SEISMIC PERFORMANCE OF BRIDGE PIERS MADE OF RECYCLED CONCRETE -

AN EXPERIMENTAL STUDY

by

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SEISMIC PERFORMANCE OF BRIDGE PIERS MADE OF RECYCLED CONCRETE - AN EXPERIMENTAL STUDY

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Abstract

Many of the civil infrastructure facilities in Canada have passed their service lives, and are structurally or functionally deficient. Some of these structures will need to be demolished and rebuilt. However, demolition will generate waste whereas new construction will require a huge amount of raw materials. Recycling of construction and demolition (C&D) wastes could be a sustainable solution as it will reduce the economic burden associated with concrete waste disposal, which increases the pressure on landfill capacity and the negative environmental impact. Reusing waste concrete as coarse aggregate, often termed as recycle coarse aggregate (RCA) in the construction industry could reduce the negative environmental impact associated with natural aggregate extraction and transportation. However, before this new recycled concrete is introduced in real life project, it is essential to investigate its properties and performance in structural elements.

This study experimentally investigates the seismic performance of reinforced concrete bridge piers made of recycled coarse aggregate (RCA). The conducted test compares the performance of bridge piers made of 100% and 50% RCA with that of 100% natural coarse aggregate (NCA) bridge pier. The bridge under study is a major route bridge located in Vancouver area and is designed as per the Canadian standard. Then the specimen was scaled down to 1/3 of the original bridge pier. The scaled specimens had 300 mm diameter and 1730 mm height. The specimens were tested under quasi-static reverse cyclic loading. Each specimen was subjected to a combination of constant axial loading equal to 10% of its axial capacity and cyclic lateral loading developed by a computer controlled hydraulic actuator. Performance of recycled aggregate concrete (RAC) piers were compared in terms of lateral load capacity, drift, ductility, and energy dissipation capacity with those of conventional concrete pier. Results of this study showed that RAC specimens had
improved ductility and lower residual displacements than the control specimen. Moreover, the energy dissipation capacity for 100% RCA specimen was about 11% higher than the control specimen. The results showed that RAC can be effectively used in structural elements in seismic regions.
Lay Summary

About 40% of the civil infrastructure including highways bridges in Canada have passed their service lives, thus some of them will need to be demolished and rebuilt. This will generate waste and require a large amount of raw materials. Recycling of concrete waste is sustainable solution for this problem. This research is one of few experimental studies investigating the seismic performance of reinforced concrete bridge piers using various recycled concrete aggregate replacement ratios (0%, 50% and 100%). Three 1/3-scaled pier specimens where tested under reversed cyclic lateral loading. The study revealed that specimens made of recycled concrete aggregate yielded similar seismic performance compared to conventional concrete. The use of recycled concrete waste showed high potential in constructing concrete structures for high seismic regions.
Preface

Major portions of this study will be submitted for possible publication in peer-reviewed technical journals and conference proceedings as listed below. The author carried out all experimental work, analyses of results, and writing of the initial draft of all papers listed below. The contributions of his research supervisor are consisted of providing guidance, supervision, and helping in the development of the final versions of the publications.

List of publications related to this thesis

A portion of Chapter 3 and 4 will be submitted to the 16th European Conference on Earthquake Engineering (16th ECEE), to be organized in Thessaloniki, Greece between 18-21, June 2018

AL-Hawarneh, M. and Alam, M.S. “Experimental investigation of seismic performance of 100% and 0% RCA bridge”. I wrote the manuscript which is being further reviewed and edited by Dr. Alam.

A portion of Chapter 3 and 4 will be submitted to the Journal of Engineering Structures, 2018.

AL-Hawarneh, M. and Alam, M.S “Experimental investigation of seismic performance of bridge piers using various RCA percentages”. I wrote the manuscript which is being further reviewed and edited by Dr. Alam.
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I have been fortunate to get the opportunity to work with an excellent group of graduate students in the research group who have offered technical knowledge, lively discussions, and friendship. I offer my enduring gratitude to the assistance of UBC Structures Laboratory Technicians especially Alec Smith and Kim and Tim for assistance with the test setup and specimen preparations.

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Dedication

To my family,
Without whom none of my success would be possible
Chapter 1: Introduction

1.1 General

Infrastructure projects are a critical foundation for a country’s economic development. One of the most important Infrastructure projects are transportation projects. For instance, in 2016 the ministry of transportation in Ontario spent about $6 billion on infrastructure projects (Ministry of Transportation of Ontario 2016). However, Huijbregts (2012) reported that around 40% of the civil infrastructure in Canada have passed their service lives, and are structurally or functionally deficient. Some of these structures will need to be demolished and rebuilt. However, demolition will generate waste whereas new construction will require a large amount of raw materials which could rise multiple environmental and economic concerns (Huijbregts 2012). It is estimated that, construction and demolition (C&D) wastes comprise about 27.5% and 29% of the total municipal solid waste in British Columbia and Ontario respectively (Statistics Canada 2008). In Canada, concrete waste occupies the largest portion among all the C&D wastes (Huda and Alam 2014). This generates financial burden associated with concrete waste disposal and also, increases pressure on the landfill capacity. Therefore, it is very important to find a sustainable way to utilize these concrete wastes. Using recycled concrete waste as a substitution of natural resources used in industry could be a solution for that problem.

One common way of reusing concrete waste is to crush it to smaller sizes similar to natural coarse aggregates and remove all deleterious materials thereby producing recycled concrete aggregate (RCA). This manufactured aggregate could replace natural aggregate in construction projects. Incorporating RCA with natural aggregate reduced the costs of infrastructure projects by 10% to 20% in Ontario (Richmond 2013). However, RCA applications are still limited to road construction and non-structural application due to lack of information about the behavior of
structural elements made of recycled aggregate concrete (RAC). Knowing RCA influence on structure’s behavior could expand the usage of RCA, thus saving more construction projects costs. Therefore, various studies (Corinaldesi, Giuggiolini and Moriconi 2002; Katz 2003; and Alam, Slater, and Billah 2013) studied the material properties of RAC. Results showed that RAC had lower performance than conventional concrete but, still met the minimum requirements. Other researchers investigated the performance of structural elements made with RAC such as beam, column, and frame. However, very limited studies focused on the seismic performance of RAC structural elements. Highly seismic areas are more prone to face damages on infrastructures aggravating the demolition and rebuilding process, hence increases the demand of incorporating RCA in structural applications. Currently, most of the concrete design codes do not incorporate RAC element design. Therefore, this a need to experimentally investigate the seismic performance of bridge piers made with RAC.

1.2 Objective of the Study

The aim of this study is to provide deeper insights on the applicability of using RCA as a sustainable solution to replac natural aggregates for infrastructure projects including bridges in highly seismic regions. Currently, there are limited studies focusing on the behavior of structural elements made of RAC. Therefore, there is a need for more investigation in this area. In addition to that, there are no guidelines for designing and constructing structural members using RAC. The objective of this study is to determine the applicability of using RCA as a full replacement of natural coarse aggregate (NCA) in structural elements and assess the adequacy of the CSA S6-14 code for designing bridge piers made with 100% RCA replacement. This study seeks to provide a
comparison between the behavior of RAC and conventional concrete elements using experimental analysis. In particular, this study is performed to achieve the following objectives:

- Studying the influence of RCA replacement ratio on the seismic performance of the bridge piers, especially the performance of bridge piers with full RCA replacement.
- Comparing the seismic performance of conventional concrete bridge pier with that made of RAC experimentally.
- Evaluating the applicability of current CSA S6-14 code in designing RAC bridge piers for highly seismic regions.

1.3 Research Significance

Sustainable construction and infrastructure are getting higher importance in Canada’s Science & Technology Strategy. According to Mirza (2007), about 60% of Canada’s infrastructure has reached about 80% of its service life which will require $123 billion to sustain its workability by repairing and rebuilding. This will produce a huge amount of C&D wastes creating huge pressure on the landfills. The present study intends to exploit the use of concrete wastes as aggregates in new concrete production which will help to preserve substantial amounts of natural resources by compensating some of NCA demand with RCA and thus, improving the gross domestic product (GDP).

This study focusses on RAC implementation in highly seismic areas which would have a significant impact on the reconstruction effort of post-earthquake events by providing easily assessable and low-cost construction material. In addition to that, it helps in reducing the amount
of concrete waste going to landfill. All these advantages of implementing RAC in seismic regions will have huge positive environmental and economic impact (Donalson, Curtis & Najafi 2011). Moreover, it aims to provide important information on the behavior of structural elements made of RAC. The outcome of this study will provide structural designers, contracting companies, local municipalities and building codes committees more information regarding the performance of RAC elements and whether the current Canadian Standard Association (CSA) codes are satisfactory for designing structural elements with RAC.

### 1.4 Thesis Outline

This thesis is organized into five chapters. Chapter 1 gives an overview of the research. Chapter 2 provides a general introduction to RAC and literature review on RCA properties and RCA concrete in structural element. More particularly, this chapter discusses the seismic behavior of different RAC structural elements. It highlights the research gaps in the available literature on the seismic performance of RAC bridge pier and thereby, sets the research objectives. Chapter 3 describes the experimental procedure along with the results of aggregate properties. Chapter 4 present the results of material properties used in constructing test specimens including steel bar and different concrete mixes. Moreover, it discusses the seismic behavior of 1/3 scale bridge pier specimens with different RCA replacement ratios. Chapter 5 presents the conclusions derived from this study and provides recommendations for future research directions.
Chapter 2: Literature Review

2.1 General

As environmental preservation is gaining increasing attention all over the world, this led to an increasing demand for sustainable materials in construction. The idea of using recycled concrete as aggregate dates to the period after World War II, when large numbers of buildings and infrastructures were damaged resulting in huge demand of aggregates required for repairing and reconstruction, as well as the availability of large quantities of concrete debris from damaged structures (Miyamoto, Gilani, and Wong 2011)

Using concrete waste as an aggregate for producing new concrete as, RAC could preserve natural resources and eliminate efforts related to manufacturing processes such as excavation/blasting and transportation to some extent, thus reducing the energy required in producing concrete which makes it greener and more sustainable solution (Tabsh and Abdelfatah 2009). Furthermore, it could reduce the cost associated with deposing concrete waste such as material handling, dumping.

This chapter presents a comprehensive review of the existing literature on RAC, in particular, various RAC properties, the comparison between fresh and hardened properties of natural aggregate concrete (NAC) and RAC and their durability, mechanical and physical aspects. Furthermore, it provides a detailed summary of existing investigation of using RAC in structural elements, especially under simulated seismic loading.

2.2 What is RCA

Demolishing concrete structures (i.e., concrete pavement, buildings, bridges etc.) produces huge quantities of concrete pieces with various sizes. Then these pieces are separated from any existing steel reinforcement using hydraulic shears, torches, and electromagnets. According to ACI
Committee 555, RCA production process includes removal of all deleterious materials including oil droppings, wood, glass, plastic, steel, clay, plaster etc. Once the deleterious materials are removed, a jaw crusher reduces the concrete debris pieces to adequate particle size distribution required for quality RCA concrete production (Lamond 2001). Recycled aggregates could be classified according to ASTM C 33 or CSA (CSA A23.1-09) standards. Afterwards, RCA particles should be tested to find its properties such as strength, water absorption, specific gravity, sulphate content and alkali-silica reaction potential conforming to CSA and ASTM standards prior to use as an aggregate (Butler 2012). A closer look on the aggregate can show the difference between NCA and RCA. Figure 2.1 represents images of natural and recycled coarse aggregates used in this study. Figure 2.2 demonstrates microscopic images of different types of coarse aggregate used in this study. These figures showed that natural aggregate had few pores and cracks under microscope. On the other hand, recycled coarse aggregates had higher sized larger number of pores and cracks. This is one of the main reasons responsible for degradation of RCA aggregate quality compared to natural aggregates which may affect the RAC mechanical and durability properties (Huda and Alam 2014).

Moreover, mixture proportioning design should account for density and water absorption of RCA to ensure achievement of target strength and workability. Tam and Tam (2009) suggested that combining RCAs from different sources during crushing and classification process could influence its overall quality which limits its application. Nagataki et al. (2004) experimentally showed that crushing concrete blocks using a jaw crusher and an impact crusher followed by further mechanical grinding could produce high-quality coarse RCA.
Figure 2.1 (a) Natural coarse aggregates, (b) Recycled coarse aggregates

Figure 2.2 Microscopic view of different types of coarse aggregate magnified 100 times

a) natural coarse aggregate, b) recycled coarse aggregate
2.3 Properties of Recycled Aggregate

Concrete properties are significantly influenced by aggregate properties as it occupies the largest portion of the concrete volume. RCA usually comes from different sources and sometimes the source is unknown which causes high uncertainty in getting exact idea about (RAC) quality. Using RCA in new concrete would be very challenging because it comes from various sources which might contain some impurities along with the adhered mortar content. This could influence RCA properties hence, difficulty in accurately predict new concrete properties (Smith 2009). Because of that, the German committee of the reinforced concrete structure set limits for the maximum permissible deleterious ingredients that RCA could contain (Dhir et al. 1998). Table 2.1 shows the permissible limits of various permissible deleterious ingredients that can be present in RCA.

