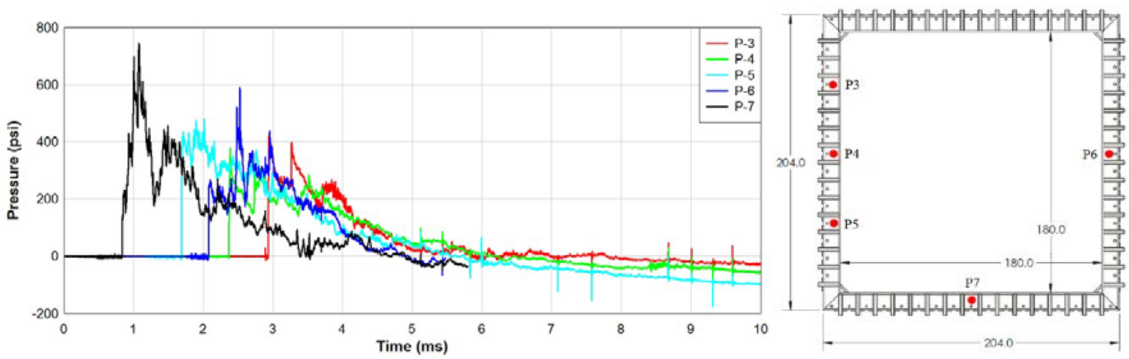

Figure 3. Measured reflected pressure P3 to P7 on the supporting steel frame during Test 1

Maximum recorded reflected pressure at the different gauges was P3: 2.97 MPa (431 psi), P4:2.6 MPa (380 psi), P5:3.3 MPa (480 psi), P6: 4.1 MPa (590 psi) and P7: 5.15 MPa (750 psi) (Figure 3). The impulses in the middle of the slab at P4 and P6 were 3.3 MPa-ms (480 psi-ms). The maximum displacement was 120 mm which corresponds to 3.0 degrees of support rotation. Residual displacement was 100 mm. There was no rebound of the slab indicating that the cables had lost the tensile forces probably due to the failure of the pre-stressing cables. The front face experienced heavy damage with concrete crushing through roughly 1/3 of the thickness of the section and buckled reinforcement. The rear face was cracked only without any concrete scabbing (Figure 2). The cable anchorage zone was damaged with through thickness cracking and a crack along the slab edge. These cracks are due to the insufficient burst reinforcement at the slab edges, which was not designed for the forces in pre-stressing cables beyond the yield stress. The crack parallel to the edge at one slab thickness may have been caused by direct shear failure due to the insufficient longitudinal reinforcement over the support lines (Figure 4). Due to the level of damage on the front face it was decided not to increase the amount of explosive and to stay limited to 3 degrees of support rotation instead of the originally targeted 4 degrees.

Figure 4. Through-thickness crack at the anchorages of the pre-stressing cables and cracked area at the edges of slab in Test 1

Test 2a, Slab Design B

Considering the concrete damage on the front face in Test 1, it was decided to reduce the amount of explosive and to see whether the concrete on the front face would stay below its ultimate limit state. The same slab design was used as in Test 1. The concrete compressive strength was $f_c^{'}$ = 51.8 MPa. The chosen amount of explosive was 770 kg of ANFO (74% of previous amount) at the stand-off distance of 13.7 m. Peak recorded pressure was in the range of 1.7 MPa (240 psi) to 3.0 MPa (430 psi). Peak recorded impulse was 3.2 MPa (470 psi-ms). The maximum measured deflection was 16 mm, which corresponds to a 0.4 degree support rotation. The slab rebounded and slab oscillations were recorded. There was no observed damage on the slab other than hairline cracks on the rear side. No through thickness cracks at the cable anchorages were observed and no crack was visible at the support zone. The behaviour of the slab was elastic. The reduction of 26% in the amount of explosive resulted in a dramatic difference in the slab response and the level of damage.

