ALKALI AGGREGATE REACTION IN NUCLEAR CONCRETE STRUCTURES: PART 3: STRUCTURAL SHEAR WALL ELEMENTS

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ABSTRACT

The experimental structural part of the program is focused on testing of steel-reinforced walls made with ASR concrete and evaluating their performance against similar walls made with normal concrete. A total of six walls were constructed, four made with ASR concrete and two with normal concrete. All the walls and accompanying small specimens such as cylinders and prisms were conditioned in the same environment in an especially-constructed high temperature and high humidity chamber to accelerate the reaction and deterioration. This paper presents results from structural testing of two walls, one made with ASR concrete and the other regular concrete. The walls were tested under lateral displacement excursions while simultaneously subjected to axial load simulating seismic loads.

INTRODUCTION

Alkali-Aggregate Reaction (AAR) problem is common in many structures such as bridges, roadways, airport runways, and nuclear power plants. As an example, the licence extension of Seabrook Station nuclear power plant is delayed by United States NRC because of the discovery of concrete degradation due to AAR in four buildings (NRC Safety Evaluation Report, 2012).

The Alkali-Aggregate Reaction progresses with time in concrete between the alkaline cement paste and reactive amorphous silica. The reaction uses the moisture in the atmosphere and produces a gel that keeps dilating. The dilating gel causes cracks in the concrete mass thus possibly compromising the integrity of concrete. This can cause a number of issues with regard to the performance of the concrete structures caused by deteriorating concrete properties such as lowering of tensile strength and stiffness and deterioration of bond characteristics.

A comprehensive study is currently under way at the University of Toronto that was initiated and is funded by the Canadian Nuclear Safety Commission (CNSC). It has three components that are described by Orbovic et al. (2015). The first part is aimed at investigating the material aspects of the problem (Gautam et al., 2015) while the second part deals with the structural implications of ASR. The third part is focused at developing analytical tools for simulating structural behaviour using the models developed in the first two parts of the program (Jurcut et al., 2015). This paper focuses on the structural aspects of the program and presents results from the tests of two shear walls (one made with regular concrete and one with ASR concrete) and associated control specimens. At the age of 28 days, the walls were stored in an environmental chamber for 7 months to accelerate the alkali silicate reaction. The temperature in the chamber was maintained at 50°C and the relative humidity was greater than 95%.
EXPERIMENTAL PROGRAM

Six wall specimens and a large group of control specimens were cast using two different types of concrete, one containing reactive aggregate (ASR Concrete) and the other normal aggregate (Regular Concrete). Control specimens included 21 cylinders, 3 Modulus of Rupture prisms, three expansion prisms, and three Dog-Bone specimens for each type of the concrete mix used. The concrete mix design for the shear wall and the control specimens were according to the ASTM C1293 (2008). Thus the coarse aggregate was sieved and graded to meet the gradation requirement as per ASTM C1293 (2008).

The geometry and the reinforcement of the six shear walls were identical and similar to those of the wall specimens tested by Bouchon et al (2004). Each wall specimen contained two columns as boundary elements as well as two beams, one at the top and the other at the bottom of the wall. The shear wall element of the specimen was reinforced in both vertical and horizontal direction with reinforcement ratio of 0.77% and 0.80%, respectively. Both beams and the boundary elements were designed with high reinforcement ratio to ensure that no premature failure occurred in these elements before the shear wall failure. The reinforcement details also ensured that the failure did not happen in shear-friction at the interface between the wall and the beams. See Figure 1 for a schematic of the shear wall specimen. Table 1 Error! Reference source not found. provides relevant properties of the reinforcing steel used in the shear wall specimens. Table 2 shows the concrete mix design for both regular and ASR concretes as received from the supplier.