<table>
<thead>
<tr>
<th>Substance</th>
<th>As</th>
<th>Pb</th>
<th>Cd</th>
<th>Cr</th>
<th>Cu</th>
<th>Ni</th>
<th>I</th>
<th>Zn</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limit (μg/l)</td>
<td>50</td>
<td>100</td>
<td>5</td>
<td>100</td>
<td>200</td>
<td>100</td>
<td>2</td>
<td>400</td>
</tr>
</tbody>
</table>

2.3.1 Gradation, Shape, and Texture

RAC properties are significantly influenced by the RCA properties such as gradation, shape, and texture. These RCA properties could significantly vary with variation in sources of RCA. According to Katz (2003) crushing strength and age of parent concrete do not have an influence on attached mortar content and gradation of RCA. The primary crushing phase includes reducing RCA size to 50 mm and removal of all metallic impurities right before starting the secondary phase. In the secondary crushing phase, RCA particle sizes are furthermore reduced to about
14-20 mm (Corinaldesi, Giuggiolini, and Moriconi 2002). Table 2.2 shows the variation in attached mortar contents with various RCA sizes.

Table 2.2 Variation in attached mortar contents with various RCA sizes

<table>
<thead>
<tr>
<th>Particle size</th>
<th>Attached mortar (by volume)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>20-30 mm</td>
<td>20%</td>
<td>BCSJ 1978</td>
</tr>
<tr>
<td>14-20 mm</td>
<td>25%-6.5%</td>
<td>Katz 2003</td>
</tr>
<tr>
<td>8-16 mm</td>
<td>40%</td>
<td>Hansen and Narud 1983</td>
</tr>
<tr>
<td>4-8 mm</td>
<td>60%</td>
<td>Hansen and Narud 1983</td>
</tr>
</tbody>
</table>

2.3.2 Specific Gravity

The typical specific gravity value for NCA is around 2.7. However, RCA has lower specific gravity compared to NCA. Salem, Burdette and Jackson (2003) and Katz (2003) showed that adhered mortar on RCA particle surface results in the reduction of specific gravity compared to that of natural aggregate. RCA specific gravity values increase with size and it ranges between 2.0 and 2.6 in the saturated surface dry (SSD) conditions (ACPA 1993, Katz 2003). Table 2.3 shows a comparison between specific gravity values of RCA and NCA used in various studies. It can be observed that the reduction in RCA specific gravity values ranges between 4% and 11% compared to that of NCA.

Table 2.3 Comparison of specific gravity values used in various studies RCA and NCA

<table>
<thead>
<tr>
<th>Reduction %</th>
<th>Specific gravity</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>4%</td>
<td>2.7</td>
<td>2.59</td>
</tr>
<tr>
<td>9%</td>
<td>2.65</td>
<td>2.4</td>
</tr>
<tr>
<td>10%</td>
<td>2.67</td>
<td>2.4</td>
</tr>
<tr>
<td>9%</td>
<td>2.74</td>
<td>2.5</td>
</tr>
<tr>
<td>11%</td>
<td>2.71</td>
<td>2.42</td>
</tr>
<tr>
<td>4%</td>
<td>2.11</td>
<td>2.03</td>
</tr>
</tbody>
</table>
2.3.3 Absorption

Typical NCA absorption capacity ranges between 0.3% and 1.25% depending on the aggregate source as shown by many studies (Table 2.4). Katz (2003) observed that the adhered mortar content on the RCA particles causes a significant increase in absorption capacity due to higher porosity of the mortar paste compared to that of NCA. He found that the absorption capacity of RCA is between 3% and 12% which is about 10 times more than that for NCA. Various studies such as (Salem et al. 2003; Gómez-Soberón 2002; Rao 2005) found that the absorption capacity of RCA was higher for smaller aggregate sizes. This could be explained by the results shown in Table 2.2 where the adhered mortar content increases for smaller sizes. Table 2.4 shows absorption capacities for RCA and NCA measured by different studies.

<table>
<thead>
<tr>
<th>Absorption</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCA</td>
<td>RCA</td>
</tr>
<tr>
<td>2.28%</td>
<td>4.35%</td>
</tr>
<tr>
<td>0.30%</td>
<td>4.70%</td>
</tr>
<tr>
<td>0.9-1.1%</td>
<td>5.8-6.8%</td>
</tr>
<tr>
<td>0.34%</td>
<td>3.3-5.4%</td>
</tr>
<tr>
<td>2.17%</td>
<td>5.23%</td>
</tr>
<tr>
<td>1.24-1.25%</td>
<td>6.28-7.56</td>
</tr>
</tbody>
</table>

2.3.4 Abrasion resistance

Abrasion resistance could provide insights on aggregate quality and weathering resistance. Sagoe-Crentsil, Brown, and Taylor (2001) concluded that abrasion resistance of RCA was about 12% lower than that for NCA. Abrasion resistance could range between 20% and 45% and it might be as high as 50% (ACPA 1993). Moreover, Abou-Zeid et al. (2005) showed that aggregate abrasion resistance is not influenced by RCA replacement level. Table 2.5 shows the abrasion resistances of NCA and RCA. It reflects the percentage of the mass reduction before and after testing.
2.4 Properties of Recycled Aggregate Concrete (RAC)

There are various constraints on using RCA in structural elements especially the percentage of RCA replacement in new concrete. Current codes allow using a small portion of RCA between 20%-30% for normal strength concrete up to 40 MPa compressive strength. According to Pepe (2015), this is due to lack of proper technical specification and guidelines regarding RCA quality uncertainties associated with production process of RCA (Huda 2014). As a result, many studies are investigating RAC properties using various RCA properties and different RCA replacement percentages. These results will increase RCA potential to replace NCA in structural elements encouraging industrial production of recycle concrete. Some studies, (Oikonomou 2005; Kuroda and Hashida 2005; Noguchi 2005; Li 2009; Abu Zeid and Kamel 2008) presented some of the existing guidelines and specifications of RCA usage in new concrete. Guidelines for using RCA in structural elements have been developed by few countries. These include, Chinese technical code (DG/TJ07-008), RILEM standards (RILEM 1994), British Standard Institution (BS8500 2006), Japanese Industrial Standards (JIS A 5023 2007), American Concrete Institute (ACI E-01 2014), Cement Concrete and Aggregate Australia (CCAA 2008) and Euro Code (EC8).

<table>
<thead>
<tr>
<th>Aggregate type</th>
<th>Abrasion resistance</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCA</td>
<td>80%</td>
<td>Nassar and Soroushian 2012</td>
</tr>
<tr>
<td>RCA</td>
<td>20-45%</td>
<td>Nassar and Soroushian 2012</td>
</tr>
<tr>
<td>RCA</td>
<td>20-50%</td>
<td>ACPA 1993</td>
</tr>
</tbody>
</table>
2.4.1 Properties of Fresh RAC

2.4.1.1 Workability

Sagoe-Crential et al. (2001) found that commercially produced RCAs are more spherical and smoother than that commonly produced for laboratory work. These RCA properties increase the workability of the commercial RAC compared to the laboratory one. According to Salem et al. (2003), higher RCA absorption capacity results in stiffer concrete mixes and a reduction in concrete mix workability compared to conventional concrete. It has been observed by Huda et al. (2014), and Leite, Figueire do Filho, and Lima (2013) that RCA quality significantly influences the concrete workability and a 5-10% of additional free water is required to obtain the same workability as conventional concrete.

2.4.1.2 Slump

Slump value provides an insight into fresh concrete consistency and workability. Topçu and Şengel (2004) investigated the influence of RCA amount on concrete workability at a fixed water to cement (w/c) ratio. They found that increasing RCA amount decreases the slump values resulting in lower workability. In addition to that, they concluded that using dry RCA causes a higher reduction in slump values. However, Yang, Chung, and Ashour (2008) suggested the use of admixtures to compensate for the reduction in slump values. Poon et al. (2004) showed that adjusting water content for the dry RCA to the actual moisture state produced a slump value of 100 mm for RAC consisting 50% RCA whereas it was 110-100 mm for NAC.
2.4.2 Properties of Hardened RAC

Hardened concrete properties could be classified into two categories: Mechanical properties and Physical. Tavakoli and Sorouchian (1996) experimentally found that there are various factors influencing RAC strength such as RCA source concrete and replacement level. In addition to that, they observed that RAC flexural, compressive and splitting tensile strength values did not match with the conventional concrete results. The following sections discuss the different RAC properties including physical and mechanical properties.

2.4.2.1 Physical properties

2.4.2.1.1 Permeability

RCA concrete has about 10-45% higher permeability than that of NCA. RAC permeability is significantly influenced by RCA source (Zaharieva et al. 2003, Abou-Zeid et al. 2005). As RCA has higher water absorption than NCA, more pores are formed due to water evaporation in the hardening stage causing higher porosity for RAC compared to conventional concrete. Zaharieva et al. (2003) found that extending curing period reduces RAC permeability by about 50% which limits the fine pores formation.

2.4.2.1.2 Porosity of concrete

Gómez-Soberón (2002) investigated various porosity properties of RAC including threshold ratio, critical pore ratio, average pore ratio, and theoretical pore radius of concrete. These properties were measured at the age of 7, 28, and 90 days. Results showed that replacing NCA with RCA increases concrete porosity. Pepe (2015) found that the RAC properties including modulus of elasticity, tensile, flexural and compressive strengths decreased with increased porosity. However,
it is difficult to formulate a direct relation between the total porosity and RAC properties (Huda et al. 2013).

2.4.2.1.3 Coefficient of thermal expansion

The coefficient of thermal expansion (CTE) is an important material property. It is a measurement of material’s expansion or contraction with temperature. Yang et al. (2008) found that RAC’s CTE value is higher than conventional concrete. They reported that the CTE value for RAC was $8.9 \times 10^{-6}/\degree C$ compared with $11.6 \times 10^{-6}/\degree C$ for conventional concrete. However, Smith et al. (2008) studied CTE of RAC and concluded that increment of RCA replacement percentages improves the thermal performance of concrete. Zega and Di Maio (2011) results supported Smith (2009) findings regarding the better thermal performance of RAC with low w/c ratios than that of conventional concrete.

2.4.2.2 Mechanical Properties

The Idea of using recycled concrete as aggregate dates to the period after World War II, since that time several studies (Etxeberria et al. 2007, Koenders, Pepe, and Martinelli 2014, Huda 2014, Moallemi Pour and Alam 2016) showed that RAC could achieve comparable mechanical properties of the conventional concrete. Frondistou-Yannas and Dietz (1977) reported that RAC had similar mechanical properties compared to conventional concrete when using RCA with less adhered mortar content. The following sections discuss the mechanical behavior of RAC.
2.4.2.2.1 Compressive strength

Compressive strength is one of the most important mechanical properties of concrete. Earlier studies investigating the influence of RCA on concrete mainly focused on RAC compressive strength and the factors affecting it.

Frondistou-Yannas and Dietz (1977) compared the mechanical behavior of RAC with conventional concrete. He observed a reduction of 24% for compressive strength and 20% to 40% for that of the modulus of elasticity of RAC compared to conventional concrete. Moreover, Hansen (1992) investigated the influence of w/c ratio of RCA source concrete on RAC compressive strength considering other factors being constant. They found that the w/c ratio of RCA source concrete significantly influences RAC compressive strength and that RAC could have similar or even higher compressive strength compared to conventional concrete when w/c ratio of source concrete is lower or at least similar to that of the conventional concrete. However, a later study conducted by Sagoe-Crentsil, Brown, and Taylor (2001) found small difference between the compressive strengths of RAC and conventional concrete. Yang, Chung, and Ashour (2008) used different RCA replacement levels (30%, 50%, and 100%) with 0.5 w/c ratio in producing 40 MPa RCA. They found that different RCA replacement levels yielded similar compressive strengths as conventional concrete. This supported Sagoe-Crentsil et al. (2001) findings which suggested that no matter what the RCA replacement levels are, the compressive strength remains almost constant.

Ulloa et al. (2013) reported that the RAC compressive strength is significantly influenced by the RCA replacement percentage and the effective w/c ratio. They observed a significant reduction in the compressive strength for 100% RCA replacement, with lower reductions for low RCA replacement levels such as 20% to 50%. Alam et.al (2013) found that RAC with RCA replacement ratio up to 50% experienced a 15% reduction in compressive strength compared to conventional
concrete. Furthermore, Gutiérrez and Juan (2004) recommended that for producing quality RCA adhered mortar content shouldn’t exceed 44%. They showed that the compressive strength of RAC with such RCA is generally higher than 25MPa.

Yet, there are limited studies that investigated RCA influence on High Strength Concrete (HSC). Acker (1996) used three different replacement levels of RCA (5%, 10%, and 25%) in producing HSC with a target strength of 75 MPa. He found that RAC with 10% RCA replacement had almost identical compressive strength as the control mix which has no RCA. Nevertheless, the concrete with 25% RCA replacement experienced about 17% reduction in compressive strength compared to control mix. On another study, Limbachiya, Leelawat, and Dhir (2000) used three different RCA replacement levels (30%, 50%, and 100%) while varying w/c ratio from 0.23 to 0.45. The study obtained compressive strengths of 50 MPa and 80 MPa. They concluded that HSC with up to 30% RCA replacement yielded similar mechanical and durability properties to HSC with NCA in it. However, HSC properties started to deteriorate with increasing RCA replacement beyond 30%.