Test 2b, Slab Design B

Considering there was no visible damage on the slab tested in Test 2a, it was decided to test the same slab with an increased amount of explosive of 910 kg of ANFO (18% increase compared to the previous test), expecting the level of damage to fall in between Tests 1 and 2a. Peak reflected pressure at the different gauges was P3: 2.5 MPa (370 psi), P4: 2.3 MPa (340 psi), P5: 2.8 MPa (410 psi), P6: 3.4 MPa (490 psi) and P7: 3.8 MPa (545 psi). The impulses at the mid-height of the slab were P4: 2.2 MPa-ms (315 psi-ms) and P6: 3.9 MPa-ms (570 psi-ms). Once again, a very small variation of the loading results in a large difference in the slab response and the level of damage indicating a cliff-edge effect mainly due to the brittle concrete failure in compression and probable loss of pre-stressing in the cables. The maximum displacement was 109 mm, which corresponds to a 2.7 degree support rotation, and the residual displacement was 90 mm. The damage is, like Test 1 (Figure 6) concrete cracking on the front face and cracking on the rear face. No slab rebound was recorded, as in the first test. The cracks on the rear face on Figure 6 are highlighted with a marker. For this reason, they are more visible than the cracks shown in Figure 2. The through thickness cracking at the level of pre-stressing anchorage and damaged slab support zone are present, aimilar what is presented in Figure 4.

Figure 5. Front and rear side of the slab after Test 2a

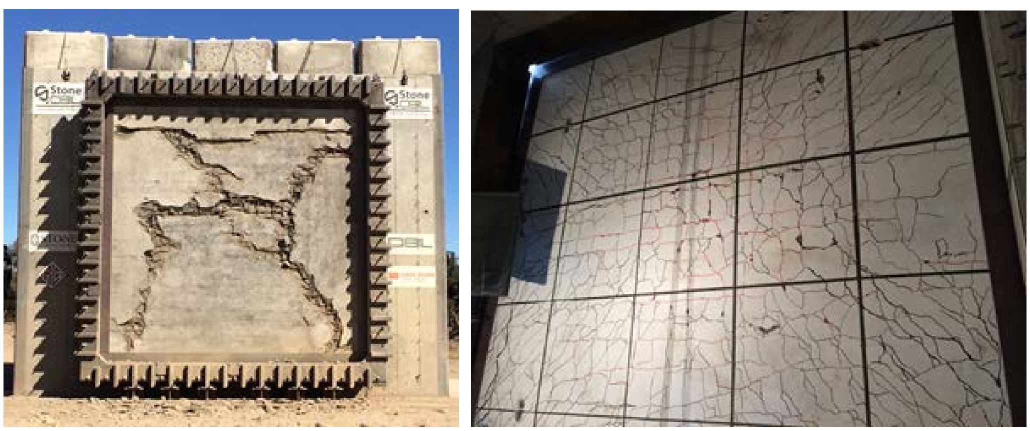

Figure 6. Front and rear side of the slab after Test 2b

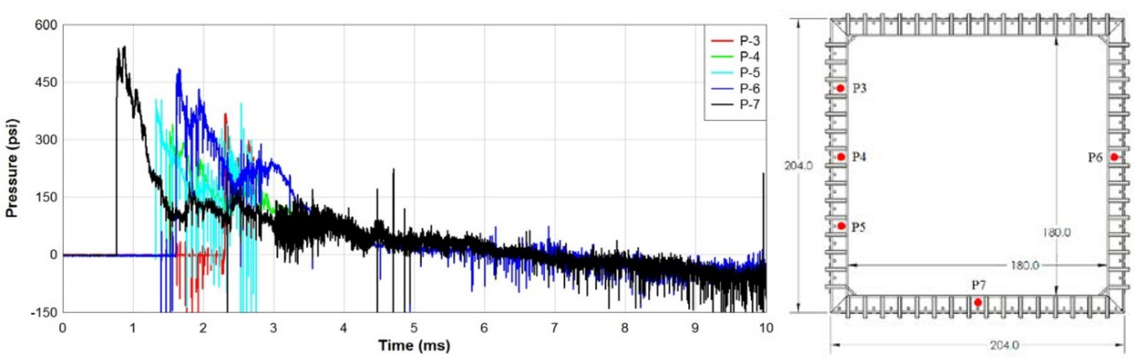

Figure 7. Measured reflected pressure P3 to P7 on the supporting steel frame during Test 2b

Test 3, Slab Design A

Test 3 was performed with Slab Design A, using 5 MPa pre-stressing and 220 kg/m^3 passive reinforcement. The concrete compressive strength was $f_c^{'}$ = 57.2 MPa. More ductile behaviour was expected than in Tests 1 and 2 due to the lower level of pre-stressing and the high amount of the passive reinforcement. The higher reinforcement ratio on the compression side is expected to improve the ductility of the slab. At the same time, higher passive reinforcement ratio was expected to improve the direct shear behaviour and reduce cracks along the slab support. The panel was subjected to an explosion of 1040 kg of ANFO at a standoff of 13.7 m, the same as in Test 1.