<table>
<thead>
<tr>
<th>Rebar type</th>
<th>Area $A_s$ (mm$^2$)</th>
<th>Yield strength $f_y$ (MPa)</th>
<th>Ultimate strength $f_u$ (MPa)</th>
<th>Strain hardening $\varepsilon_{sh}$</th>
<th>ultimate strain $\varepsilon_{u}$</th>
<th>Elastic modulus $E_s$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10M</td>
<td>100</td>
<td>430</td>
<td>638</td>
<td>8,000</td>
<td>150,000</td>
<td>182,000</td>
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<tr>
<td>20M</td>
<td>200</td>
<td>465</td>
<td>550</td>
<td>15,000</td>
<td>200,000</td>
<td>190,000</td>
</tr>
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</table>

Table 1: Properties of Steel Bars

<table>
<thead>
<tr>
<th></th>
<th>ASR Concrete</th>
<th>Regular Concrete</th>
<th>Mass Ratio ASR / Regular</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total concrete volume (litre)</td>
<td>3500.0</td>
<td>2000</td>
<td>1.75</td>
</tr>
<tr>
<td>Total mass (kg)</td>
<td>8538.5</td>
<td>4879.1</td>
<td>1.75</td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>1470.00</td>
<td>840.0</td>
<td>1.75</td>
</tr>
<tr>
<td>Coarse aggregate (kg)</td>
<td>3931.76</td>
<td>2246.72</td>
<td>1.75</td>
</tr>
<tr>
<td>Water (kg or litre)</td>
<td>646.80</td>
<td>389.60</td>
<td>1.66</td>
</tr>
<tr>
<td>Sand (kg)</td>
<td>2489.94</td>
<td>1422.83</td>
<td>1.75</td>
</tr>
<tr>
<td>Alkali pallet (kg)</td>
<td>3.80</td>
<td>2.17</td>
<td>1.75</td>
</tr>
<tr>
<td>Superplasticizer (litre)</td>
<td>12</td>
<td>7</td>
<td>1.71</td>
</tr>
</tbody>
</table>

Table 2: Concrete Mix Design
This project also involved the construction of an acceleration chamber capable of maintaining up to 60°C temperature and 95% or higher relative humidity. This chamber was used to accelerate the effects of ASR on the shear walls and control specimens. All the specimens placed in the chamber were monitored every month to measure the damage caused by the ASR. The temperature of the chamber was maintained at...
50°C with relative humidity of 95%. Strains of internal reinforcements in the shear wall specimens were monitored and measured monthly. Many of the strain gauges did not seem to show accurate measurements likely due to high temperature and high humidity. In general, the walls with ASR aggregate showed higher strains than normal concrete walls. For example, after six months of being in the chamber, the maximum strain measured from the horizontal steel in the regular concrete walls is about 0.0049 while it is over 0.0072 in walls with ASR concrete. Similarly, the maximum strain in the vertical bars was measured to be 0.0036 and 0.0066 in the regular and ASR walls, respectively.

It was decided to test the shear walls in two phases; moderate damage phase and severe damage phase. Moderate damage is referred to as the stage of damage where ASR cracks on the specimens can be identified by visual inspection. Before conducting the structural tests for the first phase, surface of the shear wall specimens were polished to clearly illustrate any cracks due to ASR. As seen in Figure 2 the shear wall specimen containing reactive aggregate showed significant signs of ASR reaction which included alkali spots over the specimen’s surface along with map cracking and silica gel. On the other hand, the shear wall specimen constructed with regular concrete showed none of the above mentioned signs and had a very smooth and fine surface as shown in Figure 2.

![Figure 2 - Surface of the Regular shear wall (Left) and ASR shear wall (Right) before testing](Image)

Results from physical observations and monitoring of the specimens indicated that the first phase of structural testing could initiate in about 6 months after conditioning of the specimens in the curing chamber. For the first phase, two shear walls, one Regular and one ASR along with some of their control specimens were due for the structural testing.

