

An ACI Technical Publication

SYMPOSIUM VOLUME



Durability of Concrete Structures
Incorporating Conventional
and Advanced Materials

SP-331

Editors:
Yail J. Kim, Isamu Yoshitake,
and Mark F. Green



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Durability of Concrete Structures Incorporating Conventional and Advanced Materials

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The Concrete Convention and Exposition
March 26-30, 2017
Detroit, MI, USA

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First printing, February 2019

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Printed in the United States of America

Editorial production: Gail Tatum

ISBN-13: 978-1-64195-054-1

Preface

Durability of Concrete Structures Incorporating Conventional and Advanced Materials

Sustainability is one of the salient requirements in modern society. Structures frequently deteriorate because of aggressive service environments; consequently, federal and state agencies expend significant endeavors to maintain the quality of the structures. Among many factors, durability plays a major role in accomplishing the concept of sustainability. Extensive research has been conducted to understand the deterioration mechanisms of concrete and to extend the longevity of concrete members. Over the past decades, the advancement of technologies has resulted in durable construction materials such as advanced composites. This Special Publication (SP) contains nine papers selected from two technical sessions held in the ACI Spring Convention at Detroit, MI, in March 2017. All manuscripts were reviewed by at least two experts in accordance with the ACI publication policy. The Editors wish to thank all contributing authors and anonymous reviewers for their rigorous efforts. The Editors also gratefully acknowledge Ms. Barbara Coleman at ACI for her knowledgeable guidance.

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TABLE OF CONTENTS

SP-331-1:

Thermal Detection of Subsurface Delaminations in Reinforced Concrete
Bridge Decks Using Unmanned Aerial Vehicle..... 1-14
Authors: Tarek Omar and Moncef L. Nehdi

SP-331-2:

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM
Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles..... 5-33
Authors: Ian Shaw, Hang Zhao and Bassem Andrawes

SP-331-3:

Cyclic Loading Behavior of CFRP-Wrapped Non-Ductile Beam-Column Joints.....34-54
Authors: Ali S. H. Zerkane, Yasir M. Saeed, and Franz N. Rad

SP-331-4:

Implications of High Plasticity Soils in Bridge MSE Wall Embankments 55-68
Author: Mark E. Williams

SP-331-5:

Long-Term Durability of GFRP Internal Reinforcement in Concrete Structures.....69-79
Authors: Omid Gooranorimi, Doug Gremel, John J. Myers, Antonio Nanni

SP-331-6:

Improving the Durability of Impact Damaged PC Bridge Girders Using
CFRP Rod Panel Retrofit.....80-100
Authors: Abheetha Peiris and Issam Harik

SP-331-7:

Experimental Investigation of CFRP Prestressed Concrete Beams 101-121
Authors: Yasir M. Saeed and Franz N. Rad

SP-331-8:

Behavior of Full-Scale Damaged Beams Repaired Using a Steel Reinforced
Polymer (SRP) Technique 122-135
Authors: Hayder Alghazali, Zuhair Al-Jaberi, Zena Aljazaeri, John J. Myers

SP-331-9:

Durability Investigations into CFRP-confined Concrete in H₂SO₄ 136-150
Authors: Yongcheng Ji and Yail J. Kim

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

Tarek Omar and Moncef L. Nehdi

Synopsis: Bridge deck condition assessment is commonly conducted through visual inspection by bridge inspectors. Considering the colossal backlog of aging bridge structures, there is a need to develop cost-effective and innovative solutions to evaluate bridge deck conditions on regular time intervals, without interrupting traffic. This makes remote sensing technologies viable options in the field of bridge inspection. This paper explores the potential for applying infrared thermography (IRT) using unmanned aerial vehicle (UAV) to detect and quantify subsurface delaminations in concrete bridge decks. The UAV-borne thermal sensing system focuses on acquiring thermal imagery using a UAV and extracting information from the image data. Two in-service concrete bridge decks were inspected using a high resolution thermal camera mounted on a UAV. The captured images were then enhanced and stitched together using a tailored procedure to produce a mosaic view of the entire bridge deck, indicating the size and geometry of the detected delaminated areas. The results were validated by conducting hammer sounding and half-cell potential testing on the same bridge decks. The findings reveal the capability of the technology to provide measurements comparable to those derived from traditional hands-on inspection methods. Thus, it can be an excellent aid in efficient bridge maintenance and repair decision-making.

Keywords: bridge deck, condition assessment, delamination, infrared, thermal image, unmanned aerial vehicle

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

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INTRODUCTION

Concrete is the most used construction material in civil infrastructures. Its service life is considered synonymous to its mechanical strength, durability and serviceability. Durability of concrete can be defined as its ability to resist weathering action, chemical attack, abrasion, or any other processes of deterioration to retain its original shape, dimensions, quality and serviceability. However, concrete structures are vulnerable to a number of factors that can cause deterioration and result in loss of strength and unsafe or failure conditions. Existing concrete bridges represent strategic components of infrastructural networks. Bridge failures can be catastrophic, both in terms of human life and economic loss, rendering the task of managing this important infrastructure asset a complex endeavour that attracts growing attention. Among all reinforced concrete (RC) bridge components, bridge decks are rapidly deteriorating and have been the leading contributor to most deficient bridges in the United States [1]. A major challenge associated with inspecting RC bridge decks is that defects are often concealed subsurface mechanisms which increase in severity until the damage becomes too severe for cost-effective repair. Therefore, condition assessment tools capable of detecting subsurface anomalies, such as voids, delaminations and cracks are required to ensure bridge safety and define maintenance and repair needs. Current bridge deck inspection practices rely on visual inspection, which heavily depend on the experience and engineering judgment of bridge inspectors. A number of hand tools, including hammers, steel rods and chains, have been widely used to detect subsurface delaminations and voids in RC bridge decks. Hammer sounding involves tapping the surface of a concrete member with a hammer at multiple locations, while chain dragging involves dragging a chain over the bridge deck surface. In both cases, the user listens to and interprets the distinctive sounds produced. A dull or hollow sound indicates delaminated concrete, and a distinct ringing sound designates non-delaminated concrete. The advantages of these methods include simplicity, portability and low operating cost. However, such techniques require hands-on access and can be labor-intensive and time-consuming for large areas of concrete, while being ineffective for detecting subsurface anomalies in decks having overlays. Traffic control must also be in place so that inspectors can safely access the concrete element. In addition, the interpretation of the sound produced is subject to the operator's judgment and experience, typically yielding primarily qualitative and subjective decisions.

These limitations have motivated the pursuit of advanced non-destructive testing (NDT) techniques for more effective and reliable bridge inspection. NDT methods used to assess subsurface defects in RC bridge decks vary in complexity and reliability and often applied only when severe defects and deficiencies are observed. Infrared thermography (IRT) is a NDT technology that has gradually gained wider acceptance as a condition evaluation tool to detect subsurface delaminations in RC bridge decks without physical contact. Such subsurface anomalies can be detected on the basis of variable concrete properties, such as density, thermal conductivity and specific heat capacity. Rapid data collection using remote sensing technology with thermal IR imagery can reduce traffic disruption and lane closures on and underneath bridge decks, thus, it is less costly than other NDT methods. Generally, IRT testing collects radiant temperature and visualizes the data in the form of real-time thermal infrared images. With the advent of newer generations of infrared cameras, IRT is evolving as an accurate, reliable and cost-effective technique that can yield both qualitative and quantitative indicators of a RC bridge deck condition. Conducting ground IRT testing in-situ on full-scale bridge decks requires mounting the IR camera on a vehicle. Collecting thermal IR images is highly dependent on the camera's field of view (FOV) and lens. The ideal option for data collection is to scan one traffic lane on each pass. However, obtaining such horizontal FOV is not always achievable in the field as it may require to mount the camera at a high level. Thus a number of survey passes should be adopted to cover the entire bridge deck area, which requires the arrangement of traffic control.

The generic definition of remote sensing technologies allows for a variety of deployment platforms including satellites, aerial inspection using fixed wing aircrafts, or vehicle mounted systems, making remote sensing potentially valuable in the field of inspection and monitoring [2]. In recent years, the unmanned aerial vehicle (UAV) technology has increasingly been used in various application areas, such as monitoring construction and operation of buildings and other types of civil engineering systems. For instance, UAVs can be equipped with different high definition cameras to offer several mapping applications in the field of photography, geology, geography, meteorology or in agriculture and forestry [3]. The improvements in navigation and sensor technology have made UAVs more reliable in terms of flight control. In addition, the advanced telecommunication technologies permit their flying at different altitudes over considerable distances [4]. Therefore, UAV equipped with high definition photo and video cameras can facilitate the inspection tasks of civil infrastructure, such as bridges as they are able to operate at low altitudes and acquire images with high resolutions. The limitations of applying ground IRT motivated the pursuit of this technology for bridge inspection. A UAV-borne imaging system with a high resolution thermal camera can provide a much safer and cost-efficient way for data acquisition. This study attracts the application of this system for evaluating the condition of full-scale deteriorated RC bridge decks.

RESEARCH SIGNIFICANCE

There is need for further research to develop more efficient bridge condition assessment for evaluating in-service bridges without interrupting traffic. Thus, remote controlled UAV equipped with a high resolution thermal imagery could be a good option to simplify the inspection tasks of RC bridge decks. This can provide rapid condition assessment and enable monitoring of deterioration progression through periodical surveys and thus, allow the surveys of hundreds of RC bridge decks to become feasible. Consequently, considerable reductions in costs associated with the inspection processes of bridge decks and in the frequency and duration of traffic interruptions can be achieved. In addition, the majority of IRT testing performed in previous research studies was based on testing specimens prepared in the laboratory environment with simulated defects, then exposing them to solar energy. Hence, several processing algorithms to extract information were developed based on the temperature of the pre-defined defects' locations and sizes with respect to the temperature of the surrounding sound concrete. However, when conducting IRT testing in-situ on full-scale RC bridge decks, the ability to extract quantitative information of unknown subsurface anomalies from images is still needed to fully realize its potential in detecting these defects.

RESEARCH OBJECTIVES AND METHODOLOGY

The main goal of the present study is to evaluate the reliability of using a UAV-borne thermal imaging system as an inspection tool for the condition assessment of RC bridge decks. The primary objectives of the study are: (i) studying the capability of UAV-borne thermal imaging system in detecting the delaminated areas in RC bridge decks; (ii) applying a rational procedure to analyze the thermal images captured by the system; (iii) creating a mosaicked thermogram of the entire bridge deck; (iv) producing a condition map indicating the geometry and sizes of delaminated areas. The methodology adopted for achieving these objectives consists of: (1) utilizing a UAV to conduct passive IRT testing in-situ on two deteriorated full-scale RC bridge decks using a thermal infrared camera; (2) enhancing the thermal contrast of the captured images using a commercial imaging software; (3) stitching the images to produce a thermal map for the entire bridge decks using especially developed Matlab codes; (4) identifying the defected areas based on the recorded thermal contrast values; (5) quantifying the delaminated areas within the bridge decks; and (6) validating the achieved results through data obtained using other technologies on the same bridge decks.

BACKGROUND

Deterioration of RC bridge decks

Deterioration of RC bridge decks is caused by multiple reasons and the distinction among their major causes is purely qualitative since such mechanisms can act in synergy. Different deterioration processes lead to different types of structural defects (e.g. delamination, spalling, cracking, rebar size reduction) or material alterations (e.g. reduced modulus, changed electrical and chemical properties). However, deterioration of RC bridge decks is commonly initiated by reinforcing steel corrosion often instilled by carbonation of concrete, chloride ions intrusion, or a combination thereof. As the steel corrodes, rust occupies increased volume, and cracking is initiated when the internal tensile stresses exceed the tensile strength of the concrete. Cracking accelerates the damage mechanisms by providing easy access for chloride ions, oxygen and moisture, resulting in subsurface fracture planes (delaminations). Such defects typically occur at or near the level of the top steel reinforcement layer. They also could occur below the bottom steel reinforcement layer as a result of chemical diffusion through construction joints. As corrosion damage advances, these delaminations can mechanically de-bond and separate from the concrete

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

structure that can lead to structural and functional failures, which are catastrophic, both in terms of human life and economic loss [5]. The spalling of the concrete further exposes the reinforcing steel to the ambient environment, resulting in accelerated deterioration rates. In addition, spalling in the soffit area of overpass concrete bridges can lead to falling debris, creating hazards to the traffic below. Therefore, the delamination-induced spalling of concrete represents the main challenge in RC bridge inspection and has been a concern for transportation agencies due to the related serviceability and safety considerations.

Detection of delaminations in RC bridge decks

Inspection of an existing bridge aims at determining whether the bridge will function safely over a specified residual service life. Guidelines for inspection of existing bridges have been developed in many countries. Visual Inspection is the predominant inspection technique used in bridge inspection manuals and serves as the baseline to which other techniques are often compared. Routine VI is often conducted within 24-month intervals depending on the condition of the bridge. While visual inspection can provide useful information on the condition of a bridge deck, the observed deterioration or damage is typically in its last stage of development. Moreover, visual inspection is effective in finding external defects such as surface cracks, scaling and spalls, yet it cannot detect subsurface flaws such as voids, internal cracks, delamination, or rebar corrosion. Application of NDT technologies has shown promise in detecting subsurface deterioration in RC bridges. Conventional acoustic methods, such as sounding with hammers, rods or chain dragging, are widely used to detect subsurface anomalies in RC bridge decks. However, initial or incipient delamination often produces oscillations outside the audible range and thus, cannot be detected by such basic techniques as they rely on the operator interpretations of the sound heard [6]. Due to such access requirements and inherent subjectivity of sounding techniques, a number of more advanced NDT techniques have been motivated. These NDT methods are capable of identifying different types of damage in RC bridges and can be applied alone to evaluate certain aspects, or can be combined to cover a wider range of testing capabilities in a complementary manner. The most commonly used NDT methods in onsite assessment and evaluation of RC bridge decks are: Impact Echo, Ultrasonic Pulse Echo, Half-cell Potential, Ground Penetrating Radar, and Infrared Thermography. Generally, such techniques learn about the characteristics of the medium from its response to an applied acoustic, seismic, electric, thermal, or electromagnetic excitation [7]. Further details about their theoretical bases, instrumentations, applications, and data analysis techniques are provided in a variety of sources such as the AASHTO manual for bridge evaluation [8], and in a report of the American Concrete Institute, ACI 228.2R [9]. Only the implemented IRT technology in this study is presented below.

Infrared thermography technology

Heat transfer takes place in three modes: conduction, convection and radiation. The internal properties of a material, such as the density, thermal conductivity and heat capacity play important roles in governing its temperature at equilibrium with its surroundings. For instance, heat capacity is defined as the amount of heat needed to raise the temperature of a unit mass of a material by one degree and describes its ability to store heat. Thermal conductivity is the rate at which heat flows through a material. Thermal diffusivity, on the other hand, is the rate at which temperature changes occur in a material. A high thermal diffusivity means that heat transfer through a material will be rapid and the amount of storage will be small. Conversely, low thermal diffusivity indicates a slower rate of heat transfer and a large amount of heat storage. Infrared thermography (IRT) is a technology to detect thermal radiation emitted from materials. It creates an image of surface temperatures based on the emitted radiation. Thermal IR radiant energy is emitted from all objects that have a temperature greater than absolute zero. The rate at which this energy is emitted is a function of the temperature of the material and its emissivity. A material's emissivity is the ability of its surface to emit energy by radiation relative to a black body and has a value between 0 and 1. This value is typically greater than 0.92 for concrete [10]. There are two testing approaches for IRT based on the source of heat. The active approach uses an external thermal stimulus to induce the required heat flow condition on the concrete under testing. The passive approach uses natural heat sources such as solar heating and ambient temperature changes.

The concept behind the application of IRT in concrete bridge evaluation is that subsurface defects such as voids, cracks and delaminations disrupt heat transfer through the concrete. When the temperature of the concrete increases, such as during daytime when the sun and ambient environment are heating the concrete, the surface area above a subsurface delamination warms up at a faster rate than that of surface areas where the concrete is intact. Delaminations can be detected as "hot spots" on the surface of the material, relative to intact concrete. Conversely, during nighttime, the air temperatures usually decrease and the material cools. Thus, the surface area above the delaminations cool at a faster rate than the intact concrete and appear as "cold spots" relative to the intact concrete.