The peak pressures recorded ranged from P3: 3.2 MPa (470 psi) to P7: 5.2 MPa (750 psi). The positive phase impulse recorded ranged from P3: 3.0 MPa-ms (430 psi-ms) to P7: 5.2 MPa-ms (760 psi-ms). The maximum deflection recorded with the laser sensors was 88 mm, which translates to a 2.2 degree support rotation. The permanent set near the slab center point was 57 mm. The overall damage was lower than in Test 2b despite the higher amount of explosive used. This can be attributed to the higher passive reinforcement ratio in this test. However, the damage at the slab edges was very similar to Test 2b (Figure 9). The damage at the slab edges can be attributed primarily to inadequate burst reinforcement ratio for this level of pre-stressing. Higher level of passive reinforcement reduced the deflection of the slab, compared with Test 1.

Figure 8. Front and rear side of the slab after Test 3

Figure 9. Through-thickness crack at the cable anchors and a crack along the edge at one slab thickness from the edge in Test 3

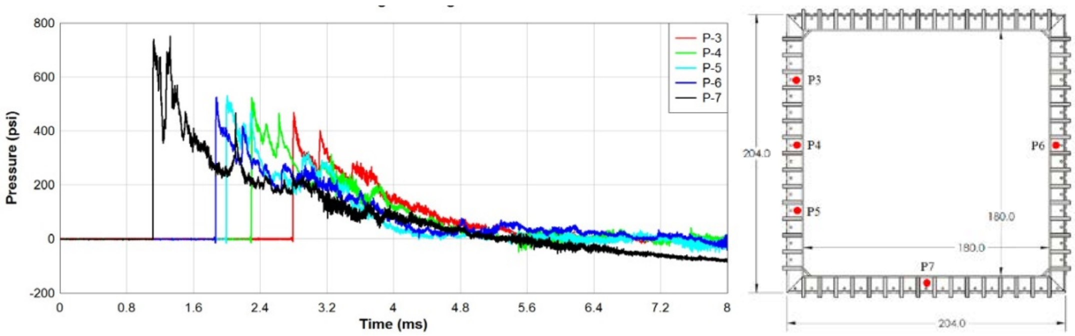

Figure 10. Measured reflected pressure P3 to P7 on the supporting steel frame during Test 3

Test 4, Slab Design C

Test 4 was performed with Slab Design C, with 10 MPa of pre-stressing and a reinforcement ratio of 220 kg/m^3. The concrete compressive strength was $f_c^{'}$ = 62.0 MPa. The ANFO amount was 910 kg. The measured maximum reflected pressure at the different gauges was P3: 2.6 MPa (380 psi), P4: 2.6 MPa (380 psi), P5: 2.6 MPa (370 psi), P6: N/A and P7: 3.0 MPa (430 psi). The impulse at the mid-height of the slab was P4: 3.0 MPa-ms (430 psi-ms). The tests were performed to compare their results with the results of Test 3, to assess the influence of pre-stressing level to the overall damage. Test 4 is directly comparable with Test 3 as the only difference is in pre-stressing level. The maximum displacement was 21 mm, which corresponds to a 0.5 degree support rotation. The slab behaved quasi-elastically with some superficial damage on the front face.