Ajudkiewicz and Kliszczewicz (2002) used RCA produced from 40-70 MPa concrete. They produced recycled high-performance concrete (RHPC) having up to 80 MPa compressive strength using 10% silica fume which outperformed concrete with the original aggregate source that had about 60 MPa. In addition to that, they found that RHPC had similar mechanical properties to the one with NCA. They suggested that the water content should be modified in order to obtain similar mechanical properties as the HSC. In addition to that, their results supported the conclusion drawn by Tavakoli and Soroushian (1996) that reported a 10% reduction in the compressive strength for RAC compared to conventional concrete. Table 2.6 presents a comparative study of various RAC compressive strengths with different RCA replacement levels as compared to conventional concrete.
Variation in Compressive Strength as compared to natural concrete

<table>
<thead>
<tr>
<th>Replacement Level</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>7.7% Increase</td>
<td>Etxeberria et al. 2007</td>
</tr>
<tr>
<td>11% Increase</td>
<td>Alam et al. 2013</td>
</tr>
<tr>
<td>14.7% Decrease</td>
<td>Yang et al. 2008</td>
</tr>
<tr>
<td>11% Decrease</td>
<td>Kwan et al. 2012</td>
</tr>
<tr>
<td>8.9% Decrease</td>
<td>Limbachiya et al. 2000</td>
</tr>
<tr>
<td>8% Decrease</td>
<td>Ajdukiewicz and Kliszczewicz 2002</td>
</tr>
</tbody>
</table>

2.4.2.2.2 Hardness

Generally, concrete hardness value is directly related to its compressive strength. Hardness value decreases when compressive strength decreases (Huda 2014). There are limited studies that addressed RAC hardness. Topçu (1997) found RAC with 100% RCA had about 50% reduction in hardness value compared to conventional concrete. Topçu and Şengel (2004) used two RCA replacement levels (50% and 100%). They found that 50% and 100% RCA had 12.5% and 21% lower hardness respectively than conventional concrete. A more recent study by Grdíc et al. (2010) on self-compacting concrete with RCA showed comparable hardness performance up to 50% RCA replacement. They observed 2.5% and 14% lower hardness value for 50% and 100% RCA replacements respectively relative to conventional concrete.

2.4.2.2.3 Flexural strength

The literature showed a wide variation in RAC flexural strength due to variation in sources of RCA. Some studies such as Poon et al. (2002) found that RAC with 100% RCA had 13% higher flexural strength than conventional concrete. However, most of the studies (Alam et al. 2013, Kang et al. 2014, Arezoumandi et al. 2015, Arora and Singh 2016) found that RAC flexural strength...
decreases with increment of RCA replacement percentage. Alam et al. (2013) showed that using 50% RCA could significantly decrease RAC flexural strength by up to 32% compared to conventional concrete. On the other hand, Kang et al. (2014) observed 13% lower flexural strength for 50% RCA replacement than conventional concrete. Furthermore, Arora and Singh (2016) showed that the 100% RCA replacement had comparable flexural strength to that of conventional concrete where the flexural strength reduction was about 11%. Table 2.7 provides a summary of RAC flexural strength values obtained by various studies as a function of RCA replacement level.

Table 2.7 Variation in RAC flexural strength with different RCA replacement levels

<table>
<thead>
<tr>
<th>Variation in Flexural Strength as compared to natural concrete</th>
<th>100%</th>
<th>75%</th>
<th>50%</th>
<th>25%</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>13% Increase</td>
<td>10.8% Increase</td>
<td>6.25% Increase</td>
<td>2.2% Increase</td>
<td>Poon 2002</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>32% Decrease</td>
<td>16% Decrease</td>
<td>Alam et al. 2013</td>
<td></td>
</tr>
<tr>
<td>31% Decrease</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Katz 2003</td>
<td></td>
</tr>
<tr>
<td>11% Decrease</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Arora and Singh 2016</td>
<td></td>
</tr>
<tr>
<td>21% Decrease</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Arezoumandi et al. 2015</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>13% Decrease</td>
<td>12% Decrease</td>
<td>Kang et al. 2014</td>
<td></td>
</tr>
</tbody>
</table>

2.4.2.4 Tensile strength

Similar to flexural strength, various studies showed contradictory conclusions regarding the RAC tensile strength. According to Gómez-Soberón (2002), RAC tensile strength decreases with the increment of RCA owing to higher porosity of RCA. Ajdukiewicz and Kliszczewicz (2002) found the reduction in RAC tensile strength value by 10% compared with conventional concrete. On the other hand, Etxeberria et al. (2007) and Alam et al. (2013) observed that RAC had higher tensile strength. Table 2.8 provides a summary of RAC tensile strength values obtained by various studies as a function of RCA replacement level.
Table 2.8 Variation in RAC tensile strength with different RCA replacement levels

<table>
<thead>
<tr>
<th>Replacement Level</th>
<th>100%</th>
<th>60%</th>
<th>50%</th>
<th>30%</th>
<th>25%</th>
<th>15%</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>10% Decrease</td>
<td>8%</td>
<td>-</td>
<td>-</td>
<td>2.7%</td>
<td>-</td>
<td>-</td>
<td>similar</td>
</tr>
<tr>
<td>2% Decrease</td>
<td>-</td>
<td>18%</td>
<td>-</td>
<td>6%</td>
<td>-</td>
<td>-</td>
<td>Etxeberria et al. 2007</td>
</tr>
<tr>
<td>- Decrease</td>
<td>-</td>
<td>16%</td>
<td>34%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Alam et al. 2013</td>
</tr>
</tbody>
</table>

2.4.2.2.5 Modulus of elasticity

Various studies (Rao 2005, Ajdukiewicz and Kliszczewicz 2002, Oliveira and Vazquez 1996) showed that RAC modulus of elasticity could reach up to 50-70% of NAC depending on the RCA replacement level and w/c ratio. Qian et al. (2011) proposed a new technique known as shucking technique to improve RAC quality. This technique is a secondary process for enhancing RCA properties. Qian et al. (2011) reported that RAC made with 100% RCA had a modulus of elasticity about 70% of that for conventional concrete. However, the modulus of elasticity of RAC made of 100% shucked RCA was about 91% of that for conventional concrete. Furthermore, shucking improved compressive, flexural and tensile strength and other RAC properties to be comparable with commonly used concrete (Qian et al. 2012).

2.4.2.3 Durability of Recycled Concrete

Investigating durability properties could provide better insight on RAC properties. Several studies (Olorunsogo and Padayachee 2002, Zaharieva, Buyle-Bodin, and Wirquine 2004, Yang et al. 2008, Kwan et al. 2012) examined RAC durability properties. Olorunsogo and Padayachee (2002) studied RAC durability properties using three RCA replacement levels (0%, 50%, and 100%). They suggested that RAC durability property decreases when RCA level is increased. However,
RAC quality could be improved by increasing the curing time. The following section discusses some of RAC durability properties.

### 2.4.2.3.1 Freezing and thawing resistance

Various studies showed RAC freeze and thaw resistance is associated with other RAC mechanical properties. To elaborate, Gokce et al. (2004) suggested that RAC freeze and thaw resistance is significantly influenced by the source of parent concrete. Yamato et al. (1998) reported that RAC freeze and thaw resistance was lower than NAC. Yet, they showed RAC experienced a small reduction in freeze and thaw resistance for up to 30% RCA replacement. Furthermore, Abbas et al. (2009) found that RACs’ freeze and thaw resistance could be similar to conventional concrete. Salem et al. (2003), Zaharieva et al (2004), Richardson, Coventry, and Bacon (2011) concluded that RCA originated from concrete made with air entrained admixture yielded RAC with high freeze and thaw resistance. They reported that RAC made of RCA produced from air entrained concrete had higher freeze and thaw resistance than that from non-air entrained concrete. Furthermore, it was observed that RACs’ relative dynamic modulus of elasticity made of air entrained RCA was about 90% of the initial value after being exposed to 500 freeze and thaw cycles. However, it was about 60% of its initial value for RAC made with non-air entrained RCA when exposed to the same number of cycles. Similarly, Kolay et al. (2017) observed that RACs’ freeze and thaw resistance increased when air entraining admixture was used in the mix. In addition to that, Tuyan, Mardani-Aghabaglou, and Ramyar (2014) studied the mechanical properties of self-consolidating concrete (SCC) using various RCA replacement levels (0%, 20%, 40%, and 60%) with three different w/c ratios (0.43, 0.48, and 0.53). They reported that the freeze and thaw resistance for SCC with RCA reduced with increasing w/c ratio and RCA replacement content.
2.4.2.3.2 Corrosion

Corrosion is highly associated with chloride, sulphate and carbonate exposure conditions. Shayan and Xu (2003) reported that RAC had higher negative half-cell potential value than conventional concrete. However, it had similar potential corrosion risk as conventional concrete.

2.4.2.3.3 Alkali-silica resistance (ASR) and alkali carbon resistance (ACR)

Generally, adding admixture improves concrete mechanical properties. According to Huda (2014) using fly ash in RAC could improve alkali-silica reactivity, but CSA guideline limits maximum fly ash to 25% of total cementing material. Yet, Li and Gress (2006) reported that use of 15% fly ash did not cause any significant improvement. Nevertheless, Shayan and Xu (2003) observed that RCA alkali-aggregate reactivity is better than NCA.

2.5 Structural element made of RAC

Investigating RAC properties is very important to predict structural element behavior. Most studies showed that RAC had lower performance compared to conventional concrete but implementing RCA does not necessary influence the performance of structural element in the same way as it does with plane concrete as there are other factors that could change the structural behavior of RAC such as ratio of the applied axial load, reinforcement arrangement - longitudinal or transverse and design detailing. Therefore, it is very important to investigate the behavior of structural element made of RAC. Yet, there are limited studies discussing the seismic performance of RAC structural elements. This section provides an overview of the existing studies on behaviors of various structural elements using RAC e.g. beams, columns, beam-column joints and frames under
different conditions. Furthermore, it discusses in depth the experimental studies on the seismic performance of these structural elements constructed using RAC.

2.5.1 Beams

2.5.1.1 Shear behavior

González-Fonteboa and Martínez-Abella (2007) studied beam shear behavior using two concrete mixes (with 0% and 50% RCA) with four different transverse reinforcement arrangements. They found that RCA replacement had a slight influence on deflections and the ultimate loads. However, the observation showed a premature cracking and notable splitting cracks along the tension reinforcement in RAC specimens. They compared the experimental results with different building codes such as ACI, CSA and New Zealand (NZS) and modified compression field theory (MCFT). The comparisons showed that the codes under study were conservative and could be used for the shear design of RAC beams. Moreover, Etxeberria, Marí, and Vázquez (2007) experimentally studied the shear strength and behavior of RAC beams using four different RCA replacement levels (0%, 25%, 50% and 100%) and three different transverse reinforcement arrangements (without transverse reinforcement, with minimum reinforcement required and with 6 mm diameter stirrups spaced 130 mm c/c) under symmetric two-point loading. They found that higher RCA replacement levels had a greater influence on the shear strength of beams, especially for beams without transverse reinforcement. Moreover, they found that for low RCA replacement levels (up to 25%) had negligible influence on the shear capacity and that RAC had lower cracking load than conventional concrete even when having similar compressive strength as conventional concrete. For specimens with shear reinforcement, Etxeberria, Marí, and Vázquez (2007) observed that ultimate shear capacity for
beams was slightly influenced by high RCA replacement level (100% RCA) which was very close to that of conventional concrete. Similarly, González-Fonteboa and Martínez-Abella (2007) performed analytical simulations on the experimental results using MCFT and standard codes such as American standard (ACI) the Canadian standard (CSA). They found that the codes showed good correlation for beams with up to 25% RCA replacement. However, the codes overestimated the shear strength of beams with high RCA replacement (more than 50%).

González-Fonteboa and Martínez-Abella (2007) evaluated the influence of silica fume on RAC beam shear behavior using four different transverse reinforcement arrangements. They observed that using up to 8% silica fume could significantly mitigate the premature cracking phenomena experienced in their previous studies (González-Fonteboa and Martínez-Abella 2007). Furthermore, they suggested that premature cracking could be eliminated by using a maximum stirrup spacing of 200 mm. Choi et al. (2010) investigated effects of RCA replacement levels ((0, 30, 50, and 100%) on the shear strength of RAC beams with three span-depth ratios (1.50, 2.50, and 3.25) and different longitudinal reinforcement ratios (0.53, 0.83, and 1.61). They observed that the higher RCA replacement levels resulted in lower shear strength.