**TESTING**

The ultimate capacity of the shear walls was predicted to be in the range of 1,200 kN based on the analysis using VecTor2 software. One actuator of 1,000 kN capacity was thus not suitable for testing. A larger actuator of 2,000 kN capacity was available but its use required extensive preparatory work that could have delayed testing significantly. It was thus decided to use two 1,000 kN actuators, one on each side of the wall specimen. Each actuator was supported by a custom built frame at one end and a small hydraulic jack placed at middle point of the actuator. This support system allowed the actuator to move freely to allow shear deformation of the specimen (Figure 3). To apply the axial force on the walls, a third frame was installed around the specimen holding another hydraulic jack as shown in [Error! Reference source not found.4](Image). This jack was used to maintain a constant axial load of 800 kN throughout the test. The shear wall specimen was anchored to the floor, and restrained from both side to prevent any slippage.
Seven LVDTs were used to measure the horizontal displacement along the height of the specimen from bottom beam to the top beam on both sides of the specimen. The Force – Displacement plot was obtained using the summation of the forces from both actuators against displacement of top of the shear wall (bottom of the top beam) with respect to the top of the lower beam.

![Figure 3: Schematic drawing showing the test setup for shear walls](image)

Figure 3: Schematic drawing showing the test setup for shear walls

![Figure 4: Test setup of a shear wall showing the top frame and loading beam](image)

Figure 4: Test setup of a shear wall showing the top frame and loading beam

The specimens were loaded using the two actuators working simultaneously. The rate of loading began with 0.005 mm/sec and was increased to a maximum of 0.15 mm/sec as cycles progressed. The first two cycles applied 0.2 mm lateral displacement in the plane of the wall in each direction and the subsequent cycles were at maximum displacements of 0.4, 0.6, 0.8, 1, 1.4, 1.8, 2, 2.5, 3, 4, 4.5, 5.5, 6, and 7 mm. For each displacement two complete cycles were applied. For the Regular shear wall, after completing 2 cycles at 7 mm, the load was increased monotonically until the wall could not maintain the axial load of 800 kN which happened at a displacement of 8.2 mm. In the case of the ASR wall, the specimen was pushed in one direction by 7 mm in the first cycle and then in the opposite direction. During this part of the cycle, the axial load started to drop as the lateral displacement increased and needed constant increase in load. At 7 mm displacement, it was decided to continue load monotonically and at 7.1 mm, the wall failed to maintain the axial load. The test was terminated at that point.

RESULTS AND OBSERVATIONS
In the Regular concrete specimen, the first cracks appeared on the shear wall at displacement of about 1.4 mm. These cracks had a width of up to 0.4 mm. As cycles progressed, width of these cracks increased and more new cracks were introduced in the shear wall. Cracks on the columns were noticed at a displacement of about 3.5 mm. At 5.5 mm displacement, a huge crack was observed in the columns and at 6 mm displacement, spalling of concrete in column areas was observed. Within this cycle significant spalling of concrete on shear wall panel was noticed accompanied by a diagonal crack which became wider as displacement was increased. The wall carried the maximum load at about 7 mm deflection beyond which it showed a softening response while still maintaining the 800 kN applied axial load and lost its capacity at a displacement of about 8.2 mm. Figure 5 shows the Regular shear wall at failure.

The ASR shear wall specimen cracked initially at a displacement of about 1 mm. These cracks had widths of 0.1 mm to 0.3 mm. As displacement was increased, more cracks opened up along with a noticeable diagonal crack within the shear wall panel. No spalling of concrete was noticed on the shear wall. However, spalling of the concrete in the column area was observed at 6 mm displacement. At this stage the diagonal crack in the wall opened significantly and eventually the specimen failed to maintain the axial load when the lateral displacement was 7.1 mm. The specimen did not display any significant descending branch of the response. See Figure 5 for the ASR shear wall at failure.

![Figure 5: Regular shear wall (Left) and ASR shear wall (Right) at failure](image_url)

The Regular Shear wall was tested at an age of 240 days and the ASR shear wall was tested at the age of 260 days. Both specimens failed in shear after developing a diagonal crack along the diagonal of the shear wall panel. The maximum capacity of the regular shear wall was recorded as 1180 kN and the maximum capacity of the ASR shear wall was recorded as 1354.5 kN. In the Regular shear wall, buckling of the vertical reinforcements close to boundary elements was observed. However, in the ASR shear wall, no buckling of reinforcement was noticed. Lateral force vs. top deflection responses of the two shear walls are shown in Figure 6.
The mechanical properties of the concretes used in both Regular and ASR shear walls were obtained by various testing methods such as a direct tensile test, simple beam with third-point loading, and standard cylinder test. Figure 7 illustrates the results from standard cylinder test of both Regular and ASR cylinders over time. The modulus of elasticity of the concrete was also measured based on the stress-strain curves obtained from the compressive strength test of cylinders. Results from this test is shown in Figure 8.
Figure 8: Modulus of Elasticity of Regular and ASR cylinders over time