Figure 11. Front and rear face of the slab in Test 4

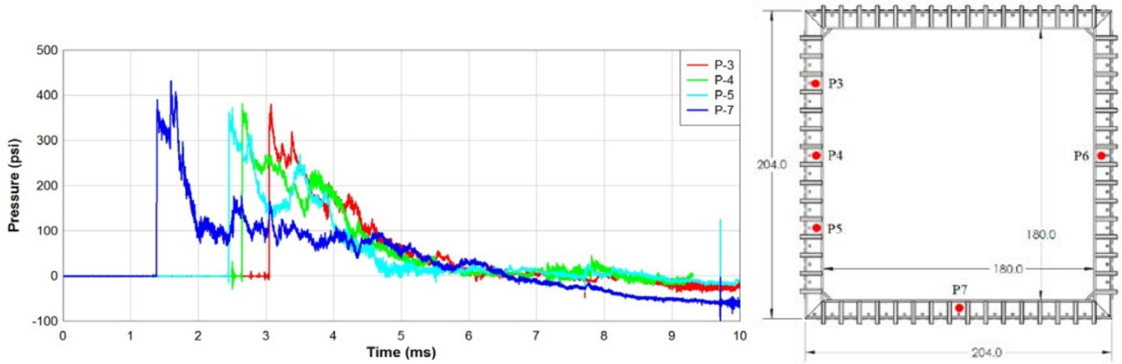

Figure 12. Measured reflected pressure P3 to P7 on the supporting steel frame during Test 4

Test 5, Slab Design C

The slab design was the same as in Test 4 and the panel was subjected to an explosion of 1040 kg of ANFO at a standoff of 13.7 m. The concrete compressive strength was $f_c^{'}$ = 50.8 MPa. The maximum reflected pressure at the different gauges was P3: 2.7 MPa (390 psi), P4: 3.6 MPa (520 psi), P5: 3.9 MPa (570 psi), P6: 5.3 MPa (770 psi) and P7: 7.5 MPa (1090 psi). The maximum impulse at the mid-height of the slab was P4: 3.3 MPa-ms (470 psi-ms) and P6: 2.2 MPa-ms (320 psi-ms). The maximum deflection recorded with the laser sensors was 85 mm, which translates to a 2.1 degree support rotation. The permanent set near the slab center point was 50 mm. The damage was similar to the damage in Tests 1 and 2b, although the maximum deflection attained was more similar to Test 3. We should note that although the pre-stressing level was different in Test 3 and Test 5, the passive reinforcement ratio was the same.