A different replacement method, equivalent mortar volume (EMV), was adopted by Fathifazl et al. (2011). They used different aggregate types (limestone, river gravel) with RCA replacement levels up to 74%. They showed higher shear strength for the RAC beams compared to conventional concrete. Whereas, Arezoumandia et al. (2014) examined the shear strength of full-scale RAC beams with 100% RCA replacement compared with conventional concrete using three different longitudinal reinforcement ratios with the absence of any shear reinforcement. The results were compared with other shear design codes e.g. ACI, BCA, CSA and EC2. The statistical test results showed that 100% RCA beams had nearly 12% lower shear strength compared with the
conventional concrete beams which imply that the current code might need some modifications to reflect on the lower shear capacity of RAC. However, the crack progression and load deflection responses were almost identical. Arezoumandi et al. (2015) extended the previous study by using 50% RCA replacement level. Their observations confirmed Arezoumandia (et al. 2014) findings for 100% RCA specimens and they observed that the shear performance of RAC specimens with 50% RCA was identical to the conventional concrete specimens. This indicates that the current codes are applicable for RAC beam shear design with RCA replacement levels up to 50%. A current study performed by Sadati et al. (2016) evaluated the influence of RCA replacement and high fly ash replacement (50%) on the beam shear capacity. They used similar parameters adopted by Arezoumandia et al. (2014) and found similar load-deflection and crack trends. However, specimens with 50% RCA and 50% fly ash replacement had about 10% and 18% reduction in shear capacity compared to conventional and 50% fly ash concrete, respectively.

2.5.1.2 Flexural behavior

Choi et al. (2009) investigated the influence of multiple parameters including (w/c ratio of the RCA source, RCA replacement level, curing condition, w/c ratio of the concrete mix and flexural reinforcement ratio) on the beam flexural behavior. They observed that RAC beams experienced greater short and long-term deflections compared to beams made of conventional concrete with the same w/c ratio and under same moment values. On the other hand, crack spacing and ultimate moment for RAC specimens were very close to that of NAC specimens. However, the RAC beams had greater crack width but still was within JSCE code limits. Furthermore, Ignjatović et al. (2013) examined the flexural behavior of beams with 100% RCA replacement. They found that beams made of 100% RCA had about 10% lower cracking loads and up to 13% higher deflection under
service loading compared to beams with conventional concrete. Yet, it had similar flexural capacity as NAC beams.

Choi and Yun (2013) studied the long-term deflection of RAC beams over one year and the influence of sustained loading on flexural behavior using four-point bending tests. The results showed that 100% RCA beams had smaller long-term to instant deflection ratios compared with NAC beams, but the total long-term deflections for RAC beam was greater than NAC beams. However, those were still satisfactory according to ACI code provisions. Unlike previous studies, they observed 20% reduction in flexural strength for RAC beams than that for NAC beams. Recently, Arezoumandi et al. (2015) investigated the flexural behavior of full-scale beams with 100% RCA using various flexural reinforcement rations under four-point loading condition. The results showed that RAC beams had about 7% lower cracking moment and approximately identical yielding and ultimate moment capacity compared to conventional concrete beams. Yet, RAC beams had lower cracking stiffness resulting in higher ultimate deflection (13%) than NAC beams. Furthermore, they showed that MCFT deflection estimation was about 35 % lower than the experimental values for both concrete types.

2.5.1.3 Behavior of RAC beams under Cyclic loading

Wang, Liu, and Zhang (2016) studied the seismic performance of 100% RCA beams under cyclic torsion. They observed that RAC and NAC beams experienced torsional failure mode with spiral cracks. Yet, RAC beams had more cracks than NAC beams. Moreover, RAC and NAC beams showed similar hysteresis loop shapes and corresponding inflection points. Despite that, RAC torsional beams experienced higher energy dissipation capacity and ductility than NAC beams by about 20% and 7% respectively. Although RAC beams under cyclic torsional loading showed
comparable stiffness deterioration to NAC beams, it was lower for RAC beams. Furthermore, (Liu et al. (2017) studied the influence of fly ash on RAC beam seismic behavior under combined flexure- shear- torsion cyclic loading. The results showed that addition of fly ash had negligible influence on the failure mode. Yet, RAC with fly ash had larger energy dissipation and ductility factor.

2.5.2 Beam-Column Joint

Corinaldesi, Letelier, and Moriconi (2011) and Letelier Gonzalez and Moriconi (2014) investigated seismic performance of 2/3 scale RAC beam-column joints using 30% RCA replacement subjected to cyclic loading. They observed similar energy dissipation and cracking patterns for both RAC and NAC beam-column joints. Although RAC specimen showed higher ductility, control specimen had higher moment capacity and greater experimental to design moment ratio. They concluded that RAC specimens with 30% RCA showed adequate seismic performance.

2.5.3 Columns

2.5.3.1 Column behavior under pure axial loading

There are limited studies investigating RAC column performance under axial loading. Choi and Yun (2012) tested 2/3 scale RAC columns with various RCA replacement ratios under monotonic uniaxial loading. They observed similar cracking patterns for all specimens and a decrease by about 6-8% in ultimate axial capacity for 100% RCA specimen compared to control specimen. However, it still satisfied the ACI 318-08 design strength criteria. Furthermore, Wang, Chen, and Geng (2015) and Xiao et al. (2012) compared the behavior of RAC confined by steel tubes (RCFS)
and RAC confined by glass fiber reinforced plastic (RCFF) tubes under axial compressive loading. Results showed that RCFS had higher axial load capacity than RCFF. In addition to that, RCFS with 100% RCA experienced about 13.7% reduction in ultimate axial strength compared to that of ordinary concrete whereas, RCFF had about 21.6% reduction compared with the control specimen. This finding supports Choi and Yun (2012) results that indicated that RAC reduces influence axial load capacity. Dong et al. (2013) extended the Xiao et. al (2012) study through investigating the influence of strengthening RCFS with carbon fiber reinforced polymer (CFRP) sheets. Unlike Xiao et. al (2012) and Choi and Yun (2012), they found that ultimate strength and stiffness for RCFS at 90-day curing period was slightly higher than the ones filled with normal concrete. Yet, hollow section tubes showed opposite results. A more recent study conducted by He et al. (2017) investigated the behavior of RAC steel-jacket retrofitted columns. Their observations showed that RCA steel-jacket retrofitted columns experienced slightly lower axial compressive strengths than the one with normal infill concrete, but the influence of RCA replacement ratio was negligible.

2.5.3.2 Seismic behavior of RAC columns

Yang, Han, and Zhu (2009) evaluated the seismic performance of circular RAC-filled steel tubular columns with two RCA replacement levels 25% and 50%. They tested 13 specimens that had 165 mm diameter, 2.57 mm steel tube thickness and 1500 mm effective height using different axial load ratios (5%, 25%, 48% and 52%) combined with cyclic lateral loading following ATC-24 1992. The adopted failure criteria were either fracture of the steel tube or 50% reduction in ultimate lateral capacity of the specimen. The results showed that RAC specimens had slightly higher ductility and energy dissipation capacity than that of NAC specimens, but RAC specimens had lower bearing capacity than NAC specimens by about 3.5% and 7.35% for 25% RCA and 50%
RCA replacement, respectively. Moreover, they found that the higher the axial load ratio, the lower the ductility ratio and the same for higher RCA replacement level. RAC and NAC specimens experienced similar failure modes under similar loading conditions. However, Xiao, Huang, and Shen (2012) investigated the influence of RCA replacement levels (0% and 100%), construction sequence (fully and partially cast-in situ) and RCA core size on the column seismic performance using six column specimens with 350x350 mm section and 1000 mm effective height with an aspect ratio of four. The specimens section was divided into external and core part. Two specimens were cast in situ where the external and internal parts were made of the same material either RCA or NCA. Another two specimens were semi pre-cast where the RAC core part was cast first then NAC external part was cast later. On the other hand, The last two specimens had the external RCA part cast first then NCA core was cast later. They found that cast in situ specimens had slightly better seismic performance where RAC specimen and NCA specimens had almost identical results. Furthermore, specimen with external precast parts had slightly higher energy dissipation and ductility factors than that of the specimens with core precast which was verified by the nonlinear FEM analysis results. This suggested that increasing the strength of RAC core could slightly increase the bearing capacity of semi-precast.

Moreover, Ma et al. (2013) evaluated the seismic performance of seven 1/2.5 scaled composite RC columns with I-steel section under combined constant axial and lateral cyclic loading. The study investigated the influence of various shear reinforcement ratios (1.02%, 1.36%, and 2.04%), RCA replacement ratio (0%, 70%, and 100%) and axial load ratio (30%, 60%, and 90%) on the seismic performance of RAC columns. Test specimens were 240x180mm with an effective height of 930 mm having a 3.25 aspect ratio. The percentages of I-steel section and longitudinal reinforcement for all the columns were 4.98% and 1.42% respectively. Their results confirmed the
findings of a similar experiment conducted by Xiao et al. (2012) and Yang et al. (2009) stating that the influence of RAC replacement level on failure and bearing capacity was negligible. Yet, Ma et al. (2013) observed that RAC specimens had lower equivalent viscous damping coefficients, ductility factors, and energy dissipation capacity than that of NAC specimens. However, increasing transverse reinforcement ratio increased bearing capacity and ductility of the columns. Furthermore, Ma, Xue, and Lin (2013) confirmed Ma et al. (2013) observations by adopting similar test procedure and fixing all parameters except the RCA replacement ratio.

In addition to the parameters investigated by previous studies, (Ma et al. 2013; et al. 2013; Ma et al. 2015), examined the influence of aspect ratio on seismic performance using different aspect ratios (1.4, 1.85, and 2.35). They tested nine short columns and one column with 2.35 aspect ratio. They observed that specimens with 1.4 aspect ratio had a brittle shear failure and spindle-shaped hysteresis loops indicating inadequate seismic performance. Furthermore, increasing axial load ratio reduced specimen ductility and energy dissipation capacity regardless of RCA replacement ratio. For short column specimen, similar observation as previous studies (Ma, Xue, and Lin 2013) on the influence of RCA replacement ratio, transverse reinforcement ratio, and axial load ratio was obtained on the seismic performance when compared to specimens with higher aspect ratios.

Recently, Soleimani et al. (2016) tested four ½ scaled square column with section dimensions of 330 mm x 330 mm and effective height of 1700 mm under combined constant axial loading (10% of gross capacity) with cyclic lateral loading to evaluate the seismic performance of RAC column with and without seismic detailing following International Building Code (IBC 200). The study used about 81% RCA replacement ratio yielding 19.3 MPa compressive strength for RAC whereas it was 31.0 MPa for control specimen. Results showed RAC non-seismically detailed columns had the lowest lateral resistance and experienced huge deterioration in lateral shear capacity by about
60% of that for conventional concrete. On the other hand, although RAC compressive strength was about 65% for that for conventional concrete, RAC seismically detailed columns had almost identical behavior as that for similarly detailed conventional concrete columns. Both seismically detailed specimen maintained their lateral shear capacity even when reaching high lateral drift ratios up to 4.0%. However, RAC specimen experienced more damages in the unconfined concrete cover only. RAC seismically detailed specimen had yielding rotation at about 0.0035 rad and ultimate rotation of 0.151 rad whereas it was 0.004 rad and 0.09 rad under similar displacement and moment values concluding that RCA specimen had 41.86 rotational ductility value compared to 21.5 for the conventional concrete specimen.

Chen et al. (2017) investigate recycled concrete-filled square steel tube (RCFST) seismic performance under constant axial compression and cyclic lateral loading using different RCA replacement ratios (0%, 30%, 70%, and 100%) and various axial compression loading ratios (23%, 27%, and 31%). Results of six specimens with a square section of 150 mm x 150 mm with 1050 mm effective height showed similar failure mode and ductility coefficient of around 3.0 for all specimens. Moreover, they found that yielding, peak, and ultimate load increased with increase in the axial compression ratio. Yet, it decreased the ultimate displacement resulting in lower ductility. The comparison of RCFST containing 100% RCA with CFST specimen under 31% axial load ratio showed that RCFST had higher energy dissipation and similar bearing loading capacity to CFST specimens indicating adequate seismic performance for RCFST.

2.5.4 Frames

Xiaoa, Suna, and Falkner (2006) experimentally studied the seismic performance of RAC frames using four 1/2 scaled frame specimens with different RCA replacement ratios (0, 30%, 50%, and
100%) under low frequency lateral quasi-static loading combined with constant axial loading. They observed similar seismic behavior for all specimens. However, the 100% RCA specimen showed about 8% and 2% reduction in yielding and ultimate loading compared to control specimen, respectively. Whereas 30% and 50% RCA specimens had about 3% and 11% reduction in yielding load and 6% and 15% for that of ultimate loading respectively than the control specimen. On the other hand, the ductility coefficients were about 4.0 for both 50% and 100% RAC specimens which were similar to that of the control specimens. However, it was about 3.7 for RAC specimens with 30% RCA in them. In terms of energy dissipation 100% RCA specimen had almost identical performance as control specimen where the other two specimens showed lower performance. Yet, it was satisfactory as defined by the Chinese standard GB 50011-2001 limits. Sun, Xiao, and Zhou (2008) extended the study done by Xiaoa, Suna, and Falkner (2006) by using two axial load ratios (15% and 30%) with identical experimental setups. The study compared the performance of 100% RCA specimen with control (0% RCA) specimen and observed similar failure modes for all specimens as obtained in previous studies (Sun, Xiao, and Zhou 2008). Unlike previous studies, they found that under the same vertical load condition 100% RCA specimen had 9.2% lower energy dissipation and slightly lower ductility coefficient approximately 4.0 compared to the control specimen having a ductility coefficient of 4.3. However, 100% RCA specimen subjected to the higher axial load ratio (30%) had 7.2% higher energy dissipation but, lower ductility coefficient value of about 3.7 than the control specimen. Moreover, Xiao et al. (2012) conducted a shake table test on a ¼ scale two bays, two spans, six-story 100% RCA concrete frame. The frame was subjected to various peak ground acceleration ranging from 0.066 g to 1.170 g. The tested frame showed that failure first occurred at beam ends and then at column bases characterizing ‘stronger columns, and weaker beams’. Furthermore, RAC
frame steeped into elastic-plastic stage after reaching PGA of 0.130 g where the influence of higher order modes increased gradually causing the seismic force distribution to follow a non-linear pattern. Although the RAC frame experienced severe damage under 1.170 g PGA, it showed adequate energy dissipation capacity, ultimate deformation capacity, maximum inter-story drift and ductility coefficient of about 4.0 leading it to withstand less frequent major earthquakes of intensity 9 without collapse.