ASTM D4788-03 [11] describes the standard procedure and equipment necessary for conducting passive IR testing for detecting delaminations in bridge decks. Although radiated energy is the parameter that is measured by a thermal IR camera, the heat conductivity within the concrete and heat convection around the concrete can also influence this measurement. Therefore, areas such as bridge soffits, which typically are not exposed to direct sunshine, can be warmed over the course of the day through convective heating if the temperature increase is sufficient. Though the temperature contrasts are smaller than those for solar exposed surfaces, subsurface delaminations can be imaged. However, this application of IRT is much less common than for solar exposed surfaces [12]. It should be noted that detection of delaminations in the presence of an asphalt overlay is difficult because the overlay could dampen the thermal contrasts achieved due to the thermal mass above a potential delamination, and the thicker the overlay, the greater the damping effect. Therefore, a major concern for the reliability of IRT testing through asphalt overlay is the possible presence of a debonded area, which could mislead interpretation. The debonded area could appear as a flaw in the IR image, in addition to the actual delaminations in the concrete. To distinguish debonding for delamination, Roddis and Maser [13] suggested that the delaminated areas appear as circular and uniform, while the debonded areas are present as large, non-circular, and non-uniform.

In addition, the environmental conditions, such as wind speed, solar energy, humidity and ambient temperature have an influence on the results from a thermal IR bridge inspection. The effects of these environmental factors can be difficult to characterize and vary over time, such that it can be difficult to determine whether environmental conditions at a given point in time are adequate to produce a high quality image. Favorable environmental conditions include sufficient solar loading or changes in ambient temperature to produce a thermal gradient in the concrete, where the concrete surface heats up at a much higher rate than the concrete core. For instance, when the ambient temperature is less than the temperature of the concrete, for example when radiant heating from the sun warms the concrete above the ambient temperature, increased wind speed in this case will result in increased energy transfer from the concrete to the surrounding environment. Accordingly, thermal gradients in the concrete decrease [12]. In other words, convective cooling for solar exposed surfaces may counteract the radiant heating of the sun and reduce the thermal gradient in the concrete. Conversely, if the ambient temperature is warmer than the concrete, for example when ambient temperatures are increasing but there is no heating from the sun, then the convective heating transfer is negative. In this case, increased wind speed will accelerate the heat transfer to the concrete, which increases the thermal gradients in the concrete, thereby increasing the detectability of a subsurface delamination.

Unmanned aerial vehicles technology

A general definition of remote sensing can be summarized as the collection and measurement of spatial information at a distance from the data source, without direct contact. Unmanned aerial vehicles (UAVs) are a remote sensing technology that have been referred to as drones, robot planes, pilotless aircraft, remotely pilot vehicles, remotely pilot aircrafts, and other terms which describe aircraft that fly under the control of an operator with no person aboard. For positioning, a UAV is commonly equipped with a GPS. Generally, a UAV remote sensing system consists of four main features: aircraft with sensor(s) for data acquisition; remote control for the entire craft; GPS for navigation; and inertial measurement unit (IMU) for attitude measurement [3]. All flight parameters (e.g. longitude, latitude, time, GPS altitude, position etc.) are usually stored in a log-file together with the aircraft status parameters (e.g. battery status, flight modus and GPS signal quality). Additionally, the camera parameters (position, image number, altitude relative to launching area etc.) are also stored in the log-file, and thus are available for processing the individual images [14]. During the UAV operations, the acquired images are stored on the SD card of the camera. Recently, ground station software releases on mobile device platforms have allowed operators to use tablets and smart phones to control aircrafts. Micro and mini UAVs are the smallest UAV technology, and a large variety of these commercial aircrafts have been recently developed. Their platforms can fly at low altitudes and are the most used for civil engineering applications. These UAVs are able to take off and land vertically, requiring less area to operate, and can hover over fixed areas. They have been applied in surveying construction sites, monitoring work-in-progress, creating documents for safety, and inspecting existing structures, especially inaccessible areas. The most critical factor in such applications is the movement of the camera directly caused by the movement of the flight system, which reacts to changes in wind conditions due to its low weight [3].

Thermal imaging with a UAV has several advantages compared to satellite and ground-based measurements. It generates spatial data with good geometric resolution without the need to interpolate the data which might cause errors. Satellite imaging, in contrast, requires very high resolution data in order to obtain a high geometric accuracy. To successfully apply a UAV-borne thermal imaging, some important parameters should be considered [14]. For instance, the camera has to be sufficiently lightweight due to the limited payload of the UAV. It also has to be robust

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

(e.g. insensitivity towards vibrations and dust). Thermal cameras for the (8–14) μm ($1\ \mu\text{m} = 3.94 \times 10^{-5}\ \text{in.}$) bandwidth generally do not have a cooled sensor, resulting in a low signal to noise ratio. However, these camera systems are relatively lightweight, which is an essential advantage for use in UAV. It should also be noted that some lightweight infrared cameras do not have an autofocus. Thus, before mounting the camera onto the UVA the focus has to be adjusted manually by focusing at an object at the same distance as the planned flight altitude. Additionally, visual images or video recorded at the same time as the thermal image allow mapping thermal subsurface anomalies and differentiating the surface defects. The flight altitude is another parameter that has to be selected via a compromise between resolution and efficiency. If the data are not recorded continuously (video), then for satisfactory mosaicking a set of thermal images, the overlap of individual neighbouring images should be at least 50 %. Thus, only the central part of the images is used, which improves the quality of the composite image. Setting ground control points for further processing is another consideration. These points should preferably be visible in both the thermal and visual bandwidths. Therefore, a material with an emissivity as well as visual contrast to the survey area has to be chosen (e.g. aluminum foil). These marker points have to be positioned in the field accurately. The images can then be stitched to fit with the markers at the respective coordinates. The flight time of a UAV depends on its payload and is also strongly influenced by wind conditions [14].

DATA COLLECTION

The UAV-borne thermal survey for this study was conducted onsite in a real-world application on two in-service RC bridge decks located in the city of London, Ontario. The bridges were scheduled, by the Ministry of Transportation Ontario (MTO) for rehabilitation because delaminations, the extent of corrosion and high chloride ions content in the bridges were recently identified by a condition survey. Thus, the bridges were considered as good candidates and selected for the UAV survey. The two bridges, (named herein bridge (1) and bridge (2)) were constructed in 1965 as single span cast-in-place RC frames passing over a creek. The frames are supported directly on the abutments, and paved with about 100 mm (4 in.) asphalt wearing surface, as indicated in the structural drawings. The bridge structures have a north-south orientation and their outer limits have concrete curbs and steel handrails. The critical deck characteristics include a total length of 15.24 m (50 ft.) with a transverse width of 12.18 m (40 ft.), which supports one lane of traffic with one side shoulder in each direction. The structural drawings of both bridges indicate a deck slab thickness varying from 0.91 m (3 ft.) at the abutments to 0.48 m (1.6 ft.) at the mid-span of the deck with a parabolic soffit. Figure 1 illustrates the surveyed bridge (2) and a view of the condition of its deck. The utilized drone, thermal camera and the data collection methodology are briefly discussed below.

The utilized aircraft in this study was Inspire 1 Pro, a quadcopter drone, which is equipped with a retractable arm system whereby the underside of each propeller motor has a foot to act as a landing pod when the arms are lowered. Thus, the propeller arms rest in a lowered position when the drone is on the ground and once in the air, they can be retracted to give the camera unobstructed views across 360-degrees. Using intelligent flight batteries allows up to 18-22 minutes of flight time and a maximum flight speed of 18 m/s (59 ft/s). The flight battery constantly supplies the flight computer with remaining voltage. Applying special algorithms, the computer supplies the operator with estimates of remaining flying distance and time to return. The drone relies on a GPS-based stabilization system that is able to hold a position even when experiencing wind interference and can automatically bring the aircraft to the home point in the event of signal loss. The system allows the drone to hold a position with $\pm 2.5\ \text{m}$ ($\pm 8.2\ \text{ft}$) horizontal and $\pm 0.5\ \text{m}$ ($\pm 1.64\ \text{ft.}$) vertical accuracy. Table 1 summarizes the specifications and features of the drone.

The FLIR Vue Pro thermal camera was utilized in this study. The camera weighs 113 grams (4 ounces), which makes it ideal for UAV applications. The camera is compatible with the utilized drone and can be mounted via an integrated 3-axis gimbal to maximize stability and weight efficiency while minimizing space. The gimbal uses brushless servo motors to keep the shot stable and the horizon level. The camera pans a full 360-degrees so that no matter which way the quadcopter turns; the camera can remain locked on the subject. Manual panning and tilting can be affected using controls on the transmitter (radio controller). Once the controller is ready to go, a tablet or smartphone can be attached, which allows the operator to see what the drone is viewing along with other statistics concerning its flight. The camera processes the temperatures by an automatic gain control (AGC) into a maximum of 256 shades of gray to display a thermal image. Since the imaging system is sensitive enough to distinguish many more than 256 different temperatures, each “shade of gray” represents a range of temperatures. The raw scene data could be stored as still images (TIFF, JPEG or in FLIR File Format (FFF)) for post processing using FLIR image processing software. It has an uncooled microbolometer detector and displays thermal images with a resolution of 640 x 512 pixels. Table 2 summarizes the specifications and features of the camera.

The UAV-borne thermal imaging in this study was conducted by a consultant specializing in aerial photography, videography and thermography who has completed UAV training, IRT training, pilot licensing, and acquired special risk aviation insurance and gained a special flight operation certificate as per the requirements of the Canadian Aviation Regulations. The setup of both the drone and the thermal camera was firstly completed by the UAV operator. This includes the installation of the propellers and downloading the drone and camera operating software. The DGI Go app controls the flight and allows for a real-time view of a live video to line up the perfect shot where the data is automatically saved on the used mobile device. The FLIR Vue Pro app allows the setup of image format, orientation, shutter speed, camera focus adjustment, shooting mode, and color palette and communication options with the mobile device. All the batteries were recharged and the camera was mounted on the drone and oriented facing straight-down to be perpendicular to the concrete deck surface being investigated. It was not possible to measure the angle while flying. Hence, an estimation of the drone's perpendicular position to the deck surface was made from the live stream video of the flight displayed in real-time on the monitor. Figure 2 illustrates the utilized drone and thermal camera.

Time of data collection is the most critical factor in a passive IR survey. In a study conducted by Washer et al [12] on concrete bridges, it was found that the effective time to perform a thermal IR test depends on the depth of the delamination. The most contrast appears on the thermal IR image approximately 4 hr after sunrise for a 50 mm (2 in.) deep delamination and 7 hr after sunrise for a 75 mm (3 in.) deep delamination. Therefore, the surveys were conducted on May 24th, 2016 at 1 PM, six hours after sunrise. During the survey, the ambient temperature was 26° C (79° F), relative humidity was 22%, and wind speed was 8 km/hr (5 mi/hr.). Bridge deck dryness was considered during data collection since surface moisture can reduce the thermal contrast on thermal IR images. Road traffic was not closed during data collection of the two bridges. While the total time taken to complete a bridge deck inspection from setup to tear-down was approximately 20 minutes, the total flight time was about three minutes for each bridge deck. A calibrated 9 mm (0.35 in.) focal lens (69 x 56) was utilized, which allowed a FOV of 13.74 m (horizontally) x 10.62 m (vertically) (45 ft. x 34.8 ft.) for each image taken at a constant 10 m (32.8 ft.) altitude and resulted in 2.5 cm (1 in.) spatial resolution. Hence, a total of four thermal images, at spacing of 5 m (16.4 ft.), were collected with an overlap of 50% to cover the entire deck length. Each image covers the entire deck width and shoulders, in addition to the sidewalks at both sides. The still images were taken at the center of each bridge deck. Figure 3 illustrates the UAV hovering process and a captured thermal image.

DATA ANALYSIS

The captured IR images of the bridge decks were firstly pre-processed. Several functions in the *ImageJ* software were utilized to improve the images' quality. For instance, the Gaussian smoothing filter was used to reduce noise whereby the software selects the size of the filter automatically. The histogram equalization function was used to distribute the intensities on the histogram, which in turn enhanced the thermal contrast. The temperature data of each image was then saved in a separate Excel file. To create a mosaic thermogram of the entire bridge deck from the individual images, a selected window from each image was extracted and stitched together with the extracted window from the next image and so on. The height of each stitch window is the number of pixels that are equivalent to the images' spacing. Hence, this appending process produces continuous data for each survey pass. The dimension in pixels depends upon the pixel resolution in each direction. For the bridges under investigation, the dimension of the mosaic for each deck was $640 \times 780 = 499,200$ pixels. A Matlab code was written by the authors to extract and stitch the selected pixels from each image. For visualization and to simplify further processing, it was necessary to scale-down the data dimension of the mosaic. This was achieved by assuming the entire deck divided into small boxes of 10 cm x 10 cm (4 in. x 4 in.) in dimension in which the temperature does not change, then computing the average of pixels, which represents the resolution within the selected dimension. This resulted in a reduced file with $128 \times 156 = 19,968$ pixels. The mean function in Matlab was modified and a new code was written to calculate the mean for specified dimension of input data. The code at first operates and returns a raw vector containing the mean of each column element, and then provides a scalar value, which represents the mean of the elements in the raw vector. Another Matlab code was also written to identify the coordinates of each pixel in the temperature data file where the origin point (0, 0) was assigned to the corner of the bridge deck, then the coordinates of all other pixels in the file were computed. The final analysis output was an Excel file including pixel information (co-ordinates and temperature) arranged in a column order. The developed Matlab codes can customize the analysis process for any other IRT survey data. It should be noted that a more rigorous analysis could be achieved based on a photogrammetric 3D reconstruction and generation of orthophotos. There are several commercial software capable of producing accurate 3D stereomodels, orthophotos and orthomosaics from data collected by various sensor systems. These orthophotos and mosaics are geographically referenced and commonly created using automated tools

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

for tie-point matching, orthorectification, relative orientation, and color-balancing. However, it should be noted that inaccuracies in input control values could cause problems in the photogrammetric modeling process, and thus it is preferable to employ the capture app belonging to the utilized software in the data acquisition process to ensure adequate quality imagery and accurate geospatial control.

Thresholding classification using the saved scaled temperature values was implemented in the present study to identify the defective areas in the concrete deck. Firstly, the thermal contrasts were calculated from the scaled temperature values of the mosaicked thermogram. It was then necessary to select a threshold value for the calculated thermal contrasts that could indicate where delamination was likely to be detectable. As per the ASTM D4788-03 [11], 0.5°C (0.9° F) is the minimum contrast to distinguish between sound and defective concrete. Because the bridge decks are asphalt paved, the thermal contrast of 0.5°C (0.9° F) could be due to the effect of pavement thickness. In addition, spalling could be hidden under the asphalt layer, and thus a higher thermal contrast value of 1°C (1.8° F) was selected. It is about 20 times higher than the sensitivity of the utilized camera such that much smaller variations in surface temperature were easily detectable. The pixels with a higher thermal contrast than this threshold value were highlighted because delaminations appear as higher temperature features. Finally, the temperature values were grouped and a condition map was plotted using a commercial mapping software to construct a high contrast image, accurately delineating the geometry and location of subsurface delaminations. Figure 4 illustrates the created maps of the two surveyed bridge decks where the delaminated areas are highlighted in red color and sound areas in blue color.

Quantitative analysis of UAV-borne thermal data enables an accurate determination of the temperature for a defective region. Thus, calculating the total delaminated areas from the produced mosaicked thermogram in Fig. 4 provides a quantified basis for powerful decision making and hence, for prioritizing alternatives in bridge deck rehabilitation plans. For this purpose, the pixels with thermal contrast higher than the threshold were highlighted inside specific boundaries. The total percentage of delaminated areas in the bridge deck was then determined by computing the total percentage of pixels in higher temperature areas over the total bridge deck. To some extent, this could be a qualitative analysis since it is difficult for different analysts to have consistent judgment in selecting the threshold. The authors are currently developing a procedure to determine objective thresholds in order to categorize the detected delaminated areas in different classes based on the severity levels of the delamination.

VALIDATION USING RESULTS OF OTHER NDT

The achieved results were validated using the results of hammer sounding and half-cell potential corrosion tests conducted on the same bridge decks. The hammer sounding was conducted by a bridge inspector from the MTO, before the UAV-borne thermal imaging survey. The hammer sounding is a qualitative evaluation of concrete and was conducted by tapping the bridge deck with a hammer. When the hammer is struck on good concrete, a ringing sound is created. Conversely, on areas where delaminations or cracks occur, the striking of the hammer produces a drum-like sound. The delamination maps created by IRT for both bridge decks were compared with the hammer sounding results. It was found that the two methods provided considerably similar sizes and shapes of defects in the two decks. The quantified delaminated areas by both techniques are presented in Table 3 below. For the first bridge, the hammer sounding results showed 17% total delaminated areas on the bridge deck, whereas the total delaminated area calculated from the thermal infrared imagery survey was 15.4% of the total bridge deck area (1.6% difference between the two methods). Similarly, the total delaminated areas on the second bridge deck identified by the hammer sounding and IRT were 32% and 29.3%, respectively (2.7% difference between the two methods). Thus, the UAV-borne thermal imagery system with the presented analysis procedure can define the location and extent of delaminations in bridge decks with considerably reasonable accuracy.