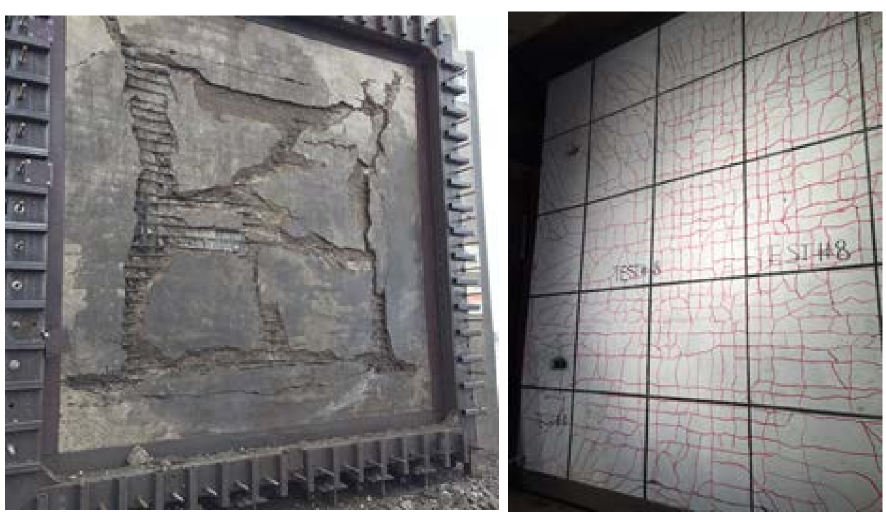

Figure 13. Front and rear face of the slab in Test 5

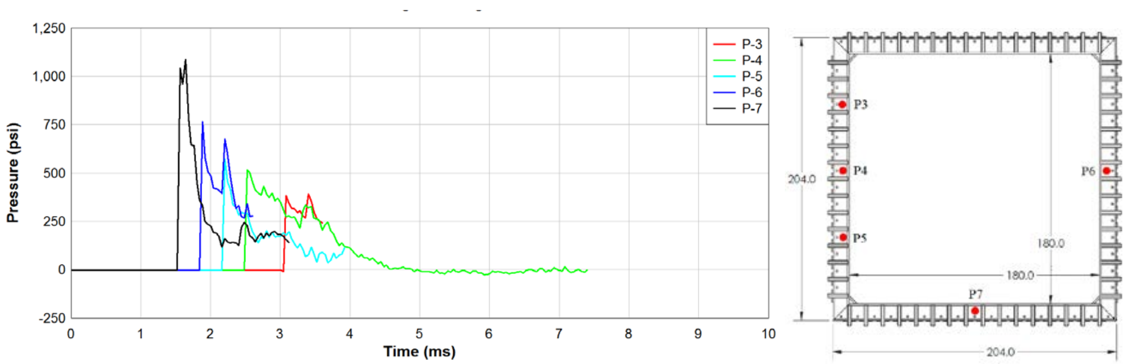

Figure 14. Measured reflected pressure P3 to P7 on the supporting steel frame during Test 5

Table 2: Recapitulated test results

Test No.	Slab Design	f_c' [MPa]	Maximum Deflection [mm]	Permanent Deflection [mm]	Support Rotation [degrees]	ANFO Weight [kg]
1	B	52.5	120	100	3.0	1040
2a	B	51.8	16	0	0.4	770
2b	B	51.8	109	90	2.7	910
3	A	57.2	88	57	2.2	1040
4	C	62.0	21	2.5	0.5	910
5	C	50.8	85	50	2.1	1040

CONCLUSIONS AND RECOMMENDATIONS

The primary failure mode for the tested slabs is flexural failure mode with concrete failure under compression on the front face. Based on the test results, the behaviour of a pre-stressed concrete slab with the pre-stressing level between 5 MPa and 10 MPa, is very different from the behaviour of a reinforcement concrete slab under blast loading. The damage is concentrated on the front face. Diagonal cracks could be observed, but the concrete damage is much wider. The depth of the disengaged concrete is roughly 7-9 cm or close to 1/3 of the concrete section. The passive reinforcement on the front face was buckled. The rear face cracked, but without back face debris or any concrete scabbing which are typical for reinforced concrete slabs under blast loading. For nuclear containment applications, it means that the liner, which is on the rear face, would not be damaged and the leak tightness would probably be preserved and the components and equipment inside the building protected. For this reason, the direct parallel with the damage levels defined for reinforcement concrete elements in PDC-TR-06-08 or in ASCE 59-11 and CSA-S850-12 are not directly applicable. However, the design criteria from PDC-TR-06-08 based on pre-stressing reinforcement index take well into account the concrete brittle failure mode under compression when high pre-stressing reinforcement index is used, as it is the case in this tsts campaign. In this test campaign all the slabs were with high pre-stressing reinforcement index (ω_p >0.3, Formula (1)), which is typical for nuclear containment buildings. Based on the performed tests under the ASME testing program, support rotations up to 3 degrees can be acceptable regarding the preservation of safety functions. However, the brittle concrete failure on the compression side remains difficult to predict as there is a sudden change from elastic behaviour to the concrete failure. With an explosive charge increase of 18%, elastic slab behaviour, with a support rotation of 0.4 degree in Test 2a, changes to concrete failure and a support rotation of 2.7 degrees in Test 2b. Based on these tests, the slabs remain elastic up to support rotations of approximately 0.5 degree. Above this limit there is a cliff-edge effect with brittle concrete failure under compression.

The damage of blast loading and soft missile loading can affect relatively large areas. However, these areas remain small compared to the size of the reactor building. In the case of greased tendons, the damaged part could be cut and reconstructed. For the case of the grouted cables this repair would be more difficult.

The secondary failure mode was at the anchorages of pre-stressed cables due to the insufficient amount of burst reinforcement provided. The tensile force in the cables due to the blast loading was above the design level at the moment of post tensioning of the cables. The width of the cracked area on the front and the rear side of the slab is roughly equal to the slab thickness. These cracks could indicate a potential direct shear failure of the slab at its supports, combined with the concrete burst due to the insufficient burst reinforcement.. However, even with more than two times higher reinforcement ratio, the crack appeared with practically the same width which is an argument that the direct shear failure was not an issue.

The authors recommend a follow up of this testing program to assess: 1) the importance and the influence of the concrete failure at cable anchorages to the overall slab behaviour, 2) the influence of the transverse reinforcement on possible concrete confinement and 3) the behaviour of the liner connected at the rear side.

ACKNOWLEDGMENT

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