Pacheco et al. (2015) experimentally simulated a pushover analysis on four three-dimensional, full-scale, RAC frame structures using different RCA replacement ratios (0%, 25%, and 100%) following Eurocode 8. It was observed that all of the specimens had ductile failure mechanism with ductile factor about 8.0 for RAC frames and a bit higher ductility value for the control specimen of about 10. Moreover, all specimens experienced similar base shear capacity about 60 kN regardless of RCA replacement ratio. Unlike previous studies, observations showed similar crack patterns, crack widths and crack spacing (between 98 mm to 109 mm) for all specimens regardless of the RCA replacement ratio.

2.6 Closure

This chapter has provided a comprehensive review of the properties of RAC for both fresh and hardened concrete and insight into current studies on the application of RAC in structural elements. Because of limited studies on structural elements made with RAC, there is lack of proper understanding of their behavior causing a lack of detailed guidelines for designing structural elements made with RAC, therefore application of RAC in commercial projects is still limited. Although a limited number of studies investigated seismic performance of RAC structural elements, researchers have tried to provide a comprehensive understanding through studying
various performance behaviors such as shear behavior, flexural behavior under various loading situation (pure axial, torsional, shear and combined loading) using different parameters including RCA replacement ratio, axial loading ratio, reinforcement detailing etc. Based on the review of studies on RAC, there is no single study that experimentally investigated seismic behavior of bridge pier having various RCA replacement levels using current building codes in North America. Moreover, there is a need for extensive research effort in this area to have a full understanding of structural element behavior involving RAC.
Chapter 3: Experimental Investigation Of RAC Bridge Piers

This chapter discusses the experimental investigation process adopted in this study. It is divided into six sub-sections including investigation of the material properties, development of mix design, designing of the tested specimen, the loading procedure adopted in this study, instrumentation and finally cyclic loading test setup. The investigation of the material properties describes the source of each material used in the test, mechanical properties such as bulk density, absorption, material gradation which follows CSA limits. Whereas the specimen designing section provides an explanation about the design method adopted in this study i.e. the forced based design. In addition to that, it describes the methodology adopted in scaling down the prototype design to produce the test specimens. The loading section describes the loading procedure used in this study based on which loading protocol was developed. The location and arrangement of instruments used in acquiring test results are illustrated in the instrumentation section. Finally, the last section discusses the cyclic test set up by providing the test layout and the way it was performed.

3.1 Materials

The concrete mixes consisted of cement, water, fly ash, recycled concrete aggregate (RCA), natural coarse aggregate (NCA), natural fine aggregate, water reducer, and air entrainment. This study used Type GA Portland cement, sea sand as fine aggregate which conformed to CSA A23.2 (Canadian Standards Association, 2014a). Figure 3.1 and Figure 3.2 shows the sieve analysis results for coarse and fine aggregates respectively. The maximum nominal size for coarse aggregates used in this study for RCA and NCA was 25 mm. Sieve analysis results of coarse aggregates presented in Figure 3.1 shows that the gradation for 100% RCA and 50% RCA combined with NCA slightly exceeded the upper CSA limits below 5 mm sieve size. This indicates
that they had more fines which will contribute to the total fine aggregate in the mix. Nevertheless, the gradation still satisfactorily meets the CSA A23.2 specifications whereas NCA gradation curve perfectly meets the CSA limits. It can be seen from Figure 3.2 that the fine aggregates gradation also falls within the CSA standard range. Lock-Block Ltd supplied the required RCA which was produced from demolished RC bridge located in Vancouver. RCA characteristics and properties have significant influence on the concrete properties. Since RCA used in this study was derived from one source, it ideally could be considered to have consistent properties for the entire RCA supplied amount. Aggregate properties obtained from material testing are summarized in Table 3.1. As shown in table 3.1, the absorption capacity of RCA was 4.79%, whereas it was only 0.7% for NCA. For reinforcement, grade W400 deformed steel bars with a yield strength of 400 MPa were used in this study.

<table>
<thead>
<tr>
<th>Results</th>
<th>RCA</th>
<th>FA</th>
<th>NCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk SSD specific gravity, SGSSD</td>
<td>2.50</td>
<td>2.59</td>
<td>2.70</td>
</tr>
<tr>
<td>Absorption capacity, AC (%)</td>
<td>4.79</td>
<td>1.9</td>
<td>0.70</td>
</tr>
<tr>
<td>Moisture content, MC (%)</td>
<td>1.91</td>
<td>1.70</td>
<td>1.31</td>
</tr>
<tr>
<td>Bulk density (kg/m³)</td>
<td>1389</td>
<td>1922</td>
<td>1633</td>
</tr>
</tbody>
</table>

![Figure 3.1 Sieve analysis of coarse aggregates](image1.png)

![Figure 3.2 Sieve analysis of fine aggregates](image2.png)
3.2 Mix Design

NCA and RCA concrete mixtures were designed, according to CSA 23.1-14, with a target strength of 30 MPa in 56 days. All concrete mixes used in this study had similar mix proportions. However, Moallemi Pour and Alam (2016) showed that 50% and 100% RCA replacement percentages had 15% and 25% reduction in compressive strength respectively. Therefore, cement proportions for the RCA mixes were increased by 5% and 15%, with respect to the NCA mix, for 50% RCA and 100% RCA mix, respectively to compensate for the compressive strength reduction. This study adopted weight replacement method for replacing RCA with NCA based on the targeted replacement percentages. Mix proportions incorporated in this study are presented in details in Table 3.2.

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Control Mix (0% RCA)</th>
<th>50% RCA</th>
<th>100% RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/c</td>
<td>0.4</td>
<td>0.385</td>
<td>0.358</td>
</tr>
<tr>
<td>water (kg)</td>
<td>152</td>
<td>152</td>
<td>152</td>
</tr>
<tr>
<td>Cement (kg)</td>
<td>300</td>
<td>315</td>
<td>345</td>
</tr>
<tr>
<td>NCA (kg)</td>
<td>1030</td>
<td>515</td>
<td>0</td>
</tr>
<tr>
<td>RCA (kg)</td>
<td>0</td>
<td>515</td>
<td>1030</td>
</tr>
<tr>
<td>Sand (kg)</td>
<td>735</td>
<td>735</td>
<td>735</td>
</tr>
<tr>
<td>Air Entraining (ml)</td>
<td>115</td>
<td>115</td>
<td>115</td>
</tr>
<tr>
<td>Water Reducer (ml)</td>
<td>456</td>
<td>456</td>
<td>456</td>
</tr>
<tr>
<td>Fly ash (kg)</td>
<td>80</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>85</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>Air content (%)</td>
<td>4</td>
<td>6</td>
<td>4</td>
</tr>
</tbody>
</table>

Concrete workability was an important factor in eliminating honeycomb formation in the test specimens. For that, the workability of fresh concrete was measured using standard slump test according to CSA A23.2-5C (Canadian Standards Association, 2014a). The recommended slump values as per CSA A23.2-5C for column elements ranges between 25 mm and 100 mm, and the initial slump values were 85 mm, 100 mm, and 80 mm for NCA, 50% RCA, and 100% RCA mixes.
respectively. All mixes fall in the recommended range of air content in fresh concrete which is 4.0 to 6.0% (Table3.2).

3.3 Design and Geometry

Design and geometric configuration of the prototype and scaled reinforced concrete bridge pier specimens according to the canadian highway bridge design code (CHBDC-2014) following a forced based design (FBD) approach are described in this section. FBD method is the traditional approach for designing bridges where simplified seismic demands could be calculated using single-mode spectral method (Zhang et al, 2016). Demand for any structure could be found using load factors that are associated with the structural plastic range capacity. Figure 3.3 illustrates FBD approach steps for designing a bridge pier. The bridge pier was assumed to be a part of a typical two span single pier major-route bridge located in Vancouver, British Columbia, Canada. It was designed to withstand seismic ground motion probability of 2% in 50 years (return period of 2475 years) without collapsing. The diameter of the bridge pier was 0.9 m and the effective height was 5.2 m as shown in Figure 3.4 with an aspect ratio of 5.8, which ensured the domination of flexure behavior (Billah and Alam 2015). A constant mass of 22.6 tons (190 kN), representing the weight of the superstructure was applied at the top which is approximately 10% of the gross pier capacity typically used in pier design (Soleimani et al. 2016). The fundamental time period of the bridge pier \( T \) was calculated using the cracked stiffness obtained from the study conducted by Priestley et al. (1996). The cracked stiffness was calculated using the elastic stiffness ratio which is a function of longitudinal reinforcement ratio and the axial load ratio Priestley et al. (1996). To specify, \( T \) was calculated as 1 sec following the equation \( T=2\pi \left( \frac{m}{k_{cr}} \right)^{0.5} \), where m is the effective mass and
$k_{cr}$ is the lateral cracked stiffness of the bridge pier. The design spectral acceleration was obtained from Vancouver uniform hazard spectrum (UHS) using the calculated $T$ and site soil class, $C$. Structural ductility was obtained using the response modification factor, $R$ based on structural configuration and the importance factor, $I$. The modified base shear demand was calculated using the above mentioned factors. Specifically, $I = 1.5$ and $R = 4$ were used for this Major-route bridge pier [CHBDC-14].

![Flow chart showing the steps in force-based design (FBD) method of bridge pier](image)

Figure 3.3 Flow chart showing the steps in force-based design (FBD) method of bridge pier
The longitudinal reinforcement ratio was calculated to be 1.91%. To elaborate, 18-30M reinforcing bars were used as longitudinal reinforcements whereas, for the shear reinforcement it was 15M steel spirals at a pitch of 70 mm with a transverse reinforcement ratio of 1.52% for plastic hinge (PH) region. In addition, 15M steel spirals were used at a pitch of 75 mm for the rest. The design moment and axial force were 1,884 kN-m and 2,300 kN respectively. The bridge pier was scaled down by 1/3 while preserving same stresses in the scaled pier as in the prototype pier. The effective scaled down pier height was 1.73 m with a diameter of 300 mm. The scaled pier longitudinal reinforcement ratio was calculated to be 1.98%, which includes 14-10M longitudinal reinforcing bars and 10M steel spirals at 75mm pitch (transverse reinforcement ratio of 2.2%). Figure 3.5 illustrates the detailing for the scaled pier. A comparative summary of the prototype and scaled down pier is shown in Table 3.3. Though (CHBDC-14) recommends using the least of
mm, ii) the greatest of the column maximum cross-sectional dimension or iii) one-sixth of clear height of the column for determining the PH length from the column base, it was estimated to be 237 mm using the equation proposed by Priestley et al. (1996). Particularly, the following equation was used to determine PH length in this study,

$$L_p = 0.08l + 0.022d_b * f_y$$  \(Eq. (3.1)\)

where, \(l\) is the column height, \(d_b\) is the longitudinal bar diameter and \(f_y\) is the yield strength of the longitudinal steel reinforcement. However, for the scaled pier, PH length was selected to be 300 mm as per CHBDC-14. The volumetric ratio of lateral reinforcement for the scaled pier was higher than the prototype pier due to difficulty in obtaining 8.25 mm rebars required to maintain the same ratio without violating the CHBDC-14 seismic requirements.