The two surveyed bridges were also investigated using the half-cell corrosion potential testing technique. Corrosion of reinforcing steel is an electro-chemical process and the behaviour of the steel can be characterised by measuring its half-cell potential to indicate the probability of active corrosion. The greater the potential the higher the risk that corrosion is taking place. An electrode forms one half of the cell and the reinforcing steel in the concrete the other. The preferred reference electrode for using in situ is silver/silver chloride in a potassium chloride solution although the copper/copper sulphate electrode is still widely used. However, the survey procedure was conducted as per the requirements of ASTM C876-09 [15] and started firstly by locating the steel and determining the bar spacing using a covermeter. The cover concrete was removed locally over a suitable bar and an electrical connection made to the steel. It was necessary to check that the steel was electrically continuous by measuring the resistance between two widely separated points. The reinforcing bar was connected to the half-cell via a digital voltmeter. Readings of half-

cell potential were taken over a grid of points to give a potential map of the area. The survey was conducted a few weeks before the UAV-borne thermal survey, by a consulting firm specializing in NDT of concrete structures, as part of a condition survey program with the MTO to evaluate the condition of 24 highway bridges. In addition, three drilled concrete core samples for visual inspection and chloride ion concentration analysis were retrieved from each bridge deck. The quantified results are presented in Table 3. The core samples indicated actual overlay thickness of about 50-60 mm (2-2.4 in.), while the delamination layers were observed at about 45-50 mm (1.8-2 in.) depth from the concrete surface.

For the first bridge, The HCP results showed probable active corrosion for 14.1% of the total deck area with corrosion potential values more negative than -0.350 V, whereas the total delaminated area calculated from the IRT results was 15.4% (about 9% difference in total defective area between the IRT and HCP surveys). Evidence of corrosion was not observed on the reinforcement encountered in the core extracted from the deck as the high corrosion potential areas were mostly found along the curb areas. The chloride content at the level of the reinforcing steel was below the chloride threshold level of 0.025%, by mass of concrete, in the three cores tested for this bridge deck. For bridge (2), the HCP results indicated probable active corrosion for 27.4% of the total deck area with corrosion potential values more negative than -0.350 V, whereas the total delaminated area calculated from the IRT results was 29.3% (about 7% difference in total defective area between the IRT and HCP surveys). The areas of potential active corrosion, as indicated by the HCP testing, have been validated by chloride ion concentration analysis of the three cores extracted from the deck and were found to be 0.033% by concrete mass. The core samples were extracted from areas prone to salt exposure, such as construction joints. In spite of the different mechanisms of the IRT and HCP techniques and their capabilities of indicating the potential of different deterioration types, it was noted that their condition maps were comparable to a high degree with respect to the location and shape of defects. It should also be noted that corrosion is among the primary reasons that could lead to delaminations in RC bridge decks, and thus could be the reason of obtaining potential corrosion areas smaller than the detected delaminated areas.

DISCUSSION AND FURTHER RESEARCH

Effective monitoring strategies are paramount in achieving the service life goals of bridge structures. Several remote sensing techniques have been used to assess conditions in different infrastructure applications. Incorporation of these techniques into the bridge monitoring and condition assessment scheme has the potential to not only enhance inspection practices, but also provide tentative assessments between inspection cycles, with minimal traffic disruptions. Vaghefi et al [16] argued that many aspects of bridge inspection could be aided by UAV remote sensing. For instance, a UAV-borne thermal imaging system deploys a UAV and utilizes an infrared thermal camera to provide spatially distributed information of the concrete surface temperature without the need to access the surface and hence, could play a major role in bridge condition assessment. The present study discusses a novel method of condition assessment for RC bridge decks based on airborne thermal photos taken by a UAV. The rationale of the analysis presented herein is that creating a plan view mosaic of the entire bridge deck facilitates the detection of defective areas. The thermal contrast developed from the concrete subsurface provided numerical values for the color contrasts that were used to process data more effectively than simply comparing multiple images. The specially written Matlab codes solved the question of how to create a mosaicked thermogram of the entire bridge deck from the individual thermal images.

The study demonstrated the applicability of this system in detecting subsurface delaminations in the surveyed RC decks. The developed methodology provided comparable condition measurements in a manner more efficient than standard practice. The achieved results comply with the results of previous studies. For example, Metni and Hamel [17] successfully utilized UAV for different types of inspections including bridges. Similarly, Brooks et al [18] investigated several applications of UAV technology including bridge inspection. They used the UAV to capture imagery, both digital and thermal, of the bridge deck and applied some algorithms to detect surface defects on the deck. Khan et al [19] collected thermal images of a mock up bridge to demonstrate the types of data that can be collected with a UAV and were able to detect possible delaminations in the concrete bridge deck.

The UAV-borne thermal imaging system could be a valuable option to inspect inaccessible bridge components. For instance, subsurface delaminations within the bridge soffits can be imaged and detected. Thus, the system has the advantage of reducing the time required to conduct inspection and the costs associated with using special access vehicles. Such detection of delaminations and damage underneath the deck and within the bridge soffits is always a challenge for the bridge inspector due to the safety measures required for passing traffic under the bridge. In spite of

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

the benefits of UAV systems, they have some limitations that require further research and development [20]. For instance, the limited payload allows only small battery packs, which cause a relatively short flight time. In addition, due to the small payload the flight system is very sensitive to changes in weather conditions, especially during high wind speeds, which can compromise image quality. Conducting cost estimation studies that consider cost parameters associated with operating UAVs during bridge inspection at the network level is also required. Furthermore, several effective image post-processing algorithms are required for accurate detection and classification of defects. The use of UAVs that could be equipped with multi-sensors along with inertial and spatial sensors to produce geo-registered 3D data for bridges should be investigated to enhance the bridge inventory management. It should also be noted that to utilize UAVs as an effective tool in bridge inspection, qualified bridge inspectors should be trained to operate the UAVs systems to provide high flight performance and objective inspection processes. Hiring private companies that offer UAV-based data collection services could also be another option.

CONCLUSIONS

The present study aims at advancing the UAV technology as a reliable and effective tool for bridge inspection, allowing improvements to the safety of highway bridges and better use of limited budgets of transportation agencies. Specifically, the application of UAV-borne thermal imaging systems for detecting delaminations in RC bridge decks was explored and a systematic data analysis of the captured images was developed. The results confirm previous findings and contribute additional evidence for the capability of the system to enhance bridge inspection methods, potentially achieving less-time consuming and more economical assessment that also creates a safer working environment. For efficient application of the system, the user should select the most compatible system of UAV and thermal camera, then plan and perform a successful field investigation, and finally derive a composite image mosaic of the concrete surface temperature patterns based on the acquired individual images. However, there are some challenges that require further research and are discussed in this study. While the attention herein has been mainly devoted to the condition evaluation of RC bridge decks, UAV-borne thermal imaging can be advantageously exploited for surveying other bridge components. Thus, by facilitating the early UAV-borne thermal imaging detection of deterioration in bridge components, transportation agencies should be able to apply proactive maintenance actions. The authors plan to explore the applicability of the deployed system to bridge soffit areas, in addition to studying the relation between the flight altitude and the accuracy of detecting subsurface anomalies in RC bridge decks. In conclusion, UAV-borne thermal imaging is a promising method to retrieve highly resolved spatial information on concrete surface temperature within a short amount of time. Once the UAV-borne thermal images become available, the introduced procedure can analyze and determine whether a defect exists, and hence is useful in quantifying the delaminated areas present in a bridge deck. This system can be implemented in bridge inspection manuals to aid safer, more frequent, and potentially less costly bridge deck inspections and enable more informed decision-making.

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Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

Table 1-Features and specifications of the utilized aircraft

Drone (Inspire 1 Pro)	
Characteristic	Specification
Aircraft Model	T600
Motor Model	DJI 3510H
Maximum Take-off Weight	3500 g (125 oz.)
Vertical Hovering Accuracy	0.5 m (1.64 ft.)
Horizontal Hovering Accuracy	2.5 m (8.2 ft.)
Maximum Aircraft Speed	18 m/s (59 ft/s)
Maximum Altitude	120 m (394 ft.)
Maximum Flight Time	18-22 Minutes
Maximum Wind S. Resistance	10 m/s (32.8 ft/s)
Operating Temperature Range	-10 ⁰ C to + 40 ⁰ C (14 ⁰ F to + 104 ⁰ F)
Propeller Model	DJI 1345 T
Maximum transmitting Distance	3.5 km (2.18 miles)

Table 2- Features and specifications of the utilized thermal camera

Thermal Camera (FLIR Vue Pro)	
Characteristic	Specification
Size	62.7 mm x 44.5 mm (2.47 in. x 1.75 in.)
Weight	101 g to 122 g (3.61 oz. to 4.36 oz.)
Detector Type	Uncooled Microbolometer
Spectral Band	7.5µm – 13.5µm
Thermal Resolution	640 x 512 pixels
Full Frame Rate	30 Hz (15 fps)
Field of View	9 mm (0.35 in.) Lens (69 ⁰ x 56 ⁰)
Input Supply Voltage	4-6 VDC
On-board Storage	MicroSD
Operating Temperature Range	-20 ⁰ C to +50 ⁰ C (-4 ⁰ F to +122 ⁰ F)
Operational Altitude	+ 12,192 m (40,000 ft.)
Polarity Control	Yes

Table 3- Percentage of defective areas in the surveyed decks

Bridge Deck	% Delaminated Areas (UAV-borne thermal)	% Delaminated Areas (Hammer Sounding)	% Potential Corrosion Areas (HCP)
(1)	15.4	17	14.1
(2)	29.3	32	27.4



Figure 1-East elevation of the surveyed concrete bridge (2) and an overview of the deck condition.



Figure 2-Aircraft (Inspire 1 Pro) and thermal camera (FLIR Vue Pro) utilized in this study.



Figure 3-Setup of UAV-borne thermal system and a snapshot during flight over bridge (2).

Thermal Detection of Subsurface Delaminations in Reinforced Concrete Bridge Decks Using Unmanned Aerial Vehicle

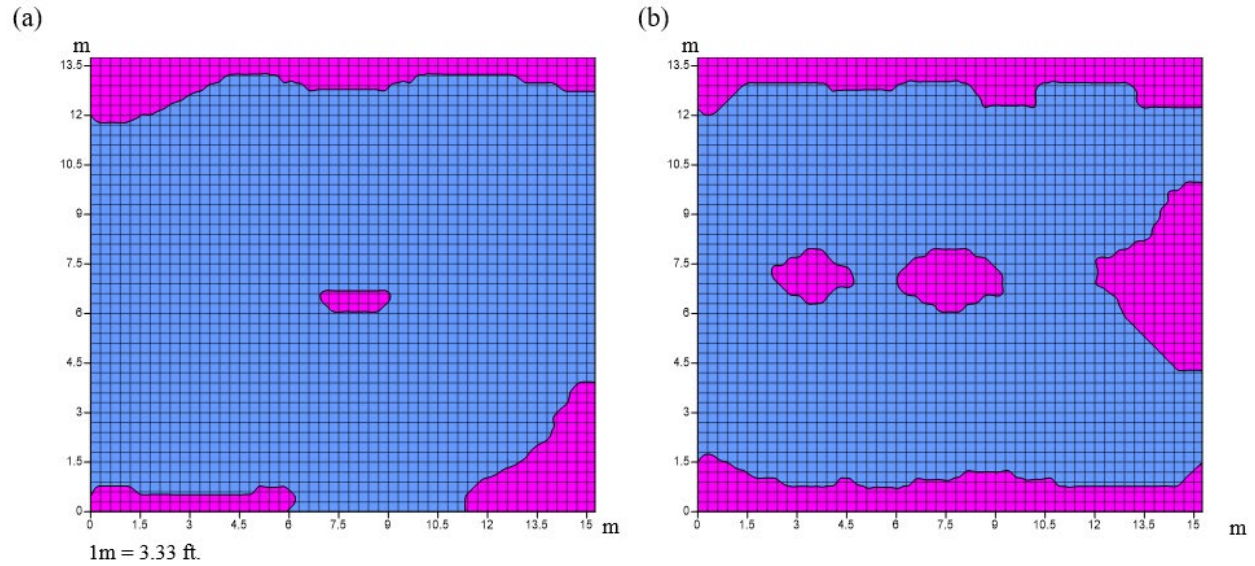


Figure 4- Delamination maps of: (a) top surface of bridge deck # 1; (b) top surface of bridge deck # 2 surveyed using UAV-borne thermal system.

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

Ian Shaw, Hang Zhao and Bassem Andrawes

Synopsis: Fiber reinforced polymer (FRP) composites have emerged as a lightweight and efficient repair and retrofit material for many concrete infrastructure applications. FRP can be applied to concrete using many techniques, but primarily as either externally bonded laminates or near-surface mounted bars or plates. This paper presents the results of direct shear pull-out tests performed on aged concrete specimens reinforced with glass FRP (GFRP) and carbon FRP (CFRP) externally bonded laminates and near surface mounted (NSM) bars. An accelerated aging scheme consisting of freeze/thaw cycling in the presence of a deicing salt solution is implemented to determine the effect of long-term environmental exposure on the FRP/concrete interface in regions that experience aggressive winter environments. The results show that the NSM bar technique is superior to externally bonded laminates in terms of efficiency in the use of FRP material and the effects of accelerated aging. Generally, the performance of GFRP is affected less than CFRP after freeze/thaw cycling for both externally bonded laminates and NSM bars. For high strength NSM FRP bar applications, a spalled or cracked concrete surface caused by freeze/thaw cycling may drastically reduce the capacity of the FRP/concrete interface by inducing failure at the concrete/epoxy filler interface.

Keywords: Accelerated aging; Concrete; Durability; Freeze-thaw; FRP; NSM.

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INTRODUCTION

As a large portion of the concrete infrastructure in the United States approaches the end of its service life, rehabilitation and repair techniques have become a popular alternative to more costly replacement. In particular, deterioration of the nation's bridges is critical, with 58,000 bridges being deemed structurally deficient in 2016¹. These bridges are largely located in areas of harsh climates, such as in the Northeast and Midwest. In these locales, bridge girders may see excessive deterioration at the end regions due to expansion joint failure, which in turn allows water to flow onto the girder ends. This water contains deicing salts, which in combination with multiple freezing and thawing cycles causes scaling and spalling of the cover concrete and eventually corrosion of the steel reinforcement. This corrosion is exacerbated by the chlorides contained in the deicing salts. Deterioration of concrete infrastructure has led to the emergence of alternative methods of rehabilitation to external steel reinforcement, which is susceptible to corrosion.

Fiber reinforced polymer (FRP) composites have emerged over the past two decades as an effective repair and rehabilitation material for many concrete infrastructure applications. Advantages of FRP materials over conventional steel reinforcement are their high strength to weight ratio and resistance to corrosion. The two primary FRP systems used for these applications are externally bonded laminates and near-surface mounted (NSM) bars or plates. Externally bonded systems are comprised of a fiber sheet or mat (typically glass or carbon), which is impregnated with an epoxy resin to create the composite material. One method of applying the laminates to the concrete surface is through a wet layup approach, in which the epoxy resin serves to both saturate the fibers and bind the sheet to the concrete surface at the same time. Another approach is to glue pre-cured laminates to the concrete surface using an epoxy resin. NSM FRP systems are installed by first cutting a groove into the concrete surface, then fastening the bar/plate into the groove with a filler material, such as an epoxy resin or cement grout. One advantage of NSM systems compared to externally bonded laminates is that they protect the FRP reinforcement from direct environmental exposure.

The tests presented in this paper are part of a larger project investigating the use of FRP composite materials as a means of repairing and retrofitting damaged ends of prestressed bridge girders. However, this study functions independently, since the results can be applied to any application of these types of FRP systems to concrete infrastructure. Use of externally bonded FRP laminate and NSM FRP bar repair and strengthening systems for concrete bridge infrastructure was originally focused in flexural applications. More recently, externally bonded laminates have emerged as an effective means of strengthening beams in shear by way of U-wraps or bonded face plies², but effectiveness of these systems is limited by a delamination failure mode, which occurs at an effective strain much lower than the ultimate strain achievable by the FRP composite materials themselves³. Research has been performed on NSM bar systems for shear applications^{4,5,6,7,8,9}, but so far these results have been less codified than externally bonded laminates in shear. Furthermore, while FRP composite materials are generally considered environmentally resistant, the bond between the FRP and concrete may be adversely affected by the exposure to high levels of water, deicing salts, and freeze/thaw cycling applied to the end regions of these concrete bridge girders. ACI 440.2R-08 gives environmental reduction factors for various FRP systems and exposure conditions which limit their design rupture strains³. However, for shear design, even the most stringent of these reduction factors (0.5 for glass FRP systems in an aggressive environment) might not actually reduce the effective strain in shear applications, which is capped at 0.004 due to loss of aggregate interlock of the concrete. Thus, it is important to investigate further the effects of environmental exposure on the bond behavior of externally bonded and NSM

FRP systems in order to determine if these systems will be effective long-term in repairing the end regions of concrete girders.