Table 3 summarizes information regarding the mechanical properties of the concretes used in the two tested shear walls just before walls were tested. For this phase of tests, the age of Regular and ASR wall specimens were about 240 and 260 days respectively of which they were stored in the acceleration chamber for 210 days.

Table 3: Mechanical Properties of the concrete used in shear walls

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Strength $f'_c$ (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Modulus of Rupture (MPa)</th>
<th>Modulus of Elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regular</td>
<td>79.0</td>
<td>4.76</td>
<td>7.26</td>
<td>47150</td>
</tr>
<tr>
<td>ASR</td>
<td>63.7</td>
<td>3.24</td>
<td>4.64</td>
<td>35750</td>
</tr>
</tbody>
</table>

DISCUSSION

Results from the control specimens have shown that compressive strength of ASR concrete did not decrease over time. Results show that although the compressive strength of the regular concrete on the day of structural testing was measured to be about 24% higher than that of the ASR concrete, the shear capacity of ASR wall was about 14% higher than that of regular shear wall. Its failure was, however, observed to be more sudden than that of the regular wall. Obviously, the reduced compressive strength of the ASR concrete does not explain higher shear strength of the ASR shear wall.
On the other hand, results from the control specimens showed that the modulus of elasticity, modulus of rupture, and direct tensile strength of the regular concrete kept increasing over time while same properties in ASR concrete were reduced beyond a certain age while its compressive strength still increased. Figure 9 compares various properties of the two concretes used in the walls at the age of 240 days.

Based on the results from structural testing and control specimens, it is believed that confinement played a critical role in determining the behaviour of ASR wall. In both walls, the top and bottom beams along with the two side boundary elements (columns), induced confinement in the specimens during testing. In the ASR specimen, this confinement was further enhanced due to the restraint developed as a result of expansion of concrete. As the shear wall panel is restrained from all sides, the internal expansion of concrete due to ASR will add significantly to the confinement effect. Since expansion of ASR concrete occurs during curing and conditioning, the reinforcement in the shear wall will undergo a pre-stressing state thus resulting in a stiffer wall response compared with that of Regular concrete wall.

These two factors are most likely responsible for the increase in the ultimate strength and increased initial stiffness of the ASR specimen compared with the Regular wall. The mode of failure is also accordingly affected differently by the state of confinement in both walls.

**CONCLUDING REMARKS**

In the first testing phase of the study, the behaviour of shear wall affected by Alkali Silica Aggregate was investigated. Two shear walls, one Regular and one ASR were constructed and tested in the Structures Laboratories of the University of Toronto under lateral displacement excursions while simultaneously subjected to axial load simulating seismic loads. Observations from the tests showed that there are several factors that affect the structural performance of the wall specimens. It was found that factors such as confinement and pre-stressing of reinforcement due to ASR expansion resulted in ultimate capacity of the ASR shear wall to be about 14% higher than that of the regular shear wall specimen. However, failure in ASR shear wall was more sudden than that of the regular wall.

Results from control material specimens showed that ASR concrete gains strength over time at slower rate in comparison with Regular concrete. However, the modulus of elasticity and tensile strength in ASR
concrete depreciated significantly with time beyond a certain age while the same properties in the Regular concrete kept increasing. These results indicate that most concrete codes such as ACI and CSA, if applied to ASR concrete, will not accurately capture its mechanical behaviour and properties. In these and other codes, most of the mechanical properties of concrete including the elastic modulus and the tensile strength are obtained through compressive strength of the concrete. Therefore, a different approach will be developed to capture the mechanical properties of concretes affected by ASR.

ACKNOWLEDGEMENTS

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