<table>
<thead>
<tr>
<th>Table 3.3 Geometric comparison of prototype and test specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description of Properties</td>
</tr>
<tr>
<td>Diameter (mm)</td>
</tr>
<tr>
<td>Effective height (m)</td>
</tr>
<tr>
<td>Clear cover (mm)</td>
</tr>
<tr>
<td>Longitudinal reinforcement ratio (%)</td>
</tr>
<tr>
<td>Volumetric ratio of lateral reinforcement (%)</td>
</tr>
<tr>
<td>Spiral pitch (mm)</td>
</tr>
<tr>
<td>Axial load, P/(\varepsilon^\Omega) (%)</td>
</tr>
<tr>
<td>Yielding Strength of Longitudinal reinforcement (MPa)</td>
</tr>
<tr>
<td>Yielding Strength of transverse reinforcement (MPa)</td>
</tr>
<tr>
<td>Compressive strength of concrete (MPa)</td>
</tr>
</tbody>
</table>

Figure 3.6 presents the reinforcement detailing of the designed scaled down section, having 25 mm concrete cover, however there was difference between the reinforcement detailing of the designed and as built section. The as built drawing were produced after the specimen were tested. Figure 3.7, Figure 3.8 and Figure 3.9 shows the reinforcement detailing of the tested specimens 0% RCA, 50% RCA and 100% RCA specimens, respectively.
3.4 Loading Protocol

The scaled down specimens’ seismic performance was investigated using a standard displacement controlled quasi-static cyclic loading depending on the yielding displacement ($\Delta_y$). It started with an increment of $0.25\Delta_y$ until it reached $\Delta_y$. After that, it was increased to $\Delta_y$ until reaching the ultimate failure. The yielding point was obtained from pushover analysis applied on a finite element model of the scaled down pier. To specify, this model was developed and analyzed using SeismoStruct 2016. Table 3.4 represents yielding displacement obtained from both the
experimental test and that from the analytical simulation described in appendix A. The displacement loading of three cycles was applied for up to $4\Delta_y$, after which it was reduced to two cycles. The rate of loading for the protocol was 5.1 mm/min until $\Delta_y$ to apply the displacement in that range accurately and observe the pier performance in the elastic range, then it was increased to 15.3 mm/min which still ensured that it was a pseudo-static test and ignored the dynamic behavior of the piers (Ghannoum et al. 2012). Figure 3.10 shows the standard loading protocol for the cyclic test and also, the protocol implemented in this study.

![Figure 3.10: Loading protocol for the applied lateral cyclic loading](image)

<table>
<thead>
<tr>
<th></th>
<th>Experimental($\Delta_y$)</th>
<th>Analytical($\Delta_y$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100% RCA</td>
<td>12.87 mm</td>
<td>15.63 mm</td>
</tr>
<tr>
<td>50%  RCA</td>
<td>11.55 mm</td>
<td>15.38 mm</td>
</tr>
<tr>
<td>0%   RCA</td>
<td>10.21 mm</td>
<td>14.75 mm</td>
</tr>
</tbody>
</table>
3.5 Instrumentation

Each specimen was equipped with electrical strain gauges to measure strains in the reinforcing steel and concrete cover and 10 Linear variable displacement transducers (LVDTs) that were supported by steel rods passing through the concrete core (placed before concrete placement) to measure the rotation of the potential plastic hinge region of specimen at five different levels starting at 100 mm and then every 100 mm up to about 500 mm above the top surface of the base. The locations of LVDTs are illustrated in Figure 3.12. Moreover, each specimen had a total of 24 strain gauges, 16 strain gauges were attached on the steel reinforcements whereas the other eight strain gauges were concrete strain gauges. To elaborate, two of the steel strain gauges were placed on the spiral, first one at 40 mm and the second one at 240 mm from the top surface of the base; 12 of them were placed on three longitudinal bars at 50 mm, 250 mm, and 450 mm from the top surface of the base. In addition, two more steel strain gauges were placed at 200 mm below the base top at the farthest longitudinal bars as shown in Figure 3.13. Eight concrete strain gauges were evenly distributed with a spacing of 100 mm placed on the column surface starting at 150 mm from the base top up to 450 mm as shown in Figure 3.11. Steel strain gauges had a gauge length of 6 mm and gauge resistance of $350 \pm 0.5 \Omega$. Whereas, concrete strain gauges had a gauge length of 75 mm and gauge resistance of $120 \pm 0.5 \Omega$. All measurements were recorded using a national instrument (NI) data acquisition system (DAQ) and LABVIEW 2014 which was used to control the hydraulic actuators.
3.6 Cyclic Loading Setup

The complete test set-up is illustrated in Figure 3.14. To elaborate, the specimens were anchored to the strong floor by post-tensioning the four corners of the footing using 35M High Strength Steel (HSS) rebar. The 10% axial load effect was simulated by post-tensioning a 25M HSS rebar going through the center duct/hole of the specimen to 190 kN and bolting it at the bottom of the base and top of the column. The applied axial load on top of the pier was monitored using a calibrated load cell. A hydraulic actuator with a load capacity of ±250 kN and ±125 mm displacement capacity was mounted to a reaction frame, which was assumed to have insignificant deformation during the test procedure. The actuator was placed in such a way which enabled it to reach its maximum displacement range during both push and pull. The lateral cyclic load was transferred to the specimen using a steel plate fixed to the column head through four strong bolts.
The lateral loading was gradually applied to the specimens in the cases of both pull and push as per the specified loading protocol (Figure 3.10). Deflections and shear force at the point of lateral load application were recorded by the MTS controller. Figure 3.15 shows the formwork constructed for casting the specimens and the curing process using wet sack after the casting. Moreover, Figure 3.16 shows 0% RCA pier under significant lateral drift during the test.

Figure 3.14 Experimental setup of bridge pier under lateral cyclic loading
Figure 3.15 Formworks for specimen casting and their curing process

Figure 3.16 0% RCA pier under significant lateral drift during the test
Chapter 4: Test Results

This chapter discusses hardened concrete properties used in this study including compressive, flexural and tensile tests for RAC and conventional concrete specimens and other concrete properties. As well steel bars yielding and ultimate strength and strain. Moreover, it discusses and compares lateral quasi-static test results of RAC specimens with conventional concrete specimen including force-displacement relation, moment-curvature, energy dissipation, residual displacement, ductility, and failure.

4.1 Material test results

4.1.1 Steel tensile test

The 10M steel rebars were tested according to ASTM A370-14 to find their mechanical properties. Figure 4.1 presents the stress-strain relationship for tested steel bars. It showed a yield strain of 0.00216 mm/mm with a yield strength of 432 MPa and a fracture strain of 0.2342 mm/mm with an ultimate strength of 745 MPa. Modulus of elasticity, $E_S$ was calculated to be 200 GPa. However, transverse reinforcement would have higher yielding strength due to strain hardening effect caused by deformation of the bars into spiral shape which shifted the yielding point to much higher values.

4.1.2 Concrete testing (Compressive, Split, and Tensile)

Three concrete mixes were used in constructing the test specimens having various amounts of RCA replacement. The control specimen had only NCA whereas, the other two RCA mixes had 50% RCA and 100% RCA replacement, respectively. In this study, three types of tests were
conducted to obtain the mechanical properties of concrete for each mix. Compression tests and split tensile tests were performed according to CSA-A23.2-9C (2014) and CSA-A23.2-13C2 (2014), respectively. Both tests used standard 100 x 200 mm cylinders for 7, 14, 28, and 56 days. Figure 4.2 illustrates compressive strength gaining for the three mixes up to 56 days.

\[ f_y = 432 \text{ MPa} \]
\[ E = 200 \text{ GPA} \]

Figure 4.1 Stress-strain relationship for steel

Figure 4.2 Compressive strength development of concrete mixtures.
All mixes had very close compressive strength results for 56 days as shown in Table 4.1. The results of compression tests on 56 days were 30.92 MPa, 28.86 MPa and 30.71 MPa for 0% RCA, 50% RCA, and 100% RCA, respectively. Figure 4.3 represents 100% RCA specimens tested under flexural tests where it shows that 100% RCA specimens had similar flexural failure pattern as conventional concrete. Figure 4.4 (a) and (b) shows specimens subjected to split tensile tests whereas Figure 4.4 (c) shows specimens under compression test. RCA and NCA specimens experienced similar failure mode under compressive and split test. Elastic modulus and Poisson ratio were calculated using stress-strain relationship obtained from the compressive strength results of the cylinders at 56 days according to ASTM C469. Figure 4.5, Figure 4.6, and Figure 4.7 show the stress-strain relationship for 100% RCA, 50% RCA and NCA concrete, respectively. In addition to that, concrete flexural strength was measured on standard 150 x 150 x 450mm beams at 56 days using CSA-A23.2-8C (2014). Table 4.1 summarizes the mechanical properties obtained from the above-mentioned tests for each mix. It was important to measure concrete properties at the time of column testing, therefore flexural and split test were conducted at 56 days.

<table>
<thead>
<tr>
<th></th>
<th>Ec'(GPa)</th>
<th>Poisson Ratio</th>
<th>Compressive Strength(MPa)</th>
<th>Tensile Strength(MPa)</th>
<th>Flexural Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100% RCA</td>
<td>18.11</td>
<td>0.1951</td>
<td>30.71</td>
<td>2.88</td>
<td>3.88</td>
</tr>
<tr>
<td>50% RCA</td>
<td>19.77</td>
<td>0.1787</td>
<td>28.86</td>
<td>2.87</td>
<td>3.38</td>
</tr>
<tr>
<td>0% RCA</td>
<td>23.51</td>
<td>0.1821</td>
<td>30.92</td>
<td>3.29</td>
<td>3.96</td>
</tr>
</tbody>
</table>

It was observed from Figure 4.5, Figure 4.6, and Figure 4.7 that 50% RCA concrete had lower axial and transverse deformations under compressive stress compared with 100% RCA and NCA concrete. However, 100% RAC specimens showed similar deformation behavior compared to
NAC specimens. The results showed that 100% RAC had the lowest modulus of elasticity value while NAC had the highest. This result supports the conclusions suggested by Moallemi Pour (2016), and Li and Zheng (2007) that RAC typically has lower elastic modulus that NAC.

![Image of specimens](image-url)

(a) Top view  
(b) Side view

Figure 4.3 Specimen subjected to flexural test

![Image of specimens](image-url)

(a)  
(b)  
(c)

Figure 4.4 (a) and (b) specimen subjected to split test, (c) specimen subjected to compressive test
(a) Compressive stress – Axial strain
(b) Compressive stress – Transvers strain

Figure 4.5 Stress-strain relationship of 100% RCA concrete

(a) Compressive stress – Axial strain
(b) Compressive stress – Transvers strain

Figure 4.6 Stress-strain relationship of 50% RCA concrete

(a) Compressive stress – Axial strain
(b) Compressive stress – Transvers strain

Figure 4.7 Stress-strain relationship of NCA concrete
4.2 Cyclic Response

4.2.1 Hysteretic behavior and backbone curves

Figure 4.8 represents the hysteretic behavior of three tested specimens (100% RCA, 50% RCA and 0% RCA) piers under quasi-static cyclic loading presented in Figure 3.6. For the 100% RCA pier specimen (Figure 4.8 (a)), the hysteresis on pushing side reached its maximum strength of 54.29 kN at 3.04% drift and was slightly reducing until 6.10% drift. After that, significant strength degradation occurred, and the pier failed under lateral loading due to longitudinal bar rupturing. The hysteresis graphs show that the strength deteriorated from its maximum capacity by 11.8% at 6.10% drift and then the strength quickly degraded by 24.7% at 6.94% drift. However, it was observed that pulling side showed more stable behavior where it reached its maximum strength of 56.37 kN at 5.20% drift. After that, the strength deteriorated slightly by 8.9% of the peak strength at a drift of 6.94% which is the maximum drift in the loading protocol.

(a) 100% RCA
Figure 4.8 Hysteretic response of piers under lateral cyclic loading
The 50% RCA pier specimen showed similar cyclic behavior as the 100% RCA pier. Figure 4.8 (b) represents the hysteretic behavior of 50% RCA pier. To describe, the hysteresis on pulling side reached its maximum strength of 55.0 kN at 6.10% drift. After that significant strength degradation occurred in the second cycle of 6.10% drift where the strength dropped by 12.4% of the peak strength. Then, the strength degradation continued but with a lower rate to drop by 16.8% of the peak strength at 6.94% drift. This sudden deterioration in pier lateral performance is due to rupturing of the longitudinal bars. Whereas, the pushing side showed more stable behavior as it reached its maximum strength of 47.5 kN at 3.50% drift. After that, the strength deteriorated slightly by 8.7% of the peak strength at a drift of 6.94%.

The control or the 0% RCA pier specimen, (Figure 4.8 (c)) shows symmetric cyclic behavior for both sides. The hysteresis on the pushing side reached its maximum strength of 53.6 kN at 3.47% drift and was slightly reducing until 5.20% drift. After that, significant strength degradation occurred at 6.10% drift due to rupturing of one of the longitudinal bars. A rapid strength deterioration continued until 6.94% drift when another longitudinal bar ruptured and the pier failed under lateral loading. Figure 10 (c) shows that the strength deteriorated from its maximum capacity by 11.6% at 5.20% drift and then the strength quakily degraded by 20.7% at 6.10% drift and by 34.6% at 6.94% drift. A similar behavior was observed at the pulling side, where it reached its maximum lateral strength of 50.9 kN at 4.34% drift and was slightly reducing until 4.5% drift. After that, significant strength degradation occurred at 6.10% drift to reach 18.7% reduction of the peak strength due to one longitudinal bar rupturing. Then the rapid strength deterioration continued until 6.94% drift to reach 35.1% reduction compared to peak strength,
as a second longitudinal bar ruptured. At that point, the lateral strength degradation exceeded the threshold for failure which was 20% reduction in lateral strength capacity.