Much experimental and analytical research has been performed on the bond-slip behavior of FRP laminates/plates^{10,11,12} and NSM bars/strips^{13,14,15,16,17}. Additionally, much research on environmental aging effects on the FRP-concrete interface has been performed, but conclusions vary significantly. Columbi et al. conducted pull-pull debonding tests on CFRP plates and wraps subjected to 100 and 200 freeze-thaw cycles and concluded that the conditioning did not significantly affect the ultimate load¹⁸. Silva et al. subjected reinforced concrete beams with CFRP and GFRP laminate systems to salt fog and wet-dry cycles for 10,000 hours¹⁹. Salt fog cycles were more detrimental than moisture cycles for either CFRP or GFRP, and in the case of GFRP the external load capacity was actually increased after the moisture cycling. Davalos et al. investigated the fracture energy behavior of GFRP-concrete interfaces subjected to freeze-thaw cycling in the presence of a 4% CaCl₂ solution. This aging protocol resulted in fracture energy decreases of 38.5%, 50.5%, and 59% for normal strength concrete specimens at 100, 200, and 300 cycles, respectively²⁰. Subramaniam et al. conducted direct shear tests on CFRP sheets bonded to concrete and saw a 17% reduction in ultimate load and a 35% reduction in fracture energy after 300 freeze-thaw cycles²¹. Al-Mahmoud et al. performed hinged beam tests with CFRP sheets and plates subjected to either 300 freeze-thaw cycles or 120 days of salt water immersion. The freeze-thaw cycles caused a 25% decrease in ultimate force for both the sheets and plates, while the salt water caused a deterioration of 48% for the sheets but negligible change in the plate specimens. Salt water immersion also changed the failure mode of the interface from debonding in a thin concrete layer to failure at the concrete/resin interface²². Garzón-Roca et al. performed pull-out tests on NSM CFRP strips with different bond lengths (60 mm (2.4 in.) vs. 90 mm (3.5 in.)), groove widths (4 mm (0.16 in.) vs. 8 mm (0.31 in.)), groove depths (15 mm (0.59 in.) vs. 25 mm (0.98 in.)), and an aging scheme with 90 wet-dry cycles. The wet-dry cycling caused a decrease of 12% in the maximum pull-out force for specimens with 15 mm (0.59 in.) groove depth, 4 mm (0.16 in.) groove width, and 60 mm (2.4 in.) bond length. However, the effect was significantly reduced or became negligible with increased bond length or groove width, respectively²³. Soliman et al. performed pull-out tests on NSM CFRP and GFRP bars subjected to 200 freeze-thaw cycles. Prior to freeze-thaw cycling, service cracks with a maximum width of 0.3 mm (0.012 in.) were developed in the specimens by tensioning up to 30% of the failure load. The conditioning caused hair cracks to form in the epoxy adhesive, which resulted in pull-out force reductions of 8-14% compared to the reference specimens²⁴. The wide range of testing setups, materials used, aging protocols, and failure modes observed in the literature make it difficult to definitively characterize the effects of long term aging on the FRP-concrete interface. Tests which consider the combination of freezing/thawing and salt water exposure on the FRP/concrete interface are lacking, specifically with regards to CFRP laminates and both GFRP and CFRP NSM bar systems. Thus, this study was focused on filling these knowledge gaps.

In this paper, the experimental results of shear bond-slip tests involving both FRP externally bonded laminate and NSM bar specimens are presented. An accelerated aging scheme was developed and applied to a set of these specimens in order to determine the effects of long-term exposure to water containing deicing salts and freeze/thaw cycling. The results are analyzed to determine the most effective and efficient FRP material type and application technique while considering long-term viability of the repair.

RESEARCH SIGNIFICANCE

The results presented in this work provide better understanding of the durability and bond behavior of both externally bonded FRP laminates and NSM FRP bars using accelerated aging method. Based on the results, it is believed that in general, the bond behavior of NSM FRP bars exceeds that of externally bonded FRP laminates especially when both are subjected to harsh environmental and chemical conditions.

MATERIAL DESCRIPTION

Both glass FRP (GFRP) and carbon FRP (CFRP) laminate and NSM bar systems were tested in this study. A wet layup approach was adopted to bind the FRP laminates to the concrete surface. The dimensions of the FRP systems used in the tests and the material properties are summarized in Table 1. The FRP laminates used in this study are unidirectional VU27G glass fabric and unidirectional TU27C carbon fabric. The saturating resin used is J300SR saturating resin. All the fabrics and resin are manufactured by QuakeWrap. For the FRP laminates and bars, both the manufacturers' properties and those acquired from laboratory tests are presented. The GFRP laminates have a lower elastic modulus and tensile strength, so based on strength only, field repairs would likely require more GFRP material than CFRP. That said, other aspects such as fatigue, creep, and durability should be considered when designing FRP materials. Since the main purpose of this study is to determine how accelerated aging will affect

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

different FRP repair systems for the same application, two plies of GFRP were used compared to one ply of CFRP for the laminate tests. The NSM bars utilized in this study are provided by Aslan FRP. The external surface of GFRP and CFRP bars used were sandblasted in order to provide increased bond with the epoxy filler. The GFRP bars had a spiral winding on its surface as well, and the sand coating was slightly coarser than that of the CFRP bars. Epoxy adhesive (HIT-RE 500-SD) was used as the groove filler material for the NSM bar specimens, per recommendation by the manufacturer of the FRP bars. The weight ratio of the composition of concrete mix is water: Portland Type I cement: coarse aggregate: fine aggregate = 0.4: 1.0: 2.0: 1.3. The average 28-day compressive strength of 152×304 mm (6 x 12 in.) concrete cylinders was 44.8 MPa (6,500 psi).

SPECIMEN DESCRIPTION AND TEST SETUP

Experimental tests on the bond-slip behavior of FRP laminates and NSM bars are generally performed using either a direct pull-out configuration or a hinged/split beam approach. The direct pull-out configuration was chosen in this study due to its simplicity and ease at which the accelerated aging scheme could be implemented. A total of 36 concrete blocks with dimensions of 305 x 89 x 89 mm (12 x 3.5 x 3.5 in.) were cast. The concrete specimen dimensions and test setup were designed to accommodate the testing of both FRP laminate and NSM bar specimens (Fig. 1). The top and bottom fixture plates included openings for both the laminates and NSM bars. The FRP laminate specimens were tested on a machine with an 89 kN (20 kip) load cell, and a set of 120 mm (4.7 in.) wide serrated grips were used to grip the FRP laminate. FRP tabs were glued the gripping end of the laminates prior to testing in order to prevent FRP rupture at the grip location. The FRP NSM bar specimens were tested on a machine with a 222 kN (50 kip) load cell with hydraulically pressurized grips. To prevent crushing of the FRP bars by the hydraulic grips, a steel pipe filled with high strength non-shrink grout was used as an anchor at the loading end of the bar. For both FRP systems, slip at the free end of the laminate/bar was measured at the center of the overhang of FRP laminate or FRP rebar using an extensometer.

In addition to FRP system and material type, two other variables were investigated: bond length and accelerated aging duration. In order to reduce the influence of the concrete compressive stresses on the FRP/concrete interface under the loading plate, all bond lengths were started 51 mm (2 in.) from the top edge of the concrete specimens. For the laminates, each FRP strip had 76.2 mm (3 in.) in width and 508 mm (20 in.) in length. Within such length, 178 mm (7 in.) overhang at the top was left for gripping and 25 mm (1 in.) overhang was left at the end to connect extensometer. Bond length was prescribed by marking the bonded area on one face of the concrete block and applying clear tape outside of the marked area. The surface of the bonded area was textured and roughened with a hard steel wire brush and was air blasted afterwards so that a clean and rough surface was produced before applying the FRP laminate. For the NSM bar specimens, a 9.5 x 9.5 mm (0.38 x 0.38 in.) groove was cut down the center of a face of the concrete block. This groove size meets the minimum groove dimensions ($1.5d_b$) and minimum clear edge distance ($4d_g$) as prescribed by ACI 440.2R-08³, where d_b is the diameter of the FRP bar and d_g is the groove depth. Bond length was prescribed by placing clay at the termination points of the epoxy filler. The complete test matrix is presented in Table 2. After the fixture was installed to the loading frame, monotonic uniaxial tensile test with loading rate equal to 0.03 mm/s (0.001 in./s) was performed.

ACCELERATED AGING PROTOCOL

The aggressive aging scheme used in these tests was designed to represent the harsh conditions that cause deterioration at the end regions of concrete bridge girders, where deicing salts leak onto the ends of the girders and multiple freezing/thawing cycles occur during the winter season. To incorporate the effect of aging in the pull-out tests carried out in this study, an aging protocol similar to that of Davalos et al. was applied²⁰. This protocol combined aspects of ASTM C666²⁵ and ASTM C672²⁶. Specimens were placed laminate/bar side down in trays containing 6 mm (0.25 in.) of solution of deicing salt and water. ASTM C672 specifies the use of CaCl_2 with a concentration of 4 g/100 ml (0.33 lb/gal) of solution, but states that other chemical deicers may be used. Since rock salt is commonly used as a deicing chemical, NaCl was used as a substitute, with an equivalent concentration of Cl^- (4.21 g/100 ml (0.35 lb/gal) of NaCl). One freeze/thaw cycle was performed per day ranging from -18 to 4 °C (0 to 40 °F). The cycle consisted of 16 hours to lower the temperature from 4 to -18 °C (40 to 0 °F), 6 hours at -18 °C (0 °F), and 2 hours to raise the temperature from -18 to 4 °C (0 to 40 °F) (Fig. 2). Freeze/thaw cycling was performed in a conventional chest freezer, with the use of an electric resistance heater to thaw the specimens. The trays were rotated throughout the cycling process in order to account for any uneven distribution of heat within the freezer. After 40 cycles, the concrete specimens showed moderate scaling, and half of the aged specimens were tested along with the unaged specimens. The rest of the specimens were continued on to 100 cycles, at which point they were tested.

TEST RESULTS

Effect of aging on specimen condition

Compared to the 40 cycle specimens, the 100 cycle specimens showed increased scaling of the concrete (moderate to severe), and in many cases coarse aggregate had been exposed and/or completely spalled off. Fig. 3(a) compares a set of concrete specimens with no aging, 40 freeze/thaw cycles, and 100 freeze/thaw cycles. The most severe of concrete damages occurred mainly at the ends of the specimens (Fig. 3(b)), which generally left the bonded portions of FRP intact. One laminate specimen (G-152-A100) had scaling on one side that resulted in loss of a minor portion of bonded area, as shown in Fig. 3(c). There were also two 100 cycle CFRP laminate specimens that had severely damaged ends that rendered the specimens untestable due to lack of bearing area against the plates in the test fixture, one of which is shown in Fig. 3(d). For this reason, the results for the 100 cycle CFRP laminates are omitted. Other than some salt depositions on the laminates, the FRP laminates, bars, and epoxy filler had no visible signs of deterioration.

FRP Laminates

Table 3 summarizes the test results for the FRP laminates. Fig. 4 presents the pull-out force data for the FRP laminates in a manner in which comparisons can easily be made between different bond lengths and aging durations. The maximum stress in the laminate $\sigma_{\max}$ was calculated by dividing the pull-out force by the laminate cross sectional area. The maximum strain in the laminate was computed by dividing $\sigma_{\max}$ by the modulus of the material. Fig. 5 shows a typical failure surface for the FRP laminate specimens. The failure mode for all specimens was sudden delamination at the FRP/concrete interface with complete debonding of the FRP laminate (FC). Typically, this type of failure mode occurs a few millimeters below the interface for normal strength concrete. In these tests, however, the delamination occurred through a layer of concrete less than 1 mm (0.039 in.). This indicates that surface preparation was likely insufficient, as the concrete failed in its weakest, outermost layer.

For the unaged specimens, the GFRP laminates saw an increase in pull-out force with increased bond length, while the CFRP laminates did not show this trend. Theoretically, the maximum pull-out force of any given FRP/concrete interface is achieved once the bonded length exceeds the effective bond length²⁷. These results imply that effective bond length of the GFRP laminates was greater than 152 mm (6 in.), while the effective bond length of the CFRP laminates was less than or equal to 102 mm (4 in.). Looking at each series of aged specimens independently, there was an increase in pull-out force with increased bond length for both GFRP and CFRP (with G-152-A40 being the only outlier). This implies that the effective bond length was increased due to aging. This aging effect coincides with most effective bond length equations used in the literature²⁷, where effective bond length is inversely proportional to concrete strength. The freeze/thaw damage likely caused reductions in the concrete strength at the FRP/concrete interface, resulting in increased effective bond lengths.

The effect of aging was more severe in the CFRP laminates than the GFRP. Across all bond lengths of GFRP specimens, 40 cycles and 100 cycles of aging resulted in average pull-out force decreases of 11.9% and 16.7%, respectively, from the unaged specimens. For the CFRP specimens, 40 cycles of aging resulted in an average pull-out force decrease of 26.4%. This discrepancy could be attributed to many factors, including differences in stiffness of the FRP laminates, thicknesses of the FRP, or material type. One likely cause was the difference in thickness. Due to the CFRP laminates being thinner than the GFRP, these concrete blocks were subjected to a 1.3 mm (0.05 in.) greater depth of salt solution on average. This could have more adversely affected the interface on the CFRP specimens. For the GFRP specimens, 2 out of 3 A100 specimens performed similarly or even outperformed the A40 specimens of the same bond length. The one A100 specimen that did not was G-152-A100 (the specimen in Fig. 3(c)), which may have underperformed due to the loss of bonded area; it also appears G-152-A40 over performed, as it did not see much reduction from the respective control. These results suggest that the decrease in the pull-out force mostly occurred in the first 40 cycles of accelerated aging. The maximum stress achieved in any GFRP laminate was 106 MPa (15,400 psi) (G-203-A0), or 18% of the manufacturer's provided tensile strength (26% of laboratory tensile strength). The maximum stress achieved in any CFRP laminate was 219.7 MPa (31,900 psi) (C-102-A0), or 24% of the manufacturer's provided tensile strength. These ratios are similar to those tested in previous research; Chen and Teng noted that the ratio of ultimate stress at failure to ultimate tensile strength in FRP bonded to concrete is 28% on average, with a coefficient of variation of 40%¹⁰.

Fig. 6 and Fig. 7 show the pull-out force vs. end-slip curves for all of the GFRP and CFRP laminate specimens. In general, the specimens with accelerated aging had a decreased stiffness and greater end-slip at failure than their respective unaged specimens. This was the case for the 102 and 152 mm (4 and 6 in.) bond lengths of GFRP and all three bond lengths of CFRP. For the 203 mm (6 in.) bond length A0 and A100 GFRP specimens, nonlinear force vs.

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

end-slip behavior resulted in a much higher end-slip at failure than any other case. Another trend that was observed was that increased bond length generally results in a stiffer interface. This result correlates with the effective bond length concept explained by Ouezdou et al.²⁷. Once the bond length exceeds the effective bond length, a portion of the bonded length of FRP closest to the free end does not see local bond stresses until the interface approaches its failure load. As the ratio of bond length to effective bond length increases further, it follows that there would be even less slip at the free end as the FRP laminate is loaded. The unaged series of CFRP laminates clearly exhibits this trend, in which the stiffness increases as the bond length increases from 102 to 203 mm (4 to 8 in.). For the unaged GFRP laminates, the 152 mm (6 in.) bond length was stiffer than the 102 mm (4 in.) bond length, but the 203 mm (8 in.) bond length did not follow this trend. The reduced stiffness of G-203-A0 was likely caused by some bending in the FRP laminate that was induced by a slight misalignment with the testing grips.

NSM FRP bars

Table 4 summarizes the test results for the NSM FRP bars, including the various failure modes. Fig. 8 presents the pull-out force data for the FRP NSM bars in a manner in which comparisons can easily be made between different bond lengths and aging durations. Fig. 9 and Fig. 10 show the post-failure surfaces of all GFRP and CFRP NSM bar specimens.