To assess the influence of RCA replacement percentages on the cyclic performance of piers, a lateral force-displacement backbone curve for each specimen was constructed (Figure 4.9). The backbone curves of the three specimens were almost parallel to each other which indicated similar general behavior. However, the difference between them was in the load values. In the pull side, specimens with 100% RCA and 50% RCA had higher lateral strength than 0% RCA by about 10.8% and 8.1%, respectively and yet, in the push side, 50% RCA showed slightly lower lateral strength compared to 0% RCA with about 11% reduction in the lateral strength. Figure 4.9 shows that the initial stiffness of the three specimens to be the same. However, 50% RCA showed slightly higher post cracking stiffness. In addition, there was a small difference in the stiffness degradation rates between the three specimens. The stiffness degradation rate is the decrease in the specimen load with the displacement increase. Strength degradation in the three specimens can be associated with concrete cover spalling which initiated the buckling of the main rebars under compression at larger drift ratios and consequent longitudinal bar rupture.

In general, all three specimens had very close results and behaved in similar ways. Table 4.2 represents the uncracked and fully cracked stiffness for the tested specimens. The cracked stiffness for the tested specimens were calculated using following equation from CSA A 23-14

\[
EI_{\text{cracked}} = 0.2 \ E_C \ast I_c + E_S \ast I_s \quad Eq. (4.1)
\]

Where \( E_C \) is concrete modulus of elasticity, \( I_c \) moment of inertia of concrete, \( E_S \) steel modulus of elasticity and \( I_s \) steel moment of inertia. As shown in Equation 4.1 concrete contribution in section stiffness in the post concrete cracking phase is low, which indicate that any variation in
steel detailing could influence the post cracking stiffness of the tested specimens. Table 4.2 shows that 0% RCA had about 9% and 17% higher pre-cracked stiffness compared to 50% and 100% RCA specimens, respectively. However, when the section started to crack the concrete contribution started to decrease and the influence of lower modulus of elasticity of 50% and 100% RCA on section stiffness reduced. For the cracked section, all specimens had very similar cracked stiffness where 100% RCA specimen had about 8% lower stiffness than 0% RCA. On the other hand, 50% RCA specimen had slightly higher post-cracking stiffness by about 6% compared to that of 0% RCA specimen. These results indicate that the all specimens had similar post-cracking stiffness regardless of the RCA replacement ratio.

The observed difference in the lateral force capacity of the hysteresis curves could be attributed to construction uncertainties such as the longitudinal bars distribution, as shown in figures 3.6, 3.7, 3.8, and 3.9, specimen preparation especially grinding longitudinal bars to attach the strain gages and material uncertainty related to concrete and steel strength. In addition to that, axial load variation was neglected which could contribute to the difference.

![Skeleton curve of piers obtained from lateral cyclic loading](image)

*Figure 4.9 Skeleton curve of piers obtained from lateral cyclic loading*
Table 4.2 Cracked and Un-Cracked Stiffness for the three specimens

<table>
<thead>
<tr>
<th></th>
<th>100% RCA</th>
<th>50% RCA</th>
<th>0% RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>K(kN/mm)</td>
<td>5.210</td>
<td>5.679</td>
<td>6.268</td>
</tr>
<tr>
<td>% of 0% RCA</td>
<td>17%</td>
<td>9%</td>
<td>-</td>
</tr>
<tr>
<td>K cracked (kN/mm)</td>
<td>1.813</td>
<td>1.971</td>
<td>1.859</td>
</tr>
<tr>
<td>% of 0% RCA</td>
<td>8%</td>
<td>6%</td>
<td>-</td>
</tr>
</tbody>
</table>

4.2.2 Strain Response

Strains in the specimens’ plastic hinge region were measured using strain gauges installed on the internal rebars and cover concrete. Some strain gauges were damaged during concrete casting. In addition, the full strain response of steel and concrete could not be obtained, as some data was lost during the tests because of gauge failure or unreliable measurements caused by highly localized strains. Figure 4.10 shows strain developed in both steel and concrete associated with the applied lateral load for each specimen. Results show that the spiral reinforcement strain was about 0.0012 mm/mm, indicating that transverse reinforcement did not reach yielding point which was 0.000216 mm/mm. Strain-load results showed that the strain response of steel reinforcement and concrete in all specimens follow the material constitutive behavior indicating that RCA replacement does not affect concrete general behavior (Figure 4.10 (j), (k), and (l)). As shown in figure 4.10 (a), (b), and (c), corresponds to the stain responses at level 1 (50 mm above base surface), first yielding in longitudinal bars occurred at lateral loads of 18.62 kN, 20.46 kN, and 26.63 kN corresponding to 10.21 mm, 11.55 mm, and 12.87 mm lateral displacements for 0% RCA, 50% RCA, and 100% RCA specimens, respectively. Figure 4.10 (d), (e), and (f) show the strain-load relationship for 0%, 50%, and 100% RCA specimens at level 2 respectively whereas, Figure 4.10 (g), (h), and (i) corresponds to the responses at level 3.
(a) Steel Strain at level 1 for 0% RCA

(b) Steel Strain at level 1 for 50% RCA

(c) Steel Strain at level 1 for 100% RCA
(d) Steel Strain at level 2 for 0% RCA

(e) Steel Strain at level 2 for 50% RCA

(f) Steel Strain at level 2 for 100% RCA
(g) Steel Strain at level 3 for 0% RCA

(h) Steel Strain at level 3 for 0% RCA

(i) Steel Strain at level 3 for 0% RCA
Figure 4.10 Strain response of steel, concrete observed on 0% RCA, 50% RCA, and 100% RCA pier
4.2.3 Moment-Curvature Response

The plastic hinge region’s curvature ($\phi$) was calculated based on the method shown in Figure 4.11. A pair of LVDTs were mounted on the extreme sides of the specimen at 100 mm above the base to measure the vertical elongation ($\Delta_t$) and contraction ($\Delta_c$). Then the average strain ($\varepsilon$) was calculated using the formula $\varepsilon = \Delta / l$ by dividing obtained vertical deformation by the gauge length ($l$). After the strain profile was constructed, the average curvature was calculated by dividing the absolute summation of average strain ($\varepsilon$) for the extreme sides of the specimen by distance between the two opposite LVDTs assuming the angle of rotation for the section as very small (Eq. 4.1). The corresponding moment ($M$) was calculated by multiplying the applied lateral load with the effective height of the specimen (Eq. 4.2).

$$\phi = \frac{|\varepsilon_t + \varepsilon_c|}{d} \quad Eq. (4.1)$$

$$M = F \times L_e \quad Eq. (4.2)$$

Figure 4.11 Method used for measuring curvature of RC piers

Results of moment-curvature at 100 mm above the base of piers under combined constant axial loading and variable cyclic lateral loading are presented in Figure 4.12. The curvature curves for the three specimens are almost parallel upto the curvature value of 0.000281 rad/ mm but with
different values indicating similar moment-curvature behavior. The 0% RCA specimen experienced a significant deterioration in moment capacity after reaching a peak moment capacity of 92.2 kN-m at curvature value of 0.000184 rad/mm until curvature value of 0.000608 rad/mm where the reduction in the moment capacity reached 34.5% of the peak. The 100% RCA and 50% RCA specimens reached peak moment capacity of 96.7 kN-m and 81.8 kN-m at the curvature value of 0.000413 rad/mm and 0.000340 rad/mm, respectively. Unlike the 0% RCA specimen, both 100% RCA and 50% RCA specimens experienced more stable behavior after reaching the maximum moment capacity where the reduction in moment capacity was 4.7% and 11.6% of the peak, respectively. This could be attributed to the lower elastic modulus of concrete with RCA replacement.

Figure 4.12 Moment-curvature response 0% RCA, 50% RCA and 100% RCA specimens
4.2.4 Ductility Analysis

This study used two types of ductility measures, displacement and curvature ductility. The ductility of the tested specimens was calculated using their yielding and ultimate points. The yield displacement and force were obtained from hysteresis curves (Figure 4.8) using the bi-linear simplified method. After that, the ultimate and yield moments were calculated from the corresponding forces. Then, the corresponding curvatures were obtained from moment-curvature relationships (Figure 4.12) obtained from tests. The result summary of the ductility analysis is shown in Table 4.3.

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>100% RCA</th>
<th>50% RCA</th>
<th>0% RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yield</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Force (kN)</td>
<td>34.8</td>
<td>36.5</td>
<td>40.0</td>
</tr>
<tr>
<td>Dis. (mm)</td>
<td>22.5</td>
<td>22.4</td>
<td>19.8</td>
</tr>
<tr>
<td>Moment (kN.m)</td>
<td>59.9</td>
<td>62.8</td>
<td>68.8</td>
</tr>
<tr>
<td>Curv. (1/mm)</td>
<td>4.73x10^{-5}</td>
<td>4.91x10^{-5}</td>
<td>4.02x10^{-5}</td>
</tr>
<tr>
<td><strong>Ultimate</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Force (kN)</td>
<td>43.8</td>
<td>51.4</td>
<td>42.3</td>
</tr>
<tr>
<td>Dis. (mm)</td>
<td>120.0</td>
<td>120.0</td>
<td>105.0</td>
</tr>
<tr>
<td>Moment (kN.m)</td>
<td>75.4</td>
<td>88.4</td>
<td>74.1</td>
</tr>
<tr>
<td>Curv. (1/mm)</td>
<td>5.81x10^{-4}</td>
<td>6.08x10^{-4}</td>
<td>4.53x10^{-4}</td>
</tr>
</tbody>
</table>

| Dis. Ductility | 5.36 | 5.33 | 5.30 |
| Curv. Ductility | 12.38 | 12.28 | 11.27 |

All the tested specimens showed good ductility performance for both displacement and curvature ductility. However, specimens with RCA replacements slightly outperformed the control specimen (0% RCA). To specify, the displacement and curvature ductility for the specimen with 50% RCA were 5.33, and 12.28, respectively and for 100% RCA were 5.36 and 12.38 respectively. Table 4.3 shows that 0% RCA specimen had the lowest yielding curvature and displacement with 4.02x10^{-5} rad/mm at 19.8 mm. However, the specimen with 50% RCA replacement had the smallest yielding...
force with a value of 34.8 kN at a displacement of 22.5 mm compared to that of the 0% RCA specimen where the yielding force was 40 kN. The 100% RCA specimen had the highest yielding curvature value of $2.69 \times 10^{-5} \text{ rad/mm}$ at 22.4 mm displacement with yielding force of 36.5 kN. Unlike the yielding measurements, the ultimate measurements showed that 0% RCA specimen had the smallest ultimate displacement, moment and curvature with values of 105 mm, 74.1 kN.m and $4.53 \times 10^{-4} \text{ rad/mm}$ as shown in Table 4.2. Moreover, specimens with RCA withstand higher curvature before failure compared with 0% RCA specimen. The ultimate displacement, moment and curvature values for the specimen with 50% RCA replacement ratio were 120 mm, 75.4 kN.m and $5.81 \times 10^{-4} \text{ rad/mm}$, respectively, whereas they were 120 mm, 88.4 kN.m and $6.08 \times 10^{-4} \text{ rad/mm}$. Ductility measures presented in table 4.3 showed that all specimens had very similar displacement and curvature ductility performance indicating that RCA replacement ratio had minor influence on specimen ductility. Ductility results indicates that all tested specimen with have very similar seismic behavior, yet RAC could undergo higher displacement before reaching failure point.

### 4.2.5 Energy Dissipation

Earthquakes generate huge amount of forces on structures. Energy dissipation capacity of a structure helps reduce the probability preventing the total collapse of the structure by dissipating forces generated by an earthquake without significant structural damage. Figure 4.13(a) shows the dissipated energy per cycle and cumulative energy dissipation for the specimens with 0% RCA, 50% RCA, and 100% RCA replacement, respectively. Energy dissipation curves of the three specimens were almost parallel to each other which indicated similar general behavior. Below 0.87% drift, the piers dissipated a very small amount of energy as the materials were behaving in
the elastic range and did not have significant residual deformation. Once nonlinearity of the materials came into action, the energy dissipation capacity of the piers started to increase significantly. Figure 4.13(b) shows that the specimen with 100% RCA replacement had the highest cumulative energy dissipation at a value of 63.96 kN-m whereas the specimen with 50% RCA replacement had the lowest value of 53.91 kN-m. The total cumulative energy dissipation of 100% RCA specimen was about 112.4% of the total cumulative energy dissipation of the control specimen, whereas for the 50% specimen it was about 94.7%. Figure 4.13(b) shows that the energy dissipation capacity per cycle for the three specimens had similar trend but with different values until 6% drift. After this, the drift control specimen energy dissipation capacity started to decay whereas the specimens with RCA replacements kept increasing. This decay in energy dissipation capacity could be attributed to the failure in the control specimen at 6.1% drift.

(a)
4.2.6 Residual Drift

Residual drift is an important attribute in assessing structural usability after a seismic event. Figure 4.14 shows the residual drift versus drift ratios for the tested specimens. It was observed that all tested specimens showed similar residual drift behavior (Figure 4.14), where the curves of the three specimens were almost parallel. The tested specimens experienced small residual drift up to 1.74% applied drift, after that, it increased with almost linear relationship but with higher slope until the maximum applied drift which was 6.98%. For the control specimen (0% RCA) the residual drift was slightly smaller (about 1.2%) than the 100% RCA specimen up to 5.23% applied drift, then it increased to overcome the drift of the 100% RCA by about 2.1% to reach a value of 72.82 mm, whereas it was about 71.35 mm for 100% RCA specimen. However,
50% RCA specimen had the lowest residual drift throughout the whole test reaching a maximum residual drift of 65.49 mm at the end of the test which is about 8.2% less than 100% RCA specimen. This could indicate that control specimen started to lose its restoring force capacity at 5.23% drift whereas the specimen with RCA replacement did not.

![Graph showing residual drift of tested specimen at different applied drift](image)

**Figure 4.14: Residual Drift of tested specimen at different applied drift**

### 4.2.7 Failure Mode

All specimens were tested to the maximum displacement capacity (±120 mm) of the hydraulic actuator corresponding to 6.98% drift. The failure modes for three specimens were characterized by the presence of horizontal cracks distributed around and along the specimen’s surface. After reaching the maximum lateral capacity, horizontal crack widths started to increase, and concrete cover started to spall for all specimens at about 90 mm displacement which initiated buckling of the longitudinal reinforcement and eventually rupture of some longitudinal bars. For 0% RCA specimen, four longitudinal bars ruptured - two at each loading side at 105 mm displacement.
Whereas, for 50% and 100% RCA only one longitudinal bar ruptured for each specimen, at about 120 mm displacement at pulling side and at pushing side, respectively. This resulted in losing the restoring force of pier and increased residual drift and energy dissipation as discussed earlier. At this point, there was a sudden drop in the lateral load capacity of all piers. The observed failure mode followed flexural failure mode for all specimens is shown in Figure 4.15.

(a) 100% RCA

Figure 4.15 Failure mode of tested
Figure 4.16 Cracking patterns for various RCA replacement ratios
Figure 4.15(a) shows the bottom section of the column after testing where it is observed the specimen experienced concrete spalling with some horizontal cracks. Figure 4.15 (b), and (c) shows the location where steel bars experienced buckling and for some bars even a rupture. Figure 4.16 shows the cracking patterns for all specimens. Figure 4.16 (a), (b), and (c) clearly show that 100% RCA specimen experienced more cracks compared to 50% and 0% RCA specimens which had a similar number of cracks. However, the visual investigation of the cracks showed that the cracks in the upper half of 100% RCA specimen where surface cracks and did not penetrate beyond the cover concrete. Crack spacing for all specimens was almost the same of about 150 mm, whereas the crack width for 100% RCA specimen was larger than the other two specimens. Moreover, 50% RCA specimen experienced similar crack width as the 0% RCA specimen. In addition to that, it was observed that 100% RCA specimen had some vertical cracks and larger concrete spallings during the test which could be associated with lower shear capacity compared to the other specimens.
Chapter 5: Conclusion and Recommendation

5.1 Summary
Every year a huge amount of waste is generated due to the construction and demolition of aging concrete structures, consequently increasing the environmental loads. Sustainable concrete (RAC) obtained from C&D wastes provides more environmentally friendly construction material that can decrease the negative environmental impact associated with concrete usage throughout its life cycle.

This study was carried out to investigate the applicability of green concrete in structural elements. Three 1/3 scale bridge pier specimens with different RCA replacement levels for each specimen were cast with a target strength of 30 MPa. The specimens were subjected to combined constant axial and lateral quasi-static loading to simulate an equivalent seismic event. The seismic performances of these pier specimens were compared to the control specimen. This chapter discusses the outcomes and limitations of the current study and provides recommendations for future research.

5.2 Conclusion
This study experimentally investigated the seismic performance of 1/3 scale bridge pier specimens made of sustainable concrete with different RCA replacement levels with seismic detailing following CSA S6-14. The following conclusions could be established based on the obtained results:
(i) Material testing showed that the quality of RCA was comparable to the NCA used in this test where it showed reduction in specific gravity and bulk density by about 8% and 14% respectively, compared to NCA. This indicates a good quality of RCA used in this study.

(ii) Decreasing w/c ratio when increasing RCA replacement ratio helped to produce comparable RAC to conventional concrete. RAC specimens showed very close compressive strength values with 30.9, 28.9 and 30.7 MPa for control mix, 50% RCA and 100% RCA, respectively. Moreover, Poisson ratios for the three mixes where very close where it was 7% higher and 2% lower for 100% RCA and 50% RCA mixes compared to control mix. However, RAC mixes showed a decrease in tensile, flexural strength and modulus of elasticity values with increase in RCA replacement levels where the reduction was about 23%, 12% and 2% for modulus of elasticity, tensile and flexural strength, respectively for 100% RCA specimen than the control mix.

(iii) RAC specimens showed similar hysteresis behavior to control specimen and even a better behavior for some specimens, especially for 100% RCA specimen with a shear value of 54.3 kN and 56.4 kN with was about 1% and 10% higher than that for the control specimen in pushing and pulling phases, respectively. The un-symmetric hysteresis in push and pull phases was due to un-uniformed longitudinal steel bar distribution. Moreover, the control specimen experienced the highest deterioration in lateral load resistance capacity with about 35% reduction compared with 17% reduction for 50% RCA specimen at high drift ratios, more than 4%, this could be associated with over grinding of the steel bars for stain gauge attachment.

(iv) The stress-strain behaviors of the three specimens were almost identical. This shows similar stress distribution in the tested specimens.
(v) The three specimens had similar initial pre-cracking stuffiness. However, both 0% and 50% RCA had higher post-cracking stiffness compared to the 100% RCA specimen. In addition to that, control specimen experienced lager stuffiness deterioration than RCA specimens.

(vi) All specimens experienced similar residual drift. However, RCA specimens showed lower residual drift, especially 50% RCA specimen which experienced about 8% lower residual drift than control specimen. Indicating 50% RCA specimen had the best self-centering capacity.

(vii) RCA specimens had superior ductility performance over the control specimen for both displacement ductility and rotational ductility. Moreover, the ductility performance of tested specimens was the highest for 50% RCA where it was higher by about 11% and 32% than 100% RCA and control specimen, respectively.

(viii) All specimens had flexure failure mode. However, 100% RCA experienced larger spalling and wider cracks in the plastic hinge region about 84% and 25% than control specimen, respectively. The general cracking patterns for the three specimens were very similar and the crack spacing was around 150 mm in average. Despite that, the 100% RCA specimen showed more horizontal cracks and initiation of vertical cracks associated with lower shear values compared to 50% and 0% RCA specimens which were almost identical.

In general, RCA specimens showed adequate seismic performance compared to the control specimen when using RCA with comparable quality as NCA. Moreover, the test results showed that the current design code (CSA S6-14) is adequate for the seismic design of bridge piers. However, shear design should be performed more critically for RAC elements.
5.3 Limitations of this study

The limitations of this study include but are not limited to:

(i) The applied axial load was assumed to be constant throughout the test, even though it could vary with the applied displacements because of column elongation and eccentricity of post-tensioning steel.

(ii) Dynamic behavior of the column was ignored as pseudo-static loading rate was applied during the test.

(iii) The test arrangement eliminated the P-delta effect associated with column slenderness as the axial load was applied using post-tensioning rod, generating axial load to inclined column head which was not always perpendicular to the column base.

(iv) The test set-up did not account for the inertial effect of the column crown, assuming a cantilever column and thus, zero rotational resistance at the column crown.

(v) Application of uni-directional lateral loading in the test.

(vi) Using higher transverse volumetric ratios for test specimens compared to the prototype due to the limitations in minimum bar diameter available in local market.

(vii) Using three RCA replacement levels 0%, 50% and 100% only with same detailing. The study tested three specimens only.

5.4 Recommendations for future research

According to this study, RCAs have the potentials to be used in structural concrete members. In order to improve the reliability of these materials in construction industry, future studies about the seismic behavior of structural elements made of RCA is recommended. Based on the findings in
this study, the interest will grow among researchers toward recycled coarse aggregate concrete as a new generation concrete.

Following are some suggestions for future investigation:

(i) Investigation of the shear effect with the combined axial and cyclic loading using various transverse reinforcement ratios.

(ii) In-depth investigation on the shear detailing adequacy of the current bridge codes such as Euro Code, ACI, and others under combined axial and cyclic lateral load.

(iii) Investigation of the seismic performance of bridge piers using a wide range of RCA replacement ratios.

(iv) Investigation of the seismic performance of bridge piers using cyclic axial and lateral loading with RCA bridge piers.

(v) Investigation of the seismic performance of bridge pier with difference RCA source and wide RCA replacement levels.

(vi) Application of bi-directional cyclic lateral loading with various axial loadings and using concrete blocks to simulate axial loading which takes P-delta and inertial effect into account.

(vii) Conducting dynamic shake table test on RC bridge pier specimens.
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Appendices

Appendix A

This appendix describes modes used in developing the loading protocol adopted in this study.

Material

It describes the material and material models used in constructing the model. For concrete material a Mander et al. nonlinear concrete model (\texttt{con\_ma}) was adopted due to the simplicity in modifying the model based on the parameters obtained from material testing. This is a uniaxial nonlinear constant confinement model, that follows the constitutive relationship proposed by Mander et al. (1988) and the cyclic rules proposed by Martinez-Rueda and Elnashai (1997). The confinement effects provided by the lateral transverse reinforcement are incorporated through the rules proposed by Mander et al. (1988) whereby constant confining pressure is assumed throughout the entire stress-strain range. Table 1 represents concrete properties obtained from experimental tests as model input for 100%, 50% and 0% RCA model, whereas Figure A.1 shows the cyclic stress-strain relationship curves for concrete Mander Model.

<table>
<thead>
<tr>
<th>Concrete parameters</th>
<th>100% RCA</th>
<th>50% RCA</th>
<th>0% RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Compressive strength (MPa)</td>
<td>30.89</td>
<td>28.2</td>
<td>30.9</td>
</tr>
<tr>
<td>Mean Tensile strength (MPa)</td>
<td>2.88</td>
<td>2.86</td>
<td>2.88</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>18,114</td>
<td>19,770</td>
<td>23,510</td>
</tr>
<tr>
<td>Strain at peak stress (mm/ mm)</td>
<td>0.002853</td>
<td>0.001981</td>
<td>0.001982</td>
</tr>
<tr>
<td>Specific weight (kN/mm$^3$)</td>
<td>2.4x10^{-8}</td>
<td>2.4x10^{-8}</td>
<td>2.4x10^{-8}</td>
</tr>
</tbody>
</table>
For steel material a Menegotto-Pinto steel model (stl_mp) was adopted due its accuracy and simplicity in modifying it. Menegotto-Pinto steel model is a uniaxial steel model based on stress-strain relationship proposed by Menegotto and Pinto (1973) coupled with the isotropic hardening rules proposed by Filippou et al. (1983). The current implementation follows that carried out by Monti et al. (1996). An additional memory rule proposed by Fragiadakis et al. (2008) is also introduced, for higher numerical stability/accuracy under transient seismic loading. Figure A.2 represents the steel cyclic stress-strain for menegotto and pinto model, whereas table A.2 shows the steel properties for longitudinal and transverse reinforcement used in this model.
Table A.2 Material Properties in put for Mander Model

<table>
<thead>
<tr>
<th>Model Parameters</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>203,000</td>
<td>200,000</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>425</td>
<td>425</td>
</tr>
<tr>
<td>Strain hardening parameter ()</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>Transition curve initial shape parameter (-)</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Transition curve shape calibrating coeff. A1 (-)</td>
<td>18.5</td>
<td>18.5</td>
</tr>
</tbody>
</table>

Figure A.2 Steel cyclic stress-strain
<table>
<thead>
<tr>
<th></th>
<th>A2 (-)</th>
<th>A3 (-)</th>
<th>A4 (-)</th>
<th>Buckling strain (-)</th>
<th>Specific weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transition curve shape calibrating coeff.</td>
<td>0.15</td>
<td></td>
<td></td>
<td>0.1</td>
<td>7.8x10^-8</td>
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<tr>
<td>Isotropic hardening calibrating coeff.</td>
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<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Isotropic hardening calibrating coeff.</td>
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<td></td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Buckling strain</td>
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<td></td>
<td></td>
<td>1</td>
<td></td>
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</tbody>
</table>

The column section was modeled as **Reinforced concrete circular section (rccs)** using 14-10M longitudinal reinforcement and 10M spiral with 70 pitch for the transverse reinforcement. An Inelastic displacement-based frame element type with 400 fibers in section was adopted as showing in figure A.3. The column models was divided into 10 structural nodes, staring at z=0 for the Node1 and with an 200 mm increment till 1400mm, then Node 8 @ 1520 mm, Node 9 @ 1720 and Node 10 @1920 mm. N0 represents the column base and it was fully restricted (rotation and movement in all directions). Finally, a 245 kN axial load was applied@ Node 10 in -Z direction, whereas an incremental displacement with max displacement of 125mm and 1000 loading steps was applied @ Node 9 in Y direction as shown in figure A.4.
figure A.3 column section meshing  
figure A.4 column under combined loading