As was observed with the FRP laminates, an increase in bond length generally corresponded to an increase in pull-out force. This was the case for unaged, 40 cycle, and 100 cycle GFRP bar series. The CFRP bars, however, did not explicitly follow this trend. Unlike the FRP laminates, which did not have any observable differences in failure mode, the multitude of observable failure modes for the NSM bar specimens provides valuable data that can account for some of the discrepancies between the expected and actual results. Due to the lower ultimate strength of the GFRP bars, the failure modes for the NSM GFRP bars fell into one of two categories: failure at the epoxy/bar interface (EB) or FRP bar rupture (FR) outside the bonded length. The EB failure mode was accompanied by concrete spalling along the bonded length (CS), splitting of the FRP bar spiral winding (FS), and removal of portions of the sand coating along the bonded length. For the unaged series, only the 102 mm (4 in.) bond length failed by EB. For the 40 cycle and 100 cycle series, the failure mode of the 152 mm (6 in.) bond length specimens switched from EB to FR, while the 203 mm (8 in.) bond length achieved FR in every case. The maximum stresses for all GFRP bar specimens ranged from 850 MPa (123 ksi) (G-b-102-A40) to 1032 MPa (150 ksi) (G-b-203-A40), or 95% to 115% of the manufacturer's provided tensile strength. This shows that even at the 102 mm (4 in.) bond length with accelerated aging, the bar was close to reaching its ultimate tensile strength. Since EB failure occurred so close to the ultimate strength of the bar, the EB failure mode in this case may actually be triggered by the splitting of the spiral winding within the bonded length. From previous tensile tests of the plain GFRP bar, the spiral winding tends to split just before the bar ruptures. The location of the initiation of FS (inside or outside of the bond length) could be a determining factor as to whether or not EB or FR failure occurred. Considering the range of maximum stresses achieved, this is likely the case.

For the higher strength CFRP bars, FRP bar rupture was not observed. Rather, the failure mode was generally either EB with CS or failure at the epoxy/concrete interface (EC) accompanied with concrete cracking (CC) and epoxy splitting (ES). For the unaged CFRP bars, EB with CS occurred for all bond lengths. For the 152 and 203 mm (6 and 8 in.) bond lengths, there was also concrete fracture at the top of the specimen (CF), perhaps due to uneven compressive forces under the top plate caused by the eccentric location of the NSM groove. As shown in Fig. 10(b), a portion of this fracture concrete affected the bond length of C-b-152-A0, which could explain the underperformance when compared to C-b-102-A0. After aging, the failure mode shifted to EC for some specimens. For the 40 cycle bond lengths of 102 and 203 mm (4 and 8 in.), which failed due to EC, the pull-out force was much lower than the 152 mm (6 in.) bond length which failed due to EB, and also much lower than their unaged counterparts (reductions of 32% and 29%). The concrete cracks associated with EC suggest the failure mode is initiated by concrete cracking, which was not observed with EB failure. The damage to the concrete surface caused by accelerated aging likely prompted the development of concrete cracking, and hence the shift in failure mode. For the 152 mm (6 in.) bond length which retained the EB failure mode, the pull-out force increased by 15% over its unaged counterpart (which further suggests the underperformance C-b-102-A0 due to CF). The 100 cycle 102 and 152 mm (4 and 6 in.) bond lengths saw only a slight decrease in pull-out force from their respective 40 cycle specimens, while retaining the same failure modes. C-b-152-A100 did have a slight mix in failure mode, as seen in Fig. 10(h); EB occurred over the middle portion of the bonded length, while EC occurred at the ends. By comparing the pull-out force with the other CFRP bar specimens, it does appear that EB was likely the primary failure mode. C-b-203-A100 shifted in the test fixture during testing due to loosened concrete aggregates spalling off of the specimen.

The bar was then pulled at a slight angle, resulting in the underperformance of this test. Disregarding this test, the maximum stresses for all CFRP bar specimens ranged from 856 MPa (124 ksi) (C-b-102-A100) to 1553 MPa (225 ksi) (C-b-203-A0), or 38% to 69% of the manufacturer's provided tensile strength.

Fig. 11 and Fig. 12 show the pull-out force vs. end-slip curves for all of the GFRP and CFRP NSM bar specimens. As with the laminates, the specimens with accelerated aging had a decreased stiffness compared to their respective unaged specimens. This was true for all bond lengths of GFRP and CFRP. For the GFRP bars with 152 and 203 mm (6 and 8 in.) bond lengths, the end-slip at failure after 40 cycles was over twice that of the unaged specimens. Even though the failure mode was EB or FR for the aged GFRP bars, the additional end-slip may be attributed to the epoxy/concrete interface, which likely suffered from the accelerated aging while the epoxy/bar interface did not. The plateaus close to the peak load seen in Fig. 12 for C-b-152-A0 and C-b-203-A0 are likely due to the concrete fracture that occurred at the top of these specimens, which caused a slight rotation of the concrete block, resulting in an increase in the displacement measured by the extensometer that is not actually end-slip.

Through comparison of reduction in pull-out force between FRP laminate specimens and NSM bar specimens subjected to same aging cycles, it is observed that the averaged reduction in the pull-out force of NSM GFRP bar specimens after 40 and 100 aging cycles are 1.9% and 0.3%, respectively, while the corresponding averaged reduction of externally bonded GFRP laminate specimens are 12.0% and 16.8%, respectively. Although the data in 100-cycle CFRP laminate specimens are lacking, the averaged reduction of NSM CFRP bar specimens after 40 aging cycles is 15.3% which is lower than 28.0% of CFRP laminate specimens under same condition. The less reduction in pull-out force of NSM bar specimens indicates that NSM bar repair system outperforms externally bonded laminate repair after being exposed to harsh environment.

CONCLUSIONS

In this study, direct shear pull-out tests were performed on concrete specimens reinforced with GFRP and CFRP laminates and NSM FRP bars. Some of these specimens were subjected to an accelerated aging protocol consisting of freeze/thaw cycling in the presence of a deicing salt solution. The purpose of these tests was to determine what effect, if any, the aging process had on the bond-slip behavior and capacity of the FRP/concrete interface. From these tests, the following conclusions were drawn:

- The NSM bar method offers more efficient usage of FRP material than externally bonded FRP laminates with regards to the percentage of ultimate strength of the FRP materials that can be achieved. This is due to the fact that the capacity of a FRP laminate interface is determined by its effective bond length, no matter bonded length of the interface, while the NSM bars can achieve strengths as high as the tensile strength of the FRP itself if the bonded length is sufficient.
- For externally bonded FRP laminates designed to achieve similar repair strength, GFRP with increased thickness performed better on average than CFRP after undergoing freeze/thaw cycling in presence of a deicing salt solution, both in terms of decrease in pull-out force and stiffness of the FRP/concrete interface.
- For externally bonded GFRP laminates, degradation of the FRP/concrete interface occurred mainly in the first 40 cycles of aging.
- GFRP NSM bars saw a negligible decline in performance after accelerated aging for the bond lengths investigated. Failure mode was either FRP bar rupture or failure at the epoxy/bar interface at stress levels close to the rupture strength of the bar and was not affected by aging.
- Some CFRP NSM bar specimens saw a shift in failure mode from the epoxy/bar interface to the epoxy/concrete interface after accelerated aging. Epoxy/concrete interface failure was caused by concrete cracking along the bonded length, which was induced by the scaling damage incurred at the concrete surface. Specimens that shifted failure mode saw a large decrease in pull-out force, while specimens that retained the epoxy/bar failure mode did not see a decrease in pull-out force.
- From these limited tests, the NSM technique for repair and retrofit of concrete structures with FRP appears to be superior to externally bonded laminates in terms of material efficiency and resistance to environmental effects.

ACKNOWLEDGMENTS

The authors acknowledge the financial support provided for this research from the Illinois Department of Transportation (IDOT) under project No. R27-156.

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Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

Table 1—Material Properties of FRP systems

Material	Size ^a , mm (in.)	Area, mm ² (in. ²)	Source	Elastic Modulus, GPa (ksi)	Tensile Strength, MPa (ksi)	Tensile strain (%)
GFRP laminate	2.54 x 76.2 (0.1 x 3)	193.55 (0.3)	Manufacturer	27.4 (3,980)	587 (85.2)	2.3
			Laboratory	22.2 (3,200)	399 (58)	1.80
CFRP laminate	1.24 x 76.2 (0.049 x 3)	94.49 (0.15)	Manufacturer	86.9 (13,000)	930 (135)	0.98
			Laboratory	91.7 (13,000)	1020 (148)	1.10
GFRP bar	6.35 (0.25)	31.67 (0.049)	Manufacturer	46 (6,700)	896 (130)	1.94
			Laboratory	50.3 (7,300)	1062 (154)	2.55
CFRP bar	6.35 (0.25)	31.67 (0.049)	Manufacturer	124 (18,000)	2241 (325)	1.81
			Laboratory	137.9 (20,000)	2461 (357)	2.13
Epoxy filler	--	--	Manufacturer	1.493 (220)	43.5 (6.31)	2.0

Note: ^aThickness by width for laminates, diameter for bars

Table 2—Test Matrix

Specimen	Material/ System	Bond Length, mm (in.)	Aging (# cycles)
G-102-A0	GFRP laminate	101.6 (4)	0
G-152-A0		152.4 (6)	0
G-203-A0		203.2 (8)	0
G-102-A40		101.6 (4)	40
G-152-A40		152.4 (6)	40
G-203-A40		203.2 (8)	40
G-102-A100		101.6 (4)	100
G-152-A100		152.4 (6)	100
G-203-A100		203.2 (8)	100
C-102-A0	CFRP laminate	101.6 (4)	0
C-152-A0		152.4 (6)	0
C-203-A0		203.2 (8)	0
C-102-A40		101.6 (4)	40
C-152-A40		152.4 (6)	40
C-203-A40		203.2 (8)	40
C-102-A100		101.6 (4)	100
C-152-A100		152.4 (6)	100
C-203-A100		203.2 (8)	100
G-b-102-A0	GFRP bar	101.6 (4)	0
G-b-152-A0		152.4 (6)	0
G-b-203-A0		203.2 (8)	0

Specimen	Material/ System	Bond Length, mm (in.)	Aging (# cycles)
G-b-102-A40	CFRP bar	101.6 (4)	40
G-b-152-A40		152.4 (6)	40
G-b-203-A40		203.2 (8)	40
G-b-102-A100		101.6 (4)	100
G-b-152-A100		152.4 (6)	100
G-b-203-A100		203.2 (8)	100
C-b-102-A0		101.6 (4)	0
C-b-152-A0		152.4 (6)	0
C-b-203-A0		203.2 (8)	0
C-b-102-A40		101.6 (4)	40
C-b-152-A40		152.4 (6)	40
C-b-203-A40		203.2 (8)	40
C-b-102-A100		101.6 (4)	100
C-b-152-A100		152.4 (6)	100
C-b-203-A100		203.2 (8)	100

Table 3—Test Results for FRP Laminates

Specimen	Pull-out force, kN (kips)	$\sigma_{\max}$, MPa (ksi)	$\epsilon_{\max}$ (%)	Difference w.r.t unaged (%)	Failure mode ^a
G-102-A0	16.92 (3.80)	87.4 (12.7)	0.32	--	FC
G-152-A0	18.37 (4.13)	94.9 (13.8)	0.35	--	FC
G-203-A0	20.51 (4.61)	106.0 (15.4)	0.39	--	FC
G-102-A40	13.50 (3.04)	69.8 (10.1)	0.25	-20.2	FC
G-152-A40	18.09 (4.07)	93.5 (13.6)	0.34	-1.5	FC
G-203-A40	17.60 (3.96)	90.9 (13.2)	0.33	-14.2	FC
G-102-A100	14.17 (3.19)	73.2 (10.6)	0.27	-16.2	FC
G-152-A100	14.42 (3.24)	74.5 (10.8)	0.27	-21.5	FC
G-203-A100	17.93 (4.03)	92.6 (13.4)	0.34	-12.6	FC
C-102-A0	20.83 (4.68)	219.7 (31.9)	0.25	--	FC
C-152-A0	18.98 (4.27)	200.2 (29.0)	0.22	--	FC
C-203-A0	20.71 (4.66)	218.4 (31.7)	0.24	--	FC
C-102-A40	13.82 (3.11)	145.7 (21.1)	0.16	-33.7	FC
C-152-A40	14.32 (3.22)	151.0 (21.9)	0.17	-24.6	FC
C-203-A40	15.45 (3.47)	162.9 (23.6)	0.18	-25.4	FC
C-102-A100	--	--	--	--	--
C-152-A100	--	--	--	--	--
C-203-A100	--	--	--	--	--

Note: ^aFC = delaminate at FRP/concrete interface

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

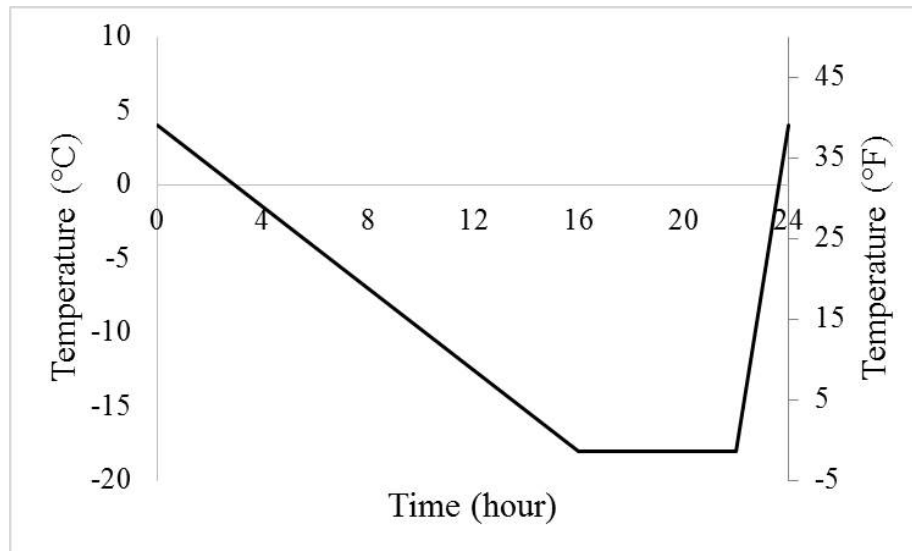
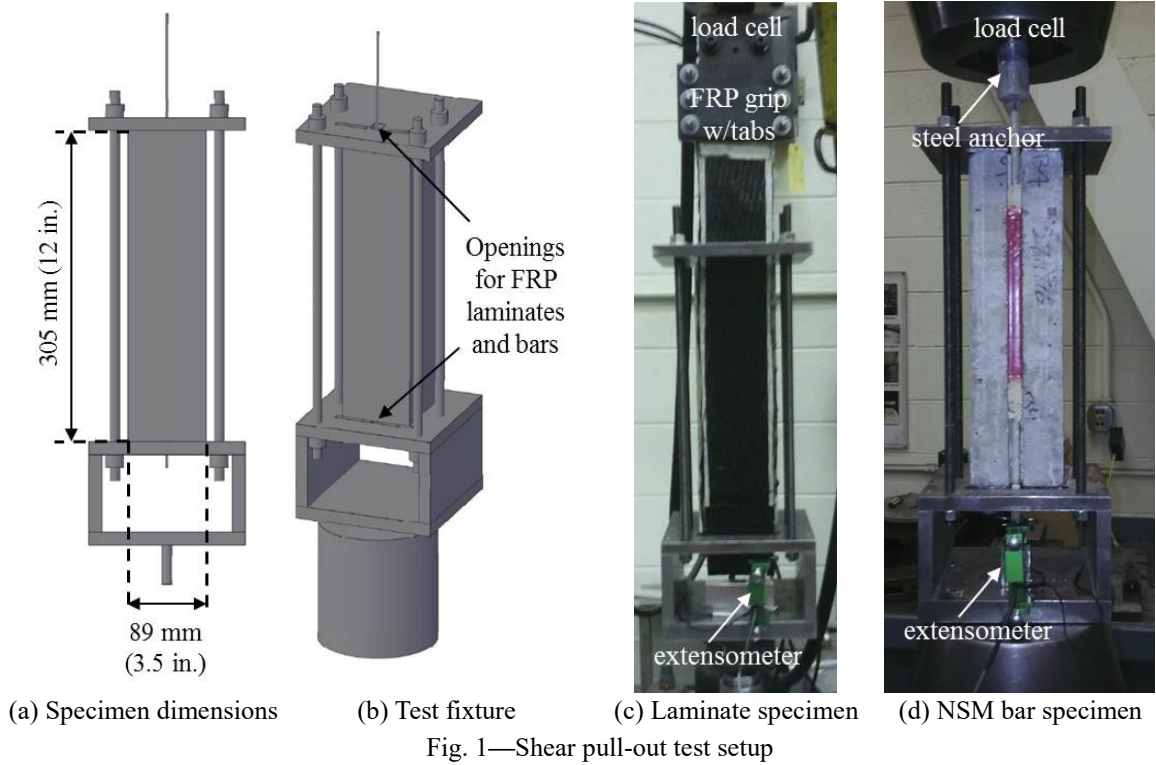
Table 4—Test Results for NSM FRP bars

Specimen	Pull-out force, kN (kips)	σ_{max} , MPa (ksi)	ϵ_{max} (%)	Difference w.r.t unaged (%)	Failure mode ^b
G-b-102-A0	28.61 (6.43)	905.1 (131)	1.96	--	EB+CS+FS
G-b-152-A0	30.15 (6.78)	953.9 (138)	2.06	--	FR
G-b-203-A0	30.68 (6.90)	970.6 (141)	2.10	--	FR
G-b-102-A40	26.88 (6.04)	850.3 (123)	1.84	-6.0	EB+CS+FS
G-b-152-A40	28.12 (6.32)	889.7 (129)	1.93	-6.7	EB+CS+FS
G-b-203-A40	32.63 (7.34)	1032.4 (150)	2.23	6.4	FR
G-b-102-A100	27.99 (6.29)	885.5 (128)	1.92	-2.2	EB+CS+FS
G-b-152-A100	29.32 (6.59)	927.6 (135)	2.01	-2.8	EB+CS+FS
G-b-203-A100	31.97 (7.19)	1011.5 (147)	2.19	4.2	FR+CF
C-b-102-A0	41.55 (9.34)	1314.6 (191)	1.06	--	EB+CS
C-b-152-A0	37.39 (8.41)	1182.8 (172)	0.95	--	EB+CS+CF
C-b-203-A0	49.10 (11.0)	1553.2 (225)	1.25	--	EB+CS+CF
C-b-102-A40	28.23 (6.35)	893.0 (130)	0.72	-32.1	CC+EC
C-b-152-A40	43.01 (9.67)	1360.7 (197)	1.10	15.0	EB+CC
C-b-203-A40	34.76 (7.81)	1099.7 (159)	0.89	-29.2	ES+CC+EC
C-b-102-A100	27.07 (6.09)	856.2 (124)	0.69	-34.9	ES+CC+EC
C-b-152-A100	41.71 (9.38)	1319.6 (191)	1.06	11.6	EB+CC+EC
C-b-203-A100 ^c	26.80 (6.03)	847.9 (123)	0.68	-45.4	ES+CC+EC

Note: ^aBond stress is calculated at the epoxy/bar interface for all specimens, regardless of failure mode

^bES = epoxy splitting; EB = failure at epoxy/bar interface with epoxy breakage; CC = concrete cracking; CS = concrete spalling; CF = concrete fracture at top of specimen; EC = failure at the epoxy/concrete interface; FS = FRP bar spiral winding splitting; FR = FRP bar rupture

^cSpecimen shifted during testing, causing bar to be pulled at an angle



Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles



(a) G-b-102 specimens: A0, A40, A100



(b) Scaling/spalling at specimen ends



(c) Scaling induced loss of bonded area



(d) Severe end damage on C-152-A100

Fig. 3—Effect of accelerated aging on concrete specimens

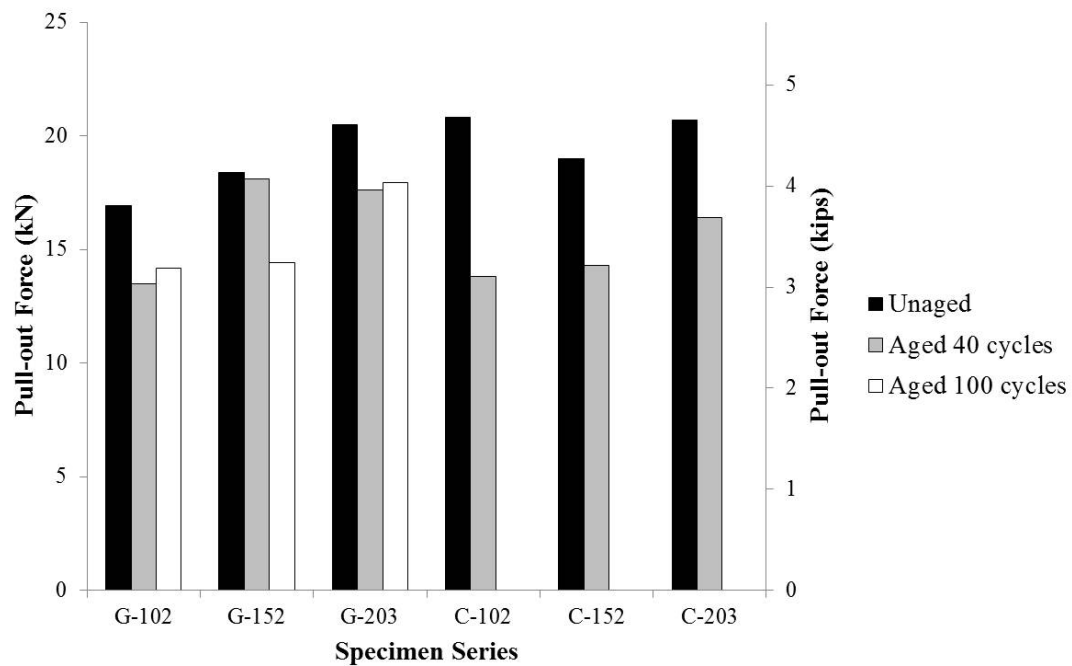


Fig. 4—Pull-out force comparison for FRP laminates

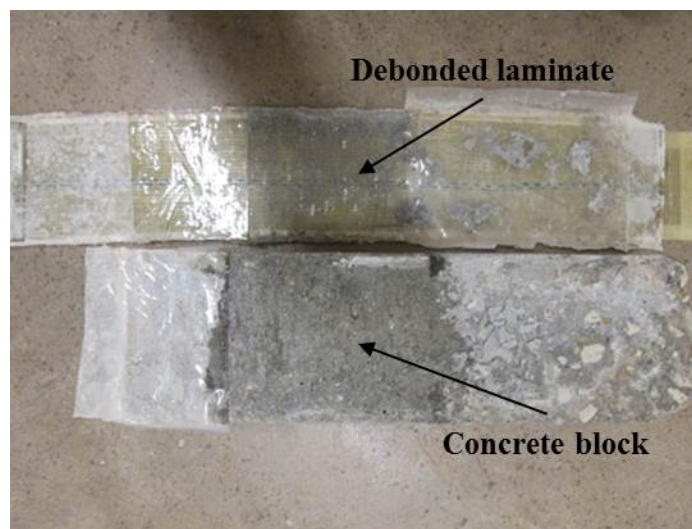


Fig. 5—Delamination failure surface of G-102-A40

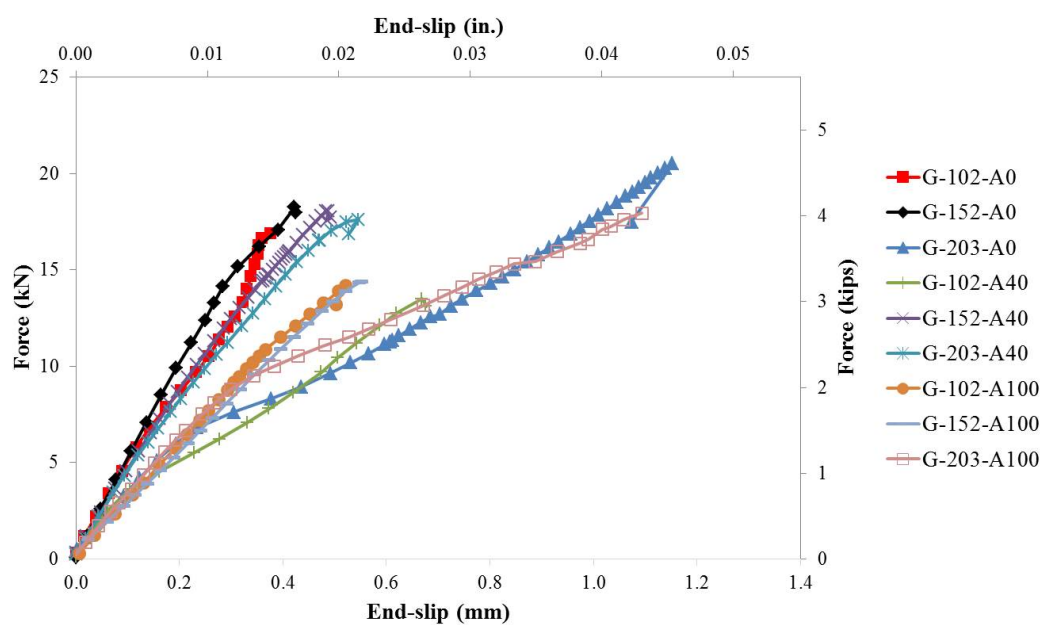


Fig. 6—Pull-out force vs. end-slip for GFRP laminates

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

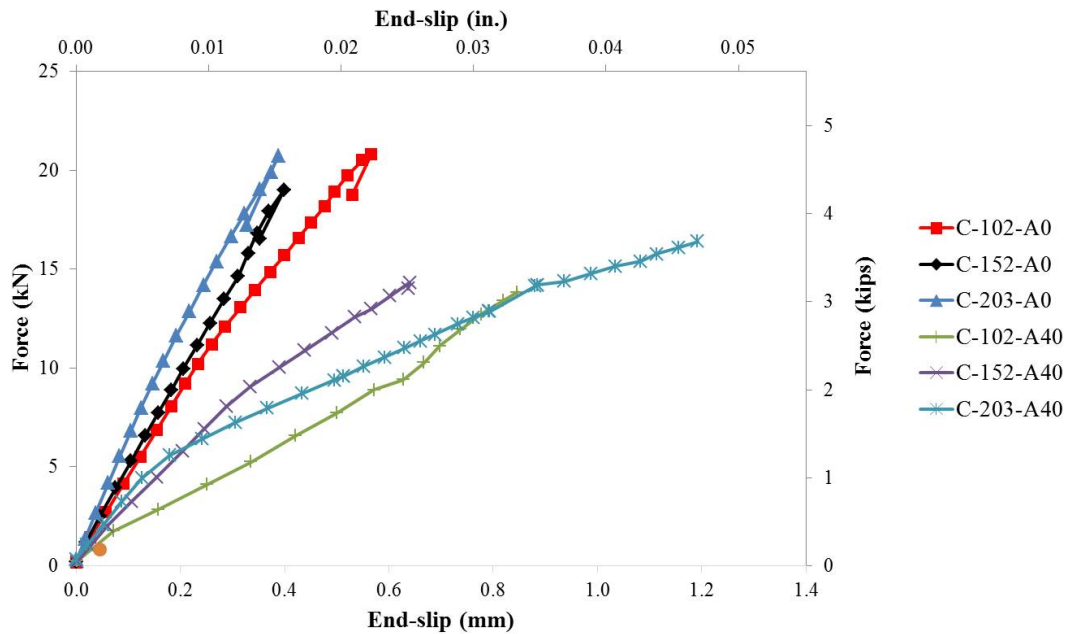


Fig. 7—Pull-out force vs. end-slip for CFRP laminates

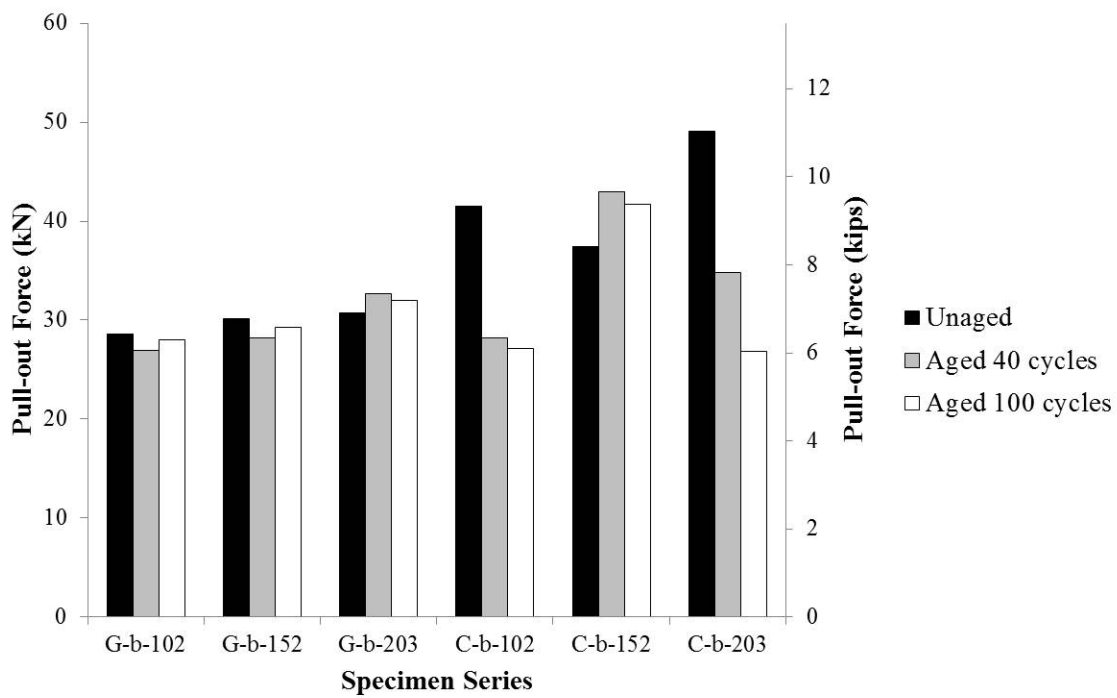


Fig. 8—Pull-out force comparison for FRP NSM bars

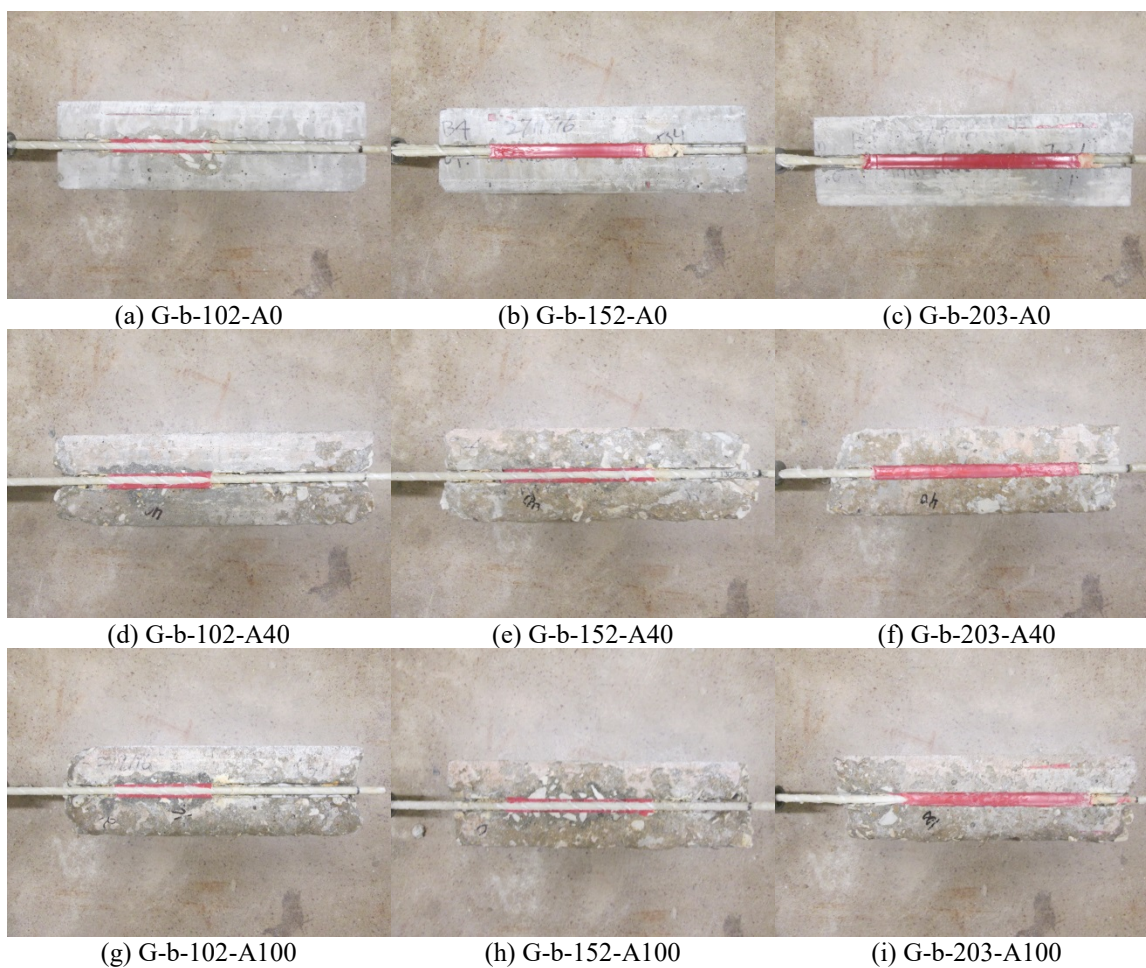


Fig. 9—GFRP NSM bar failure surfaces

Bond Behavior of Concrete Strengthened with FRP Laminates and NSM Bars Subjected to Accelerated Aging using Freeze-Thaw Cycles

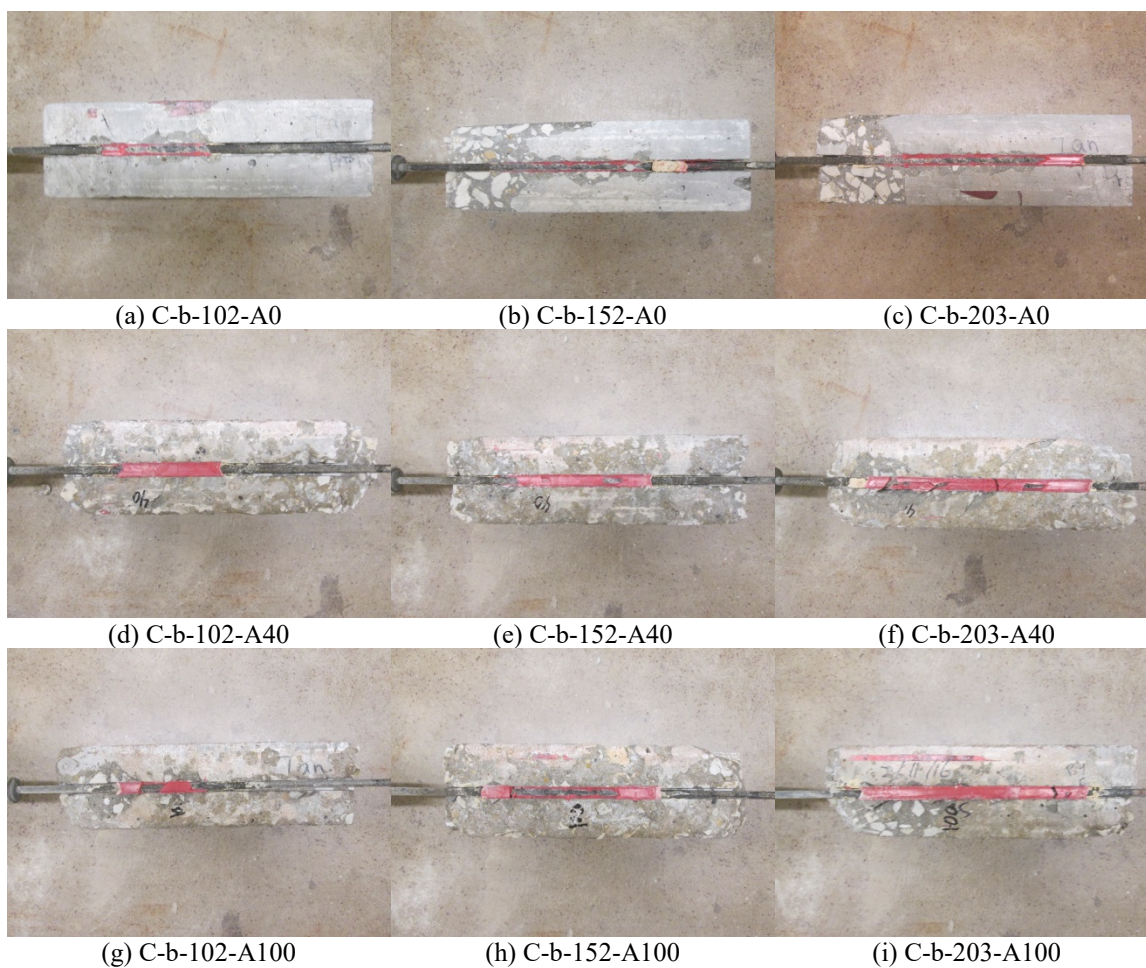


Fig. 10—CFRP NSM bar failure surfaces

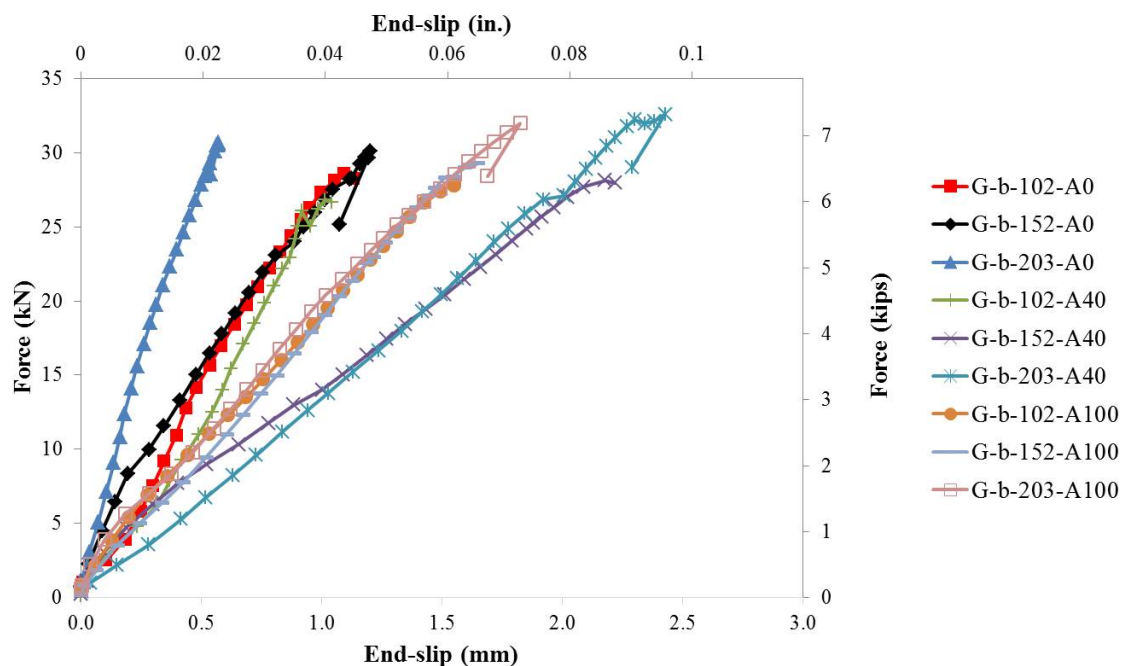


Fig. 11—Pull-out force vs. end-slip for GFRP NSM bars

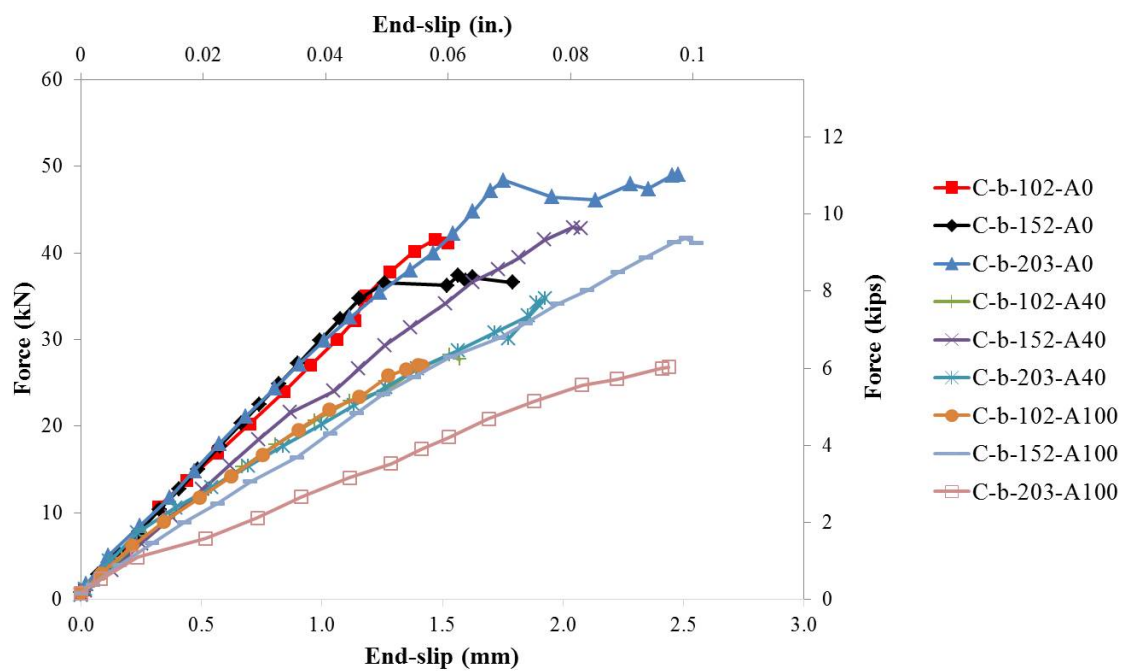


Fig. 12—Pull-out force vs. end-slip for CFRP NSM bars

Cyclic Loading Behavior of CFRP-Wrapped Non-Ductile Beam-Column Joints

Ali S. H. Zerkane, Yasir M. Saeed, and Franz N. Rad

Synopsis: Use of fiber reinforced polymer (FRP) material has served as a proper solution to overcome the weakness of concrete members caused by substandard design, changes in the load distribution, or to correct the weakness of concrete structures subjected to hostile weather conditions. Concrete beam-column joints designed and constructed before 1970s were characterized by weak joints. Lack of transverse reinforcement within the joint region, hence lack of ductility in the joints could be one of the main reasons that many concrete buildings have failed during earthquakes around the world. In the present work, carbon fiber reinforced polymer (CFRP) sheets were used as Externally Bonded FRP System to compensate for the lack of transverse reinforcement in the beam-column joints in order to retrofit the joint region and to transfer the failure to the concrete beams. Six specimens of approximately one-third scale were designed, constructed, and tested. A new technique of rehabilitation scheme is proposed for retrofitting. The scheme proved to be effective in improving the behavior of non-ductile beam-column joints, and to change the final mode of failure. The comparison between beam-column joints before and after retrofitting is presented by load versus deflection, load versus CFRP strain, energy dissipation, and ductility.

Keywords: durability of concrete, cyclic loading, retrofit, CFRP, non-ductile, beam-column joints

Cyclic Loading Behavior of CFRP-Wrapped Non-Ductile Beam-Column Joints

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INTRODUCTION

Large fraction of existing reinforced concrete frames constructed prior to the 1970s are weak in seismic resistance capacity. Most of the buildings designed and constructed based on the earlier codes were characterized by non-ductile performance during earthquakes because the earlier codes did not prescribe required reinforcement details for ductility. As a result, non-ductile behavior of these buildings during earthquakes was expected. Figure 1 shows an example of this type of damage after Kocaeli earthquake, 1999. Most of the older buildings were designed to carry gravity loads and resist wind loads. The results of the examination of collapsed buildings after earthquakes have shown the sources of weakness of these buildings to resist seismic lateral loads. Furthermore, it has been observed that one of the weakest parts in such structures has been the beam-column joints where the beams and columns stayed practically intact but the joints were crushed. Great majority of beam-column joints in existing buildings constructed before the 1970s contain non-ductile reinforcement details where inadequate or no shear reinforcement was provided in the joint region. Moreover, the design of the strong beam caused high shear force on the joint and this may have caused brittle shear failure in the joint regions.

RESEARCH SIGNIFICANCE

Concrete beam-column joints designed and constructed before 1970s lacked of transverse reinforcement within the joint region, hence lacked ductility and potentially were one of the main reasons for many concrete building failures during earthquakes. This paper offers a proposed technique to retrofit the beam-column joints that can improve the cyclic load behavior of non-ductile joints, hence durability of concrete frames. The proposed technique uses one or two layers of L-shaped CFRP sheets, plus circumferential wraps to the beam and column sections near the joint.

SCOPE OF STUDY

The scope of this study was to investigate the behavior of concrete beam-column joints under cyclic loading. Three types of joints were tested to investigate their behavior: (1) A ductile joint representing a design based on current (modern) codes, (2) A non-ductile joint representing a design based on older codes, and (3) Four non-ductile joints designed based on older codes (same as case 2) but retrofitted with CFRP sheet. Six beam-column joint specimens of approximately 1/3rd scale were designed, constructed, and tested. Each specimen contained a column section under axial load, plus a cantilevered beam projecting out of the column that was subjected to flexural loading. The first specimen was a ductile joint which was designed based on the current building design code, ACI 318-14 [1]. Test showed that it was structurally sufficient to eliminate the failure in the joint region and forming a plastic hinge in the beam. Second specimen was a non-ductile joint designed based on older codes, without transverse reinforcement. Test showed that it failed by shear in the joint. A proposed technique was used to retrofit the remaining four specimens that were the same as the second specimen, i.e., no-ductile. Two specimens used two layers of CFRP. In two of the four specimens, the columns were subjected to a higher level of axial load. In this study, the comparisons between joint behaviors before and after retrofitting are presented as load versus deflection, CFRP strain, and specimen ductility.

POTENTIAL STRUCTURAL WEAKNESS IN BEAM-COLUMN JOINTS

A beam-column joint could be defined as “that portion of the column within the depth of the beam(s) that frames into column”. Joints may be described as type I and II, based on the concrete frame design. If the concrete frame is designed to carry gravity loads and no inelastic deformations is required, the joints are called type I. If the concrete frame is designed to carry gravity loads and resistance to lateral loads and inelastic deformations are required, the joints are type II. The earlier codes designed the framing members of the joint according to their prevailing stresses, but the design of the joints was generally ignored in these codes. Many buildings were heavily damaged or destroyed during earthquakes, caused by deficiency of the shear capacity in the joint regions. Most type I joints are characterized by one or more of the following conditions: Lack of transverse reinforcement in the joint region, weak-column strong-beam condition, weak concrete. The authors of this paper consider any structural concrete with compressive strength less than 2500 psi (17 MPa) as weak. Type II joints that are constructed in accordance with modern codes are characterized by: adequate transverse reinforcement ties within the joint as shear reinforcement, adequate ductility in the joint regions, and strong-column weak-beam. These features can prevent or delay the failure in the joint and provide opportunity for the plastic hinge to form in the flexural members. The beam-column joints could also be classified based on their position as internal or external joint, or joint shape within the concrete frame. Figure 2 shows different types of beam-column joints based on position of the joint that contains different levels of confinement provided by adjoining beams.

LITERATURE REVIEW

In the earlier design codes, the structural behavior and principles of design of main structural members such as columns, beams, and slabs were well established, but provisions for design of beam-column joints to resist extreme lateral loads were absent. Joints are an essential part of columns, and it is well accepted that columns represent the overall strength and stability of framed structures. Many older buildings could be considered as containing weak-column strong-beam construction, thus leading to non-ductile behavior of joints and potential failure due in earthquakes. Over the past two decades, several techniques have been used to strengthen the joint regions in older structures [2]. Each technique has its pros and cons. An early material used as external reinforcement was steel plates as jackets that consisted of flat or corrugated steel plates. This technique was successful in increasing the capacity by attaching the plates to the concrete surface with epoxy or bolts [3]. Disadvantages were due to cost, corrosion exposure, and lack of fireproofing. Other techniques used have included concrete jacketing. Many experimental tests have been performed using concrete and reinforcement bars to confine joints. But in the field, reinforced concrete jacketing needs an abundance of labor with specialized skills. Too, concrete jackets increase the dimensions and weight of the structure [4]. Then, the use of FRP as an externally bonded reinforcement was proposed to retrofit beam-column joints [5]. FRP composites have high strength, which makes them suitable for this purpose. Use of FRP to retrofit concrete joints offers many advantages: It is light weight, resistant to corrosion, does not affect the function of the buildings, requires simple skills to do the retrofitting, and it is applied rapidly. In this section, a brief overview of a few relevant research projects is provided. Readers are encouraged to review these research projects, shown in “references” at the end of this paper, as well as other research projects.

One of the earlier studies by Park and Paulay [6] was to test and analyze the behavior of external beam-column joints under simulated severe seismic loading. Thirteen specimens were constructed in full scale. The transverse joint reinforcement and the anchorage length of beam reinforcement were the main parameters in this experiment. ACI 318-71 [7] was the main reference for the design of the specimens. The specimens were divided into two groups. The first group was designed so that the plastic hinge occurred in the beam. The second group was designed so that the plastic hinge occurred in the column. The test set up was close to the test set up conducted in this paper. The results showed poor behavior during test cycles due to the deterioration of the moment capacity and anchorage breakdown. After analyzing the failure modes and behavior, it was found that the plastic hinges did not occur where they were designed. All failures occurred at the joint region. One of the methods of strengthening beam-column joint was proposed by Biddah et al. [3]. The authors tested six specimens in one-third scale under static cyclic loading. The specimens represented an external joint of a two-story concrete frame in a nuclear power generation plant constructed in 1969. This experiment proposed a technique using corrugated steel to strengthen external joints. The investigated parameters were the shear reinforcement in the joint, the development length of the beam longitudinal bars, the retrofitting plate thickness, and the retrofitting style. The first set of specimens consisted of three specimens representing the existing external joint. One was the control specimen. The second one was strengthened by steel jackets applied to the beam and column, and in the third specimen, only the column was confined by steel jacket. One of the specimens of the second set was designed based on CSA A23.3-94 [8]. The results showed that the

existing joint performed poorly with brittle failure at the joint. The performance of the specimen designed based on CSA A23.3-94 [8] was much better and the failure happened by developing a plastic hinge in the beam. Finally, the proposed technique was considered effective and showed desirable results in terms of shear strength and energy dissipation, and the failure did not occur at the joint. Another way of retrofitting beam-column joints was suggested by Hakuto et al. [4]. Concrete jacketing was proposed to investigate retrofitting of exterior and interior joints of an existing seven-story reinforced concrete building designed and built in New Zealand at the end of the 1950s. Eight specimens in full scale were constructed, where six specimens represented internal joints, and two specimens represented external joints. The authors reported that the proposed concrete jacket technique was effective to enhance the strength, stiffness, and ductility of these joints, but it was a difficult technique and required intensive labor work.

When FRP was introduced to the market, many studies were performed to investigate new methods of strengthening concrete beam-column joints with FRP. Prota et al. [9] used FRP in forms of rods and sheets to upgrade existing beam-column concrete joints that were designed and constructed according to ACI 318-63 [10]. A new theory investigated in this study was to use FRP rods to increase the flexural strength of columns by utilizing the near surface mounted technique (NSM), and use FRP sheets as a jacket to increase the shear strength and the confinement of the joints. The study consisted of twelve specimens divided into groups. The only difference between these two groups was the axial applied load. All specimens were designed without any seismic details at the joint. Some specimens were retrofitted by wrapping CFRP sheets around the column, and the others were retrofitted by using both FRP rods and sheets. It was found that the combined action of FRP jacketing and NSM rods improved the joint strength by 60%. In the following year, Ghobarah and Said [11] used GFRP laminates to retrofit external beam-column joints. One specimen in full scale was constructed with no shear reinforcement within the joint region. It was expected that the joint would fail before the plastic hinge formation. The objective was to use GFRP laminates as seismic reinforcement to strengthen beam-column joints by providing the lateral confinement and shear resistance for the joint. After testing the control specimen, it was repaired by removing the crashed concrete, and recasting it again by using high strength concrete. Then, GFRP was used to retrofit the joint area. The free ends of the GFRP sheets were anchored by using steel plates and bolts in order to develop the maximum strength of the composite material. The retrofitted specimen showed an increase in the shear resistance, 60% increase in ductility and improvement in the system behavior by forming the plastic hinge in the beam.

In 2002, a new technique to strengthen exterior beam-column joint was proposed by using GFRP sheets and steel angles for exterior beam-column joints [12]. Three specimens in full scale were designed according to pre-1970s codes to resist the gravity loads and lateral loads. As reported by previous studies, the existing joints showed brittle joint failures. The retrofitting technique was able to keep the joint safe and change the failure mode. In addition, the new technique increased the energy dissipation factor of the joint by six times and reduced the stiffness degradation. In the same year, Ghobarah and Said [13] suggested another technique to strengthen beam-column joints by using bi-directional U-shaped GFRP sheets with and without steel plates and anchors. Four beam-column joints in full scale were constructed representing the pre-1970s design. The specimens were characterized by strong-beam weak-column, with no transverse reinforcement in the joint region. It was reported that the new method successfully improved the behavior of the joints, and moved the failure from the joint region to the beam. Other researchers [14] tested six full-scale external beam-column joints designed according to the pre-1970s codes. CFRP and GFRP sheets, in addition to steel plates and anchors were used to retrofit these joints. Both GFRP and CFRP techniques were successful to provide enough confinement to these joints, avoiding any brittle joint failure. Mukherjee and Joshi [15] also reported that both GFRP and CFRP can be effectively used for retrofitting concrete beam-column joints. However, joints with CFRP sheets showed stiffer behavior. Finally, a detailed literature review about all available strengthening techniques of beam-column joints from 1975 to 2003 was summarized and discussed by Engindeniz et al. [2]. It was observed from the literature that concrete jacketing was effective but very expensive. Also, steel jacket is corrodible and could result in undesirable capacity increase and changing the modes of failure [2,5].

EXPERIMENTAL RESEARCH IN THIS PROJECT

The experimental research, including design of specimens, materials properties, casting and curing techniques, test set-up, and instrumentation are presented in this section. The objective of this study was to investigate the behavior of a retrofitted concrete beam-column joint that was designed and constructed prior to the 1970s. Retrofitting was accomplished by using CFRP sheets as an exterior bonded reinforcement. Six specimens of approximately 1/3rd

scale were designed to represent a joint in a typical multi-story concrete building. The six specimens are referred to as J1 through J6. Table 1 shows a summary of the specimens.

Details of specimens

All specimens had identical dimensions; beam cross section was 5×6 in. (13×15 cm), its length was 20 in. (51 cm); column cross section was 5×5 in. (13×13 cm) and its total length was 46 in. (117 cm). The general shape of the specimen was “T” shape, where the two columns connected with a transverse beam at mid height to form the exterior joint. Figure 3 shows the dimensions and the reinforcement details of the Specimens J1 and J2. All specimens were tested under constant column axial load and cyclic flexural load on the beam. J1 was designed in accordance with ACI 318-14 [1], Chapter 18. The seismic provisions were included by using shear reinforcement in the joint and confinement regions within the beam and columns. This specimen was considered as a “ductile” specimen, and the plastic hinge was expected to form in the beam. This joint represents one of the control specimens. The rest of the specimens, i.e., J2, J3, J4, J5, and J6 were designed in accordance with older ACI codes (prior to 1970s), to resist the gravity loads only and the failure was expected to occur in the joints due to lack of transverse steel in the joints. J2 was the control specimen and it represented the behavior of the non-ductile joint. Specimens J3 through J6 were the same as J2, except retrofitted with CFRP.

Materials

Portland Cement Type II and pea gravel with maximum size 3/8 in. (10 mm) were used in all specimens. The grading of aggregates is not shown here for brevity. The longitudinal bars in all specimens were deformed bars Grade 60; #4 bars (No. 10) in the columns, and #3 bars (No. 13) in the beams. Plain 0.25 in. (6 mm) diameter wire was used as a transverse reinforcement in both beams and columns. Mechanical properties of the reinforcing Steel are shown in Table 2. Figures 4 and 5 show the reinforcement details of column and beam specimens. The longitudinal bars in all specimens were the same, and the differences between the specimens were the spacing of the transverse reinforcement and the shape of hooks in hoops and stirrups.

Strain gages

When the reinforcement cages were ready, for the control specimens J1 and J2 four steel strain gages were attached to the beam longitudinal reinforcement at the junction of beam to column. For the rest of the specimens, four strain gages were attached to the beam longitudinal reinforcement, and two strain gages were attached directly to the CFRP sheets.

Properties of concrete

The mix design had a normal compressive strength of 3000 psi at 28-day of age. Details of the concrete mixture are not given here for brevity. Compressive strength of concrete was determined in accordance with ASTM-C39 [16] using 6×12 in. (15× 30 cm) cylinders. For each specimen, cylinders were tested on the same day of the test. The results are shown in Table 3. Flexural strength (modulus of rupture) tests were carried out on specimens cast during casting the main specimens in accordance with ASTM- C 78 [17]. Experimental Results had a range of 390 to 450 psi (2.7 to 3.1 MPa). After casting the specimens and cylinders, three layers of natural burlap and one layer of plastic sheets were used to cover all specimens to keep the moisture in for proper curing. After five days, the wood molds were removed. Figure 6 shows specimen cages in casting molds, and after mold removal.

Carbon Fiber Reinforced Polymer (CFRP) Sheets

CFRP sheets were used as an exterior bonded reinforcement to compensate for the missing reinforcement within the joint rejoin. ACI 440.2R-17 [18] includes recommendations regarding use of CFRP sheets for strengthening concrete structures. In this experimental work, CFRP unidirectional high strength carbon fiber sheets were used; brand names are omitted in this paper. The tensile properties according to ASTM D 3039/ D 3039M-08 [19] and physical properties of the sheets, as provided by the manufacturer, are shown in Tables 4 and 5. Also, tensile tests on seven identical specimens were performed using the same ASTM Standard in the Material Science Lab at Portland State University. The results showed agreement with data from the manufacturer. ASTM D 3039/ D 3039M-08 [19] is comparable to the newer standard, ASTM D 7565/D 7565M-10 [20].

Epoxy, primer layer, putty layer, saturant layer

The epoxy that was used to attach the CFRP sheets to the concrete surface was recommended by manufacturer that supplied the CFRP material. The epoxy consisted of three layers and each one had different properties and the

instructions for use came from the manufacturer. After each specimen was prepared, the primer layer was applied to provide adhesion between the CFRP and the member. The mechanical properties of the primer layer were provided by the manufacturer, shown in the Table 6. This layer consists of two parts A and B, mixed based on mix ratio of 3:1. Before the mixing process, the concrete surface was cleaned with water, and then sun-dried. On the day of application, concrete member was cleaned using an air compressor, then primer was applied by using a paint brush. The recommended curing time for this layer was 24 hours, after which the second layer “putty layer” was applied over the primer layer. The main purpose of this layer was to level out surface defects. The mechanical properties were provided by the manufacturer, shown in the Table 7. This layer also consists of two parts A and B mixed based on the mix ratio of 3:1 by volume. The curing time for this layer was 24 hours. At this point, the joint was ready to be covered with the CFRP sheets. The sheets were pre-cut and prepared for installation. The first coat of the Saturant Layer (mechanical properties provided by the manufacturer, shown in the Table 8) was prepared using the same mixing ratio as the first two layers and applied to the joint by using a 3/8” roller. A dry sheet of CFRP was then applied at the joint region. Rolling continued until there was visible bleeding through the sheets. The CFRP sheets were left to cure for at least seven days at room temperature. This procedure was followed for one layer of the CFRP sheet as well as when two layers were applied for Specimens J4 and J6. Figure 7 shows a specimen after application of the primer epoxy, and after it was wrapped.

Rehabilitation scheme

The rehabilitation technique consisted of applying “L” shaped CFRP sheets at the corners on top and bottom of the beam. The column was then wrapped with one layer of CFRP sheet. The fiber direction was at 0 degree, which means parallel to beam direction. Finally, CFRP sheets were wrapped around the beam to provide shear capacity, confinement, and to delay potential deboning. They covered the hinging zone of concrete as substitution for steel stirrups, based on current ACI building code [1]. For all specimens, the wrapping scheme was the same, but the number of layers was different (see Table 1). Figure 8 shows the dimensions and general details of the technique that was used to retrofit the joints regardless the number of CFRP layers.

Test setup

All specimens were tested in the Concrete Lab at Portland State University. Each specimen was pinned at the ends of column vertically. The general test setup is shown in Figure 9. An axial load of $0.2 f'_c A_g$ was applied to Specimens J1 through J4, and $0.4 f'_c A_g$ to J5 and J6 by using a hydraulic ram connected to a manual pump. A load cell was used to record the load. The cyclic load was applied at the free end of the beam by using a double acting actuator connected to a manual pump. Two pressure sensors were used to measure the applied cyclic load. Two displacement transducers (LVDTs) were used to measure the displacement at the beam tip.

Loading protocol

Generally, a commonly used loading protocol is to apply a few cycles of half yield strain, followed by cycles of full yield strain, followed by cycles of two times yield strain, and so on. The idea for the joint tests in this experimental research project was to “push the system” a bit harder than the protocols commonly used, and to emulate loading closer to subduction type earthquakes that are expected to be stronger and have longer durations. For this reason, decision was made to start by loading that creates 75% of the beam capacity, followed by 90%, then follow in the “strain control” by twice the yield strain. Accordingly, the selected load sequence consisted of two phases. The first phase was load-controlled followed by a strain-controlled phase. The first phase consisted of twenty cycles. In the first ten cycles, a 2.4-kips cyclic load, which created 75% of the maximum theoretical flexural capacity of the beam, was applied at each load cycle. For the second ten cycles, a 2.8-kips cyclic load, which represented 90% of the maximum theoretical flexural capacity of the beam, was applied at each load cycle. Then the second phase (strain control) started, which consisted of 20 cycles of load applied in each direction creating a strain of 0.4% (twice the yield strain of a nominal 60 grade steel) in the beam longitudinal bars. Then the load was increased, if possible, to reach a strain of 0.6% (representing 3 times the yield strain). Strain gages were attached to all four longitudinal steel bars of the beam. The strain-controlled loading protocol was monitored by measuring the strains in the longitudinal bars of the beam. Figure 10 shows the loading protocol. After the specimen was placed in the loading frame, all strain gage and load cell wires were connected to the data acquisition system. During the test, the axial load was maintained at the upper end of the column and the cyclic load was applied at the free end of the beam, by pushing down and releasing, then up and releasing for each full cycle, till failure. The applied force and deflection were recorded at each cycle in addition to the strain gage values that were collected by a data acquisition system.

EXPERIMENTAL RESULTS, ANALYSIS AND DISCUSSION

The data gathered included information about the strength, crack patterns, ductility, failure modes, and energy dissipation.

Ductile Specimen J1

Specimen J1 represented the control specimen of a ductile concrete beam-column joint that was designed based on the seismic provisions of ACI 318-14 [1]. Figure 11 shows Specimen J1 at different stages of loading. Test started by applying an axial load of 15 kips on the column; kept constant. Beam load was then applied and cycled to produce positive and negative beam moments. Crack widths were measured and locations photographed. For brevity, details are not shown here. This specimen sustained 20 cycles in the load control regime, followed by 19 cycles in the strain control regime. At cycle number 39, the jack stroke maxed out and test was stopped. The back side of the column was still intact with no cracks. Figure 11 (d) shows the final cracking/spalling pattern. J1 was the ductile specimen and as evident by its strength and ductility, it was structurally sufficient to delay the failure within the joint region and to develop a plastic hinge in the beam. This specimen had the highest ductility factor and largest deflection. It set the standards against which the behavior of other specimens was measured.

Non-Ductile Specimen J2

Specimen J2 represented the behavior of the control specimen of non-ductile concrete beam-column joint, which was designed based on older versions of the code. Figure 12 shows Specimen J2 at the different stages of loading. The test procedure was the same as Specimen J1, i.e., 15-kip axial load on the column followed by the cyclic load on the beam. After the first ten cycles in the load control regime, a few significant cracks formed (Figure 12 (c)). By the end of cycle 11, two major diagonal cracks, “X” shape, formed at the center of the joint. After cycle 20, the load control regime ended and second stage (strain control) started. The load was applied in each direction until the strain on the beam longitudinal bars reached 0.004. At the first cycle, the strain in the beam’s top steel bars reached 0.004 as planned. The maximum applied load was 4.55 kips. During the next four cycles, this specimen showed significant degradation in the strength of the joint and the specimen lost load capacity. The test ended at end of cycle 25 when the strain reached 0.1% (indicating a steel stress of half yield capacity), and the maximum deflection was 1.25 in. The slippage of reinforcement and loss of concrete cover caused severe strength degradation, on the order of 50%. Figure 12 (d) shows the condition of the joint at the end of the test. Specimen J2 was the non-ductile specimen and as expected, it was structurally inferior, as it took less load and a lower number of cycles before failure. Its capacity was about 79% and number of cycles was 64% of the capacity and cycles resisted by J1. The result confirmed that the joint designed based on older codes was insufficient and could not handle cyclic loading protocol selected for this experimental project.

Retrofitted Specimen J3

Specimen J3 represented a non-ductile concrete beam-column joint, same as J2, but retrofitted by using one layer of CFRP. Figure 13 shows Specimen J3 at different stages of loading. Two additional strain gages were used in this test on top and bottom of the joint to measure the strains in the CFRP sheets. The same load sequences were followed as before. Because the joint and sections of beam and column were wrapped by CFRP sheets, it was not possible to track the cracks. In the first 20 cycles of load control regime which represented 75% and 90% of beam capacity, and the first 20 cycles of strain control regime (0.4% strain) there was no deterioration in the specimen’s strength. The second stage of the strain control regime was based on developing a 0.6% strain in the beam flexural bars. During the first cycle in this set, the joint showed a high level of the strength as measured by the applied load, but the CFRP sheets ruptured in X-shape form, between the beam and column. With the advance of the cycles at the same strain level, the CFRP sheets started to rupture more and gave a clear sign that the failure was imminent. After five cycles in this strain regime, the CFRP sheet ruptured completely. The concrete cover on both sides had spalled off when the maximum strain in the CFRP sheets was about 0.6%. On the other hand, the beam and the wrapped areas around the column above and below the joint were still intact. Figure 13 (d) shows Specimen J3 at the failure, at end of cycle 45. This was the non-ductile specimen retrofitted with one layer of the CFRP sheet, and its failure of was due to CFRP rupture. Its capacity was about the same as J1, and number of cycles was 13% more than of the cycles resisted by J1. The result suggests that a joint designed based on older codes but retrofitted with the proposed scheme can behave essentially the same as a joint designed based on ductility specifications of modern